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Reliability analysis of geosynthetic-reinforced steep slopes

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ABSTRACT: In slope stability analysis, the high degree of uncertainty associated with design parameters has led to increasing use of reliability-based approaches as a means of evaluating the combined effects of such uncertainties on the structure performance. In this study, the reliability level of geosynthetic-reinforced steep slopes designed according to Eurocode 7 (EC7), without any additional margin of safety, was assessed using the commercially-available *Slide* 6.0 software based on Monte Carlo simulation. To validate the EC7 partial factor design method regarding structural reliability, the estimated reliability indexes were compared with the minimum value recommended by Eurocode 0 (EC0). Additionally, through a probabilistic sensitivity analysis, the effect of variability in design parameters on slope reliability was evaluated and discussed. The results have shown that the geosynthetic-reinforced slopes designed to EC7 specifications exhibit generally an adequate reliability level according to EC0.

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The soil friction angle and the friction angle of the soil-geosynthetic interface and, secondly, the surcharge load, were found to be the most significant parameters for the reliability of the analysed slopes. For typical coefficients of variation of design parameters, the EC7 partial factor method tends to be conservative in terms of structural reliability. However, in situations of abnormal high variability, the partial factor methodology may lead to unsafe design, and thus reliability analyses should be implemented.

KEYWORDS: Geosynthetics, reliability analysis, geosynthetic-reinforced slope, Monte Carlo simulation method, probabilistic sensitivity analysis, Eurocodes, variability

1 INTRODUCTION

The absolute safety of a structure cannot be guaranteed. Uncertainty about loading and available resistance, limitations of design methods, use of simplifying assumptions and possible human errors during construction prevent accurate prediction of the structural behaviour and make it necessary to establish socially tolerable risk levels. The slope stability problem, in particular, is commonly associated with various sources of uncertainties, such as geological details missed in the site investigation phase, estimation of soil properties that are difficult to quantify (i.e. the spatial variability in the field cannot be accurately reproduced), variation in pore-water pressure, testing errors and many other important factors, which often cannot be eliminated by reasonable investigate effort or expenditure (Malkawi et al. 2000).

The conventional deterministic slope stability analyses consist of determining the global factor of safety for trial slip surfaces until the slip surface yielding the lowest factor of safety is located. These analyses are based on fixed representative values for design parameters, without explicit consideration of their inherent uncertainty and variability. The target factors of safety, empirically established (i.e. based on past experience), take into account all the uncertainties and risks involved in the design process. However, since it is common to use the same factor of safety value for a given type of application, such as long-term slope stability, without regard to the degree of uncertainty involved in its calculation, the same value of factor of safety is often applied to conditions that incorporate widely varying degrees of uncertainty

(Duncan 2000). In fact, deterministic approaches suffer from several limitations such as the impossibility of establishing a direct relationship between the global factor of safety and the level of reliability of a structure and quantifying the impact of uncertainties in random input variables on the uncertainties of the model outputs.

Most current standards (including Structural Eurocodes) are based on semi-probabilistic safety concepts, using partial safety factors for actions and resistances. The principle behind semi-probabilistic safety analyses is that uncertainties are treated right at sources with the introduction of the “characteristic value” and the “design value” of the parameters, which have a statistical background. If the calculated design value for the effect of actions is lower than the calculated design value for the resistance, the design fulfils the ultimate limit state requirements. These analyses include the concepts of uncertainty and risk since there is a correspondence between the partial safety factors and the reliability index (or probability of failure). According to Eurocode 0 (EC0), a design using EC0 with the partial factors given in Annex A1 and Eurocode 1 (EC1) to Eurocode 9 (EC9) is considered generally to lead to a structure with a reliability index value greater than 3.8 for a 50-year reference period. Holicky and Vrouwenvelder (2005) stated that the most important advantage of the partial factor design method is the possibility of taking into account the uncertainty in individual variables by calibrating the relevant partial factors and other reliability elements. Cardoso and Fernandes (2001) highlighted the importance of a consistent and rational procedure for defining characteristic values for geotechnical parameters since the safety provided by the application of Eurocode 7 (EC7) depends not only on the partial safety factors specified by the code but also on the way the characteristic values are selected.

Probabilistic approaches and reliability analyses have been increasingly applied in slope stability assessment, as a powerful way of evaluating the combined effects of uncertainty and variability associated with soil and reinforcement strength parameters and loadings (e.g. Christian et al. 1994; Kitch 1994; Chowdhury and Xu 1995; Low and Tang 1997a; Griffiths et al. 2007, 2009, 2010; Cho 2010; Fenton and Griffiths 2010; Kitch et al. 2011, Javankhoshdel and Bathurst 2014). Within a probabilistic framework, the design parameters are treated as random variables and consequently, the calculated factor of safety is also regarded as a random variable with a probability distribution. The probability of failure and the reliability index may then be determined and used as performance indicators. Furthermore, probabilistic sensitivity analyses may help to evaluate the effect of variability in individual parameters and identify the most influential variables for the structure reliability. However, it is important to note that probabilistic modelling may be associated with difficulties due to the lack of available information, typical in geotechnical engineering (Beer et al. 2013). Thus, the simplest and most obvious advantage of a probabilistic approach or reliability analysis is to complement a conventional deterministic analysis by incorporating uncertainties associated with the performance of the geotechnical structure to be analysed, thereby allowing for an enhanced assessment of the structure reliability and providing an improved basis for interaction between engineers and decision-makers (Whitman 2000; Chowdhury et al. 2010).

Based on results of reliability analyses, risk assessment is often conducted to help geotechnical engineers in making informed decisions. From an engineering point of view, the risk is associated with the exposure of recipients to hazards and may be defined as the product of the probability of an adverse event and its consequences (Baecher and Christian 2003). In the context of risk analysis, consequence is the outcome or result of a hazard being realised and may include injury or loss of life, reconstruction costs, loss of economic activity, environmental losses, among others (Modarres 2006). The process of risk assessment consists of making a decision recommendation on whether existing risks are acceptable and present control risk measures are appropriate and, if not, whether alternative measures are justified or will be implemented. Therefore, risk assessment not only includes the risk analysis, which involves the definition of scope, danger identification, estimation of probability of occurrence, evaluation of the vulnerability of the elements at risk, consequence identification and risk estimation, but also incorporates the risk evaluation, the stage at which the values and judgement enter the decision process, by including consideration of the relevance of the estimated risks and the associated social, economic and environmental consequences (Fell et al. 2005).

This paper examines the structural reliability of nine geosynthetic-reinforced steep slopes designed according to EC7 (CEN 2004), without any additional margin of safety. The probability of failure and the reliability index are estimated using *Slide* 6.0 software (Rocscience Inc. 2010) based on Monte Carlo simulation. To validate the EC7 partial factor method regarding structural reliability, the obtained reliability levels are compared with the EC0 (CEN 2002) recommended minimum value. Furthermore, through a probabilistic sensitivity analysis, the impact of variability in the input random variables on the slope reliability is investigated and the most significant design parameters are identified. This paper extends previous work on probabilistic slope stability analysis presented in Ferreira et al. (2013).

2 RELIABILITY AND DESIGN UNDER UNCERTAINTY

2.1 The concept of structural reliability

EC0 (CEN 2002) defines reliability as "the ability of a structure or a structural element to fulfil the specified requirements, including the design working life, for which it has been designed; reliability is usually expressed in probabilistic terms". Accordingly, reliability includes safety, serviceability and durability of a structure.

As a measure of reliability, EC0 introduces the reliability index, which may be related to a probability of failure. In this context, "failure" includes not only catastrophic failure – as in the case of a landslide – but also any unacceptable difference between expected and observed performance (Leonards 1975; Baecher and Christian 2003). The relationship between the reliability index (β) and the probability of failure (PF) can be expressed as (EC0):

$$\mathrm{PF} = \Phi(-\beta) \tag{1}$$

where Φ is the cumulative distribution function of the standardised normal distribution. Table 1 shows the relationship defined by Equation 1.

According to EC0, the probability of failure can be expressed through a performance function g such that a structure is considered to survive if $g > 0$ and fail if $g < 0$. Consequently, if g is normally distributed, the reliability index can be calculated as follows:

$$\beta = \frac{\mu_g}{\sigma_g} \tag{2}$$

where μ_g is the mean value of g and σ_g is the respective standard deviation. It is also noted that in this context, β and PF are only notational values that do not necessarily represent the actual failure rates but are used as operational values for code calibration purposes and for comparison of reliability levels of structures.

The required level of reliability for a certain structure depends on the consequences that may arise from a hypothetic failure scenario. EC0 establishes three different Consequences Classes (CC) based on the potential damage in terms of loss of human life and social, economic or environmental impact (Table 2). For example, agricultural buildings where people do not normally enter are comprised in the CC1 class. Residential and office buildings are included in the CC2 class and grandstands or public buildings, where consequences of failure are high, integrate the CC3 class.

The Consequences Classes (CC1, CC2 and CC3) may be associated with the respective Reliability Classes (RC1, RC2 and RC3). For each Reliability Class, EC0 establishes recommended minimum values for the reliability index, as a function of the reference period (Table 3).

2.2 Variability and uncertainty in geotechnical design

Geotechnical variability is a complex attribute that results from many sources of uncertainties. There are three primary sources of geotechnical uncertainties: inherent variability, measurement errors and transformation uncertainties. Inherent variability arises mainly from the natural geologic processes that continually modify the in situ soil mass. Measurement errors are caused by equipment, procedure and/or operator and random testing effects. Equipment effects result from inaccuracies in the measuring devices and variations in equipment geometries and systems employed. Procedure and/or operator effects derive from the limitations in existing test standards and how they are followed. Random testing errors refer to the remaining scatter in the test results which is neither assignable to specific testing parameters nor caused by inherent soil variability. The third source of uncertainties (transformation uncertainties) is introduced when field or laboratory measurements are transformed into design properties using correlation models (Phoon and Kulhawy 1999; Phoon 2004).

In the context of reliability analysis, knowledge of distribution type and characterisation of variability in design parameters are important issues. Normal and lognormal distributions have been often used in geotechnical design to characterise variability in factor of safety (Duncan 2000; Koerner 2002; Sabatini et al. 2002), permeability, friction angle and unit weight of soil (Hoeg and Murarka 1974; Lacasse and Nadim 1996; Low and Tang 1997a, 1997b; Phoon and Kulhawy 1999; Chalermyanont and Benson 2004) and tensile strength of reinforcement (Low and Tang 1997a; Chalermyanont and Benson 2004). Table 4 indicates typical coefficients of variation (COV) for design parameters of particular interest for the current study, compiled on the basis of published data. Typical coefficients of variation for a broad variety of other soil properties may also be found in the literature (e.g. Lee et al. 1983; Phoon et al. 1995; Lacasse and Nadim 1996; Baecher and Christian 2003).

2.3 Monte Carlo simulation method

Several probabilistic methodologies are available for reliability-based design, namely the Monte Carlo simulation, First Order Second Moment method, Second Order Second Moment method, Point Estimate method, Hasofer-Lind approach (FORM), among others. Each method involves different computational effort, provides a different level of accuracy and yields a different insight into the effects of the individual parameters (Baecher and Christian 2003).

The Monte Carlo method provides approximate solutions to a variety of mathematical problems by performing statistical sampling experiments. The method uses randomly generated values for the component variables to determine the probability distribution of the design variable (e.g. factor of safety). Its application requires the knowledge of the statistical distribution of the input random variables. The steps for the implementation of the Monte Carlo method may be outlined as follows (Dai et al. 1993).

1. Generation of random numbers which are independent random variables uniformly distributed over the unit interval between zero and one.

2. Transformation of the random numbers from a uniform distribution to the distribution applicable to the component variable.
3. Calculation of values of all component variables based on the appropriate random numbers.
4. Computation of the design variable (e.g. factor of safety) using the generated values of the component variables.
5. Repetition of steps 1. to 4. for a large number of times. The number of times these steps are repeated depends on the variability of the input and output parameters and the desired accuracy of the output.
6. Creation of a cumulative distribution of the design function using the data obtained from the above simulations.

The method is conceptually simple and has the capability of dealing with a wide range of functions, even those that cannot be expressed conveniently in explicit form. However, it has the disadvantage that it may converge slowly. Further details of this approach have been presented over recent decades by several authors, namely Hammersley and Handscomb (1964), Schreider (1966), Rubinstein (1981), Fishman (1996) and Baecher and Christian (2003).

2.4 Probabilistic sensitivity analysis

Sensitivity analyses have been widely applied in different areas of science and technology, such as engineering design, to investigate how a given model output depends upon the input parameters. This can be motivated simply by the wish of understanding the implications of a complex model but often arises due to the uncertainty about the true values that should be used for the input parameters (Oakley and O'Hagan 2004).

Among the different methods of sensitivity analysis, probabilistic sensitivity analysis is generally considered to be the most rigorous and is gaining widespread use. In design under uncertainty, probabilistic sensitivity analyses are typically performed to quantify the impact of uncertainties in random input variables (characterised by a probability distribution) on the uncertainty of the model output. Results from probabilistic sensitivity analyses have been used in engineering design for a range of purposes, including (Saltelli et al. 2000):

- reducing the dimension of a design problem by identifying the probabilistically insignificant factors;
- checking the validity of a model and the assumptions made on the probability distributions of the random inputs;
- obtaining insights into the design space and the probabilistic behaviour of a model response;
- investigating potential improvement on the probabilistic response by reducing the uncertainty in random inputs.

When applied to risk assessment, probabilistic sensitivity analyses can be very useful for understanding how risk estimates and, particularly, risk-based decisions are dependent on the variability and uncertainty in factors contributing to risk. In other words, sensitivity analyses can help to identify what is governing the risk estimates and, in these circumstances, contribute to risk mitigation by reducing the uncertainty related to the most relevant variables. This may be accomplished, for instance, by means of complementary geotechnical investigation (e.g. field investigation, laboratory testing, etc.).

3 GEOSYNTHETIC-REINFORCED SLOPE MODELS

In this study, nine geosynthetic-reinforced steep slopes designed according to EC7 (CEN 2004), without any additional margin of safety, were modelled and analysed using *Slide 6.0* software (Rocscience Inc. 2010). The geometry of the reinforced slopes is shown in Figure 1. All nine slopes had height H = 8.4 m and were assumed to rest on competent foundations. The reinforcement layout consisted of fourteen horizontal geogrid layers with constant length (L) and vertical spacing s = 0.6 m throughout the slope. For each slope, two geogrids with different tensile strengths were considered so that a stronger geogrid (GGR1) was used for the seven lower layers and a weaker geogrid (GGR2) was employed near the top of the structure. The backfill material was assumed to be a cohesionless granular soil with design unit weight $\gamma_d = 23$ kN/m^3.

Three different slope angles were considered: $\alpha = 60°$ (Slopes 1 to 7), $\alpha = 45°$ (Slope 8) and $\alpha = 75°$ (Slope 9). For Slopes 1 to 4, the design values of the soil internal friction angle (ϕ_d) were taken as 20°, 25°, 30° and 35°, respectively, and no surcharge load was imposed. In the case of Slopes 5 to 7, the design friction angles of the soil were respectively equal to 25°, 30° and 35°, but a uniform vertical surcharge (S_d) of 13 kPa (design value) was applied on the slope crest. Slopes 8 and 9 were also subjected to a surcharge load S_d = 13 kPa and the design friction angle of the backfill material was set at 25°. The soil-geogrid interface friction angle (δ_d) was defined in terms of a δ_d/ϕ_d ratio (δ_d/ϕ_d = 6/7) which was held constant for all slopes and both geogrid reinforcements (GGR1 and GGR2).

The design tensile strength (T_d) of geogrids GGR1 and GGR2 and the reinforcement length provided were checked, following the design procedure proposed by Jewell (1989, 1996), so that the internal and overall equilibrium of the slopes was satisfied. It should be noted that Jewell's charts apply to reinforced slopes with a level crest and resting on a competent foundation, which is the case of the slopes herein investigated.

As previously mentioned, Structural Eurocodes adopt a semi-probabilistic approach for safety verification, using design values for actions and resistances. In a common design process, the design values of the variables would be determined from their characteristic values using partial safety factors. However, in the present study, the design values were defined first, to ensure that the design load was equal to the design strength. The characteristic values were then back-calculated, using partial safety factors in accordance with the Combination 2 of the Design Approach 1 of EC7 for verification of the ultimate limit state GEO (related to failure or excessive deformation of the ground) in persistent and transient situations. Since the code does not specify which partial safety factor should be used for the determination of the design tensile strength of geosynthetics, a partial safety factor meeting the requirements of the ISO/TR 20432:2007 (ISO 2007) was considered.

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The mean values of the design parameters to be used in the probabilistic stability analysis of the reinforced slopes were then determined, assigning a statistical distribution to the design parameters which were considered as random variables (soil unit weight, soil friction angle, soil-geogrid interface friction angle, tensile strength of geogrids and surcharge load). With the exception of the surcharge load, the variables were assumed to be normally distributed. Typical coefficients of variation were assigned to each of the distributions using data reported in the literature (see Table 4). For these variables, the characteristic values were assumed as quantiles of 5% or 95% of the statistical distributions depending on whether the parameters contribute to safety or not, respectively. The surcharge load was statistically characterised by an exponential distribution and its mean value was set equal to its characteristic value.

Table 5 presents the partial safety factors (PSF) and the coefficients of variation (COV) adopted in this study. Table 6 lists the design values (DV), the characteristic values (CHV) and the mean values (MV) of the design parameters for all the slopes analysed.

Using the simplified Bishop's method, which is one of the most commonly adopted limit equilibrium methods for slope stability analysis and is widely accepted as reasonably accurate, the deterministic Global Minimum circular slip surfaces and the ratio of the design strength to the design effect of actions (the so-called over-design factor) were obtained (Figures 2 to 10). Since the over-design factors are equal to unity, Slopes 1 to 9 fulfil the EC7 safety requirements, but no additional margin of safety is established.

4 RELIABILITY ANALYSIS OF GEOSYNTHETIC-REINFORCED SLOPES

There are two types of probabilistic stability analysis which may be carried out with *Slide 6.0* software (Rocscience Inc. 2010): the Global Minimum method (fixed method) and the Overall Slope method (floating method). With the Global Minimum option, the probabilistic analysis is carried out only on the Global Minimum slip surface located by the deterministic slope stability analysis. The factor of safety of this single surface is recalculated *n* times (where *n* is the number of Monte Carlo simulations) using random values for the input parameters. The probability of failure is then computed as the number of analyses which result in a factor of safety less than unity, divided by the total number of samples. With this approach, the probability of failure (or the reliability index) of the deterministic Global Minimum slip surface is considered representative of the probability of failure for the slope.

With the Overall Slope analysis type, the entire search for a Global Minimum slip surface is repeated *n* times. For each search iteration, a new set of random variables is first determined and the Global Minimum slip surface is then located. The Overall Slope reliability is based on the distribution of factors of safety obtained for all the Global Minimum slip surfaces located by the analysis. Since several Global Minimum slip surfaces are generally encountered, the probability of failure and the reliability index calculated for the overall slope are not associated with a specific slip surface. A potential advantage of the Overall Slope method, when compared with the Global Minimum method, is that it does not assume that the probability of failure of the slope is equal to the probability of failure of the deterministic Global Minimum slip surface. However, it involves a substantially greater computation time.

The application of the Monte Carlo simulation method requires that design parameters be characterised by their probability distributions, which describe the range of possible input values along with their probability of occurrence. As mentioned before, the unit weight (γ) and friction angle (ϕ) of the soil, the soil-geogrid interface friction angle (δ), the tensile strength of the geogrids (*T)* and the surcharge load (*S*) were treated as random variables. The coefficients of variation for the input variables which were assumed to follow normal distributions (γ, ϕ, δ and T) were previously indicated in Table 5. The mean values for all the design parameters were presented in Table 6. The surcharge load considered in the design of Slopes 5 to 9 was assigned an exponential distribution defined by a mean of 10 kPa and minimum and maximum values respectively equal to 0 kPa and 30 kPa, aiming to cover the high degree of uncertainty often associated with this variable.

Figure 11 illustrates the evolution of the reliability index of one example slope (Slope 5) as a function of the number of Monte Carlo simulations, obtained from the Global Minimum and Overall Slope methods. The number of random trials adopted (n = 1 000 000) was high enough to ensure the convergence of the simulations, and hence adequate accuracy in the results. This number of Monte Carlo simulations was maintained in all the analyses performed.

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Figures 12 and 13 compare additional data from the probabilistic stability analysis of Slope 5, carried out using the Global Minimum and Overall Slope options. The probability density functions and the cumulative probability distributions of the factor of safety determined from the former methods are presented in Figure 12. Figure 13 shows the values of the mean factor of safety, probability of failure and reliability index (assuming a normal distribution of the factor of safety results). The reliability indexes estimated from the Global Minimum and Overall Slope methods were respectively 4.141 (Figure 13a) and 4.130 (Figure 13b), which exceed the target value established by EC0 for structures of RC2 and a 50-year reference period (β = 3.8). Also visible in Figure 13b are the multiple Global Minimum slip surfaces which were located throughout the Overall Slope probabilistic analysis. Nevertheless, the difference between the results obtained from both methods is not significant (Figures 12 and 13), which may be attributed to the fact that the correlation between the factors of safety of different failure surfaces is very high. As pointed out by Cornell (1967), for the case of highly correlated failure modes, the contribution to the system probability of failure from failure surfaces other than that associated with the maximum probability of failure may be small, even though they are numerous. However, it has been mentioned in the literature that for cohesive soil slopes with spatial variability in the soil parameters, the overall probability of failure may be significantly higher than the probability of failure associated with a fixed critical slip surface (Cho 2010; Javankhoshdel and Bathurst 2014).

The results from the reliability analysis of the different slopes investigated in the current study are summarised in Table 7. Regardless of the reinforced slope considered, the different analysis types (Global Minimum and Overall Slope) provided quite similar results. It can be observed that the values of the factor of safety and the reliability index increased progressively with the slope angle (Slopes 8, 5 and 9), which is in agreement with the results reported by Kitch (1994) for two geogrid-reinforced slopes with slope angles of 45° and 70°. The data presented in Table 7 also show that the reliability index decreased progressively as the friction angle of the soil was increased (Slopes 1 to 4 and Slopes 5 to 7). In other words, the reliability index was found to decrease as the relative contribution of the soil shear strength to the slope stability increased (i.e. when the soil friction angle increased, and hence the required reinforcement contribution to strength decreased) and, on the other hand, it was found to increase when the relative contribution of the reinforcement tensile strength to the slope stability increased (i.e. when the slopes became steeper and the tensile strength of the geogrids was increased so as to ensure the slope stability). This may be justified by the fact that the coefficient of variation of the soil friction angle (COV = 7%) was higher than that of the geogrid tensile strength (COV = 5%), and thus decreasing the uncertainty associated with the strength properties increases the reliability index and vice-versa.

For the number of simulations performed (n = 1 000 000), all the factors of safety obtained corresponded to safe situations (FS ≥ 1), and hence the computed probabilities of failure for Slopes 1 to 9 were equal to zero (Table 7). In any case, even considering the normal distribution fit for this zone, the number of points in the vicinity of FS = 1 was limited. To more accurately characterise the probability of failure using the Monte Carlo method, the use of a technique able to generate a relevant number of sets of values for design parameters resulting in a factor of safety close to unity would be required.

As shown in Table 7, with the exception of Slopes 4 and 7, whose soil friction angle was the highest considered in this study ($\phi_d = 35°$), the reliability indexes for the different geogrid-reinforced slopes were greater than the EC0 recommended minimum value (β = 3.8). This finding supports the idea that, for common structures, the EC7 partial factor design method tends to be conservative from the point of view of structural reliability.

5 PROBABILISTIC SENSITIVITY ANALYSIS

5.1 General

In design under uncertainty, probabilistic sensitivity analyses are commonly performed to evaluate the effect of variability in random input parameters on the probabilistic characteristics of a design performance. Results from probabilistic sensitivity analyses may be particularly useful when used as a design aid in decision making for selecting the most suitable design based on specific project constraints (e.g. target cost or schedule).

In the present study, *Slide* 6.0 software was used to perform a probabilistic sensitivity analysis of the variability associated with the design parameters of Slopes 5, 6, 8 and 9. The primary objectives of this analysis were the following: to ascertain how the variability in design parameters influences the reliability level of geosynthetic-reinforced steep slopes with different slope angles and soil friction angles; to identify the most relevant parameters regarding the reliability of these structures; and to assess the level of safety margin of the EC7 partial factor method with respect to structural reliability, which may be of particular importance in the absence of clear information concerning the variability associated with design parameters.

To perform the sensitivity analysis, different COVs around the most likely value (the value adopted in the reliability analysis presented in the previous section) were assigned to soil and reinforcement parameters (γ, ϕ, δ and *T)* on the basis of published data (see Table 4). With respect to the surcharge load (*S*), the analysis was carried out by varying the upper bound value. Table 8 indicates the COVs and surcharge limits considered. For each soil or reinforcement parameter, its COV was varied within the considered range, while the COVs corresponding to the remaining parameters and the surcharge limits were held constant at their most likely values. Similarly, for the probabilistic sensitivity analysis of the variability associated with the surcharge load, the COVs of the soil and reinforcement parameters were kept constant and equal to their most likely values. For each combination of COVs and surcharge limits, a probabilistic stability analysis (using the Global Minimum option) with 1 000 000 Monte Carlo simulations was performed and the model response was evaluated. Given the similarity between the results obtained from the Global Minimum and Overall Slope methods in the previous probabilistic analyses (see section 4), the added computation time required to carry out the sensitivity analyses using the Overall Slope option seemed unwarranted.

5.2 COV of soil unit weight

Table 9 presents the results of the probabilistic sensitivity analysis carried out to investigate how the uncertainty in the soil unit weight may affect the reliability of Slopes 5, 6, 8 and 9, in terms of the mean factor of safety, probability of failure and reliability index.

From Table 9 it can be concluded that the variation of the COV of the soil unit weight from 1% to 10% did not significantly affect the reliability level of the reinforced slopes, which could be expected since the soil weight influences both the normal and shear forces acting on each slice. In fact, although a slight reduction in the reliability index may be identified, the values of the mean factor of safety obtained for each slope remained nearly constant as the COV of the soil unit weight was increased. Furthermore, it is possible to observe that all the calculated reliability indexes were above the EC0 reference value for a structure of RC2 and a 50-year reference period (β = 3.8). These results suggest that, for conditions similar to those adopted in this study, a reinforced slope designed according to the partial factor method proposed by EC7 maintains an adequate level of reliability even if the real COV of the soil unit weight reaches 10%, provided that the variability in the remaining design parameters corresponds to the expected value (the value considered in the slope design).

5.3 COV of soil friction angle and soil-geogrid interface friction angle

The influence of the simultaneous variation of the COV of the soil friction angle and soil-geogrid interface friction angle was evaluated by using different COVs ranging from 2% to 15%. The results from these simulations are shown in Table 10. The probability of failure and the reliability index for all four slopes were significantly affected by the increase in the variability associated with the shear strength of the soil and soil-geogrid interface. When the COV of the friction angles was increased up to 10% or 15%, the values of the reliability index fell below the EC0 reference value and the probability of failure reached maximum values of about 1.2%, 1.6%, 2.1% and 0.4% for Slopes 5, 6, 8 and 9, respectively. These results suggest that the friction angles of soil and soil-geogrid interface play a decisive role on the slope reliability, regardless of the slope angle. Kitch (1994) and Kitch et al. (2011) obtained a similar conclusion regarding the effect of the variability in the soil shear strength on the reliability of geogrid-reinforced slopes, using the first-order reliability method (FORM). Thus, in the design and stability analysis of geosynthetic-reinforced steep slopes, efforts should be made in order to properly characterise the uncertainty associated with the soil and soil-geosynthetic interface shear strength, which may be accomplished by means of direct shear tests on soil and soil-geosynthetic pullout and direct shear tests (e.g. Sukmak et al. 2015; Hatami and Esmaili, 2015; Ferreira et al. 2015a, 2015b), using the specific materials to be used on the project. The obtained results also suggest that a reinforced slope designed according to EC7, without any additional margin of safety, will probably not be a reliable structure (according to EC0) if the real COVs of the soil and interface friction angles reach values higher than those adopted in the slope design.

5.4 COV of geogrid tensile strength

In order to understand how the variability in the geogrid tensile strength may affect the reliability of geogrid-reinforced steep slopes designed according to EC7, different COVs in the range of 1% to 10% were assigned to this variable (Table 11). The results presented in Table 11 demonstrate that the variation of the COV of the geogrid tensile strength over the considered range did not have a relevant influence on the mean factor of safety, probability of failure and reliability index of these particular slopes. Despite being of little importance, the influence of the variability in the geogrid tensile strength on the obtained reliability indexes increased with the slope angle (Slopes 8, 5 and 9), which is consistent with the results presented by Kitch (1994) and Kitch et al. (2011). Regardless of the variability in the geogrid tensile strength, the values of the reliability index were greater than the EC0 recommended value for structures of RC2 and a reference period of 50 years (β = 3.8). Therefore, the geogrid tensile strength is not considered to be a significant design parameter with respect to the structural reliability of the analysed slopes. However, this is probably related to the fact that the mobilised tensile strength of the geogrids is well below their design strength. In cases where the previous values are closer, the variability in the geogrid tensile strength may have much more impact on slope reliability. As shown by Kitch (1994) and Kitch et al. (2011), for internal failure modes (critical slip surfaces passing predominantly through the reinforced portion of the slope), the variability in the geogrid tensile strength may have a marked influence on the reliability level of geogrid-reinforced slopes.

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5.5 Upper limit of the surcharge load

The effect of the variability in a surcharge load (with a mean value of 10 kN/m^2) applied on the top of the slopes was studied by defining different upper bound values for the corresponding exponential probability distribution (from 15 kPa to 50 kPa). The results from this sensitivity analysis are presented in Table 12. As can be noted from the table, the increase in the variability of the surcharge load induced a small reduction in the reliability index of the slopes. Moreover, it is important to highlight that for Slopes 6 and 8, the reliability index obtained when the upper limit of the surcharge load was set at 50 kPa was slightly lower than the EC0 recommended minimum value (β = 3.8).

5.6 Limit combinations

In order to better understand to what extent the combined variability in the design parameters may affect the reliability of geosynthetic-reinforced slopes, two additional combinations were analysed in which the variability in all the input parameters was set very low or abnormally high (Table 13).

From Table 13 it can be concluded that if the variability associated with all the design parameters is very low, the reliability index of each slope more than triplicates the value obtained from the probabilistic analysis in which the most likely COVs and surcharge limits were considered. In contrast, as would be expected, if the variability is abnormally high, the probability of failure substantially increases and the reliability index undergoes a sharp reduction. As shown in Table 13, the minimum reliability index obtained for the studied slopes was about 1.8, which is less than half the value recommended by EC0 for structures of RC2 and a reference period of 50 years, corresponding to totally unacceptable probabilities of failure.

6 CONCLUSIONS

This paper investigates the structural reliability of geosynthetic-reinforced steep slopes designed according to EC7 (without any additional margin of safety), using the Monte Carlo method, and compares the estimated levels of reliability with the EC0 recommended minimum

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value. A probabilistic sensitivity analysis is then performed, enabling the evaluation of the effect of the variability associated with input random variables (i.e. soil and reinforcement parameters and loadings) on slope reliability. Based on the obtained results, the following conclusions can be drawn.

Among the nine geosynthetic-reinforced steep slopes analysed in this study, seven exhibited a reliability index greater than the EC0 recommended minimum value for structures of Reliability Class 2 and a reference period of 50 years (β = 3.8). Only those whose soil friction angle was the highest herein considered (design value of 35°) presented a reliability index slightly lower than the EC0 target value. Therefore, for usual values of design parameters, the EC7 partial factor method leads generally to a structure meeting the EC0 requirements in terms of structural reliability.

Since the variability and uncertainty associated with the reinforcement strength is typically lower that that related to the ground strength properties, the reliability index of the geosynthetic-reinforced slopes decreased as the relative contribution of the soil shear strength to the slope stability increased (i.e. when the soil friction angle increased, and consequently the required reinforcement contribution to strength decreased) and, on the other hand, it increased when the relative contribution of the reinforcement strength to the slope stability increased (i.e. when the slopes became steeper and the tensile strength of the geogrids was increased so as to ensure the slope stability).

No relevant differences between the results of the reliability analyses carried out using fixed or floating probabilistic methods (i.e. the Global Minimum and Overall Slope methods available in *Slide* 6.0 software) were observed.

The probabilistic sensitivity analysis of the variability associated with the input random variables revealed that the soil friction angle and that of the soil-geosynthetic interface and, secondly, the surcharge load, were the design parameters that had the most influence on the reliability of the investigated slopes. Therefore, in design and stability analysis of geosynthetic-reinforced slopes, efforts should be directed at reducing the uncertainty associated with such parameters.

For common variability in design parameters, the EC7 partial factor method tends to be conservative from the point of view of reliability. In any case, for situations where the variability in the input parameters reaches abnormal high values, the partial factor methodology may lead to unsafe design, and hence reliability analyses should be implemented.

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NOTATION

Basic SI units are given in parentheses.

H – slope height (m)

L – reinforcement length (m)

n – number of Monte Carlo simulations (dimensionless)

s – vertical spacing of reinforcement layers (m)

S – surcharge load (N/m^2)

S_d – design value of surcharge load (N/m^2)

T – geogrid tensile strength (N/m)

T_d – design value of geogrid tensile strength (N/m)

α – slope angle (degrees)

β - reliability index (dimensionless)

γ – soil unit weight (N/m^3)

γ_{d} – design value of soil unit weight (N/m^3)

δ – soil-geosynthetic interface friction angle (degrees)

δ_d – design value of soil-geosynthetic interface friction angle (degrees)

ϕ – soil friction angle (degrees)

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ϕ_d – design value of soil friction angle (degrees)

ABBREVIATIONS

CC – Consequences Class

CHV – characteristic value

COV – coefficient of variation

DV – design value

EC0 – Eurocode 0 (CEN 2002)

EC7 – Eurocode 7 (CEN 2004)

FS – factor of safety

GEO – ultimate limit state related to failure or excessive deformation of the ground (EC7)

GGR – geogrid

MV – mean value

PF – probability of failure

PSF – partial safety factor

RC – Reliability Class

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Table 13. Sensitivity analysis for limit combinations

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Figure 10. Global Minimum slip surface and over-design factor of Slope 9 (α = 75°, L = 5.8 m, ϕ_d = 25°, γ_d = 23 kN/m^3)

Figure 11. Convergence of the reliability index of Slope 5 (Global Minimum and Overall Slope methods)

Figure 12. Comparison of results of the probabilistic stability analysis of Slope 5 obtained from the Global Minimum and Overall Slope methods: a) probability density function of the factor of safety; b) cumulative probability distribution of the factor of safety

Figure 13. Results of the probabilistic stability analysis of Slope 5: a) Global Minimum method; b) Overall Slope method

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Table 1. Relationship between β and PF (modified from EC0; CEN (2002))

PF	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}
β	1.28	2.32	3.09	3.72	4.27	4.75	5.20

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Table 2. Definition of Consequences Classes (modified from EC0; CEN (2002))

Consequences Class	Consequence	
	Loss of human life	Social, economic and environmental
CC1	Low	Small/Negligible
CC2	Medium	Considerable
CC3	High	Very great

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Table 3. Recommended minimum values for β for ultimate limit states design (modified from EC0; CEN (2002))

Reliability Class	Minimum values for β	
	1 year reference period	50 years reference period
RC1	4.2	3.3
RC2	4.7	3.8
RC3	5.2	4.3

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Table 4. Typical coefficients of variation of design parameters

Parameter	COV (%)	Source
Soil unit weight	3 - 7	Harr (1984), Kulhawy (1992)
Soil friction angle (granular soil)	2 - 15	Singh (1971), Lumb (1974), Hoeg and Murarka (1974), Schultze (1975), Harr (1984), Kulhawy (1992), Phoon et al. (1995)
Soil-geosynthetic interface friction angle[3]	3.7 - 5.4[1] 10.2 - 16.7[2]	Sia and Dixon (2007)
Geosynthetic tensile strength[3]	1.4 - 6.8	Silvano (2005), Lopes et al. (2006), Vieira (2008), Morais (2010), Pinho-Lopes and Lopes (2013)

[1]Values corresponding to coarse grained soil-geosynthetic interfaces.

[2]Values corresponding to fine grained soil-geosynthetic interfaces.

[3]Based on repeatability testing programmes.

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Table 5. Partial safety factors and coefficients of variation of the design parameters

Parameter	PSF	COV (%)
Soil unit weight	1	5
Soil friction angle	1.25[1]	7
Soil-geogrid interface friction angle	1.25[1]	7
Long-term tensile strength of the geogrids	1.25	5
Surcharge load	1.3	-

[1]Applied to the tangent of the friction angle.

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Table 6. Design values, characteristic values and mean values of the design parameters

	Parameter	DV	CHV	MV
Slope 1 α = 60° L = 7.2 m	Soil unit weight (kN/m^3)	23.0	23.0	21.3
	Soil friction angle (°)	20.0	24.5	27.6
	Soil-geogrid interface friction angle (°)	17.1	21.0	23.8
	Geogrid tensile strength - GGR1 (kN/m)	34.6	43.3	47.1
	Geogrid tensile strength - GGR2 (kN/m)	19.8	24.8	27.0
Slope 2 α = 60° L = 5.4 m	Soil unit weight (kN/m^3)	23.0	23.0	21.3
	Soil friction angle (°)	25.0	30.2	34.2
	Soil-geogrid interface friction angle (°)	21.4	26.1	29.5
	Geogrid tensile strength - GGR1 (kN/m)	24.7	30.9	33.6
	Geogrid tensile strength - GGR2 (kN/m)	14.1	17.6	19.2
Slope 3 α = 60° L = 4.2 m	Soil unit weight (kN/m^3)	23.0	23.0	21.3
	Soil friction angle (°)	30.0	35.8	40.5
	Soil-geogrid interface friction angle (°)	25.7	31.0	35.1
	Geogrid tensile strength - GGR1 (kN/m)	17.3	21.6	23.6
	Geogrid tensile strength - GGR2 (kN/m)	9.9	12.4	13.5
Slope 4 α = 60° L = 3.4 m	Soil unit weight (kN/m^3)	23.0	23.0	21.3
	Soil friction angle (°)	35.0	41.2	46.6
	Soil-geogrid interface friction angle (°)	30.0	35.8	40.5
	Geogrid tensile strength - GGR1 (kN/m)	11.7	14.6	15.9
	Geogrid tensile strength - GGR2 (kN/m)	9.0	11.3	12.3
Slope 5 α = 60° L = 5.8 m	Soil unit weight (kN/m^3)	23.0	23.0	21.3
	Soil friction angle (°)	25.0	30.2	34.2
	Soil-geogrid interface friction angle (°)	21.4	26.1	29.5
	Geogrid tensile strength - GGR1 (kN/m)	26.3	32.9	35.8
	Geogrid tensile strength - GGR2 (kN/m)	15.7	19.6	21.4
	Surcharge load (kPa)	13.0	10.0	10.0
Slope 6 α = 60° L = 4.5 m	Soil unit weight (kN/m^3)	23.0	23.0	21.3
	Soil friction angle (°)	30.0	35.8	40.5
	Soil-geogrid interface friction angle (°)	25.7	31.0	35.1
	Geogrid tensile strength - GGR1 (kN/m)	18.5	23.1	25.2
	Geogrid tensile strength - GGR2 (kN/m)	11.1	13.9	15.1
	Surcharge load (kPa)	13.0	10.0	10.0
Slope 7 α = 60° L = 3.6 m	Soil unit weight (kN/m^3)	23.0	23.0	21.3
	Soil friction angle (°)	35.0	41.2	46.6
	Soil-geogrid interface friction angle (°)	30.0	35.8	40.5
	Geogrid tensile strength - GGR1 (kN/m)	12.5	15.6	17.0
	Geogrid tensile strength - GGR2 (kN/m)	10.0	12.5	13.6
	Surcharge load (kPa)	13.0	10.0	10.0
Slope 8 α = 45° L = 6.3 m	Soil unit weight (kN/m^3)	23.0	23.0	21.3
	Soil friction angle (°)	25.0	30.2	34.2
	Soil-geogrid interface friction angle (°)	21.4	26.1	29.5
	Geogrid tensile strength - GGR1 (kN/m)	17.1	21.4	23.3
	Geogrid tensile strength - GGR2 (kN/m)	10.3	12.9	14.0
	Surcharge load (kPa)	13.0	10.0	10.0
Slope 9 α = 75° L = 5.8 m	Soil unit weight (kN/m^3)	23.0	23.0	21.3
	Soil friction angle (°)	25.0	30.2	34.2
	Soil-geogrid interface friction angle (°)	21.4	26.1	29.5
	Geogrid tensile strength - GGR1 (kN/m)	36.3	45.4	49.4
	Geogrid tensile strength - GGR2 (kN/m)	21.7	27.1	29.6
	Surcharge load (kPa)	13.0	10.0	10.0

Table 7. Results of the probabilistic stability analysis of Slopes 1 to 9

	FS (deterministic)	Global Minimum (n = 1 000 000)			Overall Slope (n = 1 000 000)		
		FS (mean)	PF[1] (%)	β	FS (mean)	PF[1] (%)	β
Slope 1	1.450	1.453	0	4.344	1.449	0	4.464
Slope 2	1.462	1.466	0	4.228	1.461	0	4.167
Slope 3	1.487	1.492	0	3.827	1.492	0	3.901
Slope 4	1.533	1.542	0	3.631	1.543	0	3.641
Slope 5	1.476	1.489	0	4.141	1.489	0	4.130
Slope 6	1.506	1.519	0	3.838	1.516	0	3.893
Slope 7	1.542	1.559	0	3.618	1.558	0	3.639
Slope 8	1.475	1.487	0	3.852	1.485	0	3.847
Slope 9	1.484	1.497	0	4.587	1.498	0	4.591

[1]Determined as the ratio of the number of simulations with FS < 1 to the total number of simulations (n = 1 000 000).

Table 8. Coefficients of variation and surcharge limit values used in the probabilistic sensitivity analysis

COV (%)				Limits (kPa)
γ	ϕ	δ	T	S
1	2	2	1	0 - 15
3	5	5	3	0 - 30
5	7	7	5	0 - 50
7	10	10	7	-
10	15	15	10	-

Table 9. Sensitivity analysis of the COV of soil unit weight

	COV (%)			Limits (kPa)	Results		
	γ	ϕ/δ	T	S	FS (mean)	PF (%)	β
Slope 5	1	7	5	0 - 30	1.489	0	4.153
	3	7	5	0 - 30	1.489	0	4.148
	5	7	5	0 - 30	1.489	0	4.141
	7	7	5	0 - 30	1.490	0	4.125
	10	7	5	0 - 30	1.491	0	4.094
Slope 6	1	7	5	0 - 30	1.519	0	3.847
	3	7	5	0 - 30	1.519	0	3.844
	5	7	5	0 - 30	1.519	0	3.838
	7	7	5	0 - 30	1.519	0	3.832
	10	7	5	0 - 30	1.520	0	3.817
Slope 8	1	7	5	0 - 30	1.487	0	3.853
	3	7	5	0 - 30	1.487	0	3.853
	5	7	5	0 - 30	1.487	0	3.852
	7	7	5	0 - 30	1.487	0	3.851
	10	7	5	0 - 30	1.488	0	3.845
Slope 9	1	7	5	0 - 30	1.497	0	4.640
	3	7	5	0 - 30	1.497	0	4.624
	5	7	5	0 - 30	1.497	0	4.587
	7	7	5	0 - 30	1.498	0	4.534
	10	7	5	0 - 30	1.500	0	4.426

Table 10. Sensitivity analysis of the COV of soil friction angle and soil-geogrid interface friction angle

	COV (%)			Limits (kPa)	Results		
	γ	ϕ/δ	*T*	*S*	FS (mean)	PF (%)	β
Slope 5	5	2	5	0 - 30	1.486	0	10.036
	5	5	5	0 - 30	1.488	0	5.537
	5	7	5	0 - 30	1.489	0	4.141
	5	10	5	0 - 30	1.493	0.0045	2.978
	5	15	5	0 - 30	1.501	1.2325	2.021
Slope 6	5	2	5	0 - 30	1.514	0	10.439
	5	5	5	0 - 30	1.516	0	5.228
	5	7	5	0 - 30	1.519	0	3.838
	5	10	5	0 - 30	1.525	0.0085	2.732
	5	15	5	0 - 30	1.539	1.6232	1.841
Slope 8	5	2	5	0 - 30	1.484	0	10.195
	5	5	5	0 - 30	1.485	0	5.220
	5	7	5	0 - 30	1.487	0	3.852
	5	10	5	0 - 30	1.491	0.0275	2.751
	5	15	5	0 - 30	1.500	2.0566	1.863
Slope 9	5	2	5	0 - 30	1.495	0	9.657
	5	5	5	0 - 30	1.496	0	5.983
	5	7	5	0 - 30	1.497	0	4.587
	5	10	5	0 - 30	1.500	0.0001	3.351
	5	15	5	0 - 30	1.507	0.4336	2.292

Table 11. Sensitivity analysis of the COV of geogrid tensile strength

	COV (%)			Limits (kPa)	Results		
	γ	ϕ/δ	T	S	FS (mean)	PF (%)	β
Slope 5	5	7	1	0 - 30	1.489	0	4.157
	5	7	3	0 - 30	1.489	0	4.150
	5	7	5	0 - 30	1.489	0	4.141
	5	7	7	0 - 30	1.489	0	4.122
	5	7	10	0 - 30	1.489	0	4.086
Slope 6	5	7	1	0 - 30	1.519	0	3.849
	5	7	3	0 - 30	1.519	0	3.846
	5	7	5	0 - 30	1.519	0	3.838
	5	7	7	0 - 30	1.519	0	3.830
	5	7	10	0 - 30	1.519	0	3.810
Slope 8	5	7	1	0 - 30	1.487	0	3.855
	5	7	3	0 - 30	1.487	0	3.852
	5	7	5	0 - 30	1.487	0	3.852
	5	7	7	0 - 30	1.487	0	3.849
	5	7	10	0 - 30	1.487	0	3.842
Slope 9	5	7	1	0 - 30	1.498	0	4.651
	5	7	3	0 - 30	1.498	0	4.629
	5	7	5	0 - 30	1.497	0	4.587
	5	7	7	0 - 30	1.497	0	4.527
	5	7	10	0 - 30	1.497	0	4.402

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Table 12. Sensitivity analysis of the upper limit value of the surcharge load

	COV (%)			Limits (kPa)	Results		
	γ	ϕ/δ	T	S	FS (mean)	PF (%)	β
Slope 5	5	7	5	0 - 15	1.502	0	4.321
	5	7	5	0 - 30	1.489	0	4.141
	5	7	5	0 - 50	1.484	0	4.035
Slope 6	5	7	5	0 - 15	1.530	0	3.958
	5	7	5	0 - 30	1.519	0	3.838
	5	7	5	0 - 50	1.514	0	3.767
Slope 8	5	7	5	0 - 15	1.499	0	3.993
	5	7	5	0 - 30	1.487	0	3.852
	5	7	5	0 - 50	1.482	0	3.766
Slope 9	5	7	5	0 - 15	1.510	0	4.828
	5	7	5	0 - 30	1.497	0	4.587
	5	7	5	0 - 50	1.492	0	4.447

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Table 13. Sensitivity analysis for limit combinations

	COV (%)			Limits (kPa)	Results		
	γ	ϕ/δ	T	S	FS (mean)	PF (%)	β
Slope 5	1	2	1	0 - 15	1.498	0	13.009
	10	15	10	0 - 50	1.497	1.3748	1.991
Slope 6	1	2	1	0 - 15	1.525	0	12.689
	10	15	10	0 - 50	1.536	1.7600	1.823
Slope 8	1	2	1	0 - 15	1.495	0	12.342
	10	15	10	0 - 50	1.496	2.1754	1.843
Slope 9	1	2	1	0 - 15	1.507	0	14.047
	10	15	10	0 - 50	1.504	0.6034	2.227

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FIGURES

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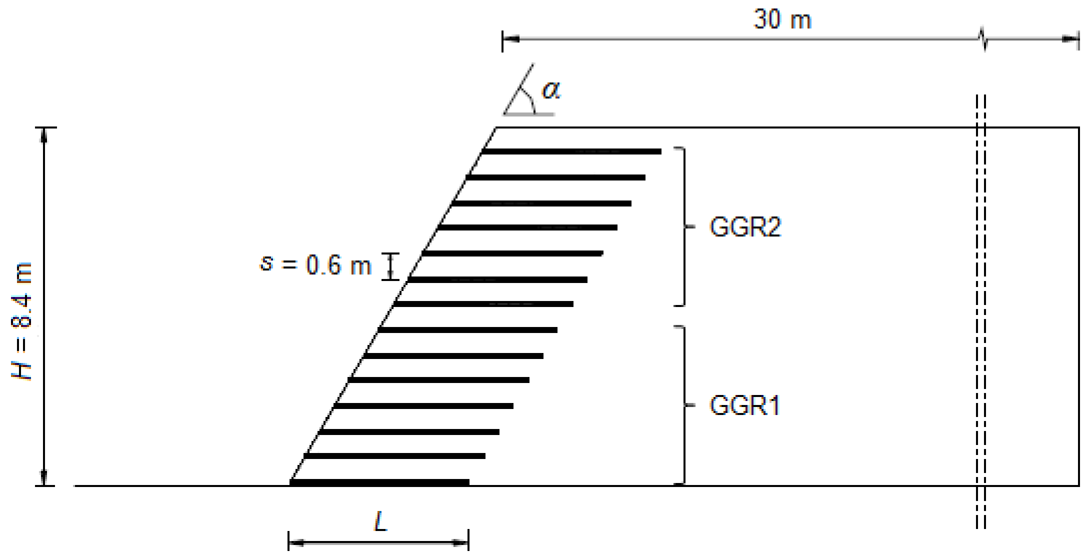

Figure 1. Schematic illustration of the geosynthetic-reinforced slopes

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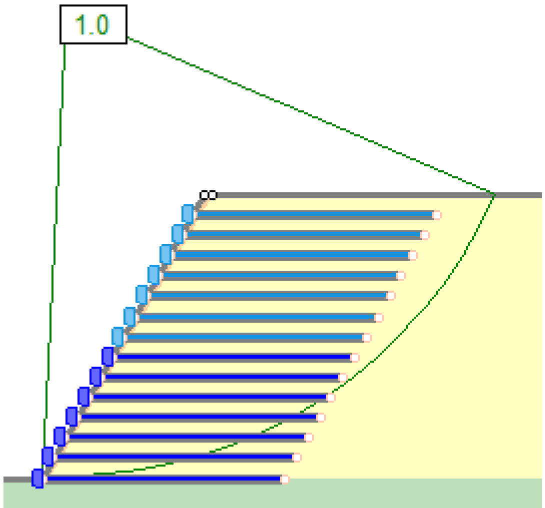

Figure 2. Global Minimum slip surface and over-design factor of Slope 1 ($\alpha = 60°$, $L = 7.2$ m, $\phi_d = 20°$, γ_d = 23 kN/m^3)

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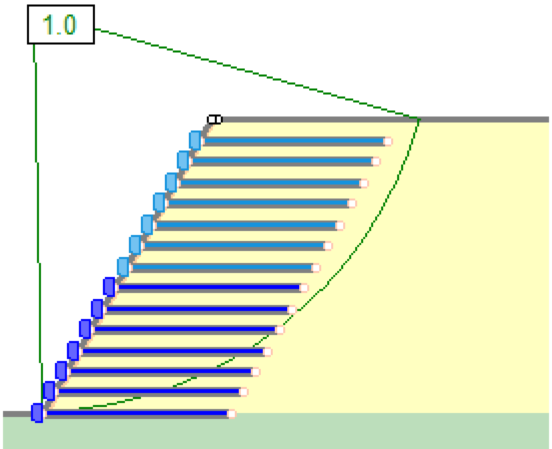

Figure 3. Global Minimum slip surface and over-design factor of Slope 2 ($\alpha = 60°$, $L = 5.4$ m, $\phi_d = 25°$, γ_d = 23 kN/m^3)

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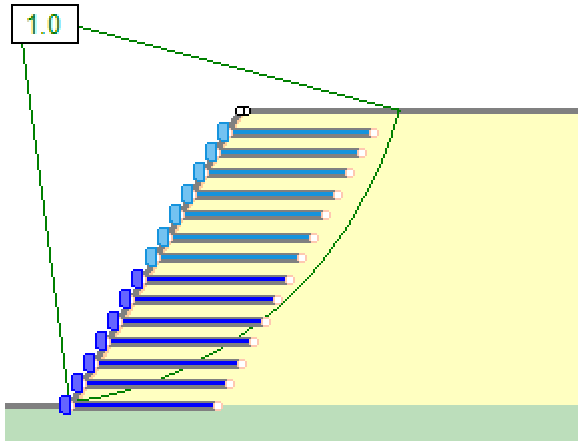

Figure 4. Global Minimum slip surface and over-design factor of Slope 3 ($\alpha = 60°$, $L = 4.2$ m, $\phi_d = 30°$, γ_d = 23 kN/m^3)

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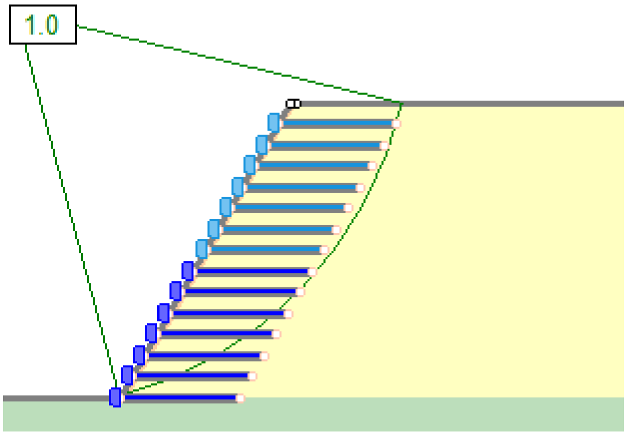

Figure 5. Global Minimum slip surface and over-design factor of Slope 4 ($\alpha = 60°$, $L = 3.4$ m, $\phi_d = 35°$, γ_d = 23 kN/m^3)

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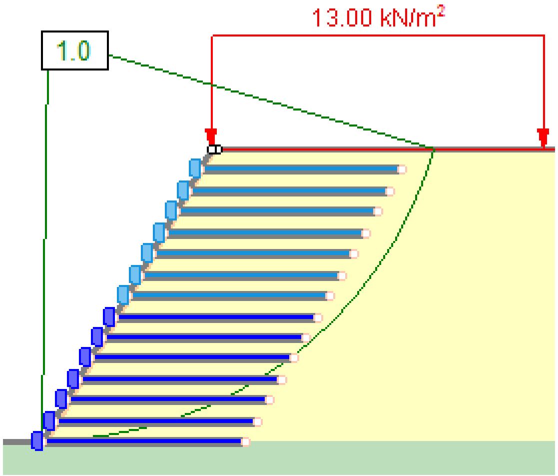

Figure 6. Global Minimum slip surface and over-design factor of Slope 5 ($\alpha = 60°$, $L = 5.8$ m, $\phi_d = 25°$, γ_d = 23 kN/m^3)

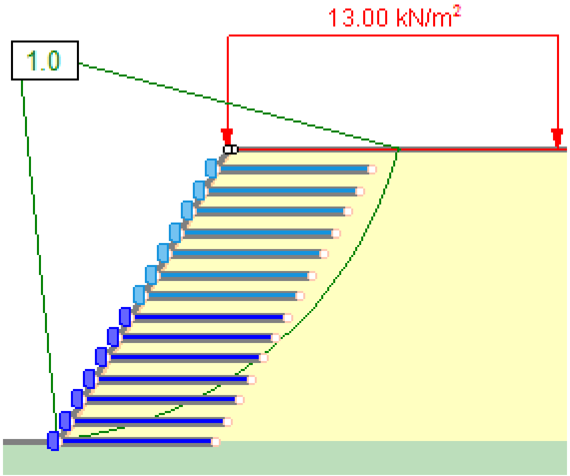

Figure 7. Global Minimum slip surface and over-design factor of Slope 6 ($\alpha = 60°$, $L = 4.5$ m, $\phi_d = 30°$, γ_d = 23 kN/m³)

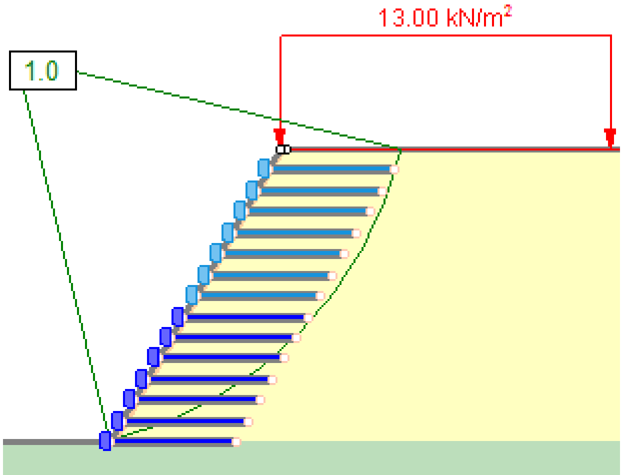

Figure 8. Global Minimum slip surface and over-design factor of Slope 7 ($\alpha = 60°$, $L = 3.6$ m, $\phi_d = 35°$, γ_d = 23 kN/m^3)

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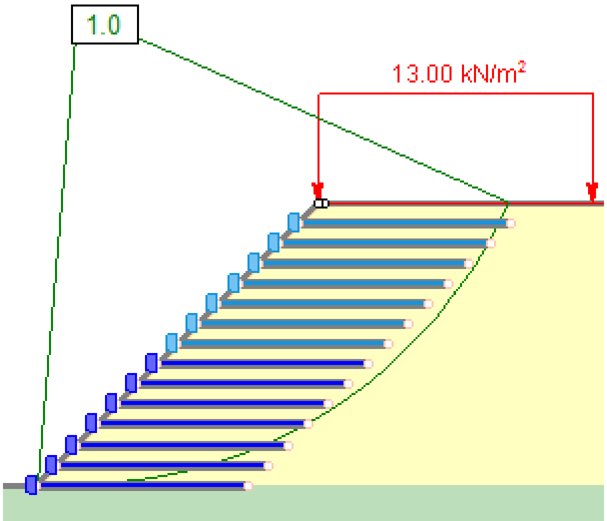

Figure 9. Global Minimum slip surface and over-design factor of Slope 8 ($\alpha = 45°$, $L = 6.3$ m, $\phi_d = 25°$, γ_d = 23 kN/m^3)

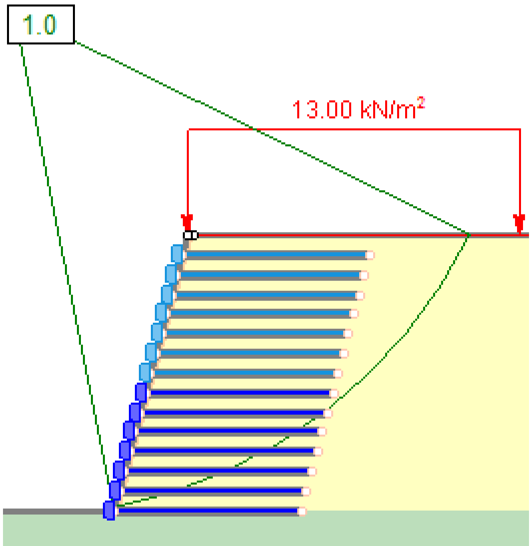

Figure 10. Global Minimum slip surface and over-design factor of Slope 9 ($\alpha = 75°$, $L = 5.8$ m, $\phi_d = 25°$, γ_d = 23 kN/m^3)

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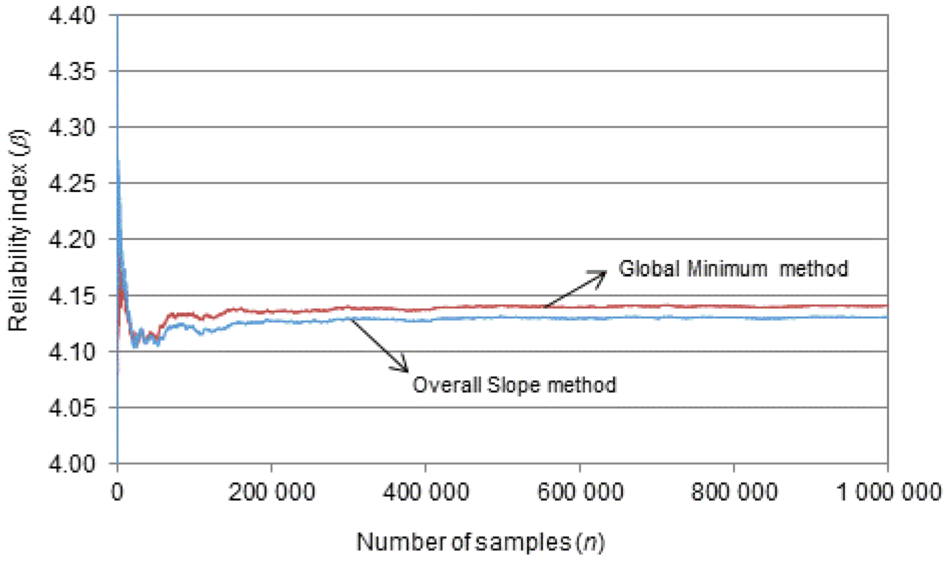
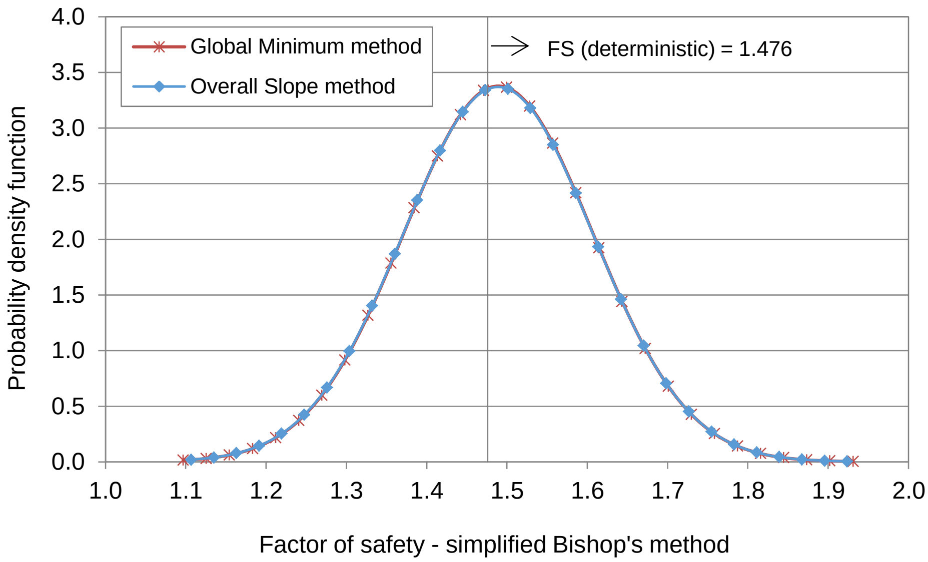
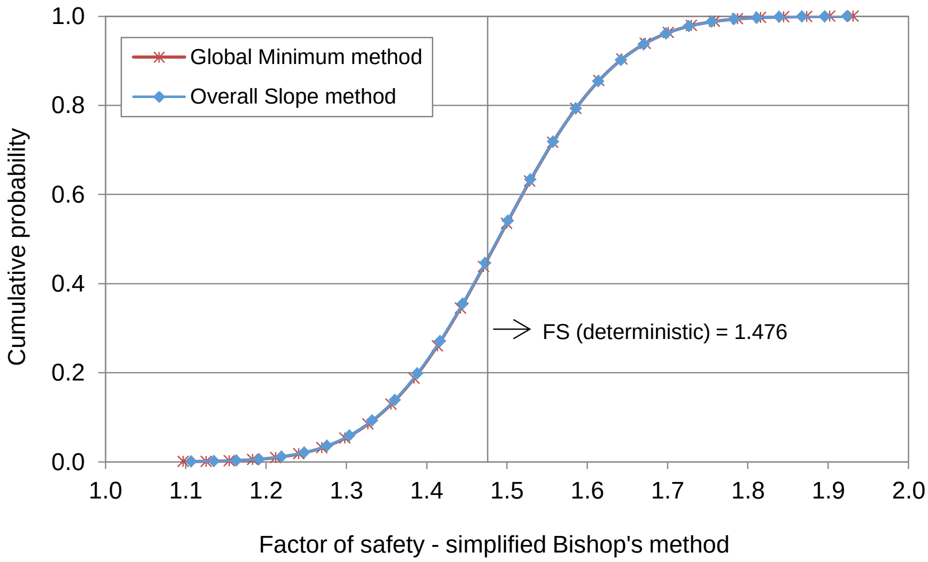


Figure 11. Convergence of the reliability index of Slope 5 (Global Minimum and Overall Slope methods)

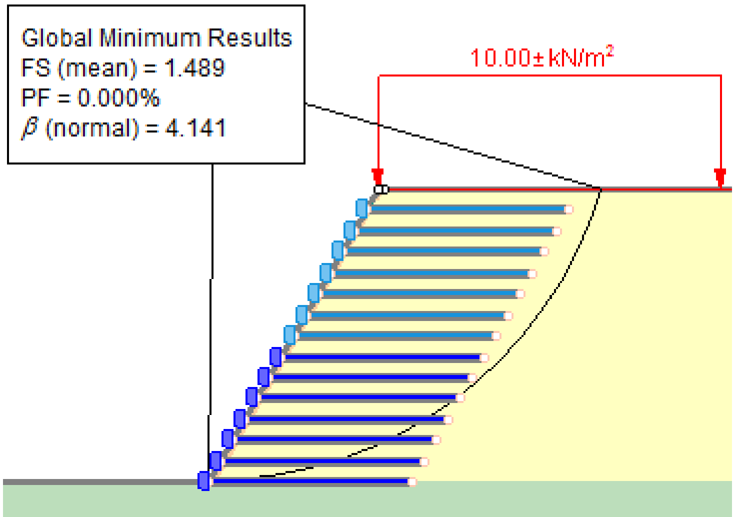


a)



b)

Figure 12. Comparison of results of the probabilistic stability analysis of Slope 5 obtained from the Global Minimum and Overall Slope methods: a) probability density function of the factor of safety; b) cumulative probability distribution of the factor of safety



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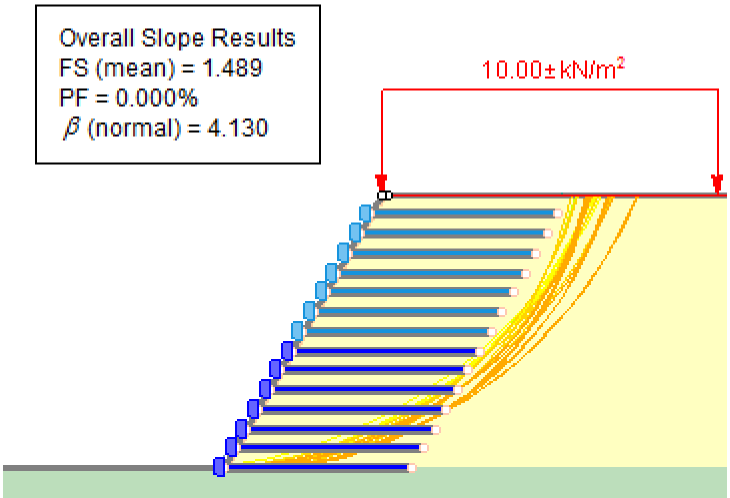

b)

Figure 13. Results of the probabilistic stability analysis of Slope 5: a) Global Minimum method; b) Overall Slope method