Eurocode 7 and rock engineering: current problems and future opportunities

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ABSTRACT: In 2010, Eurocode 7 (EC7) became the Reference Design Code for geotechnical design within the European Union. At the same time, EC7 entered into its first maintenance cycle, the aim of which was to identify and implement essential technical and editorial improvements to the Code. As part of this, a Workshop on the application of EC7 to rock engineering was arranged to be held at Eurock 2014. In this paper, following a brief explanation of both the history of EC7 and the principles of limit state design within the Eurocode suite, the papers submitted to this Workshop are unified in order to highlight the common themes they explore. These include: characterisation of discontinuous rock masses; application of rock mass classification schemes; partial factor calibration; the epistemic nature of rock mechanics parameters; benchmarking of rock engineering design calculations; the limited value of traditional deterministic analyses in the context of EC7; prescriptive measures; the observational method; and, the introduction of new forms of instrumentation. Although current problems have been found in all of these areas, suggestions have also been given for overcoming them. As the importance of EC7 for rock engineering design continues to grow, these suggestions have the potential to become future opportunities.

1 INTRODUCTION

In 2010, Eurocode 7 (EC7) became the Reference Design Code for geotechnical design within the European Union. Since then CEN, the European Committee for Standardisation, has initiated a maintenance cycle, aimed at – among other things – identifying and implementing essential technical and editorial improvements to the Code.

In early 2011, the CEN Technical Committee responsible for Eurocode 7 convened a number of small groups of experts to examine what maintenance would improve EC7. One group was given responsibility for rock engineering, and in order to obtain wider community input chose to arrange a Workshop at Eurock 2014. In this paper, and following a brief explanation of both the history of EC7 and the principles of limit state design within the suite of structural Eurocodes, the papers contributed to this Workshop are brought together in order to highlight the common themes they explore.

2 DEVELOPMENT OF EC7

Development of EC7 began in 1975 (Table 1), with 1980 being particularly important. In that year, an agreement was made between the Commission of the European Communities (CEC) and the International Society for Soil Mechanics and Foundation Engineering (ISSMFE) for the Society 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 (Simpson and Driscoll, 1998). The ISSMFE established an *ad hoc* committee for this task in 1981, which – following much international consultation – produced a ‘draft model for Eurocode 7’ in 1987. Further CEC-sponsored development of this draft took place from 1987 to 1990, whereupon the work was transferred to CEN (Comité Européen de Normalisation / European Committee for Standardisation), and in particular Technical Committee TC250, for eventual publication as the formal Eurocode 7. CEN/TC250 continues to oversee EC7.

Thus, we see that EC7 has its roots firmly in European design codes associated with foundations on and in soils. Furthermore, it seems that code de-

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Table 1: History of Eurocodes (http://eurocodes.jrc.ec.europa.eu)

Year	Event
1957	Treaty of Rome
1971	Public Procurements Directive 1971/305 issued
1975	Eurocode development started
1980	International Inquiry with regard to construction codes performed
1984	First Eurocodes published
1989	Construction Products Directive 1989/106 issued
1990	Work on draft standards (ENVs) started
1992	Publication of ENV Eurocodes commenced
1998	Conversion of ENVs to ENs initiated
2003	EC recommendation on implementation and use of Eurocodes
2004	Directive on Public Works contracts, Public Supply contracts and Public Service contracts issued
2006	Publication of ENs completed
2010	Full EN implementation; conflicting National Standards withdrawn

Table 2: Sub-committees of CEN/TC250 and their respective Eurocodes

Sub-committee	Eurocode	Title
SC1	EN1991	Actions on structures
SC2	EN1992	Design of concrete structures
SC3	EN1993	Design of steel structures
SC4	EN1994	Design of composite steel and concrete structures
SC5	EN1995	Design of timber structures
SC6	EN1996	Design of masonry structures
SC7	EN1997	Geotechnical design
SC8	EN1998	Design of structures for earthquake resistance
SC9	EN1999	Design of aluminium structures

velopment took place without any formal input from organisations such as the International Society for Rock Mechanics (ISRM) or the International Association for Engineering Geology and the Environment (IAEG). Many designers consider EC7 to be weak with regard to rock engineering practice, and this history indicates why this may be so.

2.1 *Continuing development*

The work of TC250 proceeds through a number of sub-committees (Table 2). The membership of CEN/TC250 and its sub-committees comprises delegates of the 29 CEN National Members, together with delegates from the 5 Affiliates (i.e. countries likely to become members of the EU or EFTA) participating as observers. There are nine specialist sub-committees within CEN/TC250, each of which is responsible for a particular Eurocode (Table 2). Thus, although CEN/TC250/SC7 is responsible for Eurocode 7, it works with the other sub-committees to ensure coherence of the structural Eurocodes.

A critical responsibility of TC250 is the maintenance of the Structural Eurocodes. This activity is essential in order to preserve the credibility, integrity and relevance of the Eurocodes, as well as to ensure they do not contain errors. The CEN protocol for code maintenance means that CEN/TC250 is now involved in the first of continuing five-year medium-term maintenance cycles, the output of which will include technical and editorial improvements and the resolution of questions of interpretation. For EC7, it is expected that a new edition of the Code incorporating these improvements will be published in 2019.

Maintenance and future development of EC7 to improve its applicability to rock engineering must be made in the wider context of the Eurocode suite. This is an important and major constraint, and one that in the context of rock engineering leads to particular difficulties with regard to the central tenet of the codes: namely, the use of limit state design.

3 STRUCTURAL EUROCODES AND LIMIT STATE DESIGN

All the structural Eurocodes implement limit state design (LSD), with limit states being defined as a condition "beyond which the structure no longer fulfils the relevant design criteria". Two limit states are recognised: serviceability limit states (SLS), which are "conditions beyond which specified service requirements… are no longer met"; and, ultimate limit states (ULS), defined as being "associated with collapse or with other similar forms of structural failure". These quoted definitions are taken from EN 1990: Eurocode – Basis of structural design, which, as the so-called 'head code' for the structural Eurocodes, defines key concepts that are propagated throughout the suite.

Not only does EN 1990 require designers to distinguish between SLS and ULS, it also requires them to verify that no limit state is exceeded under design conditions. In addition, and critically, EN 1990 explicitly requires designers to account for variability in attributes such as applied loads, material properties and structural geometry when performing this verification. This variability is accounted for using probabilistic models, and so the structural Eurocodes apply the concept of structural reliability in terms of probability of failure, with failure being considered as attaining a limit state. As a result, the concept of 'factor of safety' is not used, but is replaced by a reliability index, β, that represents probability of failure P_f. These are linked through the relation

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

where Φ is the cumulative distribution function of the standard normal distribution. Table 3 reproduces some target values of reliability index suggested in

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Table 3: Suggested values of reliability index (after CEN, 2002).

Consequence of attaining the ultimate limit state	Minimum values of β and associated P_f in terms of reference period	
	1 year	50 year
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, and economic, social or environmental consequences small or negligible	4.2, 1.5×10^{-5}	3.3, 5×10^{-4}

EN 1990, together with the associated probability of failure.

A number of methods are available for calculating a structure's reliability index (Baecher & Christian, 2003), but in only the simplest of cases are these straightforward in application. Thus, in order to introduce reliability concepts into customary design calculations, the Eurocode suite – in common with other limit state design codes – adopts the concept of partial factors, whereby properties such as applied loads and material strength are increased or decreased as appropriate by some factor before being used in calculation. A crucial requirement of this approach is that the partial factors required to give the target reliability index have been determined in advance. For rock engineering this is a particular difficulty, as is noted later in this paper.

The introduction of structural reliability, design working life and probability of failure to rock engineering by the Eurocode suite is highly significant, as many of the methods customarily used in rock engineering design do not consider any of these aspects. Thus, the near future of rock engineering design may witness the same sorts of changes as has been seen in geotechnical design (Orr, 2012).

In addition to the partial factor method, EN 1990 indicates that probabilistic methods may be used for limit state verification, and EC7 extends this by suggesting that (i) calculations, (ii) prescriptive measures, (iii) experimental models and load tests, or (iv) an observational method may be used – either singly or in combination – to verify the limit state. Of these approaches, those of calculation, prescriptive measures and observational methods are commonly used in rock engineering, and all are the subject of contributions to the Eurocode 7 Workshop at Eurock 2014, with authors reporting on both positive and negative aspects of their application. These contributions are highlighted below.

4 EC7 AND ROCK ENGINEERING DESIGN

The philosophy of LSD brings many advantages to the design process (Baecher & Christian, 2003), which is why modern structural design codes – including the Eurocode suite – adopt it. Unfortunately, the application of LSD to rock engineering is in its infancy, and EC7 in its current form does not correspond with most present rock engineering design procedures. Anecdotal reports suggest that EC7 is proving difficult to apply for rock engineering designs, and one of the key aims of the Eurocode 7 workshop at Eurock 2014 is to investigate this further. Using the various submitted papers as supporting material, the Workshop seeks to identify difficulties that have been encountered and what Code improvements may be useful to overcome them.

The major themes addressed by the various workshop contributions are addressed under the four subject headings of (i) general observations, (ii) uses of calculation, (iii), adoption of prescriptive measures and (iv) application of an observational method.

4.1 *General observations*

Various general shortcomings relating to the application of EC7 to rock engineering are revealed in a number of the contributions. Essentially, these all relate to the fact that, in stark contrast to engineering soils, rock masses are generally discontinuous, heterogeneous and (to varying extents) anisotropic.

The expertise gained by the rock engineering community during recent decades in undertaking and applying rock mass characterization is absent from EC7 (Ferrero et al., 2014), with specific improvements being required in the Code's treatment of discontinuity characterisation and rock mechanics laboratory and field tests (Lamas et al., 2014). In addition, more detailed guidance on scale effects due to the presence of discontinuities is thought to be necessary (Ferrero et al., 2014).

The specific limit states and failure modes applicable to fractured rock masses need greater coverage, and enhanced guidance on the determination of characteristic values for the non-linear strength criteria commonly used for rock masses and for discontinuities should be provided (Ferrero et al., 2014; Lamas et al., 2014). Similarly, the absence from EC7 of guidance regarding strength testing of anisotropic rock is noted (Bozorgzadeh & Harrison, 2014). These authors consider this significant omission should be corrected at the earliest opportunity.

One notable omission from the Code is the use of rock mass classification schemes in rock engineering design (Lamas et al., 2014). Although there are

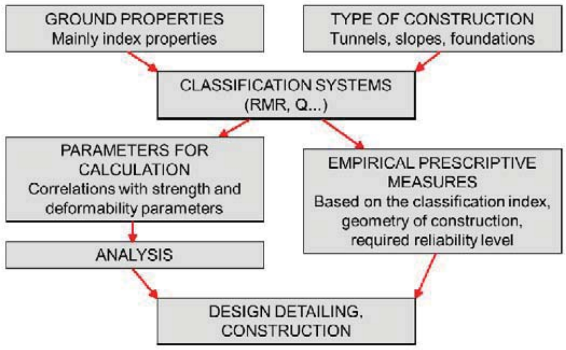

Figure 1: The role of rock mass classification systems within rock engineering design (from Lamas et al, 2014).

well-argued calls for reducing the use of these techniques (Schubert, 2012), their widespread use within both calculation and prescriptive approaches (Fig. 1) means that EC7 should nevertheless clarify their use.

A review of three geotechnical codes used in Spain (Perucho & Estaire, 2014) shows how these offer guidance on various aspects of rock engineering design, including the intensity of ground investigation in fractured rock masses for different geotechnical designs, and the use of empirical relations for ultimate bearing capacity of foundations on fractured rock that incorporate both intact rock strength and degree of fracturing. This guidance can perhaps form the basis of material for inclusion in a future version of EC7; after all, as the authors note, it is the result of many years' development and application.

Regarding the overall structure of EC7, it is concluded that the concepts of empiricism, theory, prescriptive measures and observational methods are currently separated, whereas these are often used in combination in rock engineering design suggested (Harrison et al., 2014). In addition, a key deficiency with the code is its lack of recognition of the central and necessary role that empiricism plays in rock engineering design (as indicated by Fig. 1), and these authors suggest that the immediate challenge is to align EC7 with rock engineering practice.

Recognising the wide range of designs and structures to which EC7 may be applied, the Code introduces the concept of Geotechnical Categories. However, the definitions given in the Code are seen to be unsuitable for some rock engineering designs (Harrison et al., 2014). In this vein, new Geotechnical Categories are proposed that are explicitly linked to uncertainty of ground conditions and types of construction (Olsson & Palmström, 2014) (excerpt illustrated at Table 4; see Olsson & Palmström (2014) for complete table).

Table 4: Proposed Geotechnical Categories (after Olsson and Palmström, 2014)

Excavation risk	Construction examples	Uncertainty in anticipated ground conditions		
		Low	Medium	High
Low	Foundations	1	1 – 2	2
Medium	Foundations involving blasting Tunnels, small/moderate rock cuttings	1 – 2	2	2 – 3
High	Undersea tunnels Large span caverns Underground excavations with low rock cover in susceptible areas High rock cuttings; suspension bridge anchorages Excavations with influence on nearby structures	2	2 – 3	3

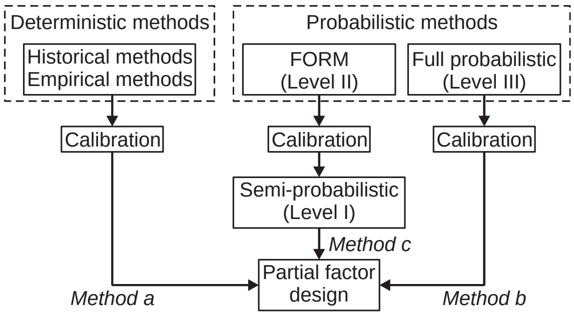

Figure 2. EN 1990 approaches to partial factor design (after CEN, 2002).

4.2 *Use of calculation*

A significant part of EC7 is devoted to limit state verification by calculation through the use of partial factors. EN 1990 presents three approaches for this (Fig. 2), and notes that method *a* has been the one most commonly used to obtain the factors presented in the Eurocode suite. The absence of partial factors specific to rock mechanics parameters is identified in four of the Workshop contributions as a particular shortcoming of EC7 (Bedi & Orr, 2014; Harrison et al., 2014; Lamas et al., 2014; Muralha & Lamas, 2014). However, as most current rock engineering design calculation approaches could potentially be re-cast as method *a*, EC7 would be improved if such calibration work were to be undertaken.

EN 1990 notes that method *b* is seldom applied due to a lack of statistical data, but that method *c* is being used for further code development. Method *c* approaches make use of statistical distributions to incorporate variability, but both Bedi & Orr (2014) and Lamas et al. (2014) note that not all rock mass parameters may be aleatory. As this would render method *c* inaccurate for rock engineering, it needs to be explored further.

This issue of the applicability of the aleatory model to fractured rock masses leads Bedi & Orr (2014) to speculate that partial factor calibration for rock engineering has not been performed because the epistemic nature of rock mass parameters precludes it. This finding raises the question, is LSD and hence the Eurocode suite applicable to rock engineering design? Clearly, this needs urgent further investigation.

Two Workshop contributions explore the issue of characteristic values for intact rock strength. Muralha & Lamas (2014) analyse specimens of granite in order to determine a cautious estimate of strength, and conclude that it is not clear how this should be done for triaxial strength criterion parameters. By analysing an extensive set of strength data for a highly anisotropic slate, Bozorgzadeh & Harrison (2014) suggest that the aleatory model is applicable to the strength of such rock, but also identify strong heteroscedasticity (i.e. non-uniform variability) of axial strength with respect to cleavage orientation. The authors point out that this has important ramifications for the assessment of characteristic strength in such rocks.

Four contributions to the Workshop report on the direct application of EC7 to rock engineering design calculations. Using distinct element modelling of a particular slope geometry in two different fractured rock masses (one a weak mudstone, the other a strong microgranite) to design a rock dowel stabilisation scheme, Koe & Ogunmakin (2014) show that Combinations 1 and 2 of Design Approach 1 give significantly different results in both cases. This discrepancy leads the authors to recommend that benchmarking using commercially available numerical analysis programs be undertaken of some exemplar rock engineering problems.

A comparison of full probabilistic (Level III of Fig. 2), semi-probabilistic using partial factors (i.e. Level II) and deterministic (i.e. global Factor of Safety) analyses for the case of single plane sliding is reported on by Nomikos & Sofianos (2014). The authors conclude that the first of these methods links directly to structural reliability, the complexity of the partial factors makes the link awkward in the second case, and in the third case is not possible. This confirms that traditional deterministic analyses will be of limited utility in the context of EC7.

SLS and ULS verification normally require different partial factors, but Avellan (2014) proposes a method by which the same factors can be used for both analyses. This may improve the direct applicability of EC7 to serviceability verification.

Finally, Estaire & Olivenza (2014) offer suggestions for applying the EC7 Design Approaches to spread foundations and both plane and wedge instability of slopes. They show how the partial factors should be included in these analyses, and suggest that partial factors for different rock mass conditions be specified in the National Annexes to EC7.

4.3 *Adoption of prescriptive measures*

Although prescriptive measures are not given extensive treatment within EC7, Olsson & Palmström (2014) note that, in conjunction with suitably conservative designs, they have been extensively used for many years by the Geotechnical Engineering Office (GEO) in Hong Kong. These authors also show how the Code in its current format may lead to misunderstandings regarding the application of prescriptive measures, and illustrate inconsistencies regarding the application of prescriptive measures and limit state verification for commonplace design situations. They conclude that the treatment within the Code of prescriptive measures for rock engineering needs substantial improvement.

4.4 *Application of an observational method*

A number of contributions to the Workshop are related to the use of observational methods (OM), which may reflect the widespread use they enjoy in rock engineering. Stille & Virely (2014) suggest that the epistemic nature of fractured rock masses leads to observational methods being a natural choice for rock engineering, as the observations reduce the uncertainty and thus allow verification that a structure satisfies the limit state. However, Spross et al., (2014) note the rarity of case studies that show formal application of OM principles, and suggest this may be that the method is to some extent considered complex and associated with low safety margins. To rectify this, these authors present a procedure for using observations from the construction phase to verify the safety of the final structure, and show that this satisfies the current definition of OM in EC7. This procedure supports the views of Stille & Virely (2014), who conclude that the OM as presented in EC7 needs further elaboration in order to become generally applicable to rock engineering design and construction. These authors also suggest that the OM should formally be applied to every Geotechnical Category 3 project, with versions of OM that rely on visual observations being developed for use with Category 1 and 2 projects.

Migliazza et al. (2014) explore further the importance of monitoring in rock engineering design and

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construction, and show how technological development both in sensors and data transmission has led to new fields of application that should be consideration in EC7.

Although the OM is usually associated with excavation and support of underground openings, Christiansson et al. (2014) report on its application to a project involving grouting to control groundwater drawdown around an underground facility comprising complex geometry and large caverns. Inflow tests prior to construction provided a model for predicting the maximum inflow that would meet the drawdown criterion. The inflow after grouting was predicted using design calculations, and by comparing this design prediction with results from the inflow tests it was established that there was an acceptable probability that the actual drawdown would be within the acceptable limits, as stipulated by the principles of the Observational Method. The authors report that the project was successful, with the actual drawdown corresponding well with the predicted behaviour.

5 CONCLUSIONS

Eurocode 7 has been under development since 1975, but during this time does not seem to have had explicit input from the rock engineering community. It has been developed using the limit state design philosophy originally developed for structural engineering, and anecdotal reports suggest it is difficult to apply to rock engineering. The Code is now undergoing a period of maintenance, and the contributions to the Eurocode 7 Workshop held at Eurock 2014 cover a diverse range of potential maintenance topics. These include: characterisation of discontinuous rock masses; application of rock mass classification schemes; partial factor calibration; the epistemic nature of rock mechanics parameters and the possible inapplicability of LSD; benchmarking of rock engineering design calculations; the limited value of traditional deterministic analyses in the context of EC7; prescriptive measures; the observational method; and, the introduction of new forms of instrumentation.

Current problems have been identified in all of these areas, but most importantly suggestions have been given for overcoming these. As the importance of Eurocode 7 for rock engineering design will continue to grow, these suggestions have the potential to become future opportunities: not only for vastly improved rock engineering design and construction techniques, but also for new and exciting research themes in support of them.

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Some key issues regarding application of Eurocode 7 to rock engineering design

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ABSTRACT: Eurocode 7 is the harmonised European standard dealing with geotechnical engineering design. Although design involving rock masses is included in the code, rock engineering principles do not seem to be fully contemplated. This document summarizes some of the most relevant issues the authors consider that need to be improved in what concerns the application of Eurocode 7 to rock engineering design. This includes topics such as the implications of the discontinuous nature of rock masses, limit states and failure modes, strength criteria, characteristic values and partial factors for rock mass parameters, rock mass characterization, use of classification systems in design, among the most significant issues.

1 INTRODUCTION

The Structural Eurocodes were conceived as a group of harmonised European standards for the structural and geotechnical design of buildings and civil engineering works, and they are a suite of ten standards concerned with the safety, serviceability and durability of structures. Eurocode 7 (EN1997-1) is one of these standards and is concerned with all aspects of geotechnical design. It deals with constructions in or on the ground, which is defined as "soil, rock and fill in place prior to the execution of the construction works". Rock engineering design is, therefore, included in the scope of Eurocode 7 (EC7), but this is often overlooked.

The Eurocodes adopt a semi-probabilistic approach of safety verification, based on rules, partially deterministic, that introduce safety at the following levels: selection of appropriate representative values of the various random parameters (actions and resistances); application of partial factors to these parameters; and introduction of safety margins in the various models used in the calculations.

Geotechnical design was slower than structural design in the application of probabilistic or semi-probabilistic approaches. Reasons for this may be the deep roots of empirical methods used in the design of structures in or on the ground. Geotechnical design does not deal with manufactured materials, with relatively well known parameter values, but with natural materials, of a great diversity as regards their origin and the condition in which they are found in nature. Geotechnical structures are not so well defined geometrically as the structure of a building or a bridge, and the actions on them are also more difficult to establish and quantify.

Introduction of EC7, bringing structural safety concepts in geotechnical design, was an important step forward in many European countries. It is interesting to note that several European countries, with well-established codes for structural design of buildings and bridges, had no code for geotechnical design before EC7. This peculiar situation generated much more discussion regarding applicability of the design principles of the Eurocodes to geotechnical structures, and also regarding its application in practice to geotechnical construction works, than it did with the other Eurocodes. At present, application of EC7, though with a number of difficulties, can be considered at a cruise speed for many types of geotechnical engineering problems.

Unfortunately, rock engineering problems cannot be considered an example regarding application of EC7. Many reasons may lie behind this fact, but the following are usually considered to be central:

- Despite of covering all aspects of geotechnical design in soil and in rock masses, problems involving soils are clearly dominant in the code. One of the reasons certainly lies in the scope of EC7, i.e. the geotechnical design of buildings and civil engineering works. The code states that "for the design of special construction works, such as dams, other provisions than those in the Eurocodes might be necessary".
- Rock engineering includes a large number of issues not covered by EC7, and not even related to civil engineering works. Often, rock engineers don't have a civil engineering background. This

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may be one of the reasons why rock engineers had little interest so far for EC7.

- A consequence of the dominant role of soil engineering was that the experts responsible for drafting the code had a soil engineering background, with reduced interest for rock engineering aspects. This was responsible for an incomplete and often incoherent treatment of rock engineering issues.
- Rock engineering presents a critical difference, when compared with soil engineering, which is the discontinuous nature of the rock masses. While design in soil mechanics typically assumes a continuous medium approach, in rock mechanics the role of the discontinuity surfaces is often dominant in the rock mass behaviour.

It is the purpose of this paper to present and discuss some key difficulties that the authors consider to exist regarding application of EC7 to rock engineering design.

2 LIMIT STATE DESIGN IN ROCK ENGINEERING

The basis of the limit state design philosophy is that, for each particular design situation, all the possible limit states for a structure, or part of it, shall be considered and that it shall be demonstrated that the likelihood of any limit state being exceeded is sufficiently small (Orr & Farrel, 1999). It is assumed that both the actions on and the resistance of a structure are aleatory and can be described by statistical distributions. However, there is evidence that this is not always the case when we deal with ground properties. In fact, the uncertainty that affects the determination of a ground property may be associated with its natural aleatory variability and with uncertainty associated with inaccuracies in our capability to predict reality (Bond & Harris, 2008).

Geomaterials are highly variable in nature and the information that we obtain from site investigations is usually incomplete and almost always insufficient (Muralha *et al.*, 2009). In many cases, by increasing the information available and improving the understanding of the processes involved, the uncertainty associated with a ground property can be reduced and it is possible to quantify its aleatory characteristics. In this case the property is said to be extrinsically epistemic. However, in other cases, ground properties cannot be assumed to have an aleatory nature, or there is not enough information to demonstrate it. In such cases, the ground property is said to be intrinsically epistemic (Bedi & Harrison, 2012).

The issue of the aleatory or epistemic uncertainty of ground properties may become critical when we deal with the properties of fractured rock masses and application of statistical methods that imply use of characteristic values and partial factors.

Bedi & Harrison (2012) demonstrated the difficulty of dealing with epistemic variability in the framework of EC7 and proposed adopting the following pragmatic approach in the immediate future:

a) fully understand which aspects of rock mechanics and rock engineering are genuinely aleatory;
b) for those aspects that are extrinsically epistemic, either ensure the required data are collected, or work to quantify the inherent variability and hence determine appropriate partial factors;
c) for those aspects that are intrinsically epistemic, eschew limit state design principles and continue with the current load and strength factor approaches.

The same authors suggest that methods should be developed, by which epistemic uncertainty may be approximated as aleatory and thus incorporated in the current limit state design paradigm.

Both approaches should be followed simultaneously. However, the current need to have mechanisms to enable the application of EC7 to rock engineering design, calls for the urgent adoption of the pragmatic approach above.

A key issue that needs to be addressed urgently has to do with the values of the partial factors to use in rock engineering design. It is very doubtful that the values of the partial factors prescribed for strength properties of soils also apply to fractured rock masses. Besides, strength parameters very common in rock engineering are not considered in EC7. This is certainly a complex task that will still require a great amount of research and time.

Another key issue regarding application of EC7 to rock engineering design has to do with which limit states to consider in design and with consideration of the failure modes applicable to fractured rock masses. A complete review of the existing text is needed, since it often refers to soils only, without attention rock masses. Consideration of important geological features, such as individual faults or shear zones, cavities or holes filled with soft material, planes of stratification or schistosity, as well as other joints grouped in sets, is of key importance for definition of failure modes in rock engineering, but this is not adequately reflected in the current EC7 wording.

When looking at limit state design and failure modes from a rock engineering perspective, it is of interest to broaden the range of applications from the strict scope of EC7 (mainly shallow and deep foundations, slopes, retaining structures and anchorages) to common types of rock engineering projects, including underground caverns, tunnels or dam foundations. Despite the problems that may arise in the application of EC7 to some of these works, such as tunnel lining support, it is useful to try to have the same limit state rationale applied to the design of a wide range of rock engineering structures.

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3 DISCONTINUOUS NATURE OF ROCK MASSES

Within the general philosophy of Eurocodes lies the assumption that failure mechanisms dictate the design methodology. For instance in Section 11 of EC7 dealing with overall stability one can read: “All limit states […] shall be considered”. The knowledge about possible relevant failure mechanisms is a requirement for the design of structures. Only then it is possible to design models and safety criteria. It can be therefore stated that parameter characterization should correspond to pertinent failure mechanisms.

Typical construction and structural materials, such as steel, wood or concrete are continuous. Their analysis follows the rules of continuum mechanics, and thus the demand is evaluated with well-established structural design procedures. The assessment of their capacity is clearly defined according to the codes. For soils such a design procedure can usually also be followed, since they may be considered macroscopically continuous.

However, rock masses as found in nature are not continuous materials. Joints and other fractures of geological origin tend to be ubiquitous features in a body of rock (Figure 1). In this way the strength and deformability features are influenced by both the properties of the intact materials and those of the ensemble of discontinuities found in the rock mass, i.e. the rock mass structure.

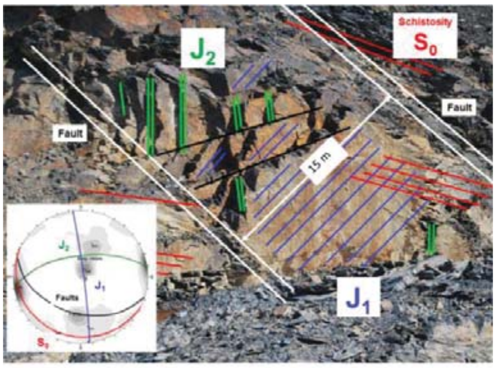

Figure 1. Typical rock mass in a slope: slate rock mass showing various discontinuity sets and stereographic projection of data.

The behavior of the rock mass depends on the size of its elementary volume and on its relation to that of the structure (Hoek & Brown, 1980). These effects may be appreciated by considering various scales of loading to which a rock mass is subjected in construction practice (Figure 2, lower left corner). The larger the elementary volume and stronger the intact rock, the more relevant are the structural features such as bedding, faults and other joints, which control the behavior of the rock mass (Barton, 1998).

Thus discontinuities must necessarily be accounted for, even if they can be considered implicit or explicitly. The approach selection is governed by the features of the joints in relation to problem scale (Figure 2) (Jing, 2003). Continuum models can be used in poor rock masses or when rock structure does not control the stress strain response. Equivalent continua can be used when the strain response depends on the global characteristics of the system intact rock-joints. Finally, discontinuous or discrete models should be used when the stress-strain response is mainly governed by the discontinuity sets.

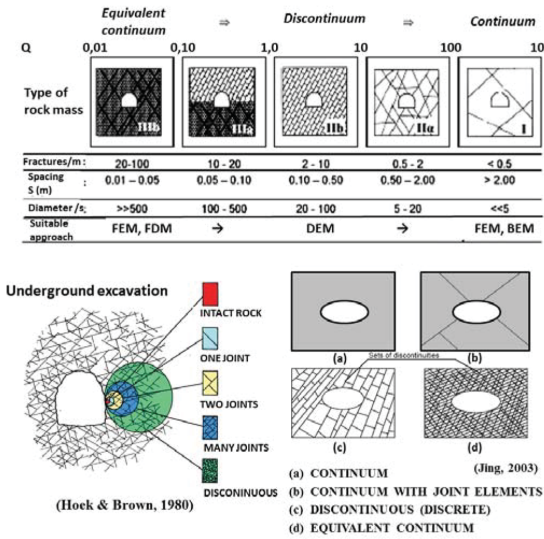

Figure 2. Relationship between behavior model and scale in rock engineering according to various authors.

The scale of discontinuity spacing versus the size of the construction work should also be considered to account for the model approach (Figure 2). It is also important to account for the role of individual large discontinuities, typically faults, since these structures may well largely control the response of the work to be carried out (Figure 3).

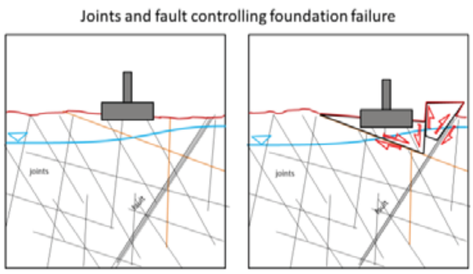

Figure 3. Foundation failure controlled by joints and a fault.

4 ROCK MASS CHARACTERIZATION

Rock mass characterization is largely overlooked in EC7. As indicated in section 2, quite often the structure or ensemble of joints found in a rock mass controls its behaviour, so this is one of the most relevant

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issues in what concerns rock mass behaviour. A reasonable way to appropriately account for structure is to carry out a quantitative description of discontinuities in rock masses, following for instance the 40-page ISRM Suggested Method (ISRM, 2007). This is certainly a key point that is not properly considered in EC7, which only states in its part 2 that “the discontinuities and corresponding infilling materials existing in the rock mass often control the strength and deformation characteristics of the material as a whole. Therefore, they shall be defined as closely as possible during the sampling operations, if such properties have to be determined.” And then it adds “Discontinuities such as bedding planes, joints, fissures, cleavages and faults shall be quantified with respect to pattern, spacing and inclination using unambiguous terms”.

So EC7 deals with rock mass structure in a vague way. In particular cases some joints may play a relevant role in limit state identification, so it would be necessary an appropriate study of this type of joints in-situ.

A proper characterization of the structure of a rock mass is needed in order to identify failure mechanisms and to obtain relevant parameters. According to the authors’ point of view, this is one of the main draws-backs of EC7 when dealing with construction in rock masses.

Additionally, the list of laboratory tests proposed by EC7 for rocks overlooks a number of tests, which could be relevant for rock mass characterization in particular cases, such as the determination of slake-durability index, Schmidt rebound hardness, shore hardness, block punch strength index (BPI) or the complete stress-strain curve for intact rock to cite a few. Moreover, and in what relates field testing, the measurement and estimate of the natural or in-situ stress field tests is completely overlooked by EC7.

So, it seems that EC7 was developed for soils, and rocks are only dealt with as secondary materials. It is relevant to remark that, as stated above, in order to obtain rock mass reasonably realistic parameters and an estimate of their variability for a large number of limit state analyses, a good knowledge of rock mass structure is of paramount relevance. And this typically complex structure usually plays a more relevant role than intact rock parameters (as derived from lab testing) in the overall strength response and behaviour of the rock mass. In this way, if the rock discontinuities are not appropriately measured in-situ, it will not be possible to have available relevant parameters.

5 ROCK MASS STRENGTH

The mechanical behaviour of rock masses may be very complex, typically much more than most soils, due to different reasons like the ones remarked in the following lines.

As it is well known, most rocks present a dilatant and brittle behaviour at low confining pressures that changes gradually to a more ductile and less dilatant behaviour with higher confining pressures. The instantaneous friction angle is not constant but higher at low confining pressures and lower at increasing confining pressures. As a result, the failure envelope of rock is non-linear but concave downward. Moreover, the intermediate principal stress seems to play an important role. A number of different non-linear strength criteria have been proposed for intact rocks by some authors during the years, such as those presented in Figure 4.

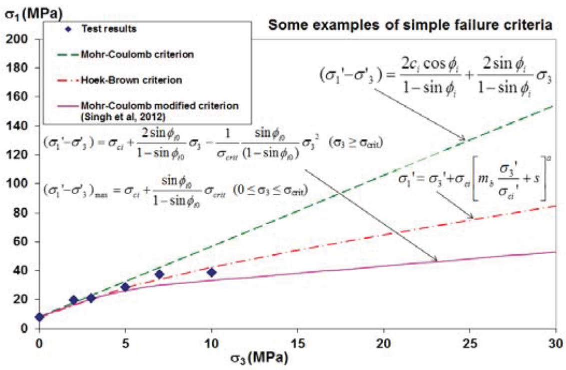

Figure 4. Some examples of simple failure criteria.

So, it can be remarked that one of the important differences between the design of soil and rock masses, from the mechanical point of view, is that while soils are usually considered to follow a linear and unique agreed strength criterion—Mohr-Coulomb—rocks follow a non-linear criterion, for which there is not a unique and universally accepted expression, but different proposed ones. Some questions arise then in relation with the EC7 philosophy: How to choose appropriate “representative values” for the parameters? How to choose partial factors for the parameters?

In this regard it is important to mention a widely used procedure to derive the strength parameters for design, which consists of using the Hoek-Brown strength criterion for interpretation of triaxial tests in intact rock in the laboratory and, through an empirical parameter obtained by in situ observation—the Geological Strength Index, GSI—, converting intact rock strength parameters in rock mass strength parameters; often not only Hoek-Brown, but also Mohr-Coulomb parameters. An important issue to be clarified for application of this procedure in the EC7 framework is how to apply partial factors and to what parameters. To the intact Hoek-Brown parameters only? Also to GSI? Or to the rock mass Hoek-Brown or Mohr-Coulomb parameters?

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Another important characteristic of rocks is their very common anisotropic behaviour. In a higher or lower degree most rocks present a different mechanical behaviour—characterized mainly by its strength and deformability—in various directions, due to a number of reasons. Referring only to the rock matrix, this anisotropy is present in metamorphic rocks, characterized by planar minerals in a plane disposition due to genetic reasons, but also in many other micro fissured rocks. Figure 5 shows an example of experimental results of triaxial strength in a shale (McLamore, 1966, taken from Serrano, 1996) and some types of adjusted models. The question about how to choose representative values of the strength arises again.

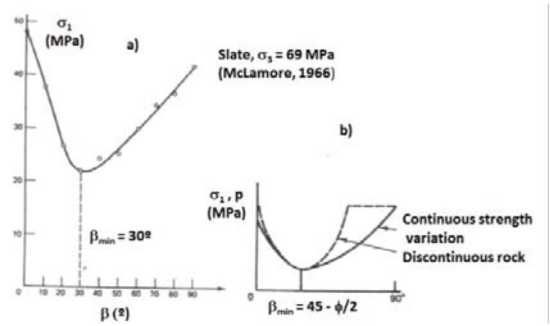

Figure 5. Triaxial strength in a shale. a) Experimental results; b) Types of adjusted models (Serrano, 1996).

Moreover, the rock mass has two main components, which are the intact rock and the fractures, having the latter a significant effect on the rock strength in most cases. The failure may be produced by different failure mechanisms in which the predominant failure may be produced inside the rock matrix or through the fractures. An example of this is shown in Figure 6 for spread foundations in rocks (Serrano & Olalla, 1998).

These fractures often determine the location and orientation of the failure surfaces, and this failure is often initiated in the weakest zones in the rock mass. Anisotropic behaviour of rocks is increased due to the presence of macro-fractures on it.

To increase even more the complexity of the failure in rocks, the failure criteria in the fractures may be also non-linear depending on the confining pressures and the strength of the rock matrix or it can be the linear strength criterion of the soil filling contained in the fractures. According to Barton (1976) simple discontinuities may be classified depending on the aperture and the filling width they have in four categories (Figure 7).

The mechanical behaviour of the discontinuities is different for each type of discontinuity: in type A it mainly depends on the rock matrix strength and the shape of the discontinuity while in type D it depends only on the strength of the filling; the behaviour of types B and C is a complex behaviour that depends on the confining pressures, being more similar to type D for low confining pressures but changing for higher pressures to a behaviour more like type A.

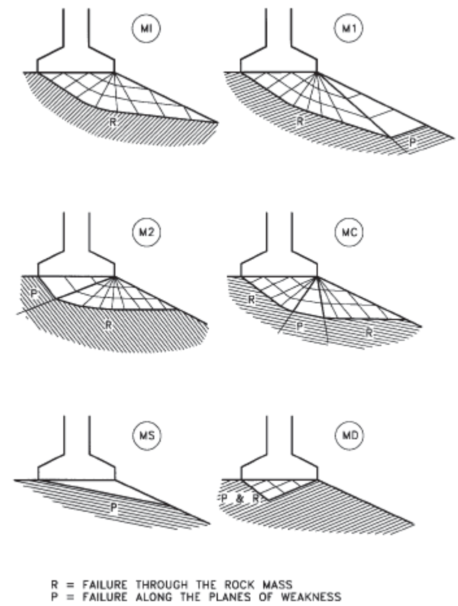

Figure 6. Possible anisotropic failure mechanisms depending on the dip of the set of weakness planes (Serrano & Olalla, 1998).

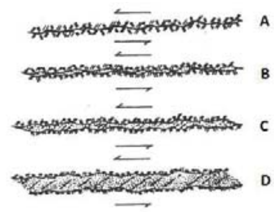

Figure 7. Types of discontinuities (Barton, 1976).

Another question arises from this: can this really complex behaviour be introduced somehow in a standard? Even if the fractures are of type A, with no filling influencing its mechanical behaviour, it is a complex and non-linear behaviour, as reflected in Figure 8.

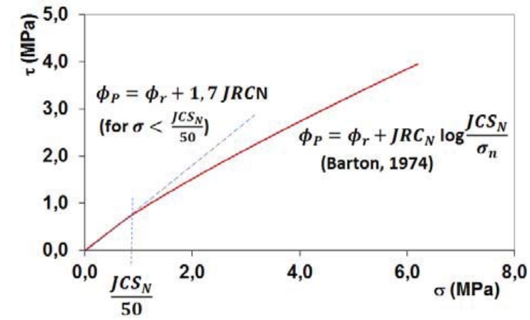

Figure 8. Strength criterion for fractures (Serrano, 1996).

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6 ROCK MASS CLASSIFICATION SYSTEMS

Rock mass classification systems, such as the RMR and the Q system, are widely used in the design process in rock engineering. They are based on the experience of the authors that developed them and imply the determination of a number of parameters that characterise the rock mass and its condition, for a specific type of project, mostly for tunnels, but also for slopes and foundations.

The result of the classification can be used in two different ways: i) to obtain, through empirical correlations, the rock mass strength and deformability parameters to be used in design calculations; or ii) to obtain, through prescriptive measures, the design of elements such as a tunnel support. These processes are illustrated in Figure 9.

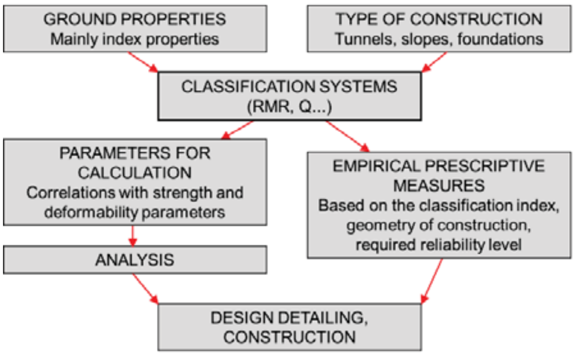

Figure 9. Rock engineering design using classification systems.

Incorporation of this design processes in the scope of EC7 should be clarified. Design by prescriptive measures is allowed in EC7 "in design situations where calculation models are not available or are not necessary". An important issue to be clarified is to which types of geotechnical structures, in terms of their complexity, does design by prescriptive measures, with the use of classification systems, apply: only to EC7 geotechnical category 1, or also to geotechnical category 2?

7 CONCLUSIONS

Application of EC7 to rock engineering design faces difficulties due to the distinct nature of the material it deals with and of the types of construction works, when compared with other Eurocodes. Some of these difficulties were identified in this paper and are summarised here:

– The assumption of the aleatory nature of rock mass parameters may not always be valid.
– Non-linear strength criteria are commonly used for rock masses and for discontinuities. Guidance on determination of characteristic values of their parameters is not available.
– Partial factors either do not exist or are not calibrated for rock masses and for discontinuities.
– Limit states and failure modes applicable to fractured rock masses need more attention in EC7.
– Rock matrix and rock mass anisotropy need to be addressed in EC7.
– Rock mass characterization needs improvement in EC7 as regards discontinuities, as well as laboratory and field tests.
– Inclusion of the widely used rock mass classifications for design of rock mass structures in the scope of EC7 needs to be clarified.

Consideration of these and other difficulties in the revision of EC7 is not an easy task and requires considerable research and time. However, it is absolutely necessary, so that rock engineers have clearer rules to follow and, as a consequence, may adhere to the application of EC7 to rock engineering design.

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The approach to rock engineering in Spanish normative documents

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ABSTRACT: In Spain there are currently three geotechnical standards: Building Technical Code (2006), for application in the field of building construction; Guide of Foundations in Road Works (2003), for application in the field of road construction; and Geotechnical Recommendation for Design of Maritime & Harbour Works (2005), for application in the field of port construction. The approach to rock mechanics in these Spanish standards is reviewed in this paper, considering different geotechnical areas: site investigation, shallow and deep foundations and slopes. It will be shown that in some cases these standards have different ways of approaching these geotechnical problems that may give out to remarkable differences in the results obtained. This reflects the fact that there are many different empirical formulae and procedures, collected in standards all around the world. Evolution in EC-7 considering more relevant aspects of rock engineering will help to unify methods and criteria used, but the variety of possible existing approaches, some of them showed in this paper, stresses the actual difficulties for that unification.

1 INTRODUCTION

There are some key aspects that make the mechanical behavior of rock masses, in general, much more complex than in most soils and therefore there are more difficulties in the application of standards to rock engineering design as it is being seen in Eurocode 7, EC-7 (Lamas et al, 2014).

In Spain there are currently three geotechnical standards, codes or guides, developed in different areas of civil engineering and construction:

1 Building Technical Code (Código Técnico de la Edificación, CTE), for application in the field of building construction. Approved in 2006.
2 Guide of Foundations in Road Works (Guía de Cimentaciones para Obras de Carretera, GCC), for application in the field of road construction. Approved in 2003.
3 Geotechnical Recommendation for Design of Maritime & Harbour Works (Recomendaciones para Obras Marítimas, ROM 0.5-05), for application in the field of port construction. Approved in 2005 (previous version in 1994).

Although only the use of CTE is compulsory as a Code, the other two are widely used in their fields and their use very often becomes compulsory as well.

The authors have been working in the preparation of the Spanish National Annex to EC-7 (Estaire et all, 2014) and in some Evolution Groups of EC-7 for what a deep analysis of the Spanish geotechnical standards have been carried out (Perucho, 2013).

The recommended site investigation, the bearing capacity for spread and pile foundations on rock and the slope stability in rocks are the aspects reviewed in this paper with the purpose of looking for differences and similarities in the way they treat the problems related to rock engineering.

The final discussion is believed to be helpful somehow in the future evolution of EC-7 as these Spanish standards are the results of a long period work, thoughts and agreements between many geotechnical and rock mechanics experts.

2 SITE INVESTIGATION

2.1 *General*

The aspects considered related to the site exploration are the intensity, the rock mass characterization and the in situ and the laboratory tests.

2.2 *Intensity of exploration*

In the field of building construction, CTE is quite precise as it indicates, for a building, the maximum separation between boreholes and their suggested depths (as it can be seen in Table 1) according to a classification previously made. This classification distinguishes among four types of construction, based on the number of floors, and three types of ground.

These tables are valid for soils and rocks as no specific distinction is made. However it can be stated that for normal outcropping rock, the ground can be classified as “T1: Favorable”.

In the field of civil engineering works, GCC and ROM 0.5-05 have the same structure in the definition of the site investigation. For each type of work (foundations, embankments, and retaining structures) they provide some tables with the number of

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transversal profiles and number of points of exploration to be done, according to some categories (between 3 or 5) of site investigation intensity. These categories depend on the site geotechnical conditions and the consequences of the work failure (from A to C, being A the category for the worst consequences and C for the least important), as it is shown in Table 2 and Table 3.

Table 1. Maximum distances (d_{max}) between site exploration points and suggested indicative depths (P) (CTE)

Construction type	Ground group			
	T1		T2	
	d_{max} (m)	P (m)	d_{max} (m)	P (m)
C-0, C-1	35	6	30	18
C-2	30	12	25	25
C-3	25	14	20	30
C-4	20	16	17	35

Table 2. Categories of intensity of site investigation (GCC)

Foundation condition	Variability		
	Heterogeneous	Normal	Homogeneous
Unfavorable	Special	Intensive	Normal
Normal	Intensive	Normal	Reduced
Favorable	Normal	Reduced	Sporadic

Table 3. Categories of intensity of site investigation (ROM)

Geotechnical conditions	Failure consequence		
	A	B	C
Unfavorable	Detailed	Detailed	Detailed
Normal	Detailed	Reduced	Reduced
Favorable	Detailed	Reduced	Minimum

There are some remarks about the role of geomechanical in situ surveys in the site investigation and the characteristics of rock ground to be classified in some of the established categories. For instance, the need for intensive explorations may be due to heterogeneities in the ground, that may be caused by faults, irregular alteration areas –like in granitic formations-, or karsts. A special type of exploration may be needed when both, heterogeneous ground and unfavorable foundation conditions, occur together, like in a pile foundation on karst.

2.3 *Rock mass characterization*

CTE has a specific clause for the rock mass characterization that indicates that a rock mass is characterized by the strength of the rock matrix, accompanied by other properties related to its discontinuities like: aperture, roughness, type of filling, spacing, fracture index, persistency, RQD or water presence. These parameters may be used to determine other indexes like RMR, which are representative of the global behavior of the rock mass. For describing the aperture, roughness, filling, spacing joint count and persistency the indications given by ISRM (1981) are used.

It is also given an additional classification of the rock matrix to estimate a value of rock uniaxial strength in the field, and some indications on the rock quality depending on RQD value and the water content in discontinuities.

Both GCC and ROM 0.5-05 include, for the rock mass characterization, a classification for estimating the preliminary characteristics of rock matrix (dry specific weight, uniaxial strength, deformation modulus) depending on the type of rock.

The description of the weathering degree of the rock is made following the same indications given by ISRM (1981).

For joint characterization GCC gives indications adapted from ISRM with little variations.

2.4 *In situ and laboratory tests*

The only in situ tests mentioned are the Lugeon test and the dilatometer. Related to laboratory tests, they basically agree in most of the more appropriate tests to be considered for rocks and their standards applicable in Spain.

3 BEARING CAPACITY FOR SPREAD FOUNDATIONS ON ROCK

3.1 *Building Technical Code (CTE)*

Rock shall be considered as a soil when the rock strength is low (Uniaxial Compressive Strength, UCS < 2,5 MPa), the RQD < 25 or the rock is strongly weathered (weathering degree > IV). In other cases, the following expression may be used to estimate the allowable bearing capacity (q_a) of a spread foundation:

$$q_a = K_{sp} UCS \quad \text{where: } K_{sp} = \frac{3 + \frac{s}{B}}{10\sqrt{1 + 300\frac{a}{s}}} \tag{1}$$

being s the joint spacing (s > 300mm), B the footing width in m (0,05 < s/B <2) and a the discontinuity opening (a < 25 mm, 0 < a/s <0,02). This formula is the same as the one included in the Canadian Foundation Engineering Manual (2007).

This Code indicates that this allowable pressure includes a safety factor of 3, and it does not give any limit to it. The following limitations are indicated:

– The ground is essentially horizontal without any problems of lateral stability.
– The resultant load inclination is lower than 10%.
– In sedimentary rocks, the strata must be horizontal or sub-horizontal.

CTE also indicates that when the spread foundation is on rock there may be problems due to the rock structure, the joint orientation, the anisotropy of the rock mass, etc. In these cases (illustrated in a figure) a specific analysis shall be done.

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3.2 *Guide of Foundations in Road Works (GCC)*

When the rock strength is very low (UCS < 1MPa) or strongly fractured (RQD < 10%) or weathered (weathering degree ≥ IV) it is recommended to consider the rock as a soil and to apply the analytical model, obtaining the strength parameters from direct shear or triaxial tests in laboratory. For stronger, less fractured and weathered rocks than stated before, the bearing capacity may be estimated from the following data: the type of rock, UCS, mean weathering degree, RQD and the joint spacing. All of these parameters shall be referred to the rock volume located under the foundation up to a distance of 1,5B*, from the foundation level.

The allowable bearing capacity (q_a) may be estimated as:

$$q_a = \alpha_1\alpha_2\alpha_3\alpha_4\sqrt{p_r UCS} \quad < 5 \text{ MPa (unless properly justified)} \tag{2}$$

Where p_r is a reference value (e. g. 1 MPa) and α_i are dimensionless parameters that depend on the rock type, the weathering degree, the joint spacing and the load inclination as follow:

$\alpha_1 = \sqrt{\dfrac{10\,UTS}{UCS}}$ (3), where UTS is the uniaxial tensile strength. A table of values of α_1 is given depending on the rock type, in case it is not calculated with the previous formulae. Its range is from 0,4 to 1.

α_2 values are given depending on the weathering degree (1,0 for sound rock (degree I), 0,7 for degree II and 0,5 for degree III). So, its range from 0,5 to 1.

$$\alpha_3 = \min(\alpha_{3a}, \alpha_{3b});\ \alpha_{3a} = \sqrt{\frac{s}{1m}}\ ;\ \alpha_{3b} = \sqrt{\frac{RQD(\%)}{100}} \tag{4}$$

where s is the joint spacing from the family of lowest value.

$\alpha_4 = (1{,}1\text{-tg}\delta)^3$ when the resultant load inclination is higher than 10% (tgδ <0,10, in this case slide and turning over must be checked) and $\alpha_4 = 1$ if lower.

The following limitations are indicated in GCC:

- The ground slope is lower than 10%.
- There is not a water flow with gradient higher than 0,2 in any direction.
- The foundation area is lower than 100 m^2. If it were higher, specific foundation movement calculation must be carried on.

3.3 *Geotechnical Recommendation for Design of Maritime & Harbour Works (ROM)*

When the rock is strongly fractured (RQD < 10% or joint spacing, s < 10 cm) or weathered (weathering degree ≥ IV) this method shall not be used as the rock shall be considered as a soil for the estimation of its bearing capacity.

The data needed to estimate the bearing capacity are the following: foundation geometrical data (width B, B*, length L, L*, depth D), load inclination (δ), general structure of the rock mass (the area of special interest is the one surrounding the foundation: 4Bx4L in horizontal and 2D in depth under the foundation level), UCS, weathering degree, mean RQD value in a depth B under the foundation level (and BxL in horizontal) and joint spacing (value corresponding to the closest family in the area of foundation).

The ultimate bearing capacity (q_{ult}) is given by:

$$q_{ult} = 3 f_D f_A f_\delta \sqrt{p_r UCS}) \quad < 15 \text{ MPa} \tag{5}$$

where p_r is a reference pressure (e. g. 1 MPa) and f_i are reducing factors depending on the fractures (f_D), the weathering (f_A) and the load inclination (f_δ), as follows:

$$f_D = \min(f_{D1}, f_{D2}) \qquad f_{D1} = 2\sqrt{\frac{s}{B^*}} \le 1 \tag{6}$$

$$f_{D2} = 0.2\sqrt{\frac{B_0\ RQD(\%)}{B^*}} < 1 \tag{7}$$

where s is the joint spacing, B* is the equivalent footing width, B_0 is a reference width (e. g. 1m).

f_A: is 1 for sound rock (weathering degree, wd=I), 0,7 for wd=II and 0,5 for wd=III).

$$f_\delta = (1.1 - tg\delta)^3 < 1 \tag{8}$$

The following limitations are indicated:

- The ground inclination is lower than 10%.
- If the foundation is close to a slope, the global stability shall be studied with a different method.
- If the foundation area is greater than 100 m^2 local failures shall be studied.

The allowable bearing capacity (q_a) is obtained by dividing q_{ult} by a global safety factor of 2,8; 2,3 or 2,1, depending on the design situation (permanent, characteristic or accidental, respectively).

4 PILE FOUNDATIONS IN ROCK

4.1 *Building Technical Code (CTE)*

This Code does not indicate anything relative to this aspect.

4.2 *Guide of Foundations in Road Works (GCC)*

The ultimate unit tip resistance (q_p) is given by $q_p = 2q_a < 20$ MPa, where q_a is the allowable bearing capacity of a spread foundation in rock [eq. 1].

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If the embedment is relevant, this tip resistance can be increased multiplying it by the embedment factor (d_f) in the area of the tip:

$$d_f = 1 + 0.4\frac{L_r}{D} \leq 2 \tag{9}$$

where L_r is the embedment depth in rock of the same or better quality than the one in the tip and D is the pile diameter.

The shaft resistance for piles in rock shall only be taken into account inside sound rock or with weathering degrees II or III, as defined by ISRM, as worst. The ultimate unit shaft resistance inside the rock embedment for bored piles (τ_f) is estimated as:

$$\tau_f = 0.1 q_p < 2 \text{ MPa} \tag{10}$$

where q_p is the unit tip resistance for that rock as stated before.

The allowable bearing capacity is obtained using a global safety factor of 3,0; 2,6 or 2,2, depending on the design situation (permanent, characteristic or accidental, respectively).

4.3 *Geotechnical Recommendation for Design of Maritime & Harbour Works (ROM)*

The ultimate unit tip resistance (q_p) is estimated as:

$$q_p = \frac{2}{3} q_{ult}\left(1 + 0.4\frac{L_R}{D}\right) \tag{11}$$

where q_h is the vertical ultimate pressure for a spread foundation as given in eq. 5 (calculated adopting as value B* the equivalent pile diameter and f_δ=1), L_R is the length of embedment in rock of the same of better quality than the rock in the tip (≤ 2.5D).

The shaft resistance in rock will be taken into account only in the areas with weathering degree equal or lower than III. It can be estimated as:

$$\tau_f = 0.1 q_{ult} < 2 \text{ MPa} \tag{12}$$

where q_{ult} is the ultimate bearing capacity given in eq. 5, calculated adopting as value B* the equivalent pile diameter and f_δ=1.

The allowable bearing capacity is obtained using a global safety factor of 2,5; 2,2 or 2,0, depending on the design situation (quasipermanent, characteristic or accidental or seismic, respectively). These values shall be applied for works whose failures would have low social and environmental impact.

5 SLOPES IN ROCK

Neither CTE nor GCC indicate anything relative to slopes in rocks. Only ROM give some indications, mainly the following ones.

- It is stated that for much fractured rocks, with small spacing between joints compared to the dimensions of works and joints in different orientations, the methods used in soils may be applied, considering an equivalent ground with a shear strength equal to that of the weakest joints. However, for sound rocks with just a few joints other more specific for rock mechanics methods shall be used.
- Different main types of instabilities are distinguished: planar and wedge failure and toppling. Some basic indications are given on how to proceed in the calculations in each of these cases. Some other types of instabilities may be produced: rock falls, bend of strata, complex slides in faulty areas or slides through weak layers.

The stability studies must follow some common principles indicated in the following paragraphs:

- The design situation must be characterized, mainly with the following data: geometry, actions and shear strength in joints. The geometry shall include the orientation, continuity and spacing of discontinuities, its water content and pressure in joints. Sensitivity calculi are recommended for the less known variables.
- Apart from the self-weight, other actions like overloading and weights of different parts of the structure must be considered.
- If the rock is to be stabilized with anchors, the active or passive forces transmitted may be represented by forces located in the head of the anchors (unless the failures surface studied is out of the anchor, in which case no force should be included). For active anchors this force should be the active force given to it, while for passive elements the force to consider will take into account the part of the element located out from the failure surface. For more accurate calculi or intermediate elements, the engineer must consider forces that may correspond to reasonable deformations.
- The strength of the rock matrix normally does not influence the global stability although it is a useful index to evaluate the shear strength of joints.
- To evaluate the global safety factor of the rock mass it is necessary to define the shear strength parameters on each joint family. The shear strength of joints must be determined either by laboratory or field tests or by using correlations. In these cases the criterion of the engineer is essential to choose the way to estimate that strength.
- When the failure surface studied intersects a structural element, its strength shall be considered as the one of a passive element.
- To estimate the global safety factor of a slope in rock all the cinematic possible failure mechanism should be considered. Values for the safety factors are given (varying between 1,4 and 1,1) depending on the combination of actions considered.

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6 DISCUSSION

Some of the similarities and differences observed may be remarked as follow:

1 Site investigation

The three standards indicate that the intensity of exploration must be defined depending on two main aspects: one related to the type of works or buildings to be constructed and the other related to the ground characteristics. The following remarks can be done:

– It is considered appropriate to define different categories of ground investigation intensity and to define such categories according to ground conditions and a category of works, defining this last taking into account the consequences of a work failure, as done in ROM.
– In order to classify the ground conditions, when it is a rock, some guidance should be given according to its discontinuities (spacing, aperture, weathering…) and its matrix quality.
– For each category of intensity and according the type of work, the site investigation needed – number and depth of boreholes, number and type of samples to get, tests to perform, etc.- must be suggested and the necessary changes in case of rock mass characterization should be indicated.

The whole process is the indicated in Figure 1.

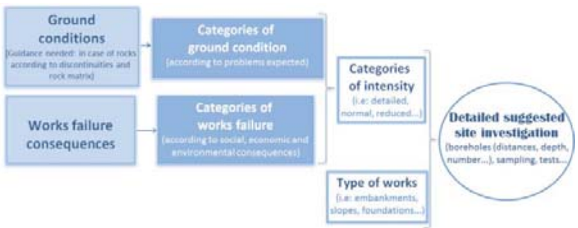

Figure 1. Process to define categories of intensity of site exploration.

The three standards give indications mainly based in ISRM recommendations (1981), for characterizing the strength of the rock matrix and its discontinuities. The authors consider it appropriate, as the indications by ISRM should be the reference.

2 Bearing capacity of spread foundations

There are some remarkable differences and a few similarities in this case:

– Each normative uses a different formula for estimating the bearing capacity of a spread foundation. This reflects the fact that there are many different formulae, collected in standards all around the world, all of them empirical. Maybe the most remarkable difference is that while GCC and ROM use formulae in which the bearing capacity depends on the square root of UCS, in the one used by CTE it depends directly on the UCS.
– There is also a great difference in the limits given for the bearing capacity: CTE does not give any limit value; GCC gives a limit value of $q_a \leq 5$ MPa unless properly justified, and ROM gives a limit value of 15 MPa for the ultimate allowable pressure, so depending on the safety factor used, the bearing capacity may vary between 5 and 7 MPa (6,5 MPa considering characteristic situation). The authors consider that giving limit values for the bearing capacity as it is done in usual practice must be reconsidered as there is a practical limitation due to the pillar concrete strength.
– All the formulae are valid for horizontal ground (less than 10% slope is indicated in GCC and ROM, no indication is given for this in CTE), and inclination of load lower than 10%. Correction factors are given for higher load inclinations in GCC and ROM. They are not included in CTE, probably due to the fact that it is not frequent in buildings to have higher inclinations.
– In GCC and ROM there is also a limit for the area of foundation of 100 m^2, being necessary to carry out a special study in case of bigger areas.
– The three standards indicate that rock should be considered as a soil when it is weak, according to the different criteria shown in Table 4. This should be unified in the future.

Table 4. Criteria to consider a rock mass as a soil

Parameter	CTE	GCC	ROM
Wd	>IV	>IV	>IV
RQD (%)	<25	<10	<10
UCS (MPa)	<2,5	< 1	---
S (cm)	---	---	<10 cm

Wd: weathering degree; S: joint spacing

– For the sake of comparison, Figure 2 shows the ranges of variation using the three standards for the following case: UCS: variable; s=300 mm; B* = 3 m and UCS<10 UTS.

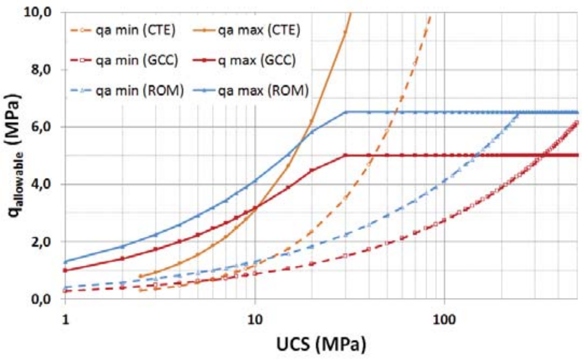

Figure 2. Ranges of variation of the bearing capacity according the three standards for a spread foundation (s=300mm, B*=3m)

It can be seen that there are relevant differences in the ranges of variation but there is a common space for the rocks of low and medium strength (UCS between 2,5 and 40 MPa). CTE gives higher values for medium and high strength rocks, mainly due to the lack of limit for the bearing capacity. Besides, it has to be noted that it states more strict conditions for the rock not to be considered as a soil, as indicated before. In addition, the lower values indicated by GCC, with respect ROM, get closer for bigger foundation widths.

– Some remarkable differences can be seen, also compared with EC-7 recommendations (Annex G, EC-7, 2004), with a simple example of a spread foundation in rock with following data: UCS= 5, 10 or 20 MPa corresponding respectively to a very marly limestone or poorly cemented sandstone (group 3 in EC-7), a metamorphic rock (group 2 in EC-7) and a sandstone (group 2 in EC-7); s = 300 mm, RQD = 100% and B* = 3 m. The results are shown in Table 5 and in Figure 3.

Table 5. Allowable bearing capacity calculated in the example according with the different standards

UCS	q_a (MPa)			
(MPa)	CTE	GCC	ROM	EC-7
5	**0.6-1.6**	**0.6-1.2**	**0.9-1.8**	**~ 1**
10	**1.2-3.1**	**0.9-1.7**	**1.3-2.6**	**~ 4**
20	**2.3-6.2**	**1.2-2.4**	**1.8-3.7**	**~ 6**

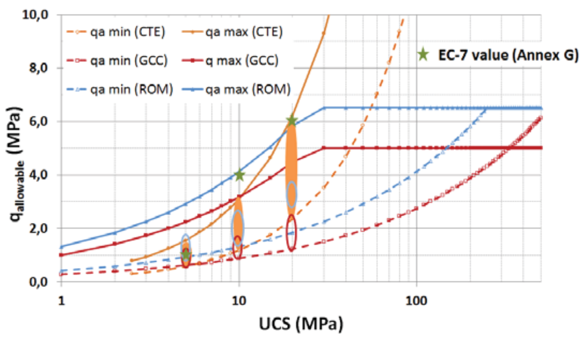

Figure 3. Bearing capacity calculated in the example according with the different standards and EC-7

It can be seen that differences may be high, most of all between the calculations done according to CTE and the two others standards for the highest strength. It can be also seen that the values given by EC-7 are the highest, except for the lowest strength for which all the values are quite similar.

3 Bearing capacity of piled foundations

Only GCC and ROM give formulae for estimating the bearing capacity of foundation on piles in rock. Both of them distinguish between tip and shaft resistance.

For calculating the tip resistance both standards use a similar formula based on the bearing capacity of shallow foundation (although this one is different in each case, as it has been shown before).

However, for calculating the shaft resistance they give slight different formulae: in GCC the unit shaft resistance is calculated as 1/10 of the bearing capacity of a shallow foundation, while in ROM it can vary, depending on the ratio between the embedded length and the diameter, between 1,5 and 0,75 the bearing capacity divided by 10. This, together with the fact that the bearing capacities calculated according one or another normative differ, may give remarkable differences in the dimensions of the designed foundations.

4 Slopes in rock. No comparison is done in this matter, as only ROM gives indications relative to this topic.

7 CONCLUSIONS

The approach to rock mechanics in Spanish standards is reviewed in this paper, considering different geotechnical areas: site investigation, shallow and deep foundations and slopes. It has been shown that in some cases these standards have different ways of approaching these geotechnical problems that may give out to remarkable differences in the results obtained.

This reflects the fact that there are many different empirical formulae and procedures, collected in standards all around the world, as there is not a unified or theoretically deduced method for each geotechnical situation. Evolution in EC-7, considering more relevant aspects of rock engineering, will help to unify methods and criteria used, but the variety of possible existing approaches, some of them showed in this paper, stresses the actual difficulties for that unification.

It is believed that the considerations given in this paper could be helpful somehow in the future evolution of EC-7. The standards analyzed in this paper are the results of a long period work, thoughts and agreements between many geotechnical and rock mechanics experts.

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Critical review of Eurocode-7 regarding rock mass characterization

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ABSTRACT: Rock mass characterization is required for the development of any engineering design since it largely defines the rock mass behavior at the scale of the operation. Rock masses are discontinuous media with the rock matrix crossed by discontinuities. These different types of joints determine weakness planes that often rule the potential induced failure mechanism. Rock blocks are formed and they can move along discontinuities depending on their geometrical and physical mechanical features. Consequently, the occurrence of discontinuities must be analyzed in a quantitative way by carrying out surveys along scan-lines or at observation windows. The traditional operations required to fulfil this aim are reported in the ISRM Suggested Methods and new approaches are under development by the application of several new developed measuring tools, such as laser scanner or photogrammetry. However, the big effort that the rock mechanics community has spent in improving rock mass characterization techniques has not been considered in EC7.

1 INTRODUCTION

Ground may be composed of soil or rock; engineering terms not clearly distinguishable and fuzzy at their borders. Strength, may characterize ground as rock if above 1 MPa, otherwise as soil. Further, in soils the elementary particles are very small in comparison with the magnitude of the structure. This is not the case with the rock masses which consist of larger elementary volumes which vary in size in comparison with the engineering structure. The strength, shape, orientation and magnitude of these elementary volumes, dictate the response of the rock mass. Ground engineering comprises structures both in soil or rock, and their design in Europe should follow the pertinent Eurocode.

1.1 *EN 199*

Eurocode EN 199 is an obligatory official document in Europe for the design of civil structures. It contains norms referring to various materials of construction. Initially it was pertinent to above ground structures such as Bridges and Buildings, and later it evolved to include other types of structures, such as those concerning ground engineering. Basic concept of the code is to separate demand with capacity of the structure. Demand is determined from the actions applied on the structures, whereas capacity is determined by the properties of the materials comprising the structure; both are factored appropriately. The latter should be larger than the former for the design to be acceptable.

EUROCODE 2 states that failure mechanisms dictate the design approach. The knowledge about possible relevant failure mechanisms is a requirement for the design of structures. Only then it is possible to design models and safety criteria. Moreover, parameters correspond to pertinent failure mechanisms.

Industrial or manmade structural materials are continuous by nature; any cracking is identified as failure. Their analysis follows the rules of continuum mechanics, and thus the demand is evaluated with standard structural design procedures. The evaluation of their capacity is straight forward according to the codes. For soils such a design procedure is also followed, as they may be considered macroscopically as continuous, and their strength and deformability properties may be determined following standard testing procedures.

However, rocks are not continuous materials. Joints and other fractures of geological origin tend to be ubiquitous features in a body of rock. Thus the strength and deformability features are influenced by both the properties of the intact materials and those of the ensemble of discontinuities found in the rock mass (Brady & Brown, 2003), i.e. the rock mass structure (Figure 1). The capacity of the rock mass depends on the size of its elementary volume and on its relation to that of the structure. These effects may be appreciated by considering various scales of load-

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ing to which a rock mass is subjected in construction practice (Fig. 2). The larger the elementary volume and stronger the intact rock, the more relevant are the structural features such as bedding, faults and other discontinuities, which control the mechanical behavior of these natural materials.

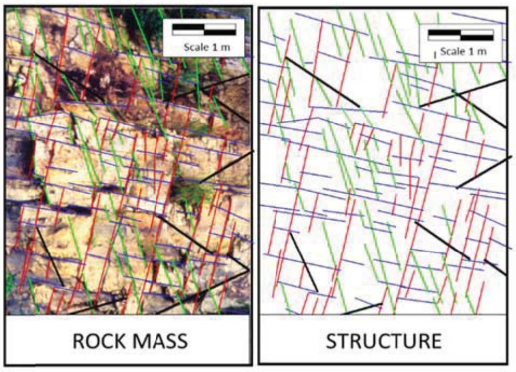

Figure 1. Concepts of rock mass, natural occurrence of rock, and rock mass structure or overall geometrical configuration of the discontinuities in the rock mass.

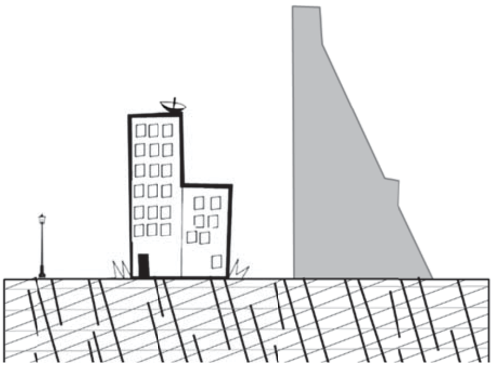

Figure 2.Various scales of loading in construction on rock masses.

1.2 *EN-1997*

The Eurocode for Geotechnical Design, EN-1997-1:2004 (EC7), was fully implemented within the European Union in 2010 (CEN, 2004). Initially it was intended to deal with the geotechnical aspects of common civil engineering works. However, it contains principles that may be used as a basis for the design of rock engineering projects, such as slopes, dams and underground constructions. It introduced in geotechnical engineering a limit state design approach. Structural adequacy of the ground is examined, as for the other materials, in terms of capacity and demand. The capacity of the ground depends on its geotechnical parameters. According to the code, the selection of characteristic values for them shall be based on results and derived values from laboratory and field tests, complemented by well-established experience. The characteristic value of a geotechnical parameter shall be selected as a cautious estimate of the value affecting the occurrence of the limit state. Characteristic values can be lower values, which are less than the most probable values, or upper values, which are greater. For each calculation, the most unfavorable combination of lower and upper values of independent parameters shall be used. If statistical methods are used, the characteristic value should be derived such that the calculated probability of a worse value governing the occurrence of the limit state under consideration is not greater than 5%.

Nevertheless, rock masses are typically jointed (Fig. 3). Their elementary volumes are larger than those of the particle size of the soil, and usually much larger than the standard specimens tested in the laboratory. Thus, representative volumes may not be tested there, disallowing the direct determination of engineering parameters in the laboratory. Engineering parameters are thus determined indirectly from individual parameters measured in the laboratory and in situ.

Moreover, EC7 only refers to discontinuity characterization in a very vague way. Accordingly, when it refers to ground properties, it states ‘Properties of soil and rock masses…. obtained from test results, either directly or through correlation, theory or empiricism and from other relevant data’. This is too vague, as for the case of rock masses, properties should not be only based on laboratory and in situ testing, but also on a thorough characterization of the rock mass in situ collecting joint data, fracturing and weathering degrees, geological features if observable, etc, in such a way that data can be derived and an estimate of rock mass quality or classification according to available techniques can be performed, if necessary.

In section 3 of EC7, related to geotechnical data, some guidelines related to characterization of soil and rock type indicates that ‘rock should be classified in terms of...jointing’ and then it reads ‘Jointing should be characterized in terms of joint type, width, spacing and fill quality’. Further in this section it is stated ‘Consideration shall be given to the following characteristics of the joints: spacing; orientation; aperture; persistence; tightness; roughness, including the effects of previous movements on the joints; filing’, which is not coherent with the previous statement, using different terms with the same meaning, such as width and aperture.

Figure 3.Typically jointed rock masses.

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2 ROCK MASS CHARACTERIZATION

The Representative Elementary Volume (REV) of the rock mass is usually much larger than in soil where the size of a test specimen in laboratory can be representative of the site. To acquire information of the rock mass at the meso-scale, the rock mass characterization must be based on experimental testing and in situ survey. Since the geometry of the discontinuities determines the block shape, size and kinematic, the in situ survey has always been a key point in rock engineering (ISRM, 1978).

The structure of the rock mass should be quantified starting from a considerable number of joint data representative of the site and including type of joint, orientation, continuity, spacing, roughness, strength, aperture if opened, fill width and nature if filled, weathering level and water (Ulusay & Hudson, 2007). The quality and the quantity of the data to be measured should be related to the rock mass quality to decrease the design uncertainties coherently with the limit state approach (Harrison and Bedi, 2013). The need to improve the rock mass characterization by improving the discontinuity survey, is witnessed by the large effort dedicated to this aim in the rock mechanics research field in the last 20 years (Lato, 2011, Ferrero et al., 2009) but is not reflected in EC7.

The "level of information" of the rock mass has also to be related to the geotechnical model to be adopted: according to the degree of fracturing we could schematize the rock mass as a continuous or as a discontinuous medium. When the rock mass must be represented by a "discontinuous model" the strength and deformability of both the intact rock, and the discontinuities have to be determined.

The failure criteria adopted and the consequent parameters required are closely related to the model used in the design stage. The parameters that are used to represent the behavior of the rock are different depending on whether this should be considered as a discontinuous medium or can be idealized as an equivalent continuous one.

If the rock mass has to be modeled as a discontinuous medium, discontinuity behavior and geometry and rock matrix behavior have to be determined and considered independently, while for the equivalent continuous an ideal "homogenized" material should be characterized.

The assessment of geotechnical parameters is one of the most important aspects for the design of works in rock masses. The input data concern the geological-geotechnical characterization of the rock that, in general, includes the estimate of (Bieniawski, 1978): deformability and strength characteristics of short and long term, permeability characteristics and the natural stress field.

The description of the experimental tests needed to determine the relevant parameters in the two different approaches is out of the scope of this paper. However, since the discontinuities are "weak planes" they are usually ruling the possible failure and the effort must concentrate their characterization both by the geometrical and the mechanical point of view. In this case, the instability is due to movement along planes and consequently the in situ measurements of the discontinuity and the definition of the joint set orientation is of crucial importance. Laboratory tests on rock matrix and discontinuity have to be carried on to determine strength and deformability features.

When systems have discontinuity spacing that isolate blocks having negligible size compared to the scale of the problem in question, it can be considered as a means of ‘equivalent continuum’. In this case it is necessary to distribute the effect of discontinuity over the entire volume of rock. The ‘homogenized’ medium is then characterized by global values of deformability and resistance.

Resistance depends on both the strength of the intact rock and of the discontinuities present in it. Hoek and Brown (1997) suggested that the rock masses of excellent quality (eg. GSI near 75) are characterized by an elastic-brittle behavior): with a significant effect of dilatancy; rocks in medium quality (GSI near 50) are characterized in the post-peak softening behavior; rock masses of poor quality (GSI around 30) are instead characterized by a perfectly plastic-elastic (zero dilatancy angle).

The parameters of deformability should be representative of a global behavior of the rock mass, and for this reason, direct tests, such as triaxial tests and shear tests conducted on a large scale in the site, may be performed on samples of rock mass which should, however, have significant volume. These tests are not only economically burdensome, but also difficult to perform.

For these reasons, in engineering practice, we resort to empirical relationships, developed over the last decades by several authors, which correlate the characteristic parameters of the mechanical behavior with the quality indexes of the rock mass.

In the case of the equivalent continuum approach, the determination of the strength and deformability features is mostly done in an indirect way through the rock mass quality determined with rock mass classification methods. An example is the Hoek-Brown criterion, the most commonly utilized for the rock mass strength, where one of the required parameters is related to the value of the rock mass quality defined by one of the available rock mass classification systems.

3 EXAMPLE OF A WIND TURBINE FOUNDATION DESIGN

The role of discontinuities in the stability of a construction founded on a fissured rock mass has been

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sometimes overlooked. However, it could be extremely relevant in particular cases. An illustrative example of such a case is presented in what follows. The design of a foundation of a wind turbine in slightly dipping natural slope (19.5º) in a fissured rock mass is studied. A section of the rock mass with the wind turbine to be located in the area is shown in Figure 4.

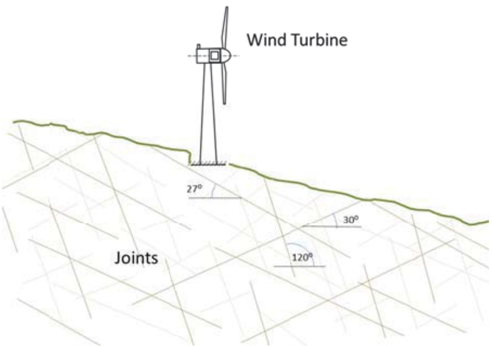

Figure 4. Cross section of the rock mass where the wind turbine is going to be installed.

The rock mass is formed by somewhat weathered granite showing four discontinuity joint sets, including the three sets observable in Fig. 4 and another sub-vertical set following the direction of the printed section. In winter, the slope can be fully saturated.

Laboratory tests on granite samples yielded an UCS value of 58 MPa and a Hoek-Brown m parameter value of 30. Density was 22.3 kN/m^3. Field characterization suggests a value of GSI around 45, and the value of D is considered equal to 1, since the area is in the slope surface and somewhat weathered. Starting from these data and for a 20 m high slope, the Hoek-Brown characterization approach (RocLab), yield cohesion 0.34 MPa and friction of 43º.

The weight of the wind turbine and its pole is 2247 kN and the wind force is estimated 787 kN. A slab foundation of concrete with dimensions 10 m x 10 m x 2 m is suggested, being able to cope with the reactions of weight and wind force.

A model on the stability of the turbine on the slope was created considering the rock mass as a homogeneous material. The FoS against circular sliding as derived from code SLIDE yields a value over 8 (Fig. 5). So even applying partial factors as suggested for soils, a sufficiently reliable stability will be derived.

However, if one considers the joints shown in Fig. 4 without boundary effects, two possible failure mechanisms are identified, consisting in the pushing of an upper block, which may produce the sliding or the toppling of the lower block (Fig. 6).

Assigning a value of the friction angle of 29º for joints, FoS 1.01 against sliding and 1.01 against overturning (Fig. 7) are derived by means of limit equilibrium approaches (Alejano et al., 2011) based on the transmission of forces between blocks, and the estimate of forces (to analyze sliding) or moments (to analyze overturning) in the lower block.

The main purpose of this example is to put forward the fact that a painstaking analysis of the role of discontinuities is in order to appropriately design foundations on fissured rock masses. Such an analysis is not recommended in this version of EC7.

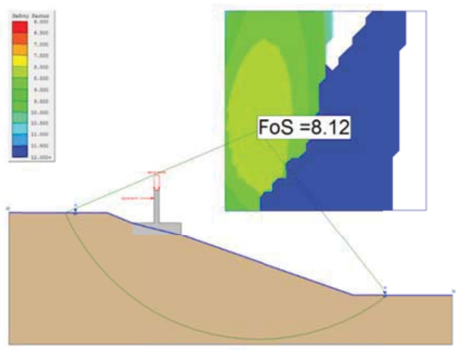

Figure 5. Stability analysis of the case study against circular failure by means of code SLIDE.

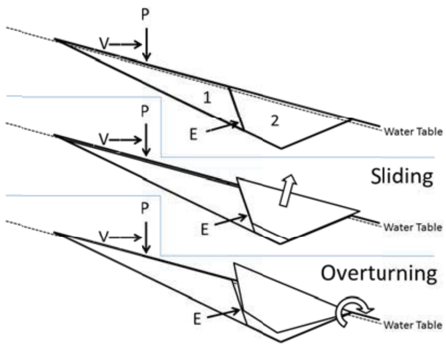

Figure 6. Two-block system of the model and external forces in the upper part. Illustration of the two identified possible failure mechanism in the lower part.

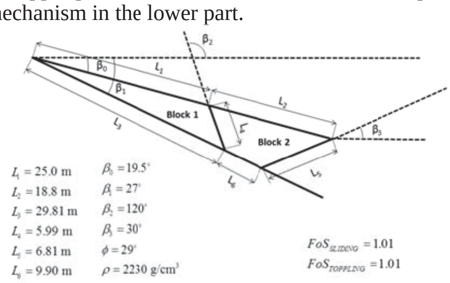

Figure 7. Diagram showing the dimensions and angles of the two block system: estimated values and derived FoS.

4 FAILURE OF THE AZNALCÓLLAR TAILINGS DAM

On the night of 24 April 1998 a breach developed in the embankment of the Aznalcóllar dam producing the release of part of the nearly 20 million m^3 of mining waste and causing one of the most relevant ecological catastrophes in Spain (Fig. 8).

After painstaking investigations it was confirmed that the rupture was due to a failure of the dam foundation at a depth of approximately 14 m below original ground level, following a bedding plane. This gave rise to an almost flat slide with a horizontal movement of 60 m.

The foundation consisted of carbonated, lightly expansive and over-consolidated silty claystone known locally as ‘Blue marls’. They normally show unconfined compressive strengths over 1 MPa and exhibit strain-softening behavior, so they can be considered as rocks (Galera et al, 2009).

As pointed out by Olalla & Cuellar (2001) the occurrence of bedding planes (not accounted for in the design calculations) and the pore pressure rise in the foundation by the weight of the tailings and that of the dam itself, had been the most significant elements finally producing the accident, whose failure mechanism is shown in Figure 9. This can be considered as a paradigmatic case in which the fact of not considering explicitly discontinuities in the design process may yield undesirable results.

Figure 8. Failure of the Aznalcóllar dam, near Seville (Spain) on 24th April 1998, where tailings were release to the Amargo river basin. Taken from http://www.pensandoelterritorio.com. Entered on 24th October 2013.

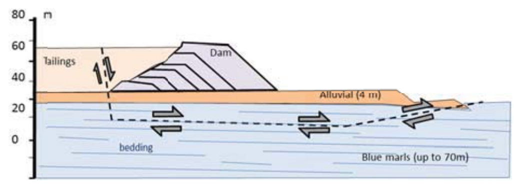

Figure 9. Cross-cut sketch of the failure mode of the Aznalcóllar dam, where the sliding line partially followed unremarked pre-existing bedding planes.

5 SLOPE STABILITY

In the case of rock slope stability, failure mechanisms are most often associated to the number, type, and features of the discontinuities found in the rock mass. Some failure mechanisms and namely plane and wedge failure and toppling instability phenomena can only be analyzed, if the structure of the rock mass is reasonably well known. Figure 10 illustrates a slope where a telecommunication tower has been built. The rock mass structure is such that there is a very persistent scarcely spaced bedding dipping against the slope. In this type of slopes toppling failures are quite common; so they should be appropriately studied starting from the design stage.

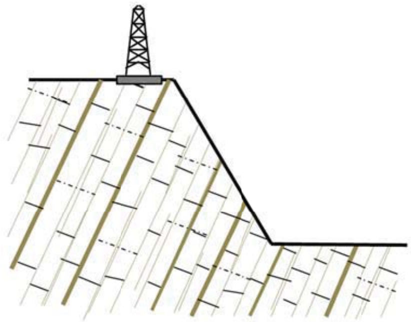

Figure 10. Slope prone to toppling.

Figure 11. A slope in a quarry in the Alps showing a typical block toppling failure mechanisms and equal area projection of the discontinuity poles of the rock mass (right hand side).

Figure 11 finally illustrates one example where toppling mechanisms have produced movement and rotation of blocks on the slope in the North West Italian Alps (Deangeli & Ferrero, 1999). As it is illustrated the rock mass structure was characterized by a joint set that determined the formation of thin slabs. In this case the failure mechanisms can be analysed by means of a discontinuous model that could reproduce the occurred instability. Tens of cases of this type can be found in the literature. The common issue is that a good knowledge of the rock mass structure is absolutely necessary to understand and control these phenomena.

6 WEATHERING

Decomposed granite and other rocks are often found in construction engineering. Rock weathering rarely produces a homogeneous altered rock mass with all material decomposed to a same degree -or even a simple weathering profile with depth (Fig. 12).

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An accurate description of the weathering degree distribution of a rock mass could be relevant to provide a good design for different construction purposes. In this case also EC7 does not contemplate adequately the characterization of weathering, but only in a vague indicative way.

Figure 12. Variable nature with all possible weathering degrees in a granite cut on a road.

7 DISCUSSION

One of the most relevant problems of EC-7 when dealing with design in rock has to do with correctly modelling the mechanism of failure, after having properly established the actual engineering geological structure. What type of guidelines do we really ask from EC7 to provide? A flow chart in EC-7 may:

- Guide us to collect appropriate engineering geological data (field characterization) and perform appropriate tests, depending on the type of ground conditions.
- Indicate the range of instability mechanisms to be analyzed, and the basic approach to contemplate (continuous vs. discontinuous).
- Provide guidance on the selection of appropriate constitutive laws, failure criteria and methods, to evaluate the appropriate design parameters (of joints, rocks and rock-masses, according to the approach).
- Suggest limiting factors of safety or acceptable displacements, and the way to evaluate them, together with guidance in monitoring and prescriptive measures,

In the examples discussed, if such a flowchart was followed, it would have been possible to identify the failure mechanisms and eventually to provide appropriate designs. The present version of EC-7 does not clearly suggest detailed joint data characterization. Further, it does not regard failure mechanism identification.

8 CONCLUSIONS

The relevant effort that the rock mechanics community has carried out during the recent decades in improving rock mass characterization techniques has not been considered in ~~the~~ the EC7 that so far only refers to characterization of continuous media (where the particle scale is negligible), such as soils.

The EC7 approach can fit the cases that deal with soils where often material samples tested at laboratory scale can supply all required parameters but it does not apply to rock masses where the meso-scale structures are extremely relevant. EC7 does not give details on rock mass characterization and on how the rock discontinuities should be considered in order to quantify the degree of fracturing and anisotropy.

Moreover, the information needed to analyze different failure mechanics and parameters that specifically apply to rock discontinuity failure criteria, which are not indicated in EC7, is also reported.

Some examples on a shallow foundation for a wind turbine in a slightly dipping slope, a well-known disaster-causing failure of a tailings dam in southern Spain and some ideas concerning various failures mechanisms of slope stability and overlooking of weathering degrees are provided in order to illustrate the authors' point of view, stating the need to better reflect rock mass characterization in EC7.

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Critical review of Eurocode-7 regarding monitoring rock masses by field instrumentation: devices and data analysis

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ABSTRACT: Monitoring in field performances of rock masses is required and widespread in engineering practice for both handmade structures (tunnels, cuttings, etc.) and natural sites (i.e. landslides). The presence of discontinuities and their role in displacement localization and hydrogeological anisotropy of rock mass are the key elements in order to define an appropriate monitoring program. The application of new nanotechnologies in the geotechnical and rock mechanic fields has increased the amount of acquired information both in space and frequency. Along with the awaited introduction in the EC7 of specific considerations regarding geo-structural characterization and failure criteria definition, it is desirable to propose specifications regarding field instrumentation monitoring devices and data analysis. A review of the rock mass key physical properties that should/can be monitored in field, according to specific projects or aims, is presented, along with the state of the art of suitable field monitoring equipment.

1. INTRODUCTION

Monitoring can be regarded as the regular observation and recording of events taking place in a certain structure. Displacements, deformations, water pressure, natural and induced stresses are among the most commonly recorded entities during monitoring.

In particular, deformation monitoring is the systematic measurement and tracking of alterations in the shape (position and altitude) of an object, resulting from the application of external forces.

Deformation monitoring and gathering of measured values are major elements for further computation of soil and rock stability, deformation analysis, prediction and alarming (Moore, 1992).

Since each monitoring project has specific requirements, the applied measuring device depends on the application features, the chosen measurement method and the required regularity and accuracy.

Therefore, monitoring of slopes or landslide areas can only be defined, designed and realized using an interdisciplinary approach (Wunderlich, 2006). A close cooperation among experts from geology, geophysics, hydrology, geotechnics together with experts from any measurement discipline such as geodesy, remote sensing and other academic fields, is an indispensable requirement.

A series of disturbances can affect the pre-existing natural state of stress of a rock or soil slope. The extent of the disturbances is usually unknown and is related to the mechanical, structural and hydrological characteristics of the slope.

Monitoring of slopes as a crucial tool for prevention and prediction of failures must be encouraged and its potential should be recognized also by non-technical or non-scientific stakeholders.

Monitoring is an important element of hazard management, which includes hazard identification, assessment and information. For active slopes, monitoring is often the only chance for a reliable prediction. Acquired data are the basis for any geomechanical interpretation. It must be stressed that financial means invested for deformation monitoring and analysis can remarkably reduce the cost for adequate retention works.

In the underground works field, monitoring is an established tool used to upgrade the design hypothesis and to control the assumptions made during the preliminary design stage; this criterion can significantly reduce geotechnical model uncertainties.

The design of a monitoring system should be done in a rational way, taking into account the key parameters that are ruling the rock mass behavior, their range, rate of variation and localization. On this basis instruments can be chosen and properly positioned.

The development of innovative nanotechnologies and their application in geotechnical and rock me-

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chanics fields has largely increased the amount of information and measurement that can be stored either in space and frequency, without any significant rising in costs.

However, digital technologies and sensors have intrinsic drawbacks regarding their sensitivity, precision and repeatability of measures, which need to be recognized and properly treated while processing the data.

Furthermore, the availability of remote sensing techniques gives an extraordinary opportunity to extend monitoring effectiveness over large areas when properly crossed with reliable local measurements. At last, the need of reliable real time monitoring, either for remote observation of the phenomena or as a risk reduction measure, requires immediate analysis and transmission of data. Appropriate statistical techniques need to be applied in order to define and update reliable threshold values for each monitored physical entity.

All these important issues, which are generally introduced in the current engineering practice, have not been dealt with in EC7, not even for the soil mechanics aspect. Along with the awaited introduction in EC7 of specific considerations regarding geo-structural characterization and failure criteria definition, it is desirable to propose specifications regarding field instrumentation, monitoring devices and data analysis.

2 MONITORING IN EC7

Eurocode 7 (EN 1997 - Geotechnical design) is an integral part of a series of European Standards, the Structural Eurocodes (EN 1990 to EN 1999), written and issued since 1980s, concerning the design of building and civil engineering works. Specifically, EC7 regards geotechnical design and therefore structures interacting with soil and rock. Those are of two main types: structures built using soil or/and rock material (e.g. embankments) or structures built on or into them (e.g. foundations, retaining structures, excavations).

The use of EN 1997 has to be intended in conjunction with the other Structural Eurocodes and, in particular, it has to be applied accordingly to what reported in EN 1991 (Eurocode 1: Action on structures) and EN 1998 (Eurocode 8: Design of structures for earthquake resistance).

Principles and requirements for safety and serviceability of soils and rocks must be respected, as for all the other construction materials considered in Eurocodes. For each geotechnical design situation and for each limit state considered (ultimate or serviceability), structural adequacy of ground condition has to be examined: the design value of the effect of actions (demand) cannot exceed the design resistance (capacity) or the design value of the effect of an action (serviceability limit).

Such effect of the actions becomes apparent through the variation of one or more physical entities that the monitoring system should record. In addition, the system could be designed to monitor some physical entities related to the current demand of the structure (e.g. forces, pressures, etc.).

The design procedures have to be conducted considering both short-term and long-term conditions concerning soil/rock, structures and overall stability conditions of the soil/rock-structure combination. The latter aspect is the most relevant when design regards rock engineering projects concerning slopes, tunnels, dams, etc.

Depending on the geotechnical complexity and the related risk level, several design tools and approaches can be used to verify the limit states: use of calculation, adoption of prescriptive measures, results from experimental models and load tests and observational method.

Among all the procedures specified to acquire the data needed for the design and the control of geotechnical works, EC7 (Part 1: General rules – Section 4 and Annex J) indicates also the monitoring systems. They can be used in pre-design phases, during construction or in post-construction phases.

In pre-design phase, monitoring systems should be used in order to acquire all the information related to the time-dependent actions that can, in any way, constitute an information basis. In such a case, monitoring can be related to the recording of the existing ground water level and its seasonal excursion, as well as the stability conditions and deformation of a potentially unstable slope.

In case of very complex geotechnical structures, uncertainties and risk associated with them increase due to the complex behaviour of the ground, the ground-support interaction, the uncertainties of the ground mechanical properties and the inevitable simplifications of mathematical models used for design. In these situations, the EC7 calls for the application of “the observational method”: the design assumptions, the geotechnical behavior, the assumed ground condition, the computational procedure and the construction methodology have to be continuously reviewed during the work construction phases.

The application of the observational method requires the identification of “significant” physical parameters during the design phase. These parameters should be representative of the post-construction behavior of the structure and their expected values and variation range should be computed. The variation range of such parameters is relative to the accuracy of the geotechnical characterization and to the simplification made during the design phase.

For this application, monitoring systems are of paramount importance for the measurement and control of the parameters chosen during the construction

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phase. In this case, monitoring systems have considerable importance in the control of ruling variables, identified in the design phase.

Depending upon the type of designed structure and to the ground interacting with it, the type of measures and their variation range to be monitored must be identified during the design phase.

The monitoring system becomes, therefore, part of the project design. It has to define not only the acceptable limits of behavior of the structures, but also a specific design containing monitoring equipment characteristics, their installation phases, frequency of readings and monitoring duration , as well as, plan of contingency action to be adopted if behavior limits are exceeded.

The monitoring system must therefore be chosen, designed and planned in order to monitor the parameters during each construction phase. If some of the recorded values exceeds the defined limits, the design assumptions must be reviewed and an alternative construction solution should be proposed and applied.

The EC7 requires that the performance of the structure persists after construction, and that the structure is adequately maintained (long-term performance). In this case, monitoring systems can be used as well, to control the operational life of the structures.

EC7 (Part 1 - Section 4 and Annex J) reports indications of some measurements that can be carried out either during construction phases or during operational life of the structure:

- deformation of the ground affected by the structure and effects on nearby buildings and utilities; values of actions;
- values of contact pressure between ground and structure; pore-water pressure; forces and displacements (vertical or horizontal movements and rotations or distortions) in structural elements;
- settlement, piezometric levels, deflection or displacement of retaining walls, temperature, vibration, etc.

In the EC7 there are no distinction between soil and rock regarding monitoring issues, but the general term used is ground. It should be emphasized that, depending upon the ground type and geotechnical structure, the key physical entities describing the acceptability limits (either ultimate and serviceability) can be different either in type, magnitude or rate of change.

3 ROCK AND SOIL MONITORING

When dealing with the design of a meaningful monitoring system, a discrimination between rocks and soil media is necessary. The hypothesis used for the estimation of the mechanical behavior of a geological system (such as a rock or soil mass) are influenced by their characterization and failure criteria definition. Furthermore, each system has some physical entities that are more significant than others for the definition of its general behavior.

The choice of a particular monitoring instrumentation should be mainly focused on the direct measurement of those “significant” quantities; other involved physical quantities, of minor direct significance for the description of the mechanical behavior of the system, may be monitored as supplementary indications and should not be confused with the others. In order to define the relative significance of each physical parameter, a preliminary sensitivity analysis could be carried out for each specific case.

In general terms, there are some peculiarities to be addressed when dealing with soil or rocks; in soil mechanics the general behavior of the mass can be assumed as an equivalent continuum and therefore is often interesting to have a diffused monitoring of some physical entities rather than concentrating measures at a specific location.

Rock mass behavior is, instead, generally governed by the variation of physical entities along discontinuities. Rock blocks generated by the intersection of those discontinuities can be considered as rigid elements in most of the cases and therefore the measurement of one or few localized physical entity (i.e. discontinuity aperture) can be adequate to describe the behavior of the entire block.

In some cases, however, due to the high degree of fracturing, rock mass can be assumed as an equivalent continuum, and can therefore be monitored with the same instruments used for the soil masses.

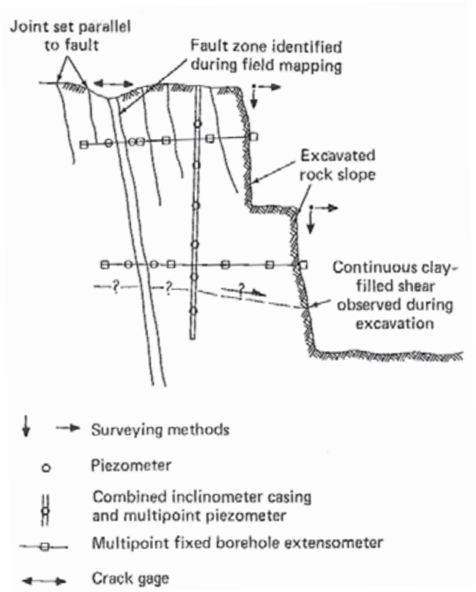

Figure 1. Monitoring scheme for soil slope (Dunnicliff, 1988).

In Figure 1 and 2, two monitoring layout, for a rock and for a soil slope respectively are reported (Dunnicliff, 1988). It can be observed as different instruments are chosen: in the case of rock slope,

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since discontinuities are ruling the displacements, crack gauges are placed across fractures on the top of the slope to measure the crack apertures due to a tension field.

Rock bolts extensimeters are also forecast to measure displacements in depth: in this case, special care should be taken to locate the displacements position in order to be able to identify the local displacement on each joint. On the soil slope, instead, inclinometers are generally used. In this case, in fact, the sliding surface location is usually unknown and needs to be identified.

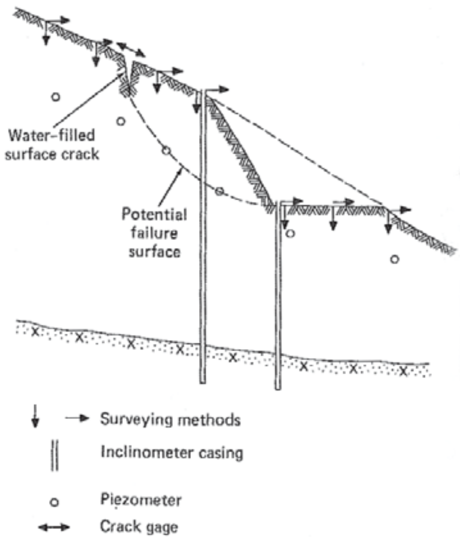

Figure 2. Monitoring scheme for rock slope (Dunnicliff, 1988).

3.1 *Monitoring technique*

A detailed description of the numerous monitoring techniques is out of the scope of this paper.

There is a main difference between the monitoring of structures interacting with the rock mass or soil and the monitoring of the natural element itself (rock mass or soil). In the first case, the monitoring devices can be those already defined in EC7 for structural elements, while in the second case EC7 should report the main differences between available monitoring devices and suggest the most reliable ones in relation to the nature of the monitored media (rock or soil).

As an example, when studying potential instabilities, one of the main objectives of monitoring is the identification of the instability phenomena size, shape and velocity (Cruden & Varnes, 1996; Hutchinson, 1988). These physical entities are directly connected with landslide hazard and, in turn, with the definition of the risk level related to the phenomena occurrence.

Consequently, displacements are a key parameter in monitoring slopes and instrumentation available for monitoring of displacements in depth can be divided in a few groups:

- instruments that measure the localized displacement and its orientation at discrete instants (i.e. standard inclinometers, bolt extensometers);
- instruments that measure the localized displacement and its orientation almost continuously in time (i.e. in place inclinometers);
- instruments that measure the occurring of a displacement without its orientation at discrete instants (i.e. TDR cables, fiber optics).

The differences between these instruments are related to their technology, precision, automation (or potential automation) and distribution within the slope depth. In some cases the presence of several measuring points is of paramount importance (i.e. unknown location of the failure surface, presence of multiple surfaces) while, in others, it is satisfactory to have a few measuring devices located in a relatively small depth range (i.e. localized failure surface). Among others, a new automated device based on MEMS (Micro Electro Mechanical Systems) for displacement monitoring has been recently released (Segalini & Carini, 2013) on the market.. The system, which can be installed at any inclination and is remotely controlled, is custom made for the specific installation, in order to maximize precision and accuracy of the measurement. These kinds of instruments are particularly indicated for rock mass monitoring, since the sensor chain is so diffuse and sensible that can properly record localized displacements along joint even if their position is not known in advance.

In rock slopes, when the displacement is expected to occur as a rigid block motion, can be monitored using the aperture of the discontinuity at the surface by means of an extensometer. However, the rate of displacements in rock mass is often so high that it cannot be recorded by standard instruments. Acoustic emission (AE) is an alternative and promising non-destructive technique for monitoring slopes in rock. AE is a natural phenomenon that occurs when a solid is subjected to stress. This stress, from an external source, causes a sudden release of sound waves resulting in micro-seismic activity, which can be detected by transducers, namely geophones.

Within a slope, stress induced by destabilizing forces causes a re-arrangement of particles along developing shear surfaces. This inter-particle friction results in a release of AE, and is an indication of straining within a rock body. In order to illustrate the meaning of these considerations, some specific examples are reported below.

EXAMPLES OF ROCK MASS MONITORING

3.2 *Madonna del Sasso rock slope*

The cliff of Madonna del Sasso is located along the western shore of Orta Lake and takes its name from the eighteenth-century sanctuary located at about 650 m a.s.l. In this area a granitic rock mass, called Granito di Alzo, outcrops.

The rock slope of Madonna del Sasso is affected by a rock instability phenomenon, highlighted by neat and long lasting episodes of slow deformation recorded by standard measurement devices such as inclinometers, topographic measurements and fissurometers. A detailed description of this case is given in Colombero et al. 2014 (Fig. 3).

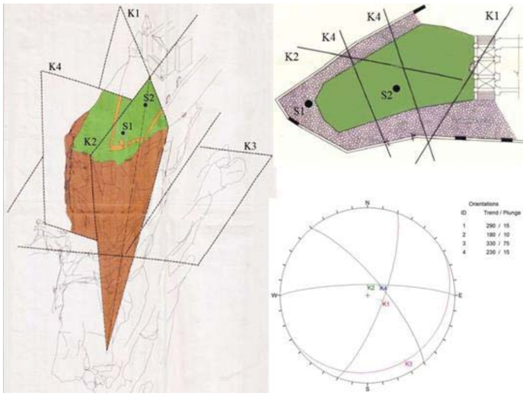

Figure 3. Slope schematic structure: a) assonometric and b) plan views together with c) surveyed joint sets (Colombero et al., 2014).

Recording and monitoring of acoustic emission were recently chosen as a strategy for forecasting dynamic ruptures. To do this the installation of a series of devices based on acoustic emission/microseismic approaches was planned; in this site standard monitoring systems have been installed several years ago and, consequently, a set of data is already available.

The devices layout (Fig. 4) aims to identify the characteristic signs of impending failure, by deploying an array of instruments designed to monitor subtle changes of the mechanical properties of the medium and installed as close as possible to the source region. A "site specific" micro-seismic monitoring system has been installed, made of 4 triaxial piezoelectric accelerometers operating at frequencies up to 23 KHz with a conventional monitoring for seismic detection (4.5 Hz seismometers) and ground deformation, provided by the University of Turin and SEIS-UK. The high-frequency equipment will allow researchers to develop a network capable of recording events with Mw < 0.5 and frequencies between 4.5 Hz and 20 kHz.

First results are already very encouraging especially in terms of sensitivity of the system; this suggests that this technique could be used during the pre-design and construction phases to contribute to the rock mass characterization, and also during post-construction phase for real time monitoring and as an element of a possible early warning system.

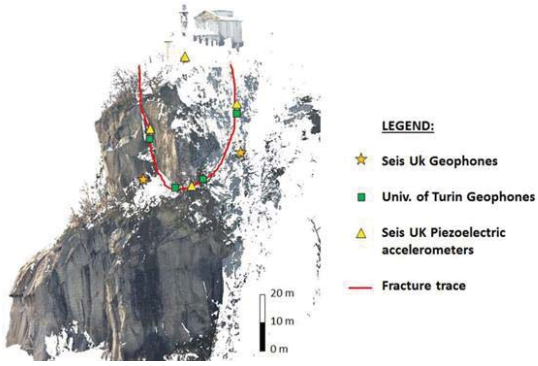

Figure 4. Instrument location in relation to the fracture.

3.3 *The Roccamurata landslide*

Roccamurata village is located on the right bank of the Taro river valley, Parma Province, Italy. This small village is threaten by an active rotational slide, which involves monogenic serpentinitic breccias of the Casanova complex and has its toe on the paved road inside the village; several buildings and the road that borders the village are severely damaged by the landslide activity (Segalini at al., 2013).

Manual inclinometer readings, carried out in the past at irregular time intervals had shown a total displacement velocity of about 8 cm/year. No indication were obtained regarding the temporal distribution of displacement during the observed period, making this monitoring outcome not useful for remedial design. An automated inclinometer device (Segalini & Carini, 2013) has been installed last year and recorded data have provided a detailed description of location and temporal distribution of displacement (Fig. 5) and average velocities (Fig. 6). A first tentative definition of an early warning threshold was proposed. The upper limit of the threshold has been exceeded during the month of August, when evidence of cracks appeared on the nearby buildings. This application demonstrates the advantages of an automated system for both the objective definition of a displacement threshold and the real time control of it. It should be advisable to review the EC7 introducing detailed indication for the application of monitoring automation in slope remediation design.

DISCUSSION

EC7 calls for the use of monitoring during pre-design, construction and post-construction phases in order to verify structural suitability both short and long term, respect to SLU and SLS. Design and planning of a proper monitoring system should answer to questions such as "which physical quantities have to be measured, what are their expected extents and their evolution velocities?", "which instruments can be used to measure them, what are the required

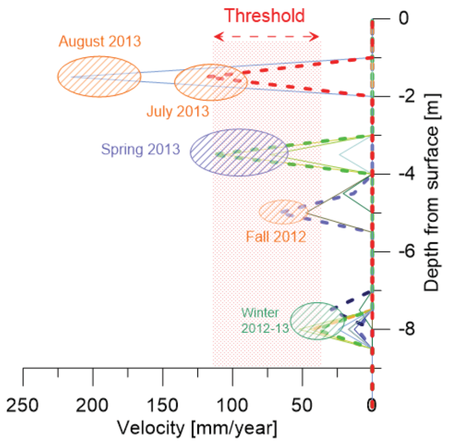

Figure 5. Total displacement recorded with the automatic system. Each line indicates the cumulated displacement in one month. Thicker lines are indicating the seasonal changes.

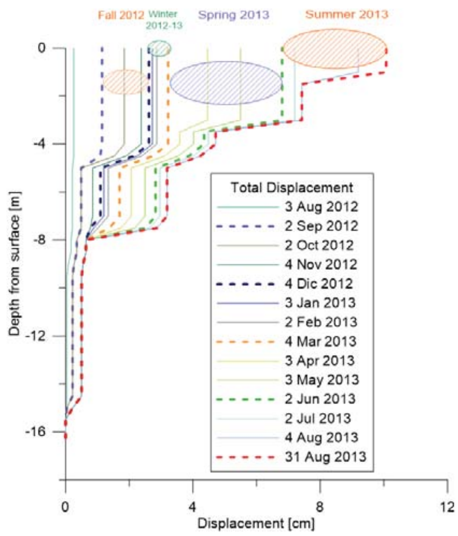

Figure 6. Velocities determined for the Roccamurata landslide at the location of displacements. During last August an anomalous velocity was recorded; during the same period many cracks were observed in the buildings.

characteristics (full scale, precision, accuracy, sensitivity, response time, etc.) for an effective and reliable measurement?", "How many instruments and in which positions?", "How long?", "How should data be recorded and treated?", "Which will be the instruments working conditions (deformations/tensions, temperature, vibrations, etc.)?".

Evidently, the answers are a function of a series of factors, such as work characteristics, type of material (rock/soil), strength-deformation mechanism, chosen geotechnical model (continuous/discontinuous) and the proper limit state to consider. Even if one cannot expect that a norm could completely answer all these questions, some indications presently not included in EC7 are desirable:

- representative quantities that can be monitored, for each geotechnical work and for each limit state, diversifying rock and soil cases;
- acceptability limits of the previously mentioned representative quantities;
- with relation to quantities to be measured (shallow and/or deep displacements, deformations, stress state, presence of water, etc.), devices and/or monitoring systems to be used in rock/soil, supplying information about their applicability field, range of measurement, precision, advantages and limits.

In any case the monitoring system should be set up in order to obtain redundant data from different devices, with the aim of eliminating bias and checking results congruence.

CONCLUSIONS

The importance of monitoring is a well-known concept in geotechnical design. Technological development both in sensors and data transmission has opened new wide application fields that need to be taken into consideration in the design practice and correctly ruled by the codes.

In this paper just a few examples are given, even if, the research activity in the field is constantly producing new proposals extremely important to improve design in rock mechanics engineering.

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Critical review of EC7 concerning prescriptive measures for rock mechanics design

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ABSTRACT: Eurocode 7 (EC7) implies that each geotechnical design situation shall be verified; that no relevant limit state conditions are exceeded. For this, one or more of four suggested design methods given in EC7 can be applied. One of these methods is prescriptive measures, which requires that documented experience and normal, well-known practice, provides satisfactory stability. EC7 allows using such design method when no calculation models are available or necessary.

Prescriptive measures involve conventional and general rules or practice in the design and may be used where comparable experience makes design by calculations unnecessary. Information of local experience is seen as particularly relevant.

As most classifications systems are systematic documentation of earlier experience of rock constructions, they qualify as prescriptive measures. The paper shows which parts of e.g. the Q-system that can be suitable for this.

The Geotechnical Category (GC) in EC7 is an important part of the rock design. The paper presents ground conditions where prescriptive measures may be used in rock engineering, design and construction.

1 INTRODUCTION

The Eurocodes were initiated for bridge construction, and have then been applied in more and more applications, such as in soil mechanics and finally in rock engineering. They serve as reference documents for the following purposes:
" - as a basis for specifying contracts for construction works and relating engineering services;
- as a framework for drawing up harmonised specifications for construction products."
Further, *"the Eurocode standards provide common structural design rules for everyday use for the design of whole structures and component products of both traditional and an innovative nature."*

Eurocode 7 (EC7) *"is intended to be applied to geotechnical aspects of the design of buildings and civil engineering works. It is subdivided into various separate parts" "EN 1997 is concerned with the requirements for strength, stability, serviceability and durability of structures. Other requirements, e.g. concerning thermal or sound insulation, are not considered."*

Though the EC7 covers both soil and rock engineering, there are significant differences between constructions in the types of materials.

Ideally, a rock mass is composed of a system of rock blocks separated by joints (discontinuities) forming a material in which all elements behave in mutual dependence as a unit.

The complicated structure of the rock mass and the wide range of its applications cause challenges and problems in rock engineering and construction involve considerations that are of relatively little or no concern in most other branches of engineering. A major challenge is the uncertainties regarding geological setting and conditions as well as the geotechnical parameters. Therefore, 'engineering judgement' and experience often play an important role in rock engineering and design.

Important in all works involving rock mechanics, rock engineering and design are the quality of the geo-data that form the basis for the calculations and estimations made.

The construction materials in rock constructions as well as the type of construction structure are different. In addition, testing of the construction materials is difficult/impossible. This causes challenges

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when rock design and construction are to be included as a part of a common geotechnical Eurocode.

A design requirement in EC7 is that no relevant limit state is exceeded for each geotechnical design situation, as defined in EN1990:2002. One, or a combination of the following design methods can be applied for this:

- calculations
- prescriptive measures,
- experimental models and load tests,
- observational method

2 WHAT ARE PRESCRIPTIVE MEASURES?

According to Merriam Webster dictionary,
- Prescriptive: rules of usage founded on long-standing custom.
- Measure: an estimate of what is to be expected (of a situation).

This shows that prescriptive measures are associated with experience, empirical methods and well-accepted practical geotechnical solutions.

Empirical methods are simply stated to be correlations between rockmass conditions and rock support and construction. Although the prediction of empirical methods is qualitative, the procedure leading to them can be either quantitative or qualitative. This procedure is important in assessing the validity of the techniques. According to Einstein (1978), empirical models are primarily found in two applications:

1. Before construction ('limited' geological information):
   - design of initial support,
   - determination of construction procedure,
   - preliminary design of final support
2. During construction (limited time):
   - determination of (details of) initial support or adaption of initial support,
   - determination of construction procedure,
   - design of final support in rare instances

Parameters are determined from boring logs, outcrop observations, maps, general knowledge of the area, and from observations in tunnels or in excavated cuttings. Some limited physical testing may also be carried out. Only a limited number of parameters can be determined from boreholes, outcrops and maps (particularly concerning conditions at tunnel grade).

In contrast, observations in the tunnel can detect the real or true rockmass conditions. Parameters, that can be easily obtained from outcrops and boreholes or quickly observed (or measured) in the tunnel, are desirable.

Rockmass classification systems are by most practitioners considered a typical empirical method (e.g. Hoek 1999, Einstein et al., 1979) as they are largely based on experience from earlier rock excavations. They have been used more and more over the last 40 years.

Pre-determined, experience-based prescriptive measures comprising suitable conservative modules of works have for many years been extensively used by the Geotechnical Engineering Office (GEO) in Hong Kong. These solutions are applied without the need for detailed ground investigations and design analyses. The following reports can be found at GEO's homepage:

- Application of Prescriptive Measures to Soil Cut Slopes (Wong et al., 1996)
- Application of Prescriptive Measures to Slopes and Retaining Walls (Wong et al., 1999),
- Prescriptive Design of Skin Walls for Upgrading Old Masonry Retaining Walls (Wang et al., 1999)
- Prescriptive Soil Nail Design for Concrete and Masonry Retaining Walls (Lui et al., 2005)
- Guidelines on the Use of Prescriptive Measures for Rock Cut Slopes (Yu et al., 2005)
- Prescriptive Measures for man-made slopes and retaining walls (Chenung et al., 2009)

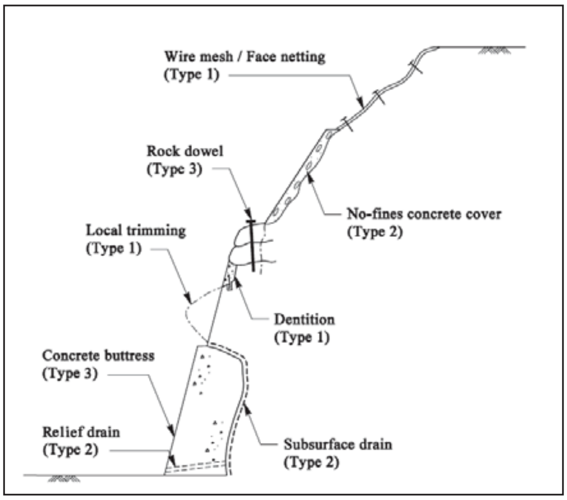

Figure 1: Schematic diagram of typical prescriptive measures for rock cuts (Yu et al., 2005).

They are all guidelines for specific geotechnical engineering areas. The background for each of them is a review of many cases, as for example for rock cuts where more than 100 slopes are reviewed. The guidelines focus on application merits and past experience. Some items for use in rock slopes are shown in Figure 1.

The guidelines give recommended procedures for applications and record sheets that has to be filled in.

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It is also stated that prescriptive measures should be designed by professional qualified engineers and experienced in Hong Kong (such as Registered Professional Engineer), and the same are also applied for construction review. All works must be properly documented.

Other examples on applications are foundation of power masts for power grid in Norway. This system was developed in 1978, based on Prescriptive Measures (NGI, 1978a,b). It is under update, and will include four ground types and four types of foundation types, where two of the foundation types are based on prescriptive measures and two on design by calculations.

3 DESIGN BY PRESCRIPTIVE MEASURES IN EC 7

Prescriptive measures in EC7 consist merely on application rules, no principles. The rules given under prescriptive measure in EC7 are relativity short compared with the other methods that may open up for understandings that are more individual.

EC7 only gives two application rules; of which the first is:

(A)"*In design situations when calculation models are not available or not necessary, exceeding limit states may be avoided by the use of prescriptive measures.* (B)*These involve conventional and generally conservative rules in the design, and attention to specification and control of materials, workmanship, protection and maintenance procedures*".

What does EC7 mean here? The first sentence (A) states that in design situations when calculation models are not available or not necessary, prescriptive measures can be used to avoid that limited states are exceeded. Some situations where calculation models are not necessary can be:

- A long tunnel with rock support, where only a few sections have been calculated, the remaining sections will be designed according to Prescriptive measures or the Observation method.
- A rock cut where a minor key block is exposed and practice is used to design the rock support.

Are these examples acceptable for using prescriptive measures? How can we be sure that none of the limited state cases exceeds? Just because the stability still is acceptable?

The (B) sentence says that these (design situations) involves conventional and generally conservative rules in the design, and attention to specification and control of material, workmanship, protection and maintenance procedures. What do EC7 mean with "*These involve conventional and generally conservative rules in the design*" ? Is this about the design situations or prescriptive measures? The rest seems quite clear.

The second application rule in EC7 is:

"Design by prescriptive measures may be used where comparable experience, as defined in 1.5.2.2 makes design calculations unnecessary. It may also be used to ensure durability against frost action and chemical or biological attack, for which direct calculations are not generally appropriate".

This is an alternative way of using prescriptive measures, and the most interesting part here is the definition of comparable experience as defined in 1.5.2.2 of EC7:

"*documented or other clearly established information related to the ground being considered in design, involving the same types of soil and rock and for which similar geotechnical behaviour is expected, and involving similar structures. Information gained locally is considered to be particularly relevant*".

The text under prescriptive measures has to be more extensive than this in the planned, updated version of EC7. This is further discussed in the following chapter.

4 WHEN CAN PRESCRIPTIVE MEASURES BE APPLIED?

According to Einstein et al. (1979), empirical methods, and consequently prescriptive measures should attempt to satisfy the following, in some way incompatible objectives:

1. They should promote economical, yet safe designs.
2. They must be correctly calibrated against test cases and those test cases must be representative of the field of application of future use.
3. They should be complete in that all relevant factors are included, yet they must be practical in that parameters can be determined and with acceptable certainty.
4. They should have general applicability and robustness, yet they must be recognised as fundamentally subjective.

Figure 2 shows how Hoek. (1999) considers the use of empirical methods (and consequently prescriptive measures) in rock engineering. The design method is selected depending on the ground composition and behaviour.

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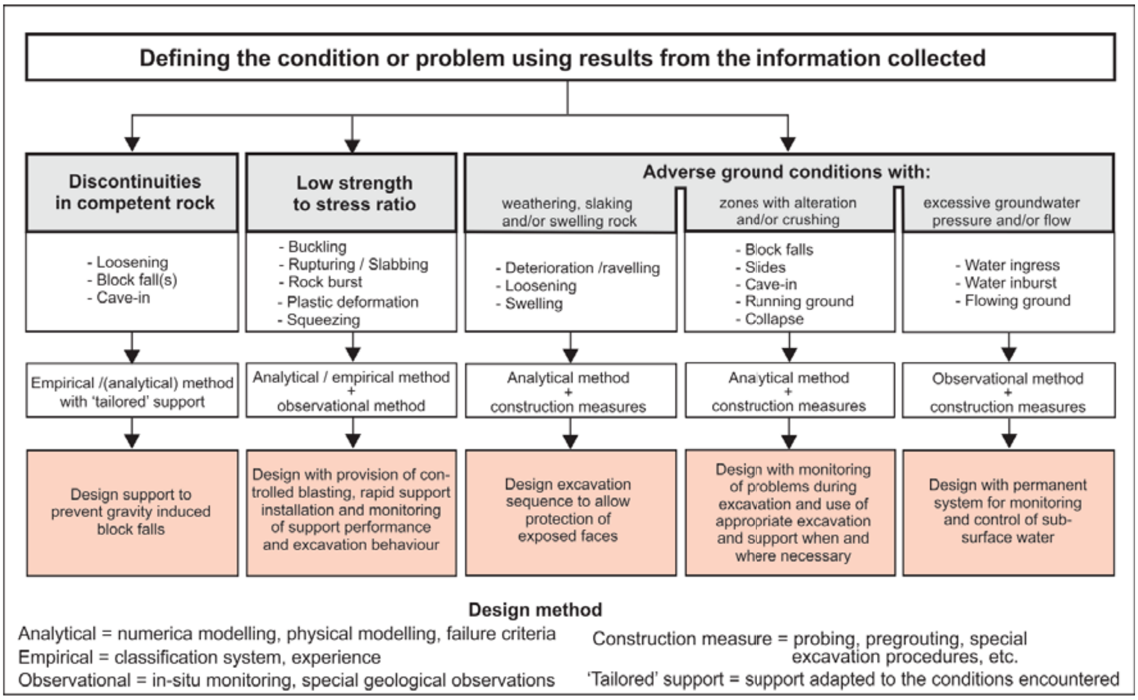

Figure 2: Design methods based on ground conditions and behaviour (developed from Hoek, 1999)

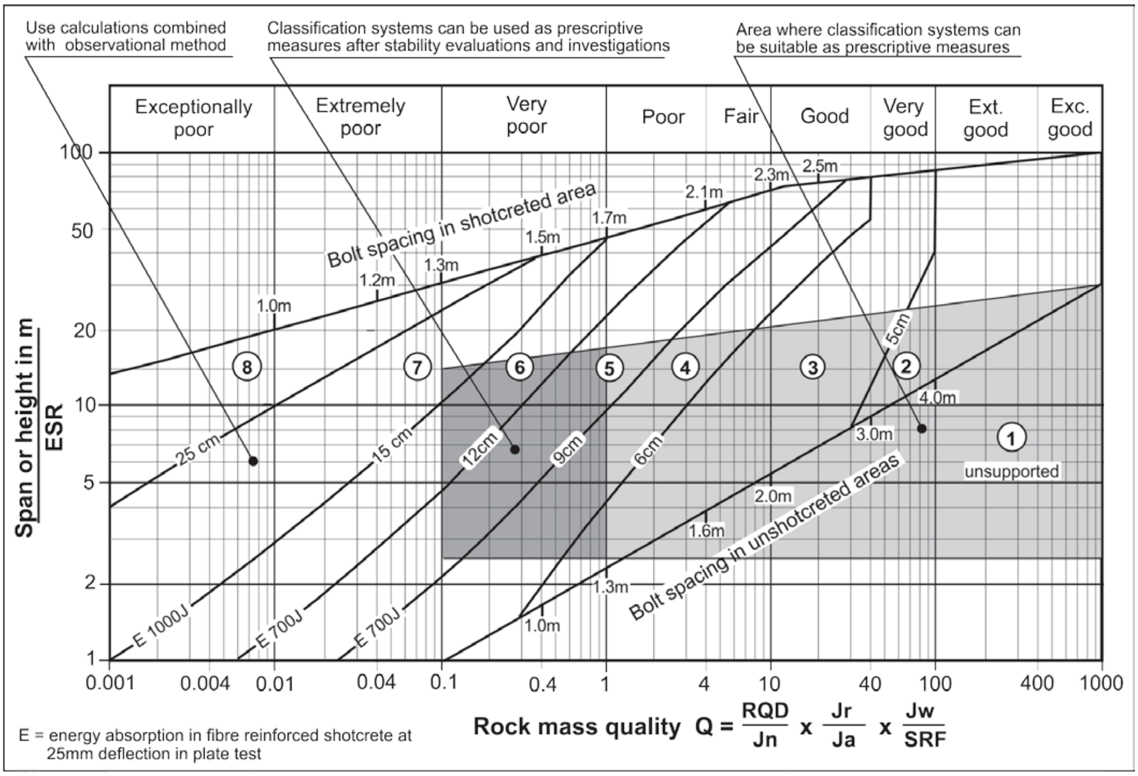

Figure 3: Where the Q classification system can best be used for design in the EC 7

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Table 1: Determination of Geotechnical Category when the ground conditions are uncertain. The ground uncertainty is assumed based on the investigations results.

Excavation Risk	**Types of Construction. Examples**	**Assessed degree of Ground Uncertainty** (before encountered)		
		Low	Medium	High
Low	- Foundations	1	1 / 2	2
Medium	- Foundations where blasting is involved - Tunnels and small - moderate rock cuttings	1 / 2	2	2 / 3
High	- Undersea tunnels - Caverns with large span - Low rock cover of underground excavations in susceptible areas - High rock cuttings; anchoring for suspension bridge - Excavation may influence on nearby settlements	2	2 / 3	3

Degree of Ground Uncertainty (before the ground has been encountered in the excavation):

Low: Clear and simple geology and ground conditions. Ground parameters can be easily found. Experience from similar ground conditions can be documented.

Medium: Clear geology and ground conditions. Methods exist to assess ground conditions and for dimensioning. Acceptable experience from other similar ground conditions and constructions can be documented.

High: Unclear geology and/or ground conditions with potential for problematic tunnel excavation. There are limited possibilities to assess the ground conditions.

Excavation Risk:

Low: No risk. Safe, straight forward excavation.

Medium: Some probability for loss.

High: Possibility for severe accident(s) and loss. Accidents and loss can be injuring incidents and/or disasters, such as collapse, water ingress, damage to nearby constructions, etc.

Table 2: Determination of Geotechnical Category when the ground conditions are known (after encountered in the excavation).

Level of Usage Requirements	**Types of Usage. Examples**	**Ground Quality**		
		Good	Fair	Poor
Low	- Simple foundations - Water tunnels, mine drifts - Moderate rock cuttings	1	1 / 2	2
Moderate	- Partly complicated foundations - Low traffic road- and railway tunnels - High rock cuttings, storage caverns in rock	1 / 2	2	2 / 3
High	- Complicated foundations; Very high rock cuttings - High velocity railway tunnels and heavy traffic road tunnels - Underground railway and hydropower stations - Areas with potential for severe landslide - The construction may cause damage on nearby settlements	2	2 / 3	3

Level of Usage Requirements:

Low: Limited requirements as long as the project functions during its lifetime. For water tunnels, e.g., downfall of fragments and single blocks are often accepted.

Moderate: Minor maintenance/control is accepted within lifetime of construction.

High: No damage or deterioration of the construction is accepted during its lifetime.

Ground Quality is defined according to a preset classification. Ground classification systems may be used for this.

Classification systems are empirical systems and include quantitative values for the rock mass quality, geological conditions and geometrical design. One of the most used classification systems is the Q-system. This system is based on review of rockmass conditions and rock support in more than thousand tunnels and caverns.

Figure 3 shows how the use of empirical methods in Figure 2, may be used with the Q-system. This means that for Q-values larger than 1, prescriptive measure can be suitable, while for Q-values between 0.1 to 1, additional stability evaluations should be applied. For Q less than 0.1 the design should be based on a combination of the observation method, preferably supported by calculations. The limit for the Q-values is not definitive. This principle will probably also be valid for several other classification systems.

In EC7, the design method is to be selected on basis of the Geotechnical Category (GC). However, also the ground conditions, i.e. the ground risks (challenges and difficulties/uncertainties) will strongly influence during excavation and the ground quality on usage or operation requirements of the project.

As the ground conditions along the tunnel cannot be determined before excavation a main issue is the geological uncertainties (and consequently excavation risks). This has to be accounted for in the GC as shown in Table 1.

After the tunnel or cavern has been excavated and the ground conditions are known, the qualities of the ground along the tunnel form the main issue in the design of the permanent support as well as in the maintenance control plans. Consequently, this should be a main input in the selection of the GC, see Table 2.

The consequences of this is that a project e.g. may use GC = 3 during planning and GC = 2 for design of permanent rock support. As the ground conditions mostly will vary along a tunnel, the GC may also vary along the tunnel.

5 CONCLUSIONS

Prescriptive measure is one of the four design methods that can be applied when no calculation models are available or necessary. It comprises predetermined, experienced-based and suitably conservative solutions, without the need for detailed ground investigations and design analysis. The EC7 application rules give very short and weak guideline for using this method.

Nevertheless, the method is very important in the design of tunnels in medium to good stability rockmasses with possibilities to use rockmass classification systems.

EC7 does not give an appropriate description for selection of the Geotechnical Category (GC). The paper shows some ideas how GC can be found and used.

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EC7 and the application of analytical and empirical models to rock engineering

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ABSTRACT: The development of EC7 seems to have proceeded with little, if any, input from the rock engineering community, such that the current version of EC7 reveals many difficulties with regard to rock engineering design. Here, following a brief review of various key aspects of the limit state design philosophy adopted by EC7, some of these difficulties in terms of empirical and analytical approaches to rock engineering design are outlined. In particular, EC7 appears to separate the concepts of empiricism, theory, prescriptive measures and observational methods, whereas these are often used in combination in rock engineering design, with empiricism being as a central theme. Similarly, the absence of partial factors for rock mechanics properties is shown to severely limit the applicability of the Design Approaches presented in EC7, and the definitions of Geotechnical Categories given in the code are seen to be unsuitable for some rock engineering designs. Overall, a key difficulty with the code seems to be the lack of recognition of the central and necessary role that empiricism plays in rock engineering design. The challenge for the immediate future is to align EC7 with rock engineering practice, so that it becomes universally applicable to rock engineering design.

1 INTRODUCTION

Development of EN 1997, Eurocode 7: Geotechnical Design (referred to here as EC7) began in 1976 when the European Commission agreed to sponsor development of a set of European codes of practice for building structures (Simpson & Driscoll, 1998). In 1980 an agreement was reached between the Commission of the European Communities and the then International Society for Soil Mechanics and Foundation Engineering to draft a model code that could be adopted as EC7. From this beginning, and after many years of drafting and development, EC7 was implemented in many European countries as the standard geotechnical design in 2010 (Orr, 2012).

From the 1980s onwards, the development of EC7 seems to have proceeded with little, if any, input from the rock engineering community, such that the current version of EC7 does not reflect customary modern rock engineering design methodologies.

Here, following a brief review of various key aspects of the limit state design philosophy adopted by EC7, some of these difficulties in terms of empirical and analytical approaches to rock engineering design are outlined.

2 LIMIT STATE DESIGN AND EUROCODE 7

EC7 is part of the suite of Eurocodes developed by CEN (European Committee for Standardisation) for structural engineering design. All of the structural Eurocodes are based on EN 1990, Eurocode: Basis of Structural Design (CEN, 2002), which applies the principle of limit state design (LSD). As the so-called 'head code', EN 1990 includes the following (Bond and Harris, 2008):

- an explanation of the fundamental engineering approach that underlies the Eurocode suite;
- establishes the principles and requirements for the safety, serviceability, and durability of structures;
- a description of the basis for their design and verification; and
- gives guidelines for related aspects of structural reliability.

EN 1990 requires structures to remain fit-for-purpose throughout their entire design working life (including the construction period), and so must possess adequate structural resistance and durability to withstand all likely actions and influences. However, in geotechnical engineering, and particularly in rock engineering involving underground construction, stability during excavation is both po-

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tentially more critical than stability in service and significantly influenced by the excavation method. As a result, the definition of limit states is critical, and so a substantial part of EC7 is devoted to this. Additionally, geotechnical engineering works encompass a wide range of complexities, from the very simple to the very intricate. Recognising this, EC7 defines geotechnical categories (see Section 2.3) to help designers apply the code.

2.1 *Limit States*

In the Eurocode implementation of LSD, satisfactory performance is ensured by the introduction of serviceability (SLS) and ultimate (ULS) limit states. The SLS applies to the proper functioning of the structure under normal conditions. It may include aspects such as the comfort of users and appearance of the structure, and may be intermittent (e.g. groundwater seepage) or permanent (cracking of a concrete retaining wall). The ULS applies to the safety of users and the structure, and represents a catastrophic loss of stability. Both SLS and ULS apply during construction, although the conditions they represent may be different from those required in service.

EC7 defines five specific ultimate limit states (clause 2.4.7.1), each referred to by a three letter acronym. These limit states are (CEN, 1997):

EQU loss of equilibrium of the structure or the ground, considered as a rigid body, in which the strengths of structural materials and the ground are insignificant in providing resistance;

STR internal failure or excessive deformation of the structure or structural elements, including e.g. footings, piles or basement walls, in which the strength of structural materials is significant in providing resistance;

GEO failure or excessive deformation of the ground, in which the strength of soil or rock is significant in providing resistance;

UPL loss of equilibrium of the structure or the ground due to uplift by water pressure (buoyancy) or other vertical actions;

HYD hydraulic heave, internal erosion and piping in the ground caused by hydraulic gradients

Not all of these states will be applicable to every design, but a designer is required to confirm the stability of a geotechnical structure with regard to all that are appropriate. It is EQU, STR and GEO that are most important for rock engineering designs.

A key principle (i.e. something that must be adopted) of EC7 is that "*For each geotechnical design situation it shall be verified that no relevant limit state… is exceeded*" (clause 2.1(1)P). Four methods for verifying a limit state are given by EC7, namely: adoption of prescriptive measures; use of experimental models and load tests; application of an observational method; and calculation (clause 2.1(4)). In the case of calculation, the designer establishes and compares load and structural resistance models, and ensures that the resistance is not less than the load. As loads cause some action, the Eurocodes use the term 'action' rather than load, and so the design problem is then to ensure the relation

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

is satisfied, where E_d is the design action and R_d the design resistance. Although trivial in appearance, application of Eqn. 1 to geotechnical structures is awkward, and leads to much complexity in EC7.

2.2 *Uncertainty in actions and material properties*

Critically, LSD explicitly recognises the variability inherent in actions and resistances. This is illustrated diagrammatically in Fig. 1, where both the action and the resistance models are shown as probability density distributions. In LSD such distributions are reduced to a single 'characteristic value', with design values of the actions and resistance, E_d and R_d, calculated from the representative action and characteristic resistance using the partial factors γ_F and γ_M. It is through the combination of characteristic values and partial factors that variability may be accounted for.

As shown in the lower diagram of Fig. 1, there is only a very small probability that the limit state will not be satisfied when using values of partial factors that lead to $E_d = R_d$. Note that for clarity of illustration the lines representing E_d and R_d are much closer to the modes of their respective distributions than is the case in reality.

Clearly, this application of LSD requires statements of the various partial factors, definitions of the representative actions and the characteristic resistance, and the means by which both E_d and R_d are calculated. As the GEO ULS represents "failure or excessive deformation of the ground", it is the calculation of R_d, the value(s) of γ_M, and the definition of X_K for rock and rock masses that are of particular importance in rock engineering design. Implications of this for rock engineering design are discussed later in this paper.

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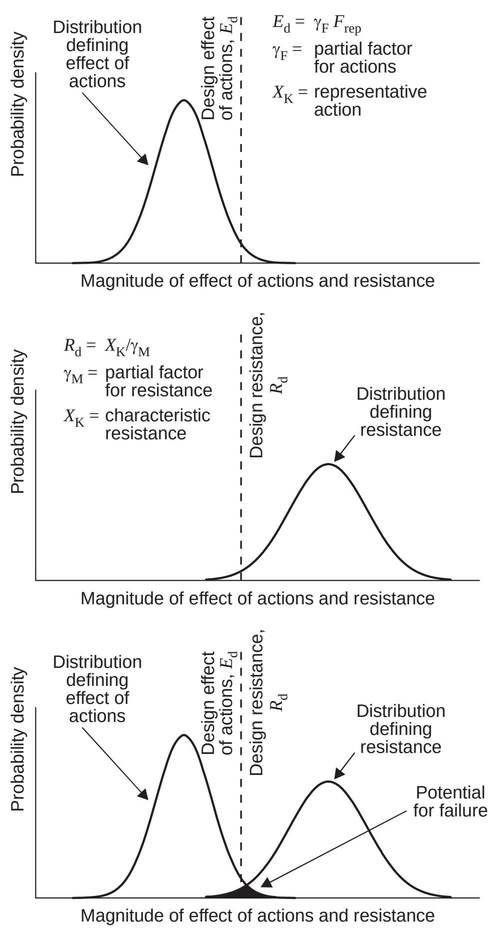

Figure 1: Principle of limit state design.

2.3 *Geotechnical Categories*

Recognising that geotechnical designs cover a wide range of complexities, EC7 introduces the concept of Geotechnical Categories, GCs (clauses 2.1(10) to 2.1(21)). Category 1 is for structures that are "*small and relatively simple"*, for which "*the fundamental requirements will be satisfied on the basis of experience and qualitative geotechnical investigations*". Category 2 represents "*conventional types of structure and foundation with no exceptional risk or difficult soil or loading conditions*", and that these designs "*should normally include quantitative geotechnical data and analysis to ensure that the fundamental requirements are satisfied*". Finally, Category 3 is for structures that "*fall outside the limits of Geotechnical Categories 1 and 2*" and includes, among others, "*very large or unusual structures*".

Although EC7 is very clear in its definitions of limit states, means by which they may be verified, and Geotechnical Categories, in Section 4 below it is shown that the application of these to rock engineering is in many ways awkward.

3 CHARACTERISTIC VALUES, PARTIAL FACTORS AND DESIGN APPROACHES

Limit state verification by calculation currently forms a substantial part of EC7, with much attention being paid to related characteristic values, partial factors and design approaches. Familiarity with these is necessary in order to apply analytical approaches to rock engineering design.

3.1 *Characteristic and derived values*

EN 1990 recognises that a stochastic model (Fig. 2) is often appropriate for characterising variability in actions and resistances, and thus suggests "*where a low value … is unfavourable, the characteristic value should be defined as the 5% fractile value; where a high value … is unfavourable, … as the 95% fractile value"* (clause 4.2(3)). A key requirement in applying this is the existence of sufficient information – say, in the form of test results – to both identify and characterise the probability density distribution. For materials made to a specification – such as steel or concrete – this information is usually available. However, and importantly, EC7 recognises that for soils and rocks only a small number of test results may be available, and as a statistical approach is not possible in such circumstances states the principles that "*The characteristic value of a geotechnical parameter shall be selected as a cautious estimate of the value affecting the occurrence*

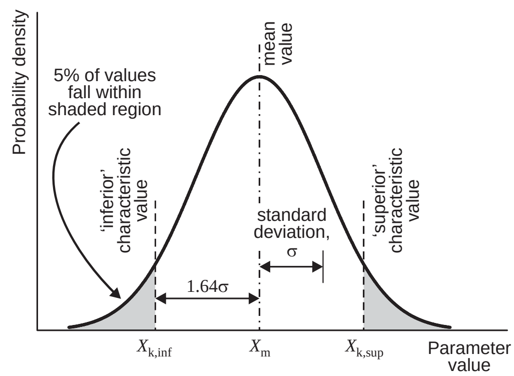

Figure 2: Statistical approach to derivation of characteristic values (after Bond & Harris, 2008).

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of the limit state" (Clause 2.4.5.2(2)P) and that "*The selection of characteristic values for geotechnical parameters shall be based on results and derived values from laboratory and field tests, complemented by well-established experience*" (Clause 2.4.5.2(1)P). Thus, for a design calculation in support of a Geotechnical Category 2 structure, because these two clauses are **principles**, they **require** appropriate testing be undertaken. However, such testing may, of course, not be feasible for rock mass properties; this is considered further in Section 4.

In many cases, test results are used to derive other values, and EC7 defines these as the "*value of a geotechnical parameter obtained by theory, correlation or empiricism from test results*" (clause 1.5.2.5). Examples of the application of theory, correlation and empiricism are readily found in rock engineering practice. Thus, for rock strength, derivation of the unconfined compressive strength and the Hoek-Brown strength parameters *m* and *s* from triaxial strength data is commonplace. Similarly, we may use correlation to obtain values of unconfined compressive strength from Point Load Index values, or use empirical relations to obtain values of rock mass elastic modulus from values of RQD (Fig. 3).

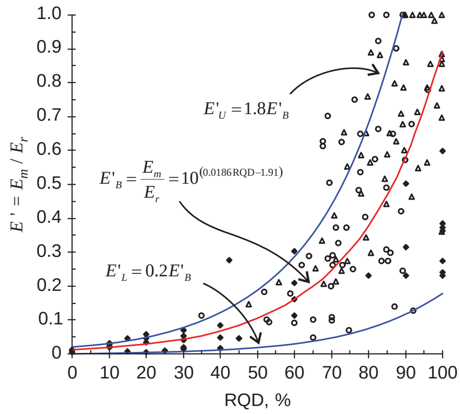

Figure 3: Empirical relations between RQD and rock mass elastic modulus (after Zhang & Einstein, 2004).

3.2 *Partial factors*

Within LSD a common method of accounting for variability in actions and resistances is through the use of partial factors (Fig. 1). For geotechnical engineering the values of these factors are intimately related to the calculation model employed, as will be shown in Section 3.3. Consequently, EC7 includes tables of partial factors that are arranged in sets, with the factors in a particular set being applicable to a given calculation model.

Table 1 lists the partial factors applicable to permanent actions for the GEO ULS. A particular difficulty for geotechnics is the fact actions may have both stabilising and destabilising effects. An example is given by the load applied to a rock foundation: this induces both minor and major principal stresses in the rock mass, with the first increasing the strength of the material and is thus being a stabilising action, and the second being a destabilising action. EC7 specifies different values of partial factors for these cases, and this can lead to the unreasonable situation of two different partial factors being applied to the same action. Such ground-structure interactions are commonplace in geotechnical engineering, and so to avoid the use of different factors EC7 introduces the single source concept: "*Unfavourable (or destabilising) and favourable (or stabilising) permanent actions may in some situations be considered as coming from a single source. If they are considered so, a single partial factor may be applied to the sum of these actions or to the sum of their effects*" (clause 2.4.2). Application of this concept that requires careful thought, and may require alternative calculation approaches in order to identify the least conservative application of a factor.

Table 1: Partial factors for GEO permanent actions

		Set	
Sense of action	Symbol	*A1*	*A2*
Unfavourable	γ_G	1.35	1.0
Favourable		1.0	1.0

For material properties, currently the only partial factors listed within EC7 are related to soils. These are shown in Table 2. No partial factors are given for rock mechanics properties.

Table 2: Partial factors for material properties

		Set	
Parameter	Symbol	*M1*	*M2*
$\tan\phi'$	$\gamma_{\phi'}$	1.0	1.25
Effective cohesion	$\gamma_{c'}$	1.0	1.25
Undrained shear strength	γ_{cu}	1.0	1.4
Unconfined strength	γ_{qu}	1.0	1.4
Weight density	γ_{γ}	1.0	1.0

Partial factors for resistance are listed only for spread foundations, driven piles, bored piles, and continuous flight auger piles. For the sake of brevity, Table 3 presents only those factors for spread foundations.

Table 3: Partial factors for resistance of spread foundations

Resistance	Symbol	Set *R1*	*R2*	*R3*
Bearing	$\gamma_{R;v}$	1.0	1.4	1.0
Sliding	$\gamma_{R;h}$	1.0	1.1	1.0

3.3 *Design approaches*

Recognising that many different design approaches are used across Europe, EC7 presents three Design Approaches (DA) for applying partial factors in geotechnical calculations. These use the sets of factors presented in Tables 1 to 3, and are for general design (specifically, not for slopes, piles or anchorages).

Design Approach 1

Combination 1: $A1 \oplus M1 \oplus R1$

Combination 2: $A2 \oplus M2 \oplus R1$

Symbol '$\oplus$' represents 'combined with'; partial factor sets *A1* and *A2* apply to actions.

Design Approach 2

Combination: $A1 \oplus M1 \oplus R2$

A1 applies to either actions or effects of actions.

Design Approach 3

Combination: $\left[A1_{\mathrm{STR}} \text{ or } A2_{\mathrm{GEO}}\right] \oplus M2 \oplus R3$

A1 applies to structural actions, *A2* applies to geotechnical actions.

The National Annexes to EC7 may state which DA is to be applied to a given design, but in the absence of this a designer should apply all DAs in order to identify the most critical. The application of DAs to rock engineering is discussed in Section 4.

4 EMPIRICAL AND ANALYTICAL APPROACHES IN ROCK ENGINEERING DESIGN

4.1 *Empiricism in rock engineering design*

The difficulty of characterising and modelling fractured rock masses means that rock engineering practice has evolved to make wide use of empiricism, such that it is found within characterisation, analysis and design. In parallel with this, analytical approaches have been developed for the design of slopes, foundations and underground excavations in rock. Indeed, in many cases rock engineering design involves a combination of empiricism, theory, prescriptive measures and observational methods, with empiricism being present to varying degrees in latter three of these. As currently formulated EC7 appears to separate these four aspects, and also does not seem to recognise the presence of empiricism as a central theme in design. In order for EC7 to be universally applicable to rock engineering design, this needs to be changed.

4.2 *Design Approaches*

Being part of the Eurocode suite, EC7 suggests that variability may be accounted for through partial factors, and from this defines the various Design Approaches for use in calculation. Although EC7 presents many partial factors, as Table 2 shows none – apart from unconfined compressive strength (assuming that this property can be considered equivalent in geotechnical meaning to unconfined strength γ_{qu}) – are currently given for rock mechanics properties.

The absence of these factors means that only those DAs which use set *M1* – for which all factors are unity – can be utilised in rock engineering design. In other words, only DA2 is applicable (note that DA1 cannot be used, because application of this approach requires both combinations 1 and 2, and combination 2 requires set *M2*). Unfortunately, DA2 requires partial factor set R2 and, as Table 3 shows, these factors are currently available only for various types of foundations.

This suggests that none of the three DAs may currently be applied to general rock engineering design. Clearly, a concerted effort is required by the rock engineering community to develop the necessary partial factors in order to change this. However, it is not easy to see how partial factors could be developed to account for the discontinuous, heterogeneous and anisotropic attributes of rock masses. Thus, it may be that a partial factor approach is not generally applicable to rock engineering, and that the Level III full probabilistic reliability method defined in EN 1990 is required. This needs further investigation.

4.3 *Geotechnical Categories*

Although EC7 introduces the useful concept of Geotechnical Categories, as currently laid out they present difficulties for rock engineering design.

GC1 represents "*small and relatively simple*" structures that can be designed using "*experience and qualitative geotechnical investigation*". This exemplifies empiricism in the form of rock mass classification schemes, as all major schemes in current

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use (e.g. GSI, Q, RMR) rely, at least in part, on subjective and qualitative assessments. However, although the use of rock mass classification schemes seems well suited to GC1 designs, EC7 requires verification that the fundamental limit states are satisfied. As things stand, this can only be done pointing to the fact that such schemes have previously been satisfactory in similar circumstances.

However, a better approach might be to recognise that for some geotechnical designs the ULS is more likely to be encountered during construction than service (as noted earlier, this is particularly the case for underground construction). In such cases, application of empiricism together with an observational approach and appropriate construction techniques allows verification of the ULS during the critical construction period. This is an example of how a fundamental change to the content of EC7 is required to allow its ready application to rock engineering.

As noted earlier, EC7 states that GC2 designs "*should normally include quantitative geotechnical data and analysis*". The situation here with regard to rock engineering is particularly awkward. Firstly, it is generally unfeasible to obtain quantitative data for the properties of rock masses, which are the scale of interest, through field or laboratory testing. As a result, empirical correlations such as those shown in Fig. 3 are often used to provide estimates of design values. Secondly, although the statement seems to exclude application of empiricism in the form of rock mass classification schemes, many analytical tools used in rock engineering are based on empiricism: the Hoek-Brown empirical strength criterion is a case in point. The question is then, is empiricism permitted within analysis? Arguably it is, because in the spirit of EC7 the fitting of the Hoek-Brown criterion to triaxial strength data is an application of theory in the same way that the fitting of the Mohr-Coulomb criterion is, and yet both of these criteria are empirical. Clearly, clarification of this issue is required in EC7.

Finally, EC7 gives a rock engineering example of a GC2 design as "*tunnels in hard, non-fractured rock and not subjected to special water tightness or other requirements*". This implies that all tunnels in fractured rock – in other words, the vast majority – are either Category 1 or 3! This is ridiculous, as rock engineering designers would regard most tunnels as being GC2: "*conventional types of structure… with no exceptional risk or difficult… conditions*". Furthermore, the complexity of geology and the difficulties of performing relevant ground investigation in advance of construction it many cases make it impossible to base the design on "*quantitative geotechnical data and analysis*". In reality, a combination of design based on prescriptive methods and extended usage of an observational method during construction is often applied, but it is not clear that EC7 supports the use of this combination for Category 2 designs. This needs to be rectified.

These simple examples indicate that the definitions of Geotechnical Categories need to be revised to suit current rock engineering practice.

5 CONCLUSIONS

This review has shown, particularly with regard to empirical and analytical approaches to rock engineering design, there is a lack of alignment between the content of EC7 and standard industrial practice. This needs to be changed.

EC7 embraces limit state design, but does not offer the guidance required to allow its application to rock engineering. In particular, there is a lack of partial factors for rock engineering, and this renders the various analytical Design Approaches inapplicable to anything except spread foundations.

The definitions of Geotechnical Categories need revision in order to more accurately reflect rock engineering practice, particularly with regard to the methodologies applied to designs in each category.

A key difficulty with the code seems to be the lack of recognition of the central and necessary role that empiricism plays in rock engineering design. The challenge for the immediate future is to align EC7 with rock engineering practice, so that it becomes universally applicable to rock engineering design.

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On the applicability of the Eurocode7 partial factor method for rock mechanics.

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ABSTRACT: The use of partial factors in rock engineering designs is currently debatable: one school of thought is that the limit state design method with partial factors in Eurocode 7 is inapplicable to rock engineering; another is that this method needs to be modified for use in rock engineering. This paper presents a discussion on the applicability – or rather, inapplicability – of the limit state method presented in Eurocode7, when undertaking rock engineering design by calculation. This paper discusses the issues involved and makes some recommendation for developments and changes to Eurocode 7 so that design by calculation may be undertaken more appropriately by practitioners engaged in rock engineering designs.

1 INTRODUCTION

For a large part of the history of geotechnical engineering, deterministic methods such as 'Factors of Safety' or 'Working Stress Design' have been utilised to account for uncertainty when undertaking geotechnical design by calculation. In recent years the Eurocodes have been introduced in Europe for the design of structures and construction work, including geotechnical designs involving soil and rock. The Eurocodes are based on the limit state design (LSD) method set out in EN 1990:2002 *'Basis of structural design'* (CEN, 2002). LSD is a semi-probabilistic method in which partial factors are applied to characteristic parameter values in order to account for parameter uncertainty and achieve designs with a certain target reliability. The limit state method in EN 1990 is aimed mainly at manufactured materials and hence defines the characteristic value of a parameter as the 5% fractile of an unlimited test series having a known statistical distribution, i.e. the parameter is a random variable defined by a stochastic function – it follows the so-called aleatory model. However, for many rock mechanics parameters uncertainty is primarily due to insufficient knowledge – so called epistemic uncertainty (Becker, 1996a; Becker, 1996b; Bedi, 2013; Bedi & Harrison, 2013). This lack of information necessitates subjective estimation of parameters used in any analysis, an example of which is the Geological Strength Index (Hoek *et al*., 1995) or Joint Roughness Coefficient (JRC) (Barton & Choubey, 1977).

The magnitude of the partial factors used in LSD are governed by the probability and consequence of failure and, in EC7 have been derived in accordance with EN 1990 by calibration to a long experience of building tradition, i.e. with conventional total factors of safety, or by semi-probabilistic studies of resistance, i.e. assuming uncertainty is aleatory. At present EC7 does not present any factors for rock mechanics parameters. This paper investigates the potential significance of this shortcoming with respect to the debate for extending the partial factor method to rock mechanics problems.

This paper begins with a brief review of LSD and the partial factor method as embodied in the 'Design by calculation' procedures in EC7. It is shown that the calculation model in EC7 assumes that parameter uncertainty is aleatory; however, in the case when parameter uncertainty is epistemic, the application of LSD remains unproven (Christian, 2004). This paper follows with discussion of a wider, and arguably more significant, issue which is the applicability of the partial factor method for rock engineering designs; that is its application when the parameters used to characterise fractured rock masses are

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subjectively estimated or assessed qualitatively through expert judgement (i.e. the uncertainty is epistemic). As such, this paper is able to draw conclusions on whether such parameters can be aligned with the LSD method in EC7.

2 LIMIT STATE DESIGN METHOD

In order to quantify the risk of adverse effects prior, to inception and construction of a project, design by calculation has become one of the most commonly applied procedures for uncertainty quantification and checking the avoidance of limit states, and thus forms a large part of EC7 (Frank *et al.,* 2005).

This section details the basis of the LSD method adopted by EC7 and explains how partial factors are implemented within this framework.

2.1 *Basis of the LSD calculation model*

EC7 embodies the LSD principles – as defined by EN 1990 – in prescribing means to undertake design by calculation. EC7 requires verification that the design value of effect of actions (loads), E_d, is less than or equal to the design resistance, R_d, of the structure ($E_d \leq R_d$) to demonstrate the occurrence of the limit state in question is sufficiently unlikely.

The uncertainties in the effect of actions and resistances are generally assumed to follow an aleatory model (as shown by the probability distributions of E and R, respectively, in Figure 1). The design effects of actions are derived from 'representative actions' , F_{rep}, multiplied up by a partial factor γ_F (i.e., $E_d = E\{\gamma_F F_{rep}\}$, while the design resistance is determined either from 'characteristic values' (X_K) of the parameters for the material properties in question divided by a material partial factor, γ_M, or from the characteristic

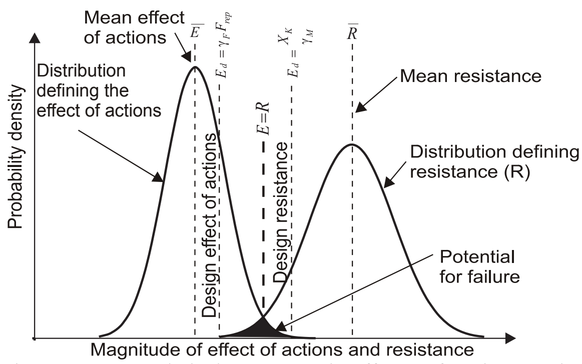

Figure 1: Geotechnical LSD: Both effects of actions and material resistance are considered as aleatory variables, modelled using statistical distributions (after Becker, 1996b).

resistance calculated using the characteristic values of the parameters divided by a partial resistance factor γ_R; i.e. $R_d = R\{X_K / \gamma_M\} / \gamma_R$, where either γ_M or γ_R is unity. The characteristic values of geotechnical parameters are selected to take account of the variability of the measured values (EN 1997-1 2.4.5.2(4)P) while the partial factors are intended to provide the level of safety required. As illustrated in Figure 2, in EN 1990, the partial factor γ_M is used to capture model uncertainty in structural resistances (γ_{Rd}) and in material properties (γ_m), which for structural materials that follow an aleatory model may be calibrated to attain a target level of reliability based on probability theory.

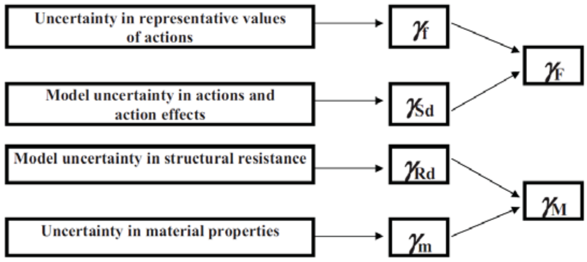

Figure 2: Relation between individual partial factors (from EN 1990 CEN 2002)

As the Eurocodes were originally conceived for manufactured materials, for which an aleatory model is appropriate, the LSD method in EN 1990 defines the characteristic value of a material property or resistance as generally corresponding to a specified fractile, which, unless otherwise stated, should be defined as the 5% fractile of the assumed statistical distribution of the particular property. This philosophy is captured by EC7, which states that if statistical methods are used to determine the characteristic value of a geotechnical parameter, it "*should be derived such that the calculated probability of a worse value governing the occurrence of the limit state under consideration is not greater than 5%*" (EN 1997-2 §2.4.5.2(11)). However, EN 1990 also states that in some circumstances, the characteristic value may be a nominal value fixed on non-statistical bases, for instance on acquired experience or on physical conditions. On this basis, EC7 recognises that the uncertainty in many geomaterial parameter values cannot be characterised by statistical distributions – because it is epistemic – and hence states that "*the characteristic value of a geotechnical parameter shall be selected as a cautious estimate of the value affecting the occurrence of the limit state*" (EN 1997-1 §2.4.5.2(2)P). It is perhaps this subjective estimate of geotechnical parameters values that

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makes the derivation of characteristic values particularly difficult.

2.2 *Basis of the partial factors*

EC7 *"…is intended to be used in conjunction with EN 1990:2002, which establishes the principles and requirements for safety and serviceability, describes the basis of design and verification and gives guidelines for related aspects of structural reliability"* (EN 1997-1 §1.1.1(1)). EN 1990 provides the partial factors for the different types of structural actions that must be considered in design calculations for various limit states, the magnitudes of which have been determined on the basis of an aleatory model to ensure a limit state is not exceeded at a pre-determined target probability of failure. EC7 provides the requirements for determining geotechnical actions and resistances as well as the values of the partial action, material and resistance factors for use in geotechnical designs for ultimate limit states involving failure of the ground, i.e. GEO ultimate limit states (see Table 1).

EC7 recognises the epistemic nature of uncertainty in determining the characteristic values of geotechnical parameters and so, as noted above, the recommended partial factor~~s~~ values given in EC7 have been chosen mainly on the basis of previous experience or calibration with the traditional factors of safety.

The limit state design procedure, defined in EC7 §2.4.1, involves:

a) Establishing characteristic values of actions, which may be either imposed loads or imposed displacements;
b) Establishing characteristic values of ground properties and properties of the structural materials;
c) Defining limiting values of deformation, crack width, vibrations, etc.;
d) Setting up calculation models for the relevant ultimate and serviceability limit states that predict the effect of actions, the resistance and/or the deformations of the ground and in which the various design situations are considered using design values of the parameters; and
e) Showing, by use of appropriate calculation models, that the limit states will not be exceeded in the design situations.

EC7 then presents three Design Approaches for GEO ultimate limit states with different combinations of recommended partial factors, shown in Table 1, that are applied to the actions and material properties or resistances depending on the Design Approach being adopted. The recommended partial material factors in EC7 are intended to indicate the minimum margin of safety for conventional designs (EN 1997-1 §2.4.6.2(2)P) and also account for uncertainty in the calculation model. Three noteworthy aspects of the partial factors presented in the Eurocodes are apparent:

Table 1: Sets of partial factors on actions, material properties and resistances for GEO ultimate limit states given in EC7 (from Bond & Harris, 2008)

Parameter			Actions or effects		Material properties		Resistance		
			A1	A2	M1	M2	R1	R2	R3
Permanent actions (G)	Unfav'ble	γ_G	1.35	1.0					
	Favourable	$\gamma_{G,fav}$	1.0	1.0					
Variable actions (Q)	Unfav'ble	γ_Q	1.5	1.3					
	Favourable	$\gamma_{Q,fav}$	0	0					
Coefficient of shearing resistance (tan φ)		γ_φ			1.0	1.25			
Effective cohesion (c′)		$\gamma_{c'}$			1.0	1.25			
Undrained strength (c_u)		γ_{cu}			1.0	1.4			
Unconfined compressive strength (q_u)		γ_{qu}			1.0	1.4			
Weight density (γ)		γ_γ			1.0	1.0			
Bearing resistance (R_v)		γ_{Rv}					1.0	1.4	1.0
Sliding resistance (R_h)		γ_{Rh}					1.0	1.1	1.0
Earth resistance … retaining structures … slopes		γ_{Re}					1.0	1.4 1.1	1.0
Prestressed anchorages		γ_a					1.1	1.1	1.0

1. Firstly, the LSD method, as embodied in EN 1990 and adopted by EC7, generally assumes that, in so far as characteristic values can be fixed on statistical bases, the characteristic values of actions (loads) and material parameters, chosen to account for the uncertainty in actions (loads) – whether structural or geotechnical – and ground properties (items 'a' and 'b', in the above list), are defined using a stochastic (aleatory) model. Thus, to apply this approach it is assumed that appropriate characteristic parameter values can be selected at a 5% fractile level from ~~a~~ probability distributions that the variabilities in the parameters are assumed to follow and these are divided by prescribed constant partial factors to obtain the design values of the parameters in question, which form the inputs into the calculation model (item 'd', in the above list).

2. To account for the epistemic nature of uncertainty in the estimation of geotechnical parameter values, EC7 also allows design values of geotechnical parameters to be either *derived from characteristic values using* the equation~~s~~ in Section 2.1, *or assessed directly"* (EN 1997-1 §2.4.6.2(1)P). As the majority of rock engineering parameters (the inputs), used to characterise the rock mass properties, for ultimate limit state calculations often cannot be assumed to be aleatory (as will be discussed in the following section), the rock resistance cannot be represented by a probability distribution curve, as in Figure 1, and hence, in accordance with EC7, direct estimation of design value~~s~~ may be the appropriate means to determine rock mass parameter values. However, this approach could be perceived as means of by-passing the use of the partial factor method, and being more akin to the traditional working stress methods with the factor of safety simply applied in a different way (Pells, 2011).
3. Thirdly, unless the partial factor for the unconfined strength and frictional shear strength parameters (q_u, c and ϕ in Table 1, respectively) of soil can also be used for rock, EC7 does not present partial factors for the parameters used to characterise fractured rock masses. One reason may be that the work necessary to develop the partial factors has never been undertaken, but – as will also be discussed in the following section – it could simply be that the parameters used to characterise the strength of fractured rock masses are epistemic in nature, and this precludes their determination.

3 ROCK MASS PARAMETERS: ALEATORY OR EPISTEMIC?

Two useful acronyms to describe rock masses are CHILE (Continuous, Homogeneous, Isotropic, Linear, and Elastic) and DIANE (Discontinuous, Inhomogeneous, Anisotropic, Non-linear Elastic) (Hudson & Harrison, 1997). The first of these is the simplifying assumption commonly adopted when undertaking design of rock engineering structures, whereas the second is the physical nature of the material in which engineering takes place.

Undertaking rock engineering in CHILE rock masses is straightforward: material properties are determined through objective means such as laboratory or field tests undertaken on small scale samples of the rock and used to characterise the variability in the rock mass. Such objectively measured rock mass parameters usually follow a stochastic distribution and can be characterised using an aleatory model (Bedi, 2013). However, the heterogeneity of DIANE rock masses makes it difficult to undertake objective or precise measurements on samples that are representative of the rock mass as a whole. Consequently, geotechnical engineers often rely on empiricism or expert judgement – for example, estimates made by geologists through field observations using various exploration methods such as outcrop, core or tunnel mapping – as well as some objective measurements to determine the values of rock mass parameters for use in design calculations, and this introduces subjectivity and hence epistemic uncertainty (Bedi, 2013).

The combination of objective and subjective assessment of DIANE rock mass parameters means that the total unpredictability in its characterisation results from both epistemic and aleatory components. With reference to the discussion in the preceding section, this interaction of aleatory and epistemic uncertainty introduces a complexity in the selection of characteristic parameter values and consistent application of the partial factor method to characterise, model and propagate the unpredictability when undertaking design by calculation in DIANE rock masses. In the following, it is discussed whether or not partial factor**s** can be calibrated for objectively and subjectively assessed DIANE rock mass parameters.

3.1 *Partial factors for objectively measured parameters*

Many rock mechanics properties have been shown to follow stochastic distributions, i.e. they are aleatory. Well known examples include intact rock strength (Yamaguchi, 1970; Ruffolo & Shakoor, 2009) (Figure 3), the discontinuity spacing (Priest &

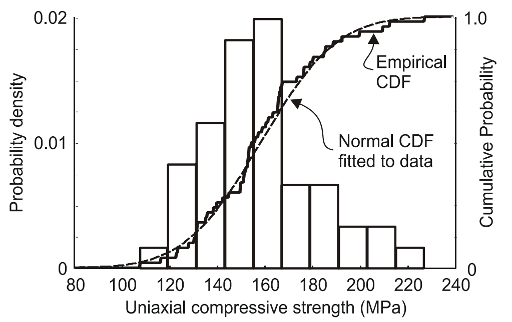

Figure 3: Normal distribution associated with uniaxial compressive strength of intact rock – Milbank granite (data from Ruffolo & Shakoor, 2009)

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Hudson, 1976) and the discontinuity orientation (Priest, 1985). Of these, the intact rock strength is usually determined through uniaxial compressive (UCS) tests on intact rock samples and so the UCS may be well suited for incorporation into the LSD model through development of calibrated partial factors. In fact, as depicted in Table1, EC7 presents a partial factor, γ_{qu}, for unconfined strength of soil that could be similarly be determined for UCS of intact rock.

However, a study using statistical analyses on UCS test data of five different rock types (Ruffolo & Shakoor, 2009) showed that uncertainty in the UCS and hence the number of tests required to make reliable estimates of the mean strength varies with rock type, as shown in Figure 4. In this figure, the degree of anisotropy and heterogeneity in the rock type (sandstone to schist) increases from left to right.

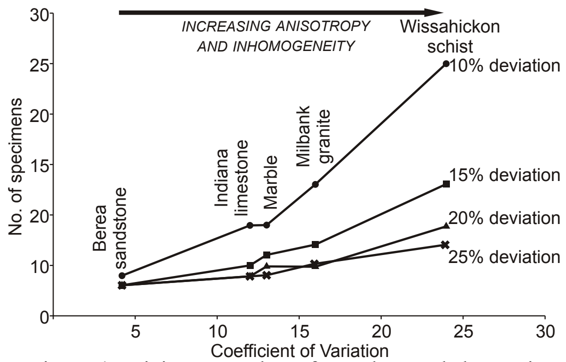

Figure 4: Minimum number of samples needed to estimate the mean unconfined compressive strength (after Ruffolo & Shakoor, 2009).

This suggests that there may be a geological link between the variability of the rock and the number of samples required to select reliable estimates of the characteristic strength, and implies that the minimum number of strength tests required may not be the same for all rock types. If true, this will have important ramifications for the codification of testing requirements in order to characterise rock strength as aleatory and thus the selection of characteristic values for the rock type in question, to which a universal partial factors can be applied.

Another parameter that can be objectively measured, though the measurements are often imprecise or erratic is the coefficient of in-situ stress, k, which defines the ratio of the in-situ horizontal stress (σ_h) to the in-situ vertical stress (σ_v). Figure 5 shows an example of the imprecision in the correlation of the measured in-situ stress ratio with depth, which suggests that k is epistemic. Due to the epistemic nature of the uncertainty stemming from the subjective estimation of a value for a parameter such as k, direct determination of a design value of the action calculated using k would be required in designs to EC7.

Additionally, in many rock engineering computations, k can act as both an action and a resistance and thus contributes towards both the effect of actions and resistance, and so, using the 'single-source principle' a partial factor of unity could be applied to both the weight density and the value k when they are used to calculate both an unfavourable action and a component of the resistance

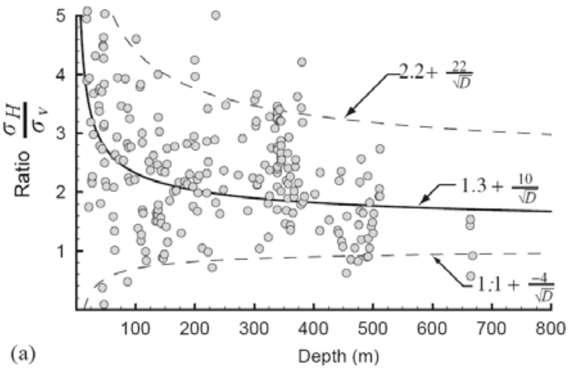

Figure 5: In-situ stress ratios determined from the Scandinavian database (from Martin et al., 2003)

This direct determination of the design values introduces significant epistemic uncertainty due to the subjectivity involved in their estimation. It is this direct estimation of design values which is perceived as inconsistent with the aleatory LSD method on which structural members are designed, and so there appears to be an inconsistency in application of the partial factor method when soil-structure interaction problems involve aleatory parameter values for structural members and epistemic rock mass properties.

3.2 *Partial factors for subjectively estimated parameters*

For rock engineering purposes, a method of quantifying rock surface roughness is the Joint Roughness Coefficient (JRC) (Barton & Choubey, 1977). JRC is arguably the most commonly-used measure of the roughness of rock joint surfaces in current use, and forms an important part of the Barton-Bandis rock joint shear strength criterion (Barton & Bandis, 1990). Despite over three decades of research aimed at developing objective assessment methods for roughness assessment, it is generally undertaken by visual comparison to a series of exemplar profiles, which inevitably introduced epistemic uncertainty due to observer subjectivity.

This is exemplified by the work of Beer *et al*. (2002), which describes the results of an online test of the visual assessment of rock profile roughness in terms of JRC. In this test, individuals involved in geotechnical engineering were asked to visually assess the JRC values of three surface profiles obtained from the same granite block; the results are presented in Figure 6. Through various statistical hypothesis tests, and as can be clearly observed in Figure 6, the authors concluded that the observations could not be defined by any one stochastic function.

It is important to recognise that in the study of Beer *et al*. (2002), the number of estimates of the JRC value was high (122-125), and even for joints in this one rock type no one stochastic distribution could be defined. In general this will not be the case.

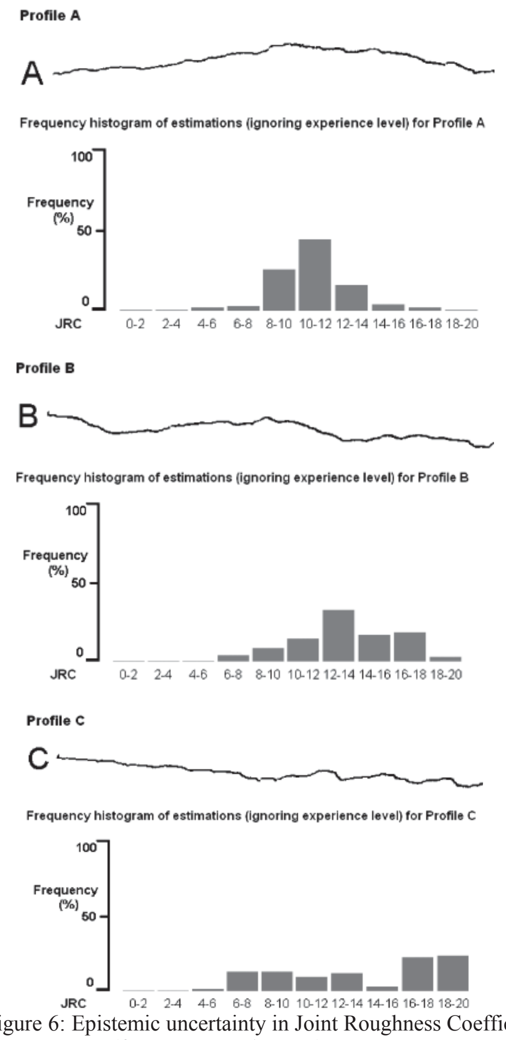

Figure 6: Epistemic uncertainty in Joint Roughness Coefficient (from Beer *et al*., 2002)

For example, in practice a single or small team of design engineers would agree on a value or range of values of JRC to be adopted for design, which introduces a great deal of subjectivity into the characterisation process. This leads to the question: 'how does one provide universal guidance to select characteristic values or partial factors for such DIANE rock mass parameters that require a great deal of subjectivity in their determination?-

4 INADEQUACIES OF LSD APPROACHES TO ROCK ENGINEERING

The concept of LSD was initially developed for engineering with man-made materials (e.g. materials employed in structural engineering) in which the material properties follow an aleatory model (Christian, 2004; Bond & Harris, 2008). The partial factor method, embodied in EC7, is based on the principle that variability in both 'load' (i.e. effect of actions, in Figure 1) and 'resistance' follow known probability density distributions, to which an appropriate prescribed partial factor may then be applied. However, as we have demonstrated so far, for rock engineering designs, where the distribution of load and resistance may be derived from input parameters exhibiting epistemic uncertainty, this assumption may not be valid.

Becker (1996b) has stated that *"to date, only the variation in loads and material strengths have been considered explicitly in reliability based design and LSD using partial factors. The other sources of uncertainty are less amenable to systematic treatment either because the information required to characterise them is lacking, or by their nature, they do not lend themselves readily to statistical analysis"*. Here, in mentioning the lack of knowledge, Becker is referring to epistemic uncertainty, for which it has been shown that other, non-probability based models are more appropriate (Bedi, 2013; Bedi & Harrison, 2013). As such, Becker's statement can be paraphrased as: *'aleatory variability is the basis of the development of LSD, but quantification of DIANE rock mass parameters, which exhibit epistemic uncertainty, in terms of characteristic values and partial factors calibrated from stochastic methods is not valid'*.

Therefore, we suggest that EC7, in its current form, may not be sufficiently developed for application to rock engineering designs. Indeed, where the uncertainty can be shown to be aleatory, EC7 should recommend calibrated partial factors (γ_m in Figure 2) to be applied to statistically determined characteristic values of the rock mass parameter in

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question. Consequently where uncertainty in the parameter values is epistemic, EC7 should explicitly state that the value of γ_M that accounts for unfavourable deviations in rock parameter values due to epistemic uncertainty, or if γ_M is set to unity, the design value of the parameter is chosen directly to provide the required level of safety. As EC7 does not currently provide an LSD model that consistently integrates the partial factors given for aleatory parameter values, such as those in the structural Eurocodes, with its implantation of the LSD method that allows direct determination of epistemic parameter rock mass parameter values when undertaking design by calculation, the question that arises is: how might epistemic uncertainty be defined using alternative models such that it may be incorporated into LSD, and in the interim what approach may be most appropriate when undertaking design by calculation in DIANE rock masses?

4.1 *Interim recommendations for design by calculation*

Whilst LSD principles assume the underlying uncertainty in the processes being modelled to be aleatory, it has been shown that some uncertainty in rock mechanics is epistemic (Bedi, 2013). This suggests that there is a fundamental discrepancy between what rock mechanics is, and what LSD assumes it to be, and this, perhaps, is the reason for anecdotal evidence suggesting that EC7 is difficult to apply to rock engineering designs. In fact, the authors of EC7 recognise this difficulty by stating that design by calculation requires the designer to understand: *"…that knowledge of the ground conditions depends on the extent and quality of the geotechnical investigations. Such knowledge and the control of workmanship are usually more significant to fulfilling the fundamental requirements than is precision in the calculation models and partial factors"*. (EN 1997-2004 §2.4.1(2)P)

In the context of aleatory and epistemic uncertainty, it is the authors' view that this statement refers to the designers' obligations to understand that they may be faced with epistemic uncertainty and so cannot apply the aleatory LSD methodologies defined in structural Eurocodes. As such, the designer must consider the suitability of the current LSD model and partial factors in EC7 given available information available at any particular design stage, in order to be able verify that the occurrence of a limit state is sufficiently unlikely. Should the level of information imply that the LSD model and partial factors are not suitable, perhaps consideration should be given to an alternative approach.

In this regard, there are only a few approaches open to us. Perhaps the trivial approach is to abandon LSD for rock engineering, and continue with the traditional approaches. Whilst appealing, this does nothing to integrate rock engineering and structural designs. Another approach would be to develop methods by which epistemic uncertainty may be approximated as aleatory and thus incorporated in the current LSD paradigm. This may, in the short term, be the most appropriate. Finally, a new LSD paradigm could be developed that encompasses both epistemic uncertainty and aleatory variability. How and whether this might be achieved is not clear.

Perhaps, in the immediate future, the pragmatic approach is as follows:

a) Fully understand which aspects of rock mechanics and rock engineering are genuinely aleatory, and for those develop the LSD model of EN 1990 to extend to such rock engineering parameter values;
b) For those aspects that are extrinsically epistemic (i.e. epistemic simply because sufficient objective information has never been collected to quantify the aleatory characteristics), it is necessary to either ensure the required data are collected, or work to quantify the inherent variability and hence determine how the characteristic values of rock parameters should be selected; and
c) For those aspects that are intrinsically epistemic (they are determined entirely subjectively and thus no amount of data collection would allow one to characterise them as aleatory, an example may be visual estimation of JRC), design strength parameter values should be selected directly, in accordance with EN 1997-1 §2.4.6.2(1)P, rather than being obtained by dividing characteristic values, which are difficult to select, by partial factors, which are difficult to calibrate. However, and as previously stated, this approach is more analogous to the traditional working stress approach rather than the structural LSD model in envisaged in the development of EN 1990.

5 CONCLUSIONS

The discussion presented in this paper has shown that the inherent problem in applying LSD principles to rock engineering is the issue of handling the uncertainty associated with the geotechnical input

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parameters required to characterise fractured rock masses.

The introduction of Eurocode 7 for geotechnical engineering recognises the need for rock engineering design methodology to become consistent with the LSD method set out in EN 1990, in order to produce robust designs with a target level of reliability. This LSD model assumes that both the effect of actions (i.e. loads) on, and resistance of, the structure are aleatory in nature. However, EC7 recognises that the uncertainty in geotechnical material parameter values is often epistemic due to the subjectivity required in their estimation. Hence, while EC7 presents an LSD method that generally involves calculations to verify that the occurrence of an ultimate limit state is sufficiently unlikely with design strength parameters obtained by dividing characteristic parameter values by prescribed partial factors, it also accommodates situations where the parameter values do not follow an aleatory model and for these situations allows the design value to be determined directly, i.e. with a partial factor of unity. This direct estimation of design values which is perceived as inconsistent with the aleatory LSD method on which structural members are designed, and so there appears to be an inconsistency in application of the partial factor method when soil-structure interaction problems involve aleatory parameter values for structural members and epistemic rock mass properties.

It is noted that while some rock mass parameter values may indeed follow an aleatory model, EC7 does not provide any partial factors specifically for them. This may be because the work necessary to develop such partial factors has never been undertaken, and so is required to incorporate such partial factors in future revisions to EC7.

Finally, recognising that some rock mechanics properties are epistemic, it has been shown that at present the pragmatic approach is generally to select the design values of such rock properties directly rather than by the application of partial factors to characteristic values. Whilst this may be perceived as being inconsistent with the aleatory basis and genesis of the LSD method set out in EN 1990, in the short term this appears to be the only means to undertake design by calculation in rock engineering that is compliant with the requirements of EC7.

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How to refine the Observational Method as described in EC7 in applied rock mechanics

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ABSTRACT: The design following EC7 must be based on a combination of calculations, prescriptive measures, experimental model and observational method. The observational method must rely on the best design that can be done in advance taken in account all prevailing uncertainties, aleatoric or epistemic. In rock mechanics, as we don't usually know the state of stress or strain in the rock, active or interactive design shall rely on the deformations measures as well as on good knowledge of the geology and structure of the rock mass to define limit states. The peculiarities due to rock mechanics in the enforcement of the observational method are emphasized. Some directions for active design will be given to better define the method and insure safe design.

1 INTRODUCTION

EN 1997-1 named Eurocode 7 (EC7) CEN (2004) deals with geotechnical design. In this design it is fundamental to check that the structure fulfils relevant criteria. These can be divided in two types, ultimate and serviceability limit states. One or a combination of following shall check the avoidance of exceeding the limit states:

- Use of calculations
- Adoption of prescriptive measures
- Experimental models and load tests
- Observational method

In structural engineering and soil mechanics calculation is the most commonly applied procedure for checking. It requires that the ground conditions are relatively well defined. The adoption of prescriptive measures means that a well-established and proven design is adopted under well-defined ground and loading conditions without calculation. Tests on models or full scale tests can be useful for single and independent structures. The observational method is normally applied when the prediction of the geotechnical behavior is difficult like when the ground conditions are complex or not sufficiently well known.

The objective of this paper is to discuss if the observational method as defined in EC7 can be applied for rock mechanics design and if not suggest necessary changes.

2 ROCK MECHANIC DESIGN

Rock mechanics as well as soil mechanics are the basic scientific subjects of knowledge. They are applied for many different design situations. The problems could be foundation on rock, rock slopes, rock tunnels and other types where rock is an essential part of the structure as counter weight to traction forces. Rock mechanic design is thus an equal part to soil mechanic design in the family of geotechnical design.

Many design situations for rock mechanic problems are characterized by limited information of geology, uncertainties regarding rock mass properties and complex mechanics. The difficulties in determining the geotechnical behavior in rock engineering have been described by many, i.e. Goricki et al. (2004), Schubert (2004), and Palmström & Stille (2006, 2010). Stille & Holmberg (2008) stated that the following issues characterize the situations in rock engineering:

- Difficulties to assess the rock mechanical properties and behavior,
- Difficulties to foresee location of specific geology and rock quality,
- Difficulties to assess the behavior of structural elements, i.e. the proper function of measures undertaken and their interaction with the rock mass and,
- Difficulties to assess the quality of rock support measures undertaken during construction.

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The very central issue is that we cannot as in structural engineering specify the properties of the building material in advance and then produce them or a structure under industrial and fully controlled processes. We cannot either by investigation in advance totally describe the geological conditions or its behavior. Additional information is required that provides us with a better understanding and means to reduce these uncertainties during construction.

Furthermore, the uncertainties are mainly of epistemic nature since it is coming from lack of information and will not reflect any fundamental randomness of the geology and its properties. This is of special importance since it implies that the uncertainties can be reduced by further observation of the geotechnical behavior to such a degree that the design can be verified with acceptable level of safety.

The difficulties in describing the behavior in combination with epistemic uncertainties makes it suitable to apply an observational approach to verify the design and also pursue it during the course of construction.

3 THE OBSERVATIONAL METHOD

Two different approaches for applying an observational approach have been described by Peck (1969). He called them "*best way out*" and "*ab inito*". Both approaches contain active elements in the design work in which results from observations may impose changes of the preliminary design. The intention of EC7 is to be an integral part of the design process from the outset and thus "*ab inito*".

Checking that the proposed design is adequate and safe is also sometimes called observation method. This is, however, a passive design and is not meant to contain any active element and changes of proposed design. This should not be called observational method since it has no active part containing contingency actions involved in the process, only checking the observed structure. The Designer's guide to the Euro code, Frank et al. (2004), says:

"*EN1997-1 introduces design by the "observational method", in which the design is reviewed in a planned manner during the course of the construction and in response to the monitored performance of the structure. The essence of the method is a precise plan of monitoring and of the action to be taken as a result of the observations. The minimum requirements to be met before and during construction are indicated.*"

The indicated five requirements of the Observational Method, stated in Euro code EN 1997-1:2005 section 2.7, are the following:

1. *Acceptable limits of behavior shall be established.*
2. *The range of possible behavior shall be assessed and it shall be shown that there is an acceptable probability that the actual behavior will be within the acceptable limits.*
3. *A plan for monitoring the behavior shall be devised, which will reveal whether the actual behavior lies within the acceptable limits. The monitoring shall make this clear at a sufficiently early stage, and with sufficiently short intervals to allow contingency actions to be undertaken successfully.*
4. *The response time of the monitoring and the procedures for analyzing the results shall be sufficiently rapid in relation to the possible evolution of the system.*
5. *A plan of contingency actions shall be devised which may be adopted if the monitoring reveals behavior outside acceptable limits.*

Further it is also stated in Designer's guide Frank et al (2004) following:

"*EN 1997-1 leaves open the manner in which safety is introduced in the supporting calculations. This might be done by reduced value of partial factors or through a less cautious selection of the characteristic values of the soil properties. The way to introduce safety into design when using the observational method is best evaluated for each individual project, depending on the perceived reasons for using factors of safety (uncertainty or displacement control) and on the consequences of failure*".

The reference to supporting calculations indicates that the code has been written with the soil mechanics application at sight. In rock mechanics supporting measures are also often prescriptive.

4 ROCK ENGINEERING ISSUES RELATED TO THE OBSERVATIONAL METHOD

4.1 *Rock mechanical aspects*

The difficulties related to rock engineering design as discussed above make it suitable and necessary to apply an observational approach to verify and pursue the design during construction. The observational method has been used since long and will be used in the future for solving many rock mechanical design situations.

The Observational Method as stated in EC7 needs to be further refined in order to be fully applicable for rock mechanical design. Some suggestions are given below.

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The concept of Geotechnical Category has been introduced to establish design requirements. In principle Category 1 is used when the risk is negligible and category 3 for very large and unusual structures or involving abnormal risks. Observational Method is connected to the cases when prediction of the geotechnical behavior is difficult. This can be the case when the geology has not been possible to establish in advance or the mechanical response is unknown. Therefore the observational method in an extended version ought to be applied in both category 1, 2 and 3 and in combination with calculations or adoption of prescriptive measures. The application of observational Method should thus be independent to the risk situation even if the application with rock mechanical measurements will probably be mostly used in category 3.

It is not directly stated in the Euro code that there are requirements according to practical application (Stille & Holmberg 2008, 2010). They are as follows:

- The monitored behaviour must be critical for the safety of the structure
- The monitored behaviour must be related to acceptable limits

The observational method involves increased commitments for the project organization. It must be sure that observation and measurements contribute towards reducing uncertainties. This means that:

- The observational method must be an integral part of the design process from the outset ("*ab initio*").
- There must be a consensus by the parties that the final design cannot be established before the completion of the work
- The observational method must also have a clear role to play in the execution phase and be integrated into the production process.

These are important and in many ways decisive factors. The contractual situation must permit a rational and fair application otherwise the integration into the production can be hampered.

EN 1997-1 does not give any guidance about the relationship between acceptable limits of behavior and limit states. It also leaves open the manner in which safety issues are introduced in the supporting calculations. It also does not discuss at which probability of failure the observed structure should fulfill.

The acceptable probability to stay within the acceptable limits is not discussed. This level is an issue for the risk management of the project and should reflect the preparedness to use the contingency actions. This ought to be described in the code.

The design to be verified during construction can be called *à priori* design. It has to be carried out in advance based on available geological information and calculations or prescriptive measures. The complexity of the geology in combination with limitation of information implies that is not possible or practical to foresee all possible behavior or either contingency measures. There is always a possibility that other geological structures and ground behavior than predicted will be encountered during excavation. This means that we need an open group of contingency actions to be exhaustive for such unforeseen geological structures or behavior.

4.2 *Rock mechanical applications*

The characteristic issues of rock mechanics design, as discussed above, put special demands on the application of observational method.

Geological follow up is a prerequisite in rock engineering. The nature of this follow up is to verify that the assumption on which the design is based on is adequate. This should be regarded as a part of the monitoring plan and based on visual observations.

A common design situation is the application of support or excavation classes based on observation of rock mass quality after each round of excavation. Basically it is a kind of Observational Method with visual observation as the monitoring devices. The predefined contingency actions are related to the excavation or support classes. The verification of the support measures for a defined rock mass quality can be based on calculations or prescriptive measures.

Difficulties to assess the rock mechanic properties, failure mechanism and behavior of the structural elements and their interaction with the rock mass will complicate the design. Therefore, monitoring of tunnels and other types of underground openings is frequently used in rock engineering. High rock slopes will belong to this category. Standard as EN ISO 18674 (2013) "*Geotechnical monitoring by filed instrumentation*" is applicable. However, to get the best results the observations must be an active part in the design and construction work. Threshold and limit values have to be defined in advance. Since it is deformations that are observed, the limit states function must be formulated in deformations or strains. Development of such failure criterion is a challenge for rock mechanics society. The requirements according to EN 1997-1 can then be applied.

Uncertainties to assess the quality of undertaken rock support are another issue, which can be addressed by application of the Observational Method. In principle the bearing capacity of the rock support elements cannot be fully specified in advance. The achieved quality has to be checked and the design of the support elements has to be adapted to the observations. The contingency actions can be specified in advance.

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5 CONCLUSION

The epistemic nature of the uncertainties in rock mechanics makes it suitably to apply an observational approach. The uncertainties can then be reduced so that an acceptable level of safety can be achieved. An observational approach is therefore a natural and necessary tool for rock mechanics design and is used frequently.

The requirements of the Observational Method as stated in EC7 have to be further elaborated in order to be fully applicable for rock mechanics design and construction. The design to be verified during construction can be based on calculations or prescriptive measures. A general interpretation of possible behavior and monitoring is required in order to cover visual observations of geology and behavior. Contingency actions shall be adapted to the monitoring results but also contain open actions to be fully exhaustive.

It is anticipated that the Observational Method would be applied to design every rock project belonging to Geotechnical Category 3 as a part of an interactive design during construction. A more elaborated definition of Observational Method containing visual observations will make the method applicable also for Category 1 and 2.

The improvements or adaptations most sought-after for the observational method in rock mechanics are:

- This method must be specified as an *ab initio* method due to the uncertainties pertaining to the rock mass,
- The acceptable limits of behavior must be related to what can be observed and also to the acceptable probability of failure of the structure. This has normally to be done in terms of deformations or strains.
- Failure criterion in terms of deformations or strain must be developed for using the Observational Method.

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Reliability against translational slip of rock slopes designed according to Eurocode 7

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ABSTRACT: A probabilistic analysis is applied to rock slope stability with an example case of a rock slope with translational potential failure mode. The capacity and the demand are represented as independent triangular random variables. The distribution of the factor of safety is found analytically and its reliability measures are evaluated, allowing for decisions to be taken in terms of risk and reliability. The same slope is examined in its limit state by applying the partial factors approach of Eurocode 7. The Eurocode design is compared to the traditional factor of safety and probability of failure.

1 INTRODUCTION

Numerous uncertainties, arising from natural variability in space and time, experimental errors, imprecise information, insufficient knowledge, up-scaling, simplistic assumptions, etc., are pervasive in rock engineering. These are commonly taken into account, indirectly, in the traditional factor of safety design, where acceptable safety factors are selected through experience or regulation, depending on application and its importance (e.g. Priest & Brown 1983).

Some drawbacks may be recognized in this approach (Yucemen et al. 1973). The same value of the safety factor is adopted (or imposed) for a particular type of application, regardless of the degree of uncertainty involved (Duncan 2000), the risk level associated with it, or the amount and quality of information available before and acquired during construction. The type of uncertainty is also relevant to rock engineering (Bedi & Harrison 2013) and its assessment essential for reliable design (Bagheri & Stille 2011).

Limit States Design (LSD) in Eurocode 7 (CEN 2004) introduced several changes to the previous geotechnical design practice. Verification at the ultimate limit state requires that the design actions, increased to reflect a low probability of occurrence, be lower than the design resistances, which have been factored down to reflect prescribed (or intended) probabilities of being exceeded. The values of the partial factors of the characteristic actions and material parameters are largely associated to variability and other uncertainties and therefore the partial factors approach may be considered as a form of reliability based design, although for complex systems the relation of the partial factors to the intended failure probabilities may be somewhat difficult to observe. Calibration of the partial factor design equations has been primarily based on method *a* of Figure 1, where the relation between the various calibration methods considered by EN 1900 (CEN, 2002) is presented. There, the probabilistic calibration procedures are divided into Level II reliability and Level III probabilistic methods.

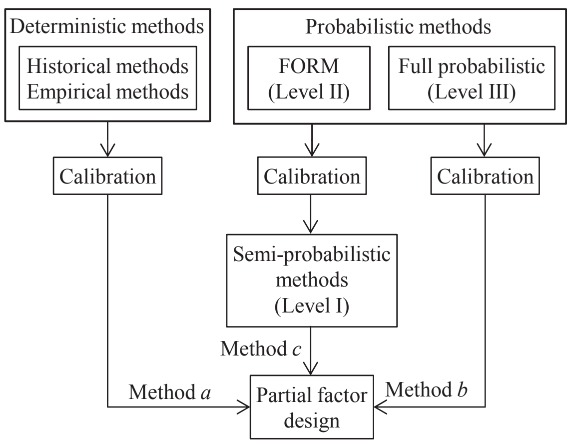

Figure 1. Overview of reliability methods considered in EN1990 (CEN 2002).

Reliability based design allows for accounting of the pertinent risks by computing the probability of occurrence of events about which there is only partial information. Thus, probabilities of failure in rock engineering, such as of rock wedges in rock

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slopes (Jimenez & Sitar 2007), may be computed systematically and quantitatively.

Probabilistic design deals primarily with the consideration of the effects of random variability. When using a probabilistic approach to design, each variable is viewed as a probability distribution. In many rock mechanics cases, particular rock properties have to be randomized in order to estimate the distribution of the resistance of a system (capacity), and similarly, the demand is the resultant of many uncertain components of the system under consideration. Nevertheless, there are also many cases where independent probability distributions may be assigned directly to capacity or demand.

2 CLOSED FORM SOLUTION

In the probabilistic formulation the reliability of an engineering system is determined by comparing the resistance of the system (Capacity, *C*) to the applied load (Demand, *D*), and failure is assumed to occur when *D* exceeds *C*. The capacity and demand may be assumed as random variables with assignable or calculated probability densities, and the reliability analysis is formed either in the form of the safety margin *M*, defined as the difference between capacity *C* and demand *D*, or the factor of safety *fs*, defined as the ratio of the capacity upon the demand, i.e.:

$$fs = C/D\ ;\ M = C - D = D \cdot (fs - 1) \tag{1}$$

By definition *fs* is also a random variable with cumulative distribution function (CDF) F_{fs}. For $C>0$ and $D>0$, F_{fs} is defined as:

$$F_{fs}(fs) = P(C/D < fs) \tag{2}$$

2.1 *Distribution of fs*

The capacity *C* and the demand *D* may be considered as statistically independent triangular random variables, with probability density functions (PDF), f_C and f_D, respectively (Figure 2). The triangular distribution is the simplest form of a two-segment piecewise linear distribution, consisting of two opposite leaning ramp functions. It can be symmetric (Figure 2a) or skewed (Figure 2b), either to the left or to the right. If the first (ascending) ramp segments are denoted by $i=j=1$ and the second (descending) by $i=j=2$, respectively, then the CDF of *fs* may be calculated by (Sofianos et al. 2013):

$$F_{fs}(fs) = \begin{cases} 0, & fs < \dfrac{L_{C1}}{U_{D2}} \\ \displaystyle\sum_{i=1}^{2}\sum_{j=1}^{2} F_{fs,ij}(fs), & \dfrac{L_{C1}}{U_{D2}} \le fs \le \dfrac{U_{C2}}{L_{D1}} \\ 1, & fs > \dfrac{U_{C2}}{L_{D1}} \end{cases} \tag{3}$$

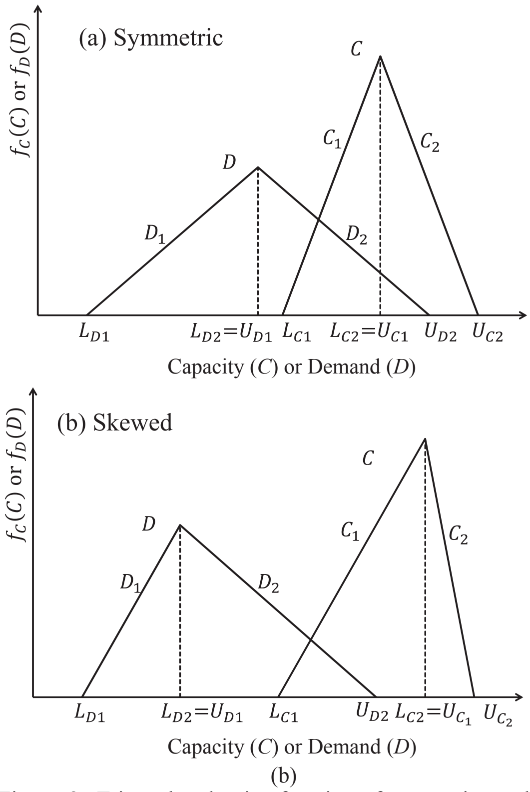

Figure 2. Triangular density functions for capacity and demand: (a) symmetric, (b) skewed.

$F_{fs,ij}(fs)$ is the probability of the combination of the *i* capacity segment with the *j* demand one, and is calculated by:

$$F_{fs,ij}(fs) = w_{Ci}w_{Dj}\begin{cases} 0, & fs < \dfrac{L_{Ci}}{U_{Dj}} \\ M_{ij} - \dfrac{N_{ij}}{fs^2}, & \dfrac{L_{Ci}}{U_{Dj}} \le fs \le \dfrac{U_{Ci}}{L_{Dj}} \\ 1, & fs > \dfrac{U_{Ci}}{L_{Dj}} \end{cases} \tag{4}$$

$$w_{Ci=}\frac{(U_{Ci} - L_{Ci})}{(U_{C2} - L_{C1})}\ ;\ w_{Dj=}\frac{(U_{Dj} - L_{Dj})}{(U_{D2} - L_{D1})} \tag{5a}$$

$$M_{ij} = h_{Ci}\frac{(R_{Uij} - L_{Ci})}{(U_{Ci} - L_{Ci})} + (1 - h_{Ci})\frac{(R_{Uij} - L_{Ci})^2}{(U_{Ci} - L_{Ci})^2}$$
$$R_{Uij} = U_{Dj}\min\left\{\frac{U_{Ci}}{U_{Dj}}, fs\right\};\ R_{Lij} = L_{Dj}\max\left\{\frac{L_{Ci}}{L_{Dj}}, fs\right\} \tag{5b}$$

$$N_{ij} = K_{4ij}Q_{4ij} + [K_{2ij}fs + K_{3ij} + K_{4ij}(L_{Dj}fs - L_{Ci})]Q_{3ij} + [K_{1ij} + K_{2ij}(L_{Dj}fs - L_{Ci})]fs\,Q_{2ij} \tag{5c}$$

$$Q_{kij} = \frac{(R_{Uij} - L_{Dj}fs)^k - (R_{Lij} - L_{Dj}fs)^k}{k}\quad ;k = 2,3,4 \tag{5d}$$

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$$
\begin{aligned}
K_{1ij} &= h_{Ci}\, h_{Dj} \left(U_{Dj} - L_{Dj}\right)^{-1} \left(U_{Ci} - L_{Ci}\right)^{-1} \\
K_{2ij} &= 2\, h_{Dj} \left(1 - h_{Ci}\right)\left(U_{Dj} - L_{Dj}\right)^{-1} \left(U_{Ci} - L_{Ci}\right)^{-2} \\
K_{3ij} &= h_{Ci} \left(1 - h_{Dj}\right)\left(U_{Dj} - L_{Dj}\right)^{-2} \left(U_{Ci} - L_{Ci}\right)^{-1} \\
K_{4ij} &= 2\left(1 - h_{Ci}\right)\left(1 - h_{Dj}\right)\left(U_{Dj} - L_{Dj}\right)^{-2} \left(U_{Ci} - L_{Ci}\right)^{-2}
\end{aligned} \tag{5e}
$$

$h_{C1}= h_{D1}=0$, $h_{C2}= h_{D2}=2$.

The PDF of *fs* is the derivative of the CDF. Particular cases of the triangular distribution are the single-segment left or right leaning ramp density functions. The ramp function is a simplification of the linear distribution, as is also the uniform density function (Nomikos & Sofianos 2011).

2.2 *Reliability parameters*

The mean and the variance of *fs* are given by (Sofianos et al. 2013)

$$\mu_{fs} = U_{C2}/L_{D1} - \int_{L_{C1}/U_{D2}}^{U_{C2}/L_{D1}} F_{fs} dfs \tag{6}$$

$$\sigma_{fs}^2 = \left(U_{C2}/L_{D1}\right)^2 - 2 \int_{L_{C1}/U_{D2}}^{U_{C2}/L_{D1}} fs F_{fs} dfs - \mu_{fs}^2 \tag{7}$$

which may be evaluated numerically.

The probability of failure $P(fs \leq 1)$ can be calculated from the CDF of *fs* (eq. (3)) for $fs=1$.

3 EXAMPLE

To illustrate the various approaches of deterministic, partial factors of EC7 and probabilistic methods, the case of the hypothetical rock slope shown in Figure 3 will be used. Such example is commonly used in rock engineering literature to illustrate deterministic, probabilistic and reliability approaches of rock slope stability design (Nilsen 1999, Bedi & Harrison 2013). The potential stability problem is a plane failure where a potentially unstable rock block rests on an inclined weakness plane dipping out of the slope face. For failure to occur, the restraint to sliding should have been overcome along both the weakness plane and the lateral sides of the block. In hard rocks, the lateral release may be provided by other discontinuities (joints) existing transverse to the crest of the slope (Goodman 1989).

The forces that enter into stability calculations include the self-weight of the block, the water forces, the earthquake forces and any support forces. These should be resolved into components parallel and normal to the sliding plane in order to calculate the resisting force F_R and the driving force F_D acting per unit width at a certain section of the slope. The stability of the slope at that particular section is analyzed by comparing F_R and F_D and the condition for limiting equilibrium is reached when F_D equals F_R.

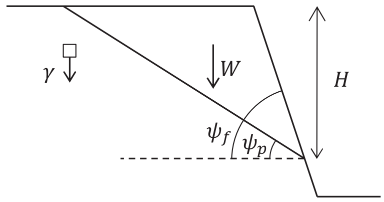

Figure 3. A typical plane sliding stability problem of a rock slope.

3.1 *Deterministic analysis*

Under gravity alone (i.e. a dry slope and no earthquake load considered) the deterministic factor of safety *FS* is:

$$FS = \frac{\tan\phi_a}{\tan\psi_p} \tag{8}$$

where ϕ_a is the active friction angle and ψ_p is the dip of the potential sliding surface.

If the sliding surface is a single, continuous, unweathered, smooth and planar rock joint, then no asperity component of the shear strength may be assumed. The active friction angle will be equal to the basic friction angle ϕ_b and the stability of the slope requires that $\phi_b > \psi_p$.

Normally, real rock joints are non-planar and have non-linear peak shear strength – normal stress criterion, which may be expressed by the scale-effect corrected form of the Barton (1973) empirical strength criterion:

$$\tau = \sigma_n \tan\left(\phi_r + JRC_n \log\frac{JCS_n}{\sigma_n}\right) = \sigma_n \tan\left(\phi_r + i\right) \tag{9}$$

ϕ_r is the residual friction angle (Barton and Choubey 1977) and JRC_n, JCS_n are the scale-dependent joint roughness coefficient and joint wall compressive strength respectively (Barton and Bandis 1982). In that case, the active friction angle ϕ_a on the potential sliding plane is strongly dependent on the normal stress and the friction parameters adopted for design should be adjusted to the actual normal stress level (Nilsen 1985).

Let us assume (for illustration purposes) for the slope of Figure 3 that σ_n is constant along the potential sliding plane. Then:

$$\sigma_n = W \sin\psi_p \cos\psi_p / H \tag{10}$$

W is the weight of the potentially unstable block. It is calculated as:

$$W = \gamma H^2 \left(\cot\psi_p - \cot\psi_f\right)/2 \tag{11}$$

Substituting (11) in (10):

$$\sigma_n = \gamma H \sin\psi_p \cos\psi_p \left(\cot\psi_p - \cot\psi_f\right)/2 \tag{12}$$

Figure 4 shows the variation of the normalized normal stress $\sigma_n/\gamma H$ acting on the potential sliding plane, for slope angle ψ_f=75°, as ψ_p varies from 30° to 60° and the associated variation of the roughness component i of the active friction angle for JRC_n=10, JCS_n=100 MPa, γ=30 kN/m³ and H=20 m. As shown, despite the large variation of $\sigma_n/\gamma H$ the roughness component only varies as much as a few degrees, indicating the large active friction angle for such a low stress environment.

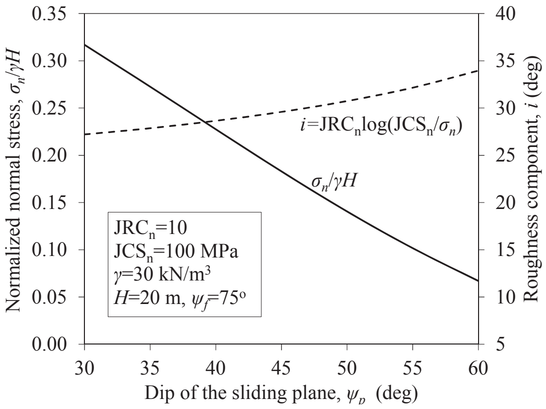

Figure 4. Variation of $\sigma_n/\gamma H$ and i the normalized normal stress and the roughness component of the active friction angle.

In Table 1, the deterministic factor of safety is evaluated for the rock slope of Figure 3 with H=20 m, ψ_f=75°, ψ_p=35°. Two cases of rock joints are examined: an idealized unweathered smooth planar joint with ϕ_b=40° and a rough joint with ϕ_r=30°, JRC_n,=10 and JCS_n=100 MPa.

Table 1. Deterministic factor of safety for the rock slope of Figure 3.

	Smooth planar joint	Rough joint
σ_n (MPa)		0.164
i (deg)		27.86°
ϕ_a (deg)	40°	57.86°
FS	1.2	2.27

These safety factors may or may not be acceptable depending on the geotechnical situation and failure consequences. In open-pit mining some degree of slope instability may be expected if not compromising safety, including unacceptable injuries or fatalities to personnel, and production; the acceptability criteria are largely site-specific. A wide range of suggested minimum acceptable factors of safety is found in the literature (Duzgun et al. 1998) although most recommendations are based on targeted probabilities of failure that may be associated with factors of safety by applying a probabilistic analysis.

3.2 *Partial factors of EC7*

According to Eurocode 7, the design effects of actions E_d should not exceed the corresponding design resistance to those actions R_d, i.e. $E_d \le R_d$. For the particular rock slope example under gravity alone, the design actions and the design resistance are:

$$E_d = \gamma_G \cdot W \sin\psi_p \tag{13}$$

$$R_d = \gamma_{G,fav} \cdot W \cdot \cos\psi_p \cdot \left(\tan\phi_{ak} / \gamma_\phi\right) / \gamma_{\mathrm{Re}} \tag{14}$$

ϕ_{ak} is the characteristic value of the active friction angle. In deriving the above expressions distinction has been made for the partial factors to the weight of the potentially sliding rock block (characteristic action) whether its effect is favourable ($\gamma_{G,fav}$) or unfavourable (γ_G). EC7 requires different partial factors to be applied to unfavourable and favourable actions. For rock slope stability this is a particular issue as the weight of the rock is the cause of the potential loss of stability but also contributes to the resistance (Bond & Harris 2006).

Three design approaches (Table 2) are foreseen in EC7. The DA 3 is the national choice for Greece in checking the overall stability of the geotechnical (GEO) ultimate limit states of natural or engineered slopes for persistent and transient design situations.

Table 2. Partial factors used in the various Design Approaches (DA) and/or combinations (Comb) of EC7.

	DA1		DA2	DA3
	Comb 1	Comb 2		
γ_G	1.35	1.0	1.35	1.0
$\gamma_{G,fav}$	1.0	1.0	1.0	1.0
γ_ϕ	1.0	1.25	1.0	1.25
γ_{Re}	1.1	1.0	1.1	1.0

The “degree of utilization” (or “utilization factor”) is defined as the ratio of the effect of actions to its corresponding resistance (Bond & Harris 2006), i.e. $\Lambda = E_d/R_d$. For the rock slope example of Figure 2:

$$\Lambda = \gamma_\phi \cdot \gamma_{\mathrm{Re}} \left(\gamma_G / \gamma_{G,fav}\right) \cdot \left(\tan\psi_p / \tan\phi_{ak}\right) \tag{15}$$

The design is acceptable if the degree of utilization is less than 1. Thus, by using the DA 3 of EC7, the limit state is verified for values of $\phi_{ak} > 41°$.

3.3 *Probabilistic analysis*

The probabilistic analysis of section 2 requires that the capacity and demand random variables be independent. If the potential sliding surface of Figure 3 is a single smooth planar joint, the active friction angle ϕ_a, which is equal to ϕ_b if the joint is also unweathered, is independent from the normal stress acting on the sliding plane. The capacity C is the tangent of ϕ_a and it may be considered as statistically independent from the demand D, which is the tangent of ψ_p.

As with any other rock mechanical parameter, a natural variability of ϕ_b may be expected, as revealed by the experimental results of Alejano et al. (2012). Assuming for the capacity a symmetric triangular distribution, with mean value M_C and coefficient of variation CV_C, then the limits of C are evaluated as:

$$L_C = M_C \cdot \left(1-\sqrt{6}\cdot CV_C\right);\, U_C = M_C \cdot \left(1+\sqrt{6}\cdot CV_C\right) \quad (16)$$

The limits of the demand triangular random variable may be assigned directly by assuming upper and lower bounds for the dip of the sliding plane ψ_p=35°±3°. A slightly skewed probability density function of D is produced, with support [L_D=tan32°,U_D=tan38°], mode M_D=tan35° and skewness a_{3C} given by (Kotz & van Dorp 2004):

$$a_{3C} = \frac{\sqrt{2}(1-2\theta)(2-\theta)(1-\theta)}{5\{1-\theta(1-\theta)\}^{3/2}} = 0.013 \quad (14)$$

where θ=(M_D-L_D)/(U_D-L_D)=0.48. The limits of the segments of C and D are evaluated in Table 3 for a mean value of the capacity M_C=tan40° and a coefficient of variation CV_C=0.05.

Table 3. Limits of the segments of capacity and demand for M_C=tan40°, CV_C=0.05, ψ_p=35°±3°.

$L_C=L_{C1}$=0.736	$M_C=U_{C1}=L_{C2}$=0.839	$U_C=U_{C2}$=0.942
$L_D=L_{D1}$=0.625	$M_D=U_{D1}=L_{D2}$=0.700	$U_D=U_{D2}$=0.781

The distribution of the factor of safety, for the limits shown in Table 3, is evaluated from equations (3) to (5); it is plotted in Figure 5 along with the density function of *fs*. A probability of failure P_f=0.0025 is evaluated from eq. (3) for *fs*=1. The mean and the variance of *fs* are evaluated from eqs. (6) and (7): μ_{fs}=1.1975 and σ^2_{fs}=0.0811.

For comparison, the results of a probabilistic numerical simulation by using the Latin Hypercube sampling technique, represented by the histogram, are also plotted in Figure 5; practically identical results may be observed.

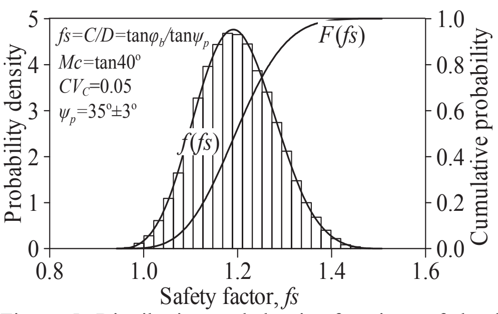

Figure 5. Distribution and density functions of the factor of safety for the rock slope example of Figure 3.

Equations (3) through (5) may be used for the parametric evaluation of the probability of failure P_f and for μ_{fs}, as the friction angle adopted to evaluate the mode of C (M_C=tanϕ_b) varies from 37° to 42° degrees. This parametric analysis is shown by the diagram of Figure 6, where the variation of μ_{fs} is plotted in the left hand vertical axis and the variation of P_f in the right hand vertical axis. The linear increase of μ_{fs} with the increase of ϕ_b and the rapid decrease of P_f for ϕ_b >40° may be observed. In the same diagram, the variation of Λ for ϕ_{ak} varying from 37° to 42° is plotted. It is observed that values of ϕ_{ak} larger than 41° are associated with very low probabilities of failure, as calculated by the probabilistic analysis.

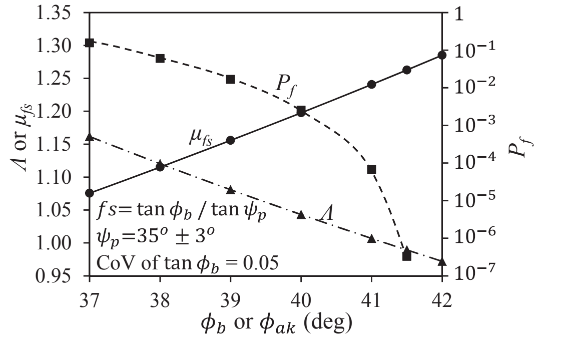

Figure 6. Variation of P_f and μ_{fs}, as the friction angle adopted to evaluate the mode of C (M_C=tanϕ_b) varies from 37° to 42°.

In Figure 7, a parametric evaluation of P_f with the coefficient of variation of C is shown for M_C=tan40° and M_C=tan41°. It is observed, that P_f is increased dramatically with the increase of the coefficient of variation; this is not addressed by the partial factors method of EC7.

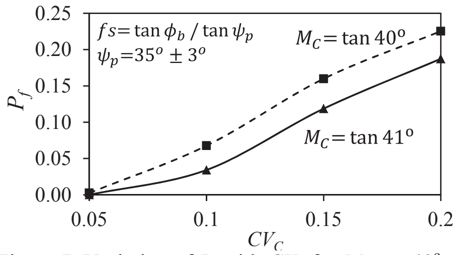

Figure 7. Variation of P_f with CV_C for M_C=tan40°. and Mc

The parametric evaluations of Figures 6 and 7 highlight the importance of ϕ_b for rock slope stability that has also been observed experimentally and numerically (Alejano et al. 2011).

In the case that the sliding plane is a single rough joint, then the active friction angle being dependent on the normal stress is also dependent on the inclination of the sliding plane. Therefore the capacity and the demand are not statistically independent random variables, and the probabilistic analysis of Section 2 may not be accurate.

4 DISCUSSION-CONCLUSIONS

A simple example of a planar translational failure is used to illustrate the application of traditional, partial factors and probabilistic methods of rock slope design. All methods yield measures of the slope stability that should be evaluated in practice

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for any site-specific situation. A factor of safety equal to 1.2 (>1.0) is evaluated for a basic friction angle of 40^{o}, indicating a stable slope, without however evaluating any reliability measure. The application of the partial factors method yields a required basic friction angle larger than 41^{o} for the verification of the overall stability of the slope. Although this might seem as a small difference, the application of the probabilistic analysis indicates a dramatic reduction of the probability of failure with only a few degrees increase of the basic friction angle. For low values of the coefficient of variation the probabilities of failure are low but they are significantly increased with the increase of the dispersion of the capacity distribution.

The traditional deterministic factor of safety may not easily be related to an imposed risk. As a result, increased factors of safety may be required to address an increased degree of uncertainty.

The partial factors method of EC 7 employs a standard procedure that incorporates to some degree the variability of the material parameters and model uncertainties. The allowance of more than one design approaches provides some flexibility for design and the ability for adopting the principles of limit states design to the experiences of the national geotechnical communities. It is normally associated with low probability of failure, which however is difficult to be estimated for complex design situations.

The probabilistic analysis presented herein employs a clear and robust methodology, from a statistical point of view, that only makes use of explicit definitions for the reliability measures; such is the probability of failure. The adoption of triangular distributions for statistically independent capacity and the demand may be reasonable for the data limited problems encountered in rock engineering, particularly since the important features of skewness and bounded physical support are preserved by the triangular density functions. Therefore, they may be used, when applicable, instead of the normal distribution or any asymmetrical smooth distributions. The selection is not restrictive, since the triangular distribution is a special form of a piecewise-linear density function, and can take the form of a trapezoidal or polygonal distribution. This is particularly important for systems with probabilities of failure that are controlled by the tail of the distribution, where the piecewise linear function may follow, by increasing the number of linear segments, as closely as desired the available data.

For the rock slope example examined here the probabilistic analysis yields the measures of reliability in terms of the probability of failure. Depending on the failure consequences, the slope may be considered safe if the probability of sliding is within prescribed limits, either provided by literature or imposed by regulations. Such closed form solutions may help, as a next step, for more reasonable Eurocode design taking into account data reliability.

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Towards an improved observational method

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ABSTRACT: The observational method is today an accepted method in Eurocode 7 for design of geotechnical structures. However, case studies with formal application of its principles are still rare. One reason could be that the method to some extent is considered complex and associated with low safety margins. In fact, the Eurocode does not give any reference to how the safety of the completed structure can be assured. This paper strives to open up a discussion on how the observational method can be improved by including a requirement for a safety margin of the completed structure. A methodology is outlined and illustrated with a simple calculation example analysing the safety of a square rock pillar. Lastly, the compatibility with the observational method is discussed.

1 INTRODUCTION

When designing underground excavations in rock, the engineer must inevitably consider the uncertainties related to the properties of the rock in one way or another. A common approach is to be conservative in the design, but this is not always economically sound or possible in practice. For situations when it is difficult to predict the geotechnical behaviour, Eurocode 7 (CEN 2004) suggests applying the observational method. This method was first outlined by Terzaghi and later defined by Peck (1969).

When applying the observational method, a preliminary design is first prepared based on any available knowledge about the conditions and properties at the site. During the construction phase, the behaviour of the structure is observed. The essence of the observational method is to establish plans in advance, which infallibly put necessary contingency actions into operation if the behaviour is unacceptable. These measures accommodate the structure to the actual conditions at the site. The crucial challenge is to relate a measurable parameter to the structural behaviour and define the acceptable range of the parameter, so that the structure still meets all requirements even after the design has been altered by contingency actions. To facilitate an economically sound construction, the preliminary design must in addition be made so that contingency actions are sufficiently unlikely to be needed.

Case studies describing and discussing the formal application of the observational method as defined in the Eurocode are still rare (Spross & Larsson, in press), even though the method at a glance can seem straightforward. It can however be argued that less strictly defined “observational approaches” sometimes are used in practice. Even so, concerns regarding low safety margins have been put forward, as reported by Powderham (2002). The concerns are not surprising; the benefit of the observational method is to permit less conservative designs than other approaches, leaving to the designing engineer to interpret how to make the best use of the method. However, this benefit is also one of the major drawbacks, as the available guidelines (Frank et al. 2004) give virtually no guidance at all regarding suitable safety requirements for the structure. This implies that the safety margin of the completed structure is at best arbitrary, but possibly totally unknown.

In this paper, we address this issue by suggesting a methodology based on Bayesian statistics for how to assess the final safety margin of a geotechnical structure that has been constructed with the observational method. Herewith, we strive to open up a discussion for an improved definition of the observational method in Eurocode 7 that includes a requirement for a safety margin of the completed structure. The methodology is illustrated with a simplified calculation example, in which observations of the structural behaviour of a rock pillar are used in a safety assessment. In the end, the compatibility with the framework of the observational method is discussed.

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2 INCLUDING MEASUREMENTS IN SAFETY ASSESSMENTS

2.1 *The Bayesian approach*

When applying the observational method, the measurements will contribute with geotechnical information that preferably should be taken into account in assessments of the structural safety. This suggests the use of a Bayesian updating procedure, which sequentially reduces the parameter uncertainty when new data is available. With a Bayesian approach, one expresses the degree of belief that something will happen, instead of interpreting probability as strictly frequentistic, see e.g. Christian (2004). The Bayesian interpretation of statistics is advantageous especially for cases that do not admit any trials to reveal the underlying probability.

To make use of the reduced uncertainty, it is necessary to apply a probabilistic approach to assess the structural safety. Common methods are for example the first-order reliability method (FORM) and Monte Carlo simulations, which both are described in most textbooks on structural reliability.

2.2 *General procedure*

The general procedure suggested in this paper begins with making a first, crude estimation of the possible range and distribution of a relevant parameter for the analysed failure mode. Bayesians call this the prior distribution of the parameter (marked $'$). The prior is then updated with measurements, forming the revised posterior distribution (marked $''$), which takes all available information into account. This implies that sequentially better assessments of the structural safety can be made during construction. The Bayesian approach is more extensively described in Ang & Tang (2007), among others, and was previously discussed in relation to the observational method in Stille & Holmberg (2008) and Stille & Holmberg (2010). Originally, it is based on Bayes' theorem:

$$P(E|A) = \frac{P(A|E)P(E)}{P(A)} \tag{1}$$

where $P(E|A)$ is the posterior distribution of E given the new information A, $P(A|E)$ is the conditional probability of obtaining the result A given the occurrence of E, $P(E)$ is the prior distribution of E, and $P(A)$ is the probability of obtaining the new information A.

2.3 *Sources of uncertainty*

The updating procedure can be interpreted such that even though it is known that the inherent variability of a parameter X is fairly small, there are still other sources of uncertainty that should be taken into account. Any pre-investigations or other prior knowledge might indicate a wide range, but with some measurements it might be possible to narrow down this uncertainty significantly.

In other words, the total uncertainty about a parameter estimated from measurements can be divided into several components in terms of variances (Baecher & Ladd 1997, Müller et al. in press):

$$\sigma_{tot}^2 = \sigma_{inh}^2 + \sigma_{\mu}^2 + \sigma_{m.e.}^2 + \sigma_{tr}^2 + \xi \tag{2}$$

where σ_{inh} is the inherent variability, σ_{μ} is the uncertainty related to the estimation of the mean of the parameter, $\sigma_{m.e.}$ is related to the measurement error, σ_{tr} is related to the error (bias) due to the transformation from the measured quantity to the parameter of interest, and ξ is the statistical model error. However, this paper only covers the updating of the mean value and the related uncertainty σ_{μ} to keep the focus on the observational method; for a complete reliability analysis, effort must likely be made to reduce also the measurement and transformation errors. Such measures are described in Müller et al. (in press). Thus, we define for this paper a "random scatter" component σ_{sc} as

$$\sigma_{sc}^2 = \sigma_{inh}^2 + \sigma_{m.e.}^2 + \sigma_{tr}^2 + \xi \tag{3}$$

2.4 *Updating a parameter from observations*

This definition implies that both the parameter of interest X and its mean μ are modelled as (normally distributed) random variables. Assuming that the pre-investigation suggests that the prior estimation of X (denoted X') is distributed $N(\mu', \sigma')$, the prior estimation of the mean is $\mu_{\mu}' = \mu'$ and the prior uncertainty related to the estimation of mean is

$$\sigma_{\mu}' = \sqrt{\sigma'^2 - \sigma_{sc}^2} \tag{4}$$

This follows from Equations (2)–(3). Consequently, X' is

$$N(\mu', \sigma') = N\left(\mu', \sqrt{\sigma_{sc}^2 + \sigma_{\mu}'^2}\right) \tag{5}$$

Following the proposed procedure (Fig. 1), σ_{μ}' is updated with n observations, recognising that a normal distribution with a known variance is its own conjugate distribution, which implies that the posterior distribution is also normal. (If this assumption is unsuitable, Markov chain Monte Carlo simulations can instead be used to numerically estimate the posterior distribution). With a normally distributed parameter, the prior distribution X' is updated to the posterior distribution X'', which is (Ang & Tang 2007)

$$N\left(\mu'', \sqrt{\sigma_{sc}^2 + \sigma_{\mu}''^2}\right) \tag{6}$$

in which

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$$\mu'' = \frac{\bar{x}\,\sigma_\mu'^{\,2} + \mu'\sigma_{sc}^2/n}{\sigma_\mu'^{\,2} + \sigma_{sc}^2/n} \tag{7}$$

$$\sigma_\mu''^{\,2} = \frac{\sigma_\mu'^{\,2}\,\sigma_{sc}^2/n}{\sigma_\mu'^{\,2} + \sigma_{sc}^2/n} \tag{8}$$

where $\bar{x}$ is the sample mean. The Equations (7)–(8) show that the posterior mean is a weighted average of the prior mean and the sample mean, depending on the respective variances. It is also evident that for large *n*, the posterior mean will approach the sample mean. Thus, the updating procedure can reduce the uncertainty about the mean by increasing the sample size *n*. The reduction of the other variance component σ_{sc}^2 is however not within the scope of this paper.

3 FICTIVE EXAMPLE: ESTIMATING THE SAFETY OF A ROCK PILLAR FROM MEASUREMENTS

3.1 *Case presentation*

To illustrate the described procedure, a fictive example is presented in the following. In an excavated room with the load from the overburden uniformly distributed over a large array of rock pillars, the probability of failure of one pillar is assessed (Fig. 2).

The pillar is formed by a sequential excavation, during which measurements of the pillar deformation is made after each step. In this example, the updating procedure is applied to the deformation modulus of the rock mass E_m. This is advantageous as the magnitude of the deformation is dependent on the current size of the pillar during the sequential excavation. The E_m on the other hand remains approximately the same, irrespectively of pillar size. This means however that the measurements of the deformation must be converted into observations of the E_m.

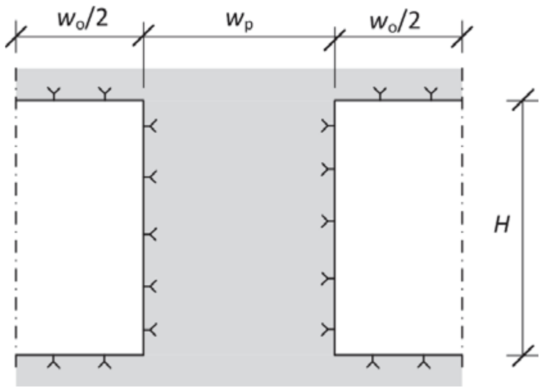

Figure 2. A vertical cross-section of the square rock pillar.

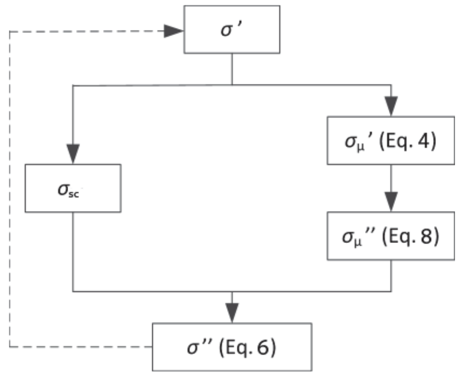

Figure 1. The updating process of the uncertainty related to a parameter. The dashed line indicates that the procedure can be done repeatedly as more data becomes available. The posterior distribution then becomes the new prior.

For a single square pillar, the average pillar stress is (Hoek & Brown 1980)

$$\sigma_n = \gamma_r z \left(1 + \frac{w_o}{w_p}\right)^2 \tag{9}$$

where γ_r is the unit weight of the rock, z is the depth below the surface, w_o is the width of the opening between two pillars in each (perpendicular) direction, and w_p is the width of the sides of the pillar.

From Hooke's law, a limit state function $G = 0$ is derived for the designed pillar, assuming there is a maximum strain ε_{acc} that must not be exceeded:

$$G = \Psi\varepsilon_{acc} - \frac{\sigma_{n,d}}{E_m} = 0 \tag{10}$$

where Ψ is the model uncertainty related to the applicability of Hooke's law, and $\sigma_{n,d}$ is the pillar stress to which the completed pillar will be exposed (Eq. 9). Thus, the probability of failure of the completed pillar can be estimated as

$$P_f = P(G < 0) \tag{11}$$

A tolerable probability of failure $P_{f,tol}$ for the completed rock pillar is here set to 10^{-3} (cf. the 2nd principle of the observational method: “acceptable limits of behaviour shall be established”).

In this example, the following simplifications are made: (1) the rock mass deforms only linear-elastically, (2) the measurements are independent of each other, (3) probability distributions are only assigned to Ψ, ε_{acc}, γ_r, and E_m; all other parameters are given constant values, (4) both the initial in situ strain and the possible scale-effect on the rock mass properties, see e.g. Palmström & Stille (2010), are neglected.

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3.2 *Indata for calculations*

The indata are summarised in Table 1 and presented in the following. The completed rock pillar will be H = 6.0 m high, and its sides w_p will be 2.8 m wide. The overburden consists of z = 40 m rock with a mean unit weight of 26.5 kN/m^3 and $COV\gamma_r$ = 0.1. The model uncertainty Ψ is assigned the distribution $N(1; 0.1)$ and ε_{acc} is set to 0.0015 with $COV\varepsilon_{acc}$ = 10%.

The prior knowledge about E_m could potentially be estimated from empirical rock mass classification methods. In this example, E_m' is assigned the distribution N(18 GPa, 5 GPa) to imitate an average quality rock mass (Hoek & Brown 1997). The non-reducible random scatter of E_m is assigned a 10% *COV* on the prior estimation of the mean, i.e. $\sigma_{sc,Em}$ = 1.8 GPa. From Equation (4), the prior uncertainty about the mean $\sigma_{\mu,Em}'$ is calculated to 4.7 GPa.

The volume of rock carried by the pillar is assigned the constant value

$$V_r = z\left(w_o + w_p\right)^2 \tag{12}$$

but for each sequence in the excavation, the horizontal cross-section area $w_p^{\,2}$ of the pillar decreases and the opening between pillars w_o widens (Table 2), which increases the pillar stress.

The deformation observed in each excavation sequence is also listed in Table 2. For this fictive example, the deformation measurements were simulated. This process is explained in Appendix A. If the methodology were used in practice, real measurements would of course be used instead.

The 9 measurements of the deformations $\delta_{obs,i}$ are converted into "observations" of E_m with Hooke's law

$$E_{m,i} = \frac{\sigma_{n,i}}{\delta_{obs,i}/H} \tag{13}$$

where $\sigma_{n,i}$ is the mean pillar stress (Eq. 9) present during excavation sequence ***i***. The average observed E_m is then calculated and used in Equations (6)–(8), to find the posterior distribution E_m''.

Table 2. Observations of E_m throughout the sequential excavation.

Excavation sequence	w_p (m)	w_o (m)	δ_{obs}* (mm)	$E_{m,obs}$ (GPa)
1	9.5	1.5	0.4	21.3
2	8.0	3.0	0.6	20.0
3	6.5	4.5	0.7	26.0
4	5.5	5.5	1.1	23.1
5	4.5	6.5	2.0	19.0
6	4.0	7.0	2.3	20.9
7	3.6	7.4	2.8	21.2
8	3.2	7.8	3.5	21.5
9	2.8	8.2	4.2	23.4

* Fictive observations of the deformation were generated as described in Appendix A.

Table 1. Parameters used in the fictive example.

Name	Denot.	Unit	Mean	Std *
Acceptable strain	ε_{acc}	-	0.0015	0.00015
Unit weight of rock	γ_r	kN/m^3	26.5	2.65
Depth below ground	z	m	40	
Side length of carried rock volume	w_o+w_p	m	11	
Pillar height	H	m	6.0	
Prior deformation modulus of rock mass	E_m'	GPa	18	5
Random scatter of E_m	$\sigma_{sc,Em}$	GPa		1.8
Model uncertainty	Ψ	-	1	0.1

* Normal distributions are assigned to all basic random parameters, for which standard deviations are given. All other parameters are constant.

To illustrate the impact of the updating process, P_f was calculated with the deformation modulus based either on the prior distribution E_m' or on the posterior distribution E_m''. The probability of failure of the completed rock pillar was calculated with a Monte Carlo simulation in the software Matlab R2013a, with a sample size of N = 10 million generated (pseudo-)random numbers. This rendered a tolerable error of at most 3%, based on (Ang & Tang 2007)

$$e_{tol}[\%] = 200\sqrt{\frac{1-P_f}{NP_f}} \tag{14}$$

3.3 *Calculation results of the fictive example*

The rather wide prior distribution of E_m was clearly narrowed through the Bayesian updating process. The various probability distributions related to the uncertainties of E_m are presented in Figure 3. Because of the updated E_m, the calculated P_f of the analysed rock pillar was reduced from 0.10 to 0.00047.

4 DISCUSSION

4.1 *How is the Bayesian updating procedure compatible with the observational method?*

The observational method implies allowing adjustments to the design of the structure, to accommodate it to the present geotechnical conditions in the ground. The presented calculation example concerns the three first requirements that must be met before the construction can be started, as stated in Eurocode 7 (CEN 2004). In the following, these requirements are sequentially quoted and commented on, with respect to their compatibility with the Bayesian updating procedure.

– *acceptable limits of behaviour shall be established;*

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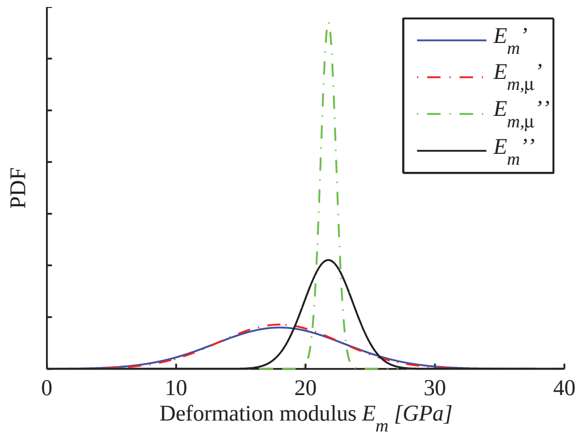

Figure 3. The distributions in the updating procedure. The distribution of $E_{m,\mu}'$ is only marginally tighter than E_m', as the uncertainty about the mean is dominant for the prior.

The first requirement is for the rock pillar interpreted as establishing a tolerable probability of failure (often called target safety index). Thus, if the pillar does not "behave" acceptably during construction, the completed pillar risks ending up with a too high probability of failure. In the general case, however, these limits do not necessarily need to be defined as probabilities of failure. Depending on what the designer believes is unacceptable behaviour, limits could also be defined as for example a required performance or even a desired aesthetic.

– *the range of possible behaviour shall be assessed and it shall be shown that there is an acceptable probability that the actual behaviour will be within the acceptable limits;*

The interpretation is that the preliminary design must be able to withstand the likely outcomes of the observations. In the calculation example, the possible behaviour is indicated by the range of the applied empirical rock mass classification method σ_{Em}' and the random scatter $\sigma_{sc,Em}$. To satisfy the requirement, the pillar must be designed to manage the outcomes of E_m'' that make the need for contingency actions reasonably low. The limit between suitable and unsuitable preliminary designs is not specified in the Eurocode; the level of probability for having to put contingency actions into operation is for the decision-maker in charge to choose. From a safety point of view, it should not matter whether contingency actions are used or not. The difference concerns in theory merely the final cost.

In the fictive example, the posterior deformation modulus turned out to be in the upper range of the prior estimation (Fig. 3). This allowed the calculated final probability of failure of the pillar to fall below the tolerable probability of failure. Hence, there was no need for any contingency actions.

– *a plan of monitoring shall be devised, which will reveal whether the actual behaviour lies within the acceptable limits. The monitoring shall make this clear at a sufficiently early stage, and with sufficiently short intervals to allow contingency actions to be undertaken successfully.*

The monitoring must be carefully planned, to make sure that it is relevant for the analysed limit state. In the example, δ is measured, although E_m is the parameter used in the updating procedure. This might seem roundabout at a glance, but as the measured parameter obviously needs to be measurable in practice, E_m can be ruled out in this regard. On the other hand, δ permits easy measurement after each excavation sequence, but would instead be more complicated to relate to the relevant limit state (P_f of the completed rock pillar). But by measuring δ and using the result to indirectly observe E_m, the monitoring can give an indication of the P_f of the final pillar design before it actually is completed. Thus, the Eurocode requirement is met.

For the general case, the plan of monitoring and the updating procedure must be adjusted to each new application, since each engineering problem has its unique features. That is, the engineer must for each application of the observational method consider which variables are more suitable for measuring and updating. This should be reflected in the choice of limit state function (Stille & Holmberg 2008). One limitation is that the parameter uncertainty must to a considerable degree be of epistemic nature; if the uncertainty is aleatoric, it will not be possible to reduce the uncertainty with measurements. Another issue is the measurement error. For example, if small changes are expected, the installed instruments must be able to capture them with sufficient precision to produce useful measurement results.

4.2 *Suggestion of a requirement for an appropriate safety margin*

The presented methodology outlines a procedure that allows an assessment of the final safety margin of a structure. This feature directly addresses the concerns regarding low safety margins associated with the observational method, reported by Powderham (2002). Although it can be argued that the need for a safety margin should be understood from the first requirement (regarding establishing acceptable limits of behaviour), we believe that this issue deserves more attention.

By explicitly requiring an appropriate safety margin for the final structure, the framework of the observational method would become more rigid. Hence, any arbitrary interpretations of how to deal with safety when applying the observational method can be avoided. Consequently, we argue for adding a new requirement to the principles of the observational method, stating to leave an appropriate safety margin with the completed structure.

5 CONCLUSIONS

The presented procedure for how measurements and observations from the construction phase can be used to assess the safety of the final structure shows good agreement with the current definition of the observational method in Eurocode 7. Further development is however needed to take any design changes from contingency actions into account in the safety assessment. More case studies from different kinds of geotechnical problems are also vital to show a general validity.

The paper highlights that the current definition of the observational method does not explicitly require any safety margin for the completed structure. To strengthen the framework of the observational method, we find it suitable that such a requirement is added to the principles of the method. By doing so, the reported concerns about low safety margins being associated with the observational method can be repudiated. The outlined procedure indicates a promising way to meet the proposed requirement. We intend to develop the concept in future research.

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8 APPENDIX A: SIMULATION OF OBSERVED DEFORMATIONS

In the calculation example, the set of deformation measurements was created from a (pseudo-)random realisation of a normally distributed "true" E_m of 22 GPa with a random scatter $\sigma_{sc,Em}$ = 1.8 GPa. From Hooke's law, the corresponding deformations for each excavation sequence i were calculated from

$$\delta_{obs,i} = \frac{\sigma_{n,i}H}{E_m} \quad (15)$$

where $\sigma_{n,i}$ is the (mean) pillar stress after excavation i. The simulated deformation measurements are presented in Table 2.

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