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Eurocode 7 and Rock Engineering Design: The Case of Rockfall Protection Barriers

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Abstract: The Eurocode 7 or EC7 is the Reference Design Code (RDC) for geotechnical design including rock engineering design within the European Union (EU). Moreover, its principles have also been adopted by several other countries, becoming a key design standard for geotechnical engineering worldwide. It is founded on limit state design (LSD) concepts, and the reliability of design is provided mainly by a semi-probabilistic method based on partial factors. The use of partial factors is currently an advantage, mainly for the simplicity in its applicability, and a limitation, especially concerning geotechnical designs. In fact, the application of partial factors to geotechnical design has proven to be difficult. In this paper, the authors focus on the way to apply EC7 principles to rock engineering design by analyzing the design of rockfall protection structures as an example. A real case of slope subjected to rockfall is reported to outline the peculiarity connected to rock engineering. The main findings are related to the complementarity of the reliability-based design (RBD) approach within EC7 principles and the possibility of overcoming the limitations of a partial factor approach to this type of engineering problem.

Keywords: rockfall; reliability analysis; Eurocode 7; block volume

1. Introduction

In mountain regions, rockfalls are one of the most dangerous landslide phenomena due to their high impacting energy and motion. Usually, countermeasures are adopted for mitigating the effect of impacting blocks on vulnerable structures and reducing economic damage and loss of human life. In the last decades, flexible barriers have been successfully used as protecting structures because of their low cost, modularity, and ease of installation. The definition of their maximum resistance and the kit components are standardized in [1]. Following [1], the design of the protection structures consists of verifying that impact energy, commonly evaluated in a deterministic way, is lower than the nominal resistance of the barrier. However, in [1], no indications about the definition of the impacting energy in terms of rock block volume or block tracks along the slope are provided. Moreover, the current design process does not take into account the uncertainties in the slope and rock mass model, and hence the uncertainties in determining rock block volume and block track along the slope [2,3].

In 2010, the Eurocode for Geotechnical Design, EN-1997-1:2004 [4], also known as Eurocode 7 or informally EC7, became the reference design code (RDC) for geotechnical design including rock engineering design within the CEN (European Committee for Standardization) member states [5]. EC7 is one standard within the comprehensive Structural Eurocode suite, which has also been adopted by several other countries that are not members of CEN. EC7 is thus becoming an essential design standard for geotechnical engineering worldwide [6].

EC7 is internationally unique in both the breadth of its application and the requirement for designers to implement limit state design (LSD) principles in all aspects of geotechnical design. However, and somewhat regrettably, EC7 has reached its current status without the benefit of formal and structured input from the wider rock mechanics community [6]. As a result, many designers are reporting that the code is complicated, and in certain circumstances, impossible to apply to rock engineering design. Fortunately, at the same time, as the code became the RDC within the CEN member states, it entered into its first maintenance cycle, the aim of which was to identify and implement essential technical and editorial improvements to the code. This cycle will culminate in 2024 with the publication of the revised version of the code.

The application of EC7 to geotechnical engineering design in general, and rock engineering design in particular, is proving to be somewhat problematic. There seem to be many reasons for this, but, as this article attempts to show, in the context of rock engineering, it appears that the principal ones are (i) a lack of understanding of LSD by rock engineers, and (ii) a divergence between customary rock engineering design practice and LSD [7].

This paper presents a new strategy to design rockfall barriers in the framework of EC7 principles, taking into account the main rock mass characteristics that govern these phenomena. Rock block volume, and consequently impact energy, are evaluated by using a stochastic approach to overcome the limitation of a deterministic approach and to allow for the computation of the probability of failure as an indicator of the structure risk. By using the proposed methodology, the probability of failure of the system impacting rock block-protection structures can be evaluated. The magnitude of the effects of actions on the barrier is defined and compared with barrier strength. The latter is considered as the sum of each barrier component strength, without going into the details of their design, which is outside the scope of this paper.

The paper begins with a review of the development of EC7 and the basis of LSD, then goes on to show how LSD is currently implemented within EC7. Then, reliability based design (RBD) is considered as a possible approach to overcome the current limitation of the LSD approach to rock engineering problems.

Following this, tools to improve valuable quantitative data and how to use the first order reliability method (FORM) to quantify failure probability and overcome the limitations of partial factors are discussed.

Their application to a real case of slope subjected to rockfall is reported to outline the peculiarity connected to rock engineering.

2. Development of the Structural Eurocodes

In the structural Eurocodes, the concept of a probabilistic approach to safety was addressed in their initial development [8]. However, many studies and discussions have highlighted:

- the need for clear definitions and differentiation between the two distinct limiting conditions of a loss of serviceability and the onset of collapse [8];
- the role that uncertainty and variability in loads and material strength play in determining safety;
- the need to develop a robust probabilistic framework and associated analytical techniques (e.g., [9,10]); and
- the need to develop and understand acceptable probabilities of failure for different types of structures [11,12].

This design philosophy is known as limit state design, LSD. The underlying principles of LSD are succinctly presented in Figure 1. This uses the structural Eurocode nomenclature, and shows that both E, the effect of actions (which are the loads or the effects of loads), and R, the resistance (i.e., strength) can both be represented by probability density distributions. The limit state is the condition $R - E = 0$, and failure is represented by that region for which $R - E < 0$. The location on the limit state line with the greatest probability of failure is known as the design point, (E_d, R_d), and this is a key feature of most LSD calculation procedures. Note that in this figure, for clarity, the characteristic values are shown as being the median values of the distributions (i.e., the 50% fractile); in the structural Eurocodes, 5% and 95% fractile values are used [13].

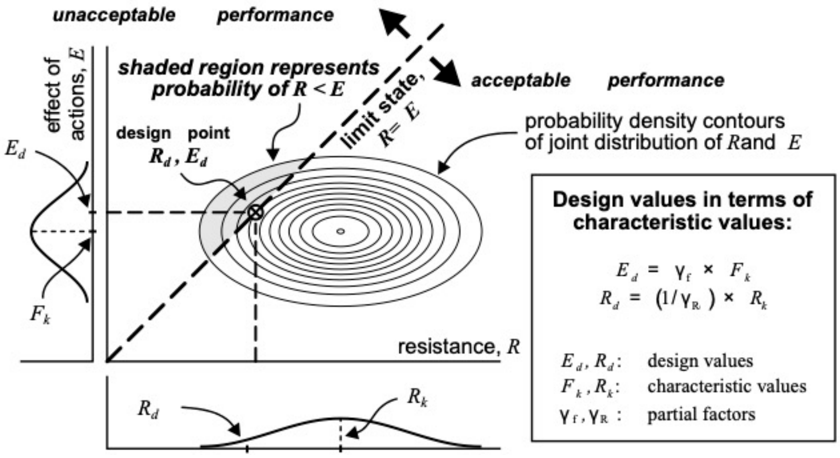

Figure 1. Schematic representation of limit state design (after [14]).

Although the kernel of LSD is probabilistic principles, for ease of use, LSD codes and standards have adopted an implementation with factors applied to individual load and strength or resistance components of the design equations. The term LRFD—load and resistance factor design—is in general use in North American design codes instead of LSD as the factors are applied to the load and resistance components. However, a key aspect of LSD is that the concept of the factor of safety does not exist. Instead, a basic design equation of the form

$$E_d \leq R_d \tag{1}$$

is used, where E_d and R_d are the design value of the effect of the actions and the resistance, respectively. These are related to the corresponding characteristic values (E_k and R_k) through

$$E_d = \gamma_f E_k \,; R_d = \frac{1}{\gamma_R} R_k \tag{2}$$

where γ_E and γ_R are the associated partial factors which, as Figure 1 shows, account for the variability in the effect of the action and the resistance. The magnitudes of the partial factors are set to ensure that the probability of falling into the unsafe region ($R_d - E_d < 0$) is less than some specified value, and this so called calibration of partial factors has been applied in some areas of geotechnical engineering (e.g., [15]). Partial factors can only be applied to properties in those systems that can be modeled in the form $R_d - E_d = 0$, and their values are dependent on both the system being modeled and the variability of the property to which they are being applied [14].

The successful use of partial factors in structural engineering is due to the fact that many systems in this field can be modeled in a linear form, and that the variability of structural loads and material properties are well understood and quantified. This has severe consequences for much (if not all) of rock engineering design [6,16,17] because of the variability, non-linearity, and uncertainty of rock properties. Moreover, no information on the value of the probability of failure of the designed structures is provided by the partial factor approach.

Development of EC7 began in 1975. For rock mechanics and rock engineering, a key event in the history of EC7 was the 1980 agreement between the Commission of the European Communities (CEC) and the International Society for Soil Mechanics and Foundation Engineering (ISSMFE) for

ISSMFE to survey existing codes of practice for foundations within the member states and hence draft a model code that could be adopted as Eurocode 7 [18]. The code development took place without any formal input from organizations such as the International Society for Rock Mechanics (ISRM) or the International Association for Engineering Geology and the Environment (IAEG). This offers an explanation as to why EC7 is currently weak with regard to rock mechanics theory and rock engineering practice. If EC7 is to be used successfully in rock engineering design, this weakness must be corrected, and this is what the current revision process is seeking to do. However, this may not be sufficient, for the reason that rock mechanics and rock engineering has developed along an essentially deterministic path, with the calculation of factors of safety rather than the probability of failure being the norm. Thus, in addition to modifying EC7, this deterministic path will need to be left in favor of a probabilistic one.

EC7 was fully implemented within the CEN member states in 2010, and is currently under revision as per the mandated working practices of CEN. CEN regards all codes and standards as 'live' documents, in that they should evolve in accordance with the users' requirements and new developments in theoretical understanding, materials, and methods.

The revision is being written by a series of project teams under contract to CEN, and supported by an extensive organization of working groups with specialisms in particular areas. The Working Group for Rock Mechanics currently comprises 18 active members from Austria, France, Germany, Italy, Norway, Portugal, Sweden, Switzerland, and the United Kingdom.

Of particular note are specific changes made to support rock engineering including:

- The recently revised version of EN 1990 [13], which provides the basis for design in the Eurocodes, now explicitly recognizes the existence of discontinuous rock masses, and notes that the geometry of discontinuities should be considered when determining the engineering properties of a rock mass. As a result, the concepts of anisotropy, heterogeneity, and potentially size effects are now embodied in [13].
- The application of explicit reliability-based approaches (e.g., Monte Carlo simulation, first order reliability methods (FORM), and first order second moment (FOSM)) is permitted when application of partial factor approaches is inappropriate. As it seems that partial factor approaches are unlikely to be applicable for design analyses in rock engineering that need to account for the presence of specific discontinuities [19], this is a change of fundamental significance.
- Enlarged and improved guidance on the use of the observational method including approaches and procedures appropriate for rock engineering.

Together, these changes represent a new start for rock engineering as far as the structural Eurocodes are concerned. While the fundamental differences between structural and geotechnical engineering have been recognized from the start of the work on the Eurocodes, the revised EC7 will explicitly include material that is directly relevant and applicable to rock engineering. As a result, it is eagerly anticipated that it will be possible to apply the design principles in EC7 to rock engineering in a way that has not been possible to date. However, this does not mean that problems will not remain: in fact, there will be many, but these are not simply due to deficiencies in [4,13], they are more related to the fundamental basis of how rock engineering design is conducted, as the following section will reveal.

Given that rock engineering practice continues largely to follow a deterministic path, in what follows, we examine how rock engineering would need to evolve in order to align with the principles of LSD.

LSD requires uncertainty in resistance and the effects of actions to be quantified in terms of probability density distributions. This indicates that uncertainty in these variables has to be regarded as aleatory (i.e., capable of being described in a probabilistic sense).

Application of LSD to rock engineering has particular ramifications for data requirements, in that an aleatory model based on sufficient high-quality data is required.

3. Rockfall Phenomena and EC7

Rockfall phenomena are commonplace in mountainous regions, and protection structures such as flexible net barriers are usually adopted to mitigate the effects of impacting blocks on vulnerable structures. Their design is an excellent example that highlights the complexity of the identification of design parameters in EC7 application. Uncertainties are involved in slope characterization (geometry, restitution coefficients), discontinuity survey methods, the definition of the design block volume, and the simulation of block trajectories along the slope.

Concerning flexible barriers, they are produced and tested according to [1] "Falling Rock Protection Kits" and following developments of EOTA (European Organization for Technical Assessment in the area of construction products). However, EAD does not define rock block volume, which is the dominant parameter for a proper design of rockfall barriers, but only provides guidance on the standardized procedure for assessing the performance of barrier structures in terms of mechanical resistance and energy absorption capacity.

Some design requirements and guidelines are provided by national standards such as UNI 11211-4 in Italy [20] and ONR 24810 in Austria [21]. In UNI 11211-4, the impacting design energy is considered as the kinetic energy where the velocity is assumed to be the 95% fractile of the velocity distribution defined by using dynamic analyses and the rock block mass is the block volume multiplied by rock mass density. Multiplicative coefficients are applied to both the velocity and the rock mass to take into account the methodology for velocity determination and the accuracy of the topographical and geomechanical surveys. Additionally, the design energy value can be increased by up to 20%, depending on the economic and cultural importance of the protected structure evaluated by ad-hoc risk analyses.

The ONR standard proposes a semi-probabilistic approach in relation to three consequence classes [13]. The design block is defined as a percentile of the site block size distribution: up to 98% fractile depending on:

- the frequency of rockfall phenomena;
- the number of rock blocks in the deposition area;
- the number of joints in the detachment area; and
- the consequence class.

Moreover, partial factors are applied to the design energy (defined as the 99th fractile of energy distribution obtained for the location of interest) and the barrier resistance. The non-uniqueness of partial factors is the reason for the need to go beyond a single factor of safety [22] by using the probability of failure as an indicator for evaluating the risk of failure of a rockfall barrier.

4. Reliability-Based Design Approach

The previous sections have highlighted the current limitation of a LSD approach to rock engineering problems: a RBD approach may overcome this and provide new insights into how concepts from EC7 may be applied to the design of rockfall protection barriers.

The reliability-based design approach deals with the relation between the loads a system must carry and the ability of the system to carry those loads. Both the loads and the resistance may be uncertain, so the result of their interaction is also uncertain [23]. Moreover, the RBD approach enables the non-normal distribution of leading variables in rockfall flow phenomena and their possible correlations to be taken into account.

The results of RBD analyses are expressed by the coordinates of the design point, x^*, and the reliability index β, which can be related to the probability of failure, P_f. P_f is defined as:

$$P_f \approx 1 - \Phi(\beta) = \Phi(-\beta) \tag{3}$$

where Φ is the normal cumulative probability function. Among the available methods for performing reliability analysis [23–28], the most widely used and consistent one is the first order reliability method (FORM) of [10], in which β is given by:

$$\beta = \min_{\vec{x} \in F} \sqrt{\vec{n}^T [R]^{-1} \vec{n}} \quad (4)$$

where $\vec{x}$ is the vector of random variables; R is the correlation matrix between the random variables; F is the failure domain, notation "T" and "−1" represent the transpose and inverse matrix, respectively, and $\vec{n}$ is a dimensionless vector defined as follows:

$$\vec{n} = \frac{\vec{x} - \mu^N}{\sigma^N} \quad (5)$$

where μ^N and σ^N are the normal mean and the normal standard deviation vectors, respectively, evaluated using [29].

By minimizing Equation (4), the tangency of the expanding dispersion ellipsoid with the failure domain surface at the most probably failure point (x^*) can be defined. In this paper, the Excel spreadsheet developed by [30] was used for performing a RBD analysis: a simple Visual Basic for Application, VBA, code automates the computation of x_i from n_i for the use of performance function g(x) = 0, via $x_i = F^{-1}\Phi[(n_i)]$, where Φ is the standard normal distribution and F is the original non-normal distribution.

Introducing random variability to R and E leads to structural safety being defined as the probability of the limit state being attained. As a result, 'factor of safety' is not used, but is replaced by a reliability index, β, that represents the probability of failure P_f. These are linked through the relation $P_f = \Phi(-\beta)$, where Φ (−) is the cumulative distribution function of the standard normal distribution.

Values of P_f can be assigned much more objectively than a factor of safety: for example, society may accept a higher probability of failure for rock slopes in remote mountainous regions than alongside heavily trafficked major roads and rail links. Table 1 lists some target values of the reliability indexes suggested in [13] and shows how these values reflect both the consequence of attaining the ultimate limit state and the design life (i.e., reference period). The values list in Table 1 was used for defining the probability of the failure of building and civil structures. Currently, however, there seem to be no recognized values for rock engineering structures, and whether the values of Table 1 are appropriate is not known. The definition of the target probability of failure for rock engineering design is also fundamental since RBD approach can provide results technically and economically difficult to reach, as discussed below.

The use of FORM, with respect to other statistical approaches such as Monte Carlo simulation (MCS), can be made to reduce computational time: in fact, MCS requires the calculations of hundreds and thousands of performance function values that increase computational time.

Table 1. Suggested values of reliability index (after [13]).

Consequence of Attaining the Ultimate Limit State	Minimum Values If Beta and Associated P_f in Terms of Reference Period			
	1 year		50 years	
	β	P_f	β	P_f
High consequence for loss of human life, or economic, social or environmental consequences very great	5.2	~1×10^{-7}	4.3	~1×10^{-5}
Medium consequence for loss of human life, economic, social or environmental consequences considerable	4.7	~1.5×10^{-6}	3.8	~7×10^{-5}
Low consequence for loss of human life, or economic, social or environmental consequences small or negligible	4.2	~1.5×10^{-5}	3.3	~5×10^{-4}

5. Reliability Based Design as A Complementary Tool for EC7 Design

Principles and limitations of the LSD approach have been highlighted in Section 2. In summary, for rock mechanics problems such as rockfalls, the use of partial factors is problematic and in addition, no indications regarding the probability of failure can be derived from their application.

For this reason, RBD analysis is required for certain complex geotechnical applications including the design of rockfall protection structures, since, as highlighted by many authors [17,31–34], applying the same partial factors in problems with different levels of uncertainty may not result in the same target failure probability. Instead, by fixing the reliability index, the probability of failure remains the same, independent of the problem type and the level of parametric uncertainty. However, as shown in the following, partial factors can be back calculated from the RBD by knowing the design point coordinates and fixing characteristic values for the random variables. High β-values correspond to safer conditions (see [13]) and a higher consequence class is assigned to the structure.

Since the RBD approach requires performing advanced statistical analysis, limited data for defining the probability distributions of the considered variables represent the main limitation of the method. In fact, large sets of high-quality data are required and this, of course, is a very different state of affairs from what exists generally in rock mechanics: small datasets of medium quality (e.g., spacing) are the norm. However, concerning rockfall studies, a great amount of data concerning rock discontinuities (in terms of orientation and spacing) can be inferred from remote sensing techniques, allowing for statistical analyses and therefore for the evaluation of uncertainties [35–37].

In summary, RBD provides insights for EC7 design when statistical information on key parameters affecting design is known, when partial factors have yet to be proposed, and when input parameters are correlated, as in the case of rockfall phenomena [3,17,34].

In the following sections, the RBD approach was applied to a hypothetical barrier placed at the base of a steep rock face along Gardesana Road, in the Gargnano-Muslone area along the western Lake Garda (Brescia Province, Italy).

6. RBD for Rockfall Barriers: A Case Study

The RBD approach was performed on a hypothetical barrier that should be placed along the Gardesana Road (Italy) to protect this infrastructure from rockfall events coming from the surrounding steep and highly-fractured rock mass.

The studied rock mass is located along the western Lake Garda where carbonatic sedimentary succession belonging to the Lombardian Basin is exposed. In particular, the observed outcrops consist of massive platform limestones of the Corna Formation (Rhaetian–Early Jurassic p.p.), cherty limestone and marly limestones of the Medolo Formation (Lower Jurassic), marlstone and marly limestone of the Concesio Formation (Middle Jurassic), siliceous sediments of the Scaglia Lombarda Formation (Middle-Late Jurassic), and bedded limestones of the Maiolica Formation (Early Cretaceous).

Regarding the structural setting, the investigated area is located between the Tremosine–Tignale thrust system and the inverted Ballino–Garda fault and is characterized by open to tight asymmetrical folds with NE–SW trending axes and axial planes variably dipping toward NW. In particular, the investigated rock slopes lie on the southern right limb of a syncline fold and mainly consist of thin-bedded marly limestone (from a few centimeters to decimetric thickness) and cherty limestone (10–40 cm thick) of the Medolo Formation, with bedding dipping toward NW and W at a moderate angle (Figure 2).

In the study area, mesoscopic low angle compressional structures (N-to-SE), subparallel to the bedding, have also been observed, together with high angle to subvertical faults and discontinuity sets, variably oriented. More information about the geological and geomorphological settings and the methodologies for obtaining the main discontinuity characteristics of this face of the rock mass can be found in [38].

Figure 2. Rock face along the Gardesana Road (Italy).

Due to limited accessibility to the study area, only a few traditional geomechanical surveys [39,40] were performed to define the main parameters for representative block volume assessment. However, a digital surface model (DSM) was obtained by merging dense point clouds from laser scanner and photogrammetric surveys. By using the software Rockscan [41] and CloudCompare, discontinuity planes were manually delimited and their dip, dip direction, and spacing were extracted.

By using the previously described methodology, a huge amount of high quality data was produced and statistically analyzed: 138 and 1580 discontinuity planes coming respectively from traditional and non-contact surveys allowed for orientation evaluation. A dataset of 666 spacing values was obtained and statistically analyzed: lognormal and gamma distributions were found to perform best in simulating the spacing frequency distributions of the discontinuity. Moreover, kinematic analyses highlighted that three of the identified discontinuity sets were most predisposed to generate rockfall events. In Table 2, the results of geomechanical and statistical analyses are reported.

Table 2. Orientation and spacing values of the discontinuity sets mostly predisposed to generate a rockfall event and their corresponding frequency distributions.

Set	Dip [°]	Dip Direction [°]	Frequency Distribution	Spacing [m]	
				Mean Value	Standard Deviation
K1	50	310	Lognormal	0.29	0.24
K3	70	80	Lognormal	0.43	0.38
K5	40	240	Gamma	3.25	4.57

Since the rock face is very steep (78° on average) and its distance from the main road is very limited (the road runs underneath the rock face), the impacting kinetic energy can be conservatively approximated to the potential energy of the falling rock block. Rockfall barrier design is based on the hypothesis that the energy retained by the block is instantaneously and completely transferred to the barrier upon impact. Consequently, this kind of structure is designed with an “energy approach”, comparing the kinetic energy of the falling blocks with the resistance energy of the barrier determined according to EAD. Even if in the EC7 Equation (1) refers to forces, in rockfall phenomena, it should be referred to as energies. Thus, the performance function for RBD analysis can be set as follows:

$$g(x) = R_B - E_P = 0 \quad (6)$$

where R_B is the barrier resistance and E_P is the potential energy given by:

$$E_P = m \cdot g \cdot H_{max/min} = \rho \cdot V_B \cdot g \cdot H_{max/min} \quad (7)$$

In Equation (7), m is the rock block mass in kg; ρ is the rock density in kg/m^3; g is the gravity equal to 9.81 m/s; $H_{max/min}$ is the considered fall height in m; and V_B is the block volume in m^3 that can be found by using [41] Palmstrøm's equation for rock block generated by three discontinuity sets. Consequently, Equation (7) can be rewritten as:

$$E_P = \rho \cdot \frac{S_{K1} S_{K3} S_{k5}}{\sin\gamma_{13} \sin\gamma_{35} \sin\gamma_{15}} \cdot g \cdot H_{max/min} \quad (8)$$

where S_{K1}, S_{K3}, S_{K5} are the spacing of each discontinuity set and γ_{13} is the angle between set 1 and set 3 (similarly for γ_{35} and γ_{15}). In Figure 3, the main characteristics observed in the geomechanical survey are shown.

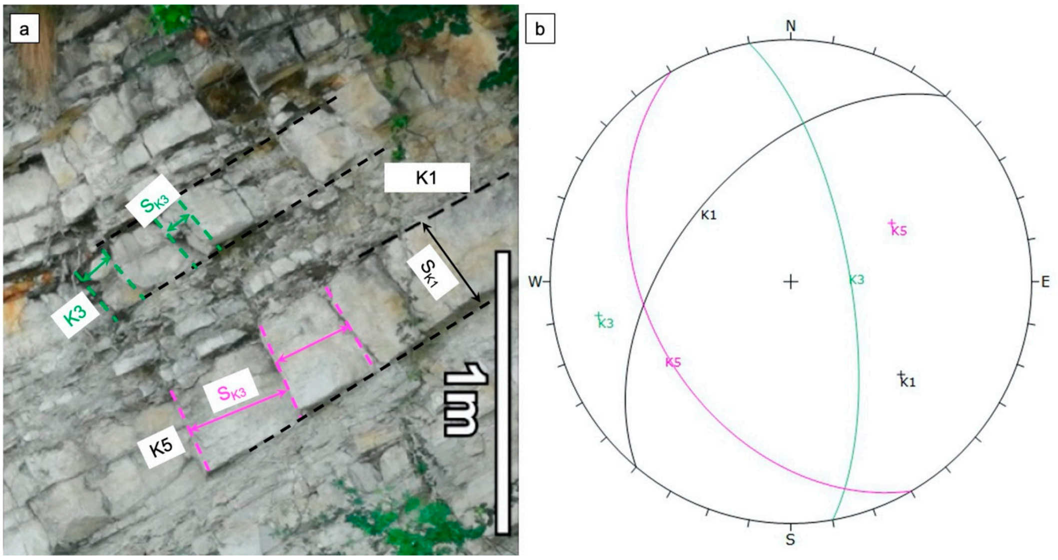

Figure 3. (**a**) Orientation and spacing of the discontinuity sets observed in the study area. (**b**) Plot reporting the main discontinuity sets.

As shown in [42], block volume estimation with Palmastrom's equation, in its common application, results in a rough calculation of the average volume. Since discontinuity spacing is evaluated in a deterministic way, it is far from representative of a complex medium such as a rock mass [38]. In the RBD approach, this limitation is overcome since the performance function uses values from frequency distributions.

Rock density and angles between sets were assumed as averaged values (Table 3), since their influence on volume variability was proven to be limited, compared to that of joint spacing [43]. Both minimum and maximum fall height values were considered for the analyses in order to evaluate their influence on the results.

Table 3. List of parameters used for RBD analysis.

Parameter	Symbol	Value
Maximum rock face height	H_{max}	155 m
Minimum rock face height	H_{min}	65 m
Rock mass density	ρ	2600 kg/m^3
Angle between K1 and K3	γ_{13}	82°
Angle between K1 and K5	γ_{15}	95°
Angle between K3 and K5	γ_{35}	100°

Since the barrier is manmade and built following standardized engineering criteria [3,17], energy absorption capacity probability distribution was assumed to be normally distributed. In a previous work [3], the authors demonstrated that the uncertainties related to barrier energy absorption capacity can be neglected since the dominant variable is the impacting kinetic energy. Consequently, the energy absorption capacity standard deviation was set equal to 3% of the mean.

A FORM analysis was undertaken to determine the reliability index from the probability of failure using the reliability index values given in Table C1 of Annex C of EN 1990 [13] as a function of the probability of failure. Four probability distributions, one for energy absorption capacity of the barrier and three for set spacings, were considered as variable parameters in the RBD analysis.

The proposed methodology allows the definition of the probability of failure of the system defined by the impacting rock block and protection structures. The barrier resistance is considered as a global value given by the sum of the strengths of single barrier components that can be defined following both [1] and [44].

In Figure 4, the [30] FORM computational approach in the Microsoft Excel spreadsheet platform is shown. The reliability index β is calculated starting from mean value (Para 1) and standard deviation (Para 2) for each involved parameter and their respective probabilistic distribution. For the Gamma distribution, mean and standard deviation are replaced by shape and rate parameters.

Frequency Distribution	Parameter	Para1	Para2	x*	μ^N	σ^N
Normal	R_B	9.00E+08	2.70E+07	8.97E+08	900000000	27000000
Lognormal	S_{K1}	0.29	0.24	2.21	-2.89	1.63
Lognormal	S_{K2}	0.43	0.38	3.61	-5.08	2.72
Gamma	S_{K5}	0.51	6.42	27.67	-20.04	17.62

Correlation matrix R				n_x
1	0	0	0	-0.13
0	1	0	0	3.13
0	0	1	0	3.20
0	0	0	1	2.71

g(x)	β
0.00	5.2

Probability of failure
0.0000083%

Figure 4. Determining the reliability index β and the coordinates of the design point x* for a hypothetical rockfall barrier.

The column of design values, x*, is automatically updated by Microsoft Excel Solver by imposing the g(x) = 0 constraint. Physically, x* represents the coordinates of the point where the four-dimensional equivalent dispersion ellipsoid is tangential to the limit state surface. These coordinates are the most probable failure combination for the analyzed rockfall parameters. The column μ^N and σ^N are the normal mean and the normal standard deviation values, respectively, evaluated using [29] starting from Para 1 and Para 2 of the considered probability distribution. In the correlation matrix R, diagonal-off values were set at null since all the parameters were considered as independent.

The design parameters as a function of the probability of failure are listed in Table 4. As expected, design spacing values were not influenced by the considered fall height that, conversely, had a huge influence on design barrier resistance. However, these differences became smoother by considering the ratio between design parameters and the corresponding mean values (Figure 5).

Table 4. Design parameters evaluated for a reliability-based design approach as a function of reliability index values proposed in EN 1990 Annex C Table C1 considering both the maximum and minimum fall height.

		H_{min}				H_{max}			
β [-]	P_f [-]	R_{B_min}* [J]	S_{K1}* [m]	S_{K3}* [m]	S_{K5}* [m]	R_{B_max}* [J]	S_{K1}* [m]	S_{K3}* [m]	S_{K5}* [m]
1.28	1.00×10^{-1}	1.83×10^{6}	0.34	0.51	6.12	4.35×10^{6}	0.34	0.51	6.11
2.32	1.02×10^{-2}	8.49×10^{6}	0.54	0.83	11.24	2.03×10^{7}	0.54	0.83	11.25
3.09	1.00×10^{-3}	2.44×10^{7}	0.77	1.21	15.42	5.79×10^{7}	0.77	1.20	15.39
3.72	9.96×10^{-5}	5.59×10^{7}	1.04	1.66	18.94	1.33×10^{8}	1.04	1.65	18.93
4.27	9.77×10^{-6}	1.13×10^{8}	1.37	2.18	22.09	2.69×10^{8}	1.37	2.19	22.07
4.75	1.02×10^{-6}	2.07×10^{8}	1.74	2.81	24.87	4.98×10^{8}	1.74	2.82	24.93
5.2	9.96×10^{-8}	3.59×10^{8}	2.16	3.54	27.44	8.97×10^{8}	2.21	3.61	27.67

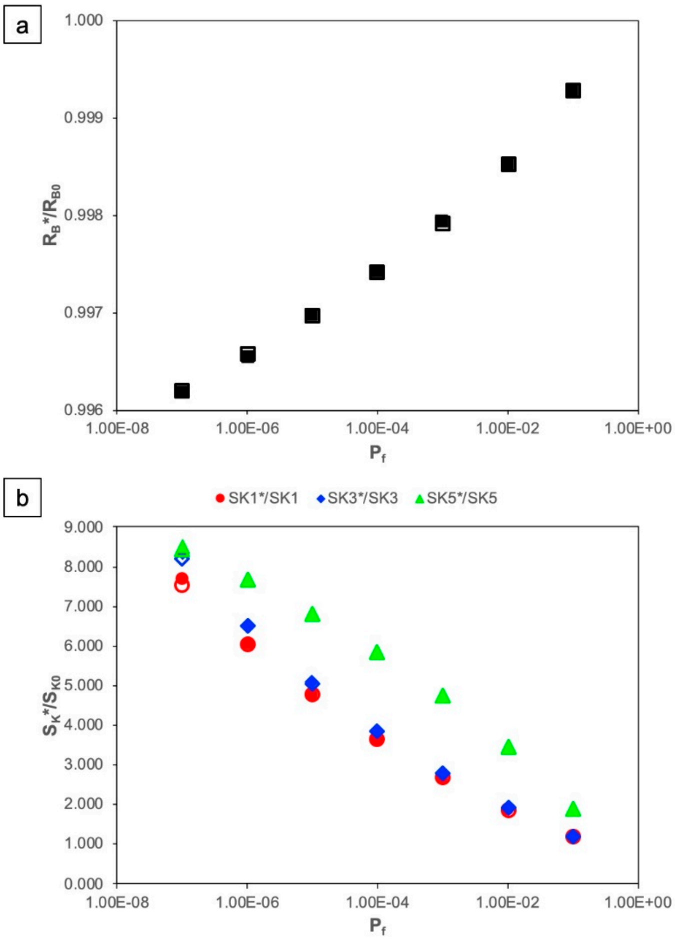

Figure 5. (**a**) Ratio between the design parameter (R_B* and S_K*) and mean value for resistance (R_{B0} and S_{K0}) and (**b**) for set spacing as a function of the probability of failure. All the parameters are dimensionless.

Considering that currently the maximum certified resistance for a flexible rockfall barrier is 10,000 kJ, just for β up to 2.32, the design values can be accepted. This does not affect the validity of the methodology, but it points out another aspect to take into account in EC7: the maximum accepted probability of failure for this type of temporary structure. However, the RDB approach is not influenced by the type of protection structure since it depends on the performance function used for evaluating the impact energy.

Figure 5 shows the trend of the ratio between the design parameters and mean parameter as a function of the probability of failure, considering both minimum (empty markers) and maximum (filled markers) height. A remarkable aspect is that K1 and K3 spacings followed a similar trend, while the distance between the design and mean values was higher for K5 spacing, reflecting the influence of high standard deviation for this set spacing compared to the other ones (cfr. Table 2). The trend of R_B*/R_{B0} showed a little variation (between 1 and 0.996) as a function of probability of failure. This result mirrors negligible uncertainties related to the energy absorption capacity of the barrier compared to those related to kinetic energy determination. Moreover, this ratio is independent from the falling height values: in fact, the two graphs reported in Figure 5a are totally overlapping.

Figure 6 shows the spacing cumulative probability distribution of each discontinuity set. Diamond markers are the average spacing values (cfr. Table 2), while the cross markers are the design points evaluated by using the RBD approach considering the minimum (Figure 6a) and maximum (Figure 6b) fall height values. The black dashed line represents the 95th fractile, commonly suggested as a characteristic value: it is possible to note that this value allows for reaching a reliability index, β, up to 3.09, which corresponds to a probability of failure equal to 10^{-3}. Greater values of β, corresponding to lower P_f values, may be reached considering higher fractile percentages.

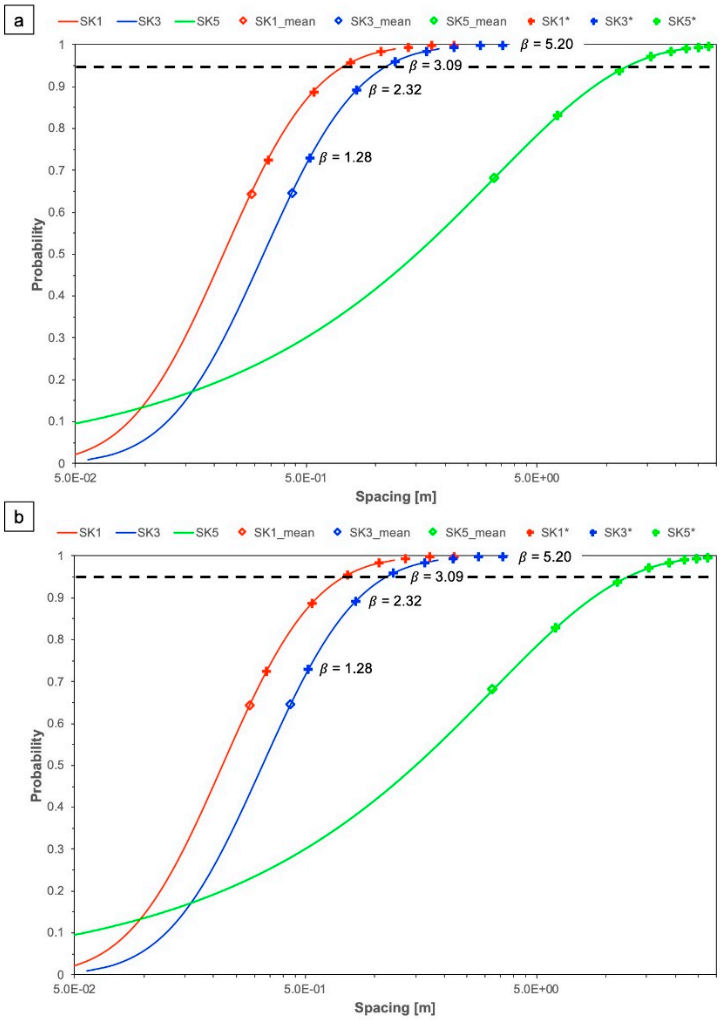

Figure 6. Spacing cumulative probability distribution of each discontinuity set. Diamond markers are the average values; cross markers are the design point values evaluated by using the RBD approach considering H_{min} (**a**) and H_{max} (**b**).

Finally, as discussed in Section 6, the RBD approach may be directly correlated with the partial factor concept introduced in EC7, with the advantage that these factors are linked with a known probability of failure. In fact, the probability distribution of the involved parameters can be used for back-calculating the partial factors by assuming the i-th percentile of this distribution and dividing by the evaluated design parameter value. In this study, partial factors were applied to the energy of the falling block and energy absorption capacity of the barrier.

Figure 7 shows the rock fall barrier partial factor trends, γ, for each impacting energy and resistance as a function of the probability of failure, P_f. Partial factors were calculated as suggested by EN 1990 [13]: partial factors for energy absorption capacity were defined as the ratio of the design parameter, R_B*, and the 5th percentile of resistance probability distribution. Analogously, partial factors for impacting energy is defined combining the design values of the set spacing and the 95th percentile values of their probability distribution.

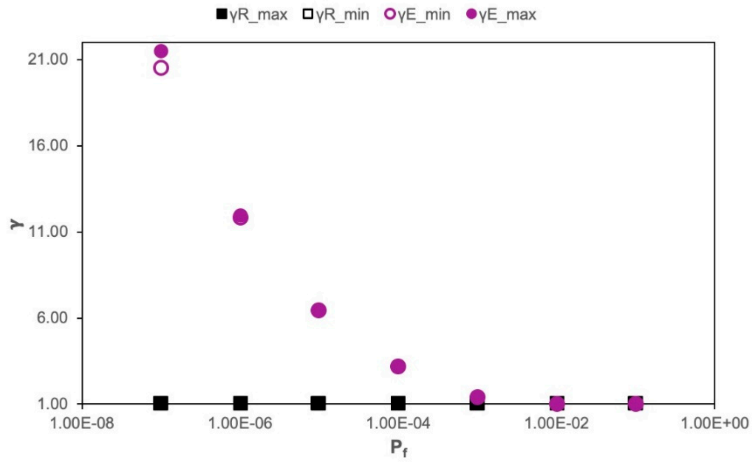

Figure 7. Partial factor dependence on barrier resistance and impact energy as a function of probability of failure considering the minimum (empty markers) and maximum (filled markers) fall height. Where not visible, maximum and minimum markers are overlapping. All the parameters are dimensionless.

As a general comment, partial factors for resistance, which have to be intended as reducing factors, are independent from the probability of failure values and are the same for both the maximum and minimum fall height values. This aspect mirrors a low degree of uncertainty in relation to barrier resistance evaluation. Partial factors for energy increase as the P_f of failure decreases: in particular, in correspondence of P_f equal to 10^{-3} (that corresponds to β = 3.09), the partial factors became greater than 1 up to the maximum value of 21.5. Although these partial factors were associated with a known target failure probability, they remained correlated to the proposed case. Consequently, their universal meaning remains debatable and there are not enough elements and accumulated experience to extend the partial factor to rockfall phenomena with some certainty of safety. Furthermore, since the values of β listed in Table C1 of Annex C of EN 1990 [13] are considered to be appropriate for buildings and civil structures, they may not be applicable for rock engineering structures such as those analyzed in this paper. For instance, the partial factors corresponding to the Eurocode 50-year target β value of 3.8 were respectively equal to 3.2 for impacting energy and 1 for absorbing energy. The partial factor for impacting energy is not appropriate since it mirrors an expected design lifetime (50 years) greater than that certified by manufacturers (20 years).

7. Conclusions

EC7 is mandatory for civil engineering geotechnical design including rock-engineering design in the CEN member states. However, rock engineering issues are treated in a deficient way. As a result, its applicability to rock engineering design is limited and hence adherence by rock engineers to Eurocode 7 is very low, and most practitioners have little contact with the code. Eurocode 7 has focused so much on the partial factor approach that designers are often unaware that LSD has a probabilistic basis.

The Eurocodes are currently undergoing a major process of maintenance that started in 2011 and the second generation of Eurocodes is being drafted. Numerous improvements will be implemented as regards geotechnical engineering, which encountered difficulties dealing with a suite of Eurocodes based on principles mainly originating from structural design.

Recognition of the need to upgrade rock engineering aspects resulted in this being a major objective of the current revision, so that the code treats soil and rock on an equal basis. Implementation of rock engineering is an issue still requiring great effort. Hopefully, enough time and human resources as well as good cooperation with those involved with the other topics will ensure that the objectives established for the second generation of Eurocode 7 are achieved.

In this paper, the authors have highlighted the complementary nature of the RBD approach with EC7, particularly for the design of rockfall protection structures, since the current guidelines do not cover the design of this type of structure and therefore do not take the rock mass block volume distribution and the properties of the rock mass in general into account. Moreover, it has been highlighted how the force approach, commonly used in Eurocodes, is not appropriate for rock fall phenomena, where an energy approach is more suitable, as demonstrated in this work.

The RBD approach allows for the direct evaluation of the probability of failure by modeling the probability distribution of the design parameters: moreover, once a target probability failure is set, it enables the best strategy to achieve it to be identified.

However, this approach presents limitations: undoubtedly, the availability of data for performing robust statistical analyses is the main one. For rockfall phenomena, it can be overcome by coupling remote sensing data with traditional geomechanical surveys for the acquisition of a large dataset. Moreover, a statistically robust block volume distribution assumes a fundamental role in the definition of block design volume.

Furthermore, one of the main challenges of the revised Eurocode 7 will be the definition of the acceptable probability of failure for rockfall protection barriers for both flexible and rigid ones. As shown in this paper, resistance design values obtained from RBD may not be technically and/or economically feasible (mainly for the flexible barrier).

As highlighted in this paper, even if the RBD approach allows back-calculation of partial factor values, it should become common practice to avoid their use in rock mechanics applications in favor of a target probability of failure as an indicator of the residual risk.

Summarizing, the findings of this paper should be considered in the revision process of Eurocodes. In particular, the energy approach should be allowed for the analysis of impulsive phenomena (such as rockfall events) in the framework of RBD analysis and appropriate target reliability indexes should be defined for the design of rockfall countermeasures.

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