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# Eurocodes in Hard Rock Engineering in Finland

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**Keywords:** Eurocodes, Hard Rock Engineering, NAD, national annex document, design method

## 1. Introduction

The Eurocodes are a set of European design principles, rules, and guidance intended for the design of load carrying structures. The codes have been compiled by the CEN Technical Committee 250 (CEN/TC250). The intent of the Eurocodes is to unify the design methodology. Parallel use with existing national building codes is allowed during the transition period (2007–2010) to ensure a smooth transition and enough time to make the necessary changes.

In Finland (and Sweden and Norway), there is no formal procedure on how to design rock spaces. The design is based on a designer's expertise, experience, views, and specific procedures. The problem here is that identical initial data could generate very different results depending on the way the initial data is interpreted and which design methods are used. The Eurocodes do not explicitly state how to design rock spaces, but they define the minimum requirements on how to design structures.

The Eurocodes are divided into 10 areas. Of these ten, only a part will affect how to design rock spaces. Eurocode 0 defines the main principles concerning the whole design process. Eurocode 1 defines the loads. Eurocode 2 defines the design of concrete structures (e.g. design of reinforced concrete structures, shotcrete, and grouted rebars). Eurocode 3 defines the design of steel structures (e.g. design of non-grouted bolts, steel columns, and pillars). Eurocode 7 defines the design of geotechnical structures. Eurocode 8 would define the design against earthquakes, but it is not required in Finland, unless otherwise stated.

## 2. Eurocodes change the design of rock spaces

Many projects already require the use of the Eurocodes. Design working life is a concept which defines an assumed period for which a structure or a part of it is to be used for its intended purpose with anticipated maintenance but without major repair being necessary. For many rock engineering projects a normal specification is 100 years while previously the design assumption has been 50 years. For practical purposes, design working life can be interpreted to mean that 95 percent of the structure must last longer than the specified time with the specified maintenance.

In the highest geotechnical class GC3 (corresponding to the highest Finnish difficulty class AA) it is no longer possible to use empirically based design only. Eurocode 7 states that prescriptive measures may be used if calculation models are not available or not necessary. Empirical methods (such as the Q-method or the RMR-method) may still be used to give a rough estimate to assess the validity of design. However, the Eurocode does allow the use of the Observational Method when prediction is difficult or impossible, but monitoring and redesign are possible. Because prediction is often difficult and monitoring is often recommended, this should be viewed more as a part of a standard design process (Ström 2009) than a stand-alone method.

The Finnish regulation for the design requirements of civil defence shelters within rock states that rock engineering calculations should be made if the span is wide or the rock conditions are difficult. This implies that empirical design methods should not be used in geotechnical class GC3. The

notes in section 2.1 of Eurocode 7 state that geotechnical class GC2 includes “tunnels in hard, non-fractured rock and not subjected to special water tightness or other requirements.” Obeying the note in the Eurocode 7 would put practically all tunnels into GC3. However, FISE (Qualification of Professionals in Building, HVAC and Real Estate Sector in Finland) has made a much more practical interpretation which is shown in the full paper.

As of 2010, many countries have withdrawn their national codes and the Eurocodes are required to be used. Rock structures are required to last longer than has been previously required, which sets high demands on material durability and/or maintenance. Currently (2010), a common specification is 100 years. Many countries lack a NAD (National Annex Document) suitable for rock engineering, and companies are doing their own research on what should be done and how. Research is being carried out on how to fulfil the design working life requirement without additional maintenance.

According to section C4 in Eurocode 0, the current design based on partial safety factors is a Semi-Probabilistic Method (Level I). Eurocode 0 does allow the use of First Order Reliability Method (Level II) or Full Probabilistic Method (Level III). The increased accuracy requires more accurate initial data which sets high requirements for geological initial data, site investigation, and data interpretation. Suitable software for stochastic rock engineering is often lacking, and calculations must be done partially with customised programs or with the aid of stochastic software. Here a semi-deterministic approach, such as the Latin Hypercube Sampling, can be of great help in order to control the complexity of the multivariable probability equations.

### **3. Eurocode based design**

The Eurocode allows for multiple approaches to design methodology. The design method presented here has been developed by Rockplan during 2007–2010. It meets all the requirements set in the Eurocodes and it is consistent with the pre-existing design methodology. For a more detailed description and more case examples please refer to master’s theses of Siren (2008) and Ström (2009).

A simplified description of the design methodology is as follows:

1. Site investigation and initial data accumulation with field and laboratory research combined with database searches and reference site research;
2. Building a geological model and a model describing constructability;
3. Rock mechanical analyses; “Limit state method”
4. Rock excavation and reinforcement plans;
5. Control measurements during construction and use; “Observational method”
6. Post analyses.

### **4. Conclusions**

Some parameters are still missing and a National Annex Document for rock engineering or a guidance document for rock engineering would clarify the situation and set common ground rules. Currently the Eurocode approach allows a variety of analysis methods. This means that the same initial data could generate a wide variety of rock engineering solutions.

The Eurocode methodology works well for rock bolts and shotcrete. Using the design methodology given in this paper it is also possible to model displacements and stresses according to Eurocodes. The design process remains the same, but the parameters are now derived differently and some formulas have changed. The Eurocode analysis can be more precise if suitable initial data exists. Comparison to design guidance from almost 30 years ago shows, that the changes introduced in the Eurocode are reasonable. For geotechnical classification purposes we suggest that the FISE table is used (see full paper), because the clarifying note given in Eurocode 7 is unreasonably restrictive. During the next revision of Eurocode 7 this note should be corrected to avoid confusion.

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# Eurocodes in Hard Rock Engineering in Finland

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## Summary

The Eurocodes replace the 27 existing national building codes with one building code covering the entire European Union. In total, there are 58 parts and 5500 pages. The design principles and methods are unified, but nationally varying conditions (such as rock mass strength and predominant stress field) can be taken into account via National Annex Documents (NADs). As of 2010, there is no NAD or guidance document for rock engineering in Finland. Many projects already require the use of the Eurocodes and by the spring of 2011, Finland will withdraw the possibility to use a national building code in parallel to the Eurocodes. In this paper, we describe how the Eurocodes have changed and will change the way we design underground spaces. Case example of implementation will be shown with comparison to the coexisting Finnish design methodology. For some parts, such as the rock bolts or the fibre-reinforced shotcrete, the Eurocode methodology works fairly well. For modern rock engineering (e.g. numerical or stochastic modelling), there is much room for interpretation and a clear need for guidelines. In Sweden, the Implementeringskommission för Europastandarder inom Geoteknik (IEG) has released two reports on the matter (6:2006 and 3:2008) and will release a third one by the end of 2010. In Finland, Rockplan has studied the implementation and effects of the Eurocodes in the rock engineering field since 2007. Two master's theses (Siren 2008 and Ström 2009) and hundreds of hours of hard work later, we present our suggestions.

**Keywords:** Eurocodes, Hard Rock Engineering, NAD, national annex document, design method

## 1. Introduction

The Eurocodes are a set of European design principles, rules, and guidance intended for the design of load carrying structures. The codes have been compiled by the CEN Technical Committee 250 (CEN/TC250). The intent of the Eurocodes is to unify the design methodology. Parallel use with existing national building codes is allowed during the transition period (2007–2010) to ensure a smooth transition and enough time to make the necessary changes. (Modified from [1] p. 11)

In Finland (and Sweden and Norway), there is no formal procedure on how to design rock spaces. The design is based on a designer's expertise, experience, views, and specific procedures. The problem here is that identical initial data could generate very different results depending on the way the initial data is interpreted and which design methods are used. The Eurocodes do not explicitly state how to design rock spaces, but they define the minimum requirements on how to design structures. (Modified from [2] p. 14)

The Eurocodes are divided into 10 areas. Of these ten, only a part will affect how to design rock spaces. Eurocode 0 defines the main principles concerning the whole design process. Eurocode 1 defines the loads. Eurocode 2 defines the design of concrete structures (e.g. design of reinforced concrete structures, shotcrete, and grouted rebars). Eurocode 3 defines the design of steel structures (e.g. design of non-grouted bolts, steel columns, and pillars). Eurocode 7 defines the design of geostructures. Eurocode 8 would define the design against earthquakes, but it is not required in Finland, unless otherwise stated. [3] [4] [5] [6] [7] [8]

## 2. Eurocodes change the design of rock spaces

### 2.1 Past changes

Many projects already require the use of the Eurocodes. Design working life is a concept which defines an assumed period for which a structure or a part of it is to be used for its intended purpose with anticipated maintenance but without major repair being necessary. For many rock engineering projects a normal specification is 100 years while previously the design assumption has been 50 years. For practical purposes, design working life can be interpreted to mean that 95 percent of the structure must last longer than the specified time with the specified maintenance.

Since 2005 in Europe (and since 2009 in Finland) a unified specification (EN 10080) is used for rebar rock bolts with classes B500A, B500B (previously A500HW in Finland), and B500C. The first letter denotes a structural steel, the number in the middle specifies the yield strength in MPa ( $\text{N/mm}^2$ ), and the last letter specifies the ductility class (C being the most ductile).

In the highest geotechnical class GC3 (corresponding to the highest Finnish difficulty class AA [15] p. 30) it is no longer possible to use empirically based design only. Eurocode 7 states that prescriptive measures may be used if calculation models are not available or not necessary ([7] s. 2.5). Empirical methods (such as the Q-method or the RMR-method) may still be used to give a rough estimate to assess the validity of design. However, the Eurocode does allow the use of the Observational Method when prediction is difficult or impossible, but monitoring and redesign are possible. Because prediction is often difficult and monitoring is often recommended, this should be viewed more as a part of a standard design process ([2] p. 64) than a stand-alone method.

The Finnish regulation for the design requirements of civil defence shelters within rock [9] states that rock engineering calculations should be made if the span is wide or the rock conditions are difficult. This implies that empirical design methods should not be used in geotechnical class GC3. The notes in section 2.1 of Eurocode 7 state that geotechnical class GC2 includes “tunnels in hard, non-fractured rock and not subjected to special water tightness or other requirements.” Obeying the note in the Eurocode 7 would put practically all tunnels into GC3. However, FISE (Qualification of Professionals in Building, HVAC and Real Estate Sector in Finland) has made a much more practical interpretation [10] shown in Table 1.

*Table 1. Interpretation of Geotechnical Classification for tunnels (modified from [10])*

Geotechnical Class GC3 equivalent difficulty class AA	Geotechnical Class GC2 equivalent diff. class A	Geotech. Cl. GC1 equiv. diff. cl. B/C
<b>Environmental demands:</b> - groundwater level must be maintained; - culture-historically valuable items or large amount of people in the immediate vicinity; - exceptionally sensitive equipment or structures.	<b>Environmental demands:</b> - conventional environmental demands.	<b>(not used)</b>
<b>Rock properties:</b> - high <i>in-situ</i> stresses or exceptionally weak strength properties in relation to dimensions.	<b>Rock properties:</b> - average rock properties.	
<b>Usability requirements:</b> - span $L \geq 20$ m; - thin rock roof ( $\leq L/2$ ); - tunnels beneath water; - highest height $h \geq 30$ m; - loads exceeding the civil defence shelter loads; - large number of users; - any other high security demand.	<b>Usability requirements:</b> - conventional usability requirements; - standard civil defence shelter loads.	

Table 2 presents the Geotechnical Classification guidelines in EN 1997-1 [7]. By comparing the two tables, it can be stated that the FISE interpretation is in accordance with the principles and application rules set in the Eurocodes.

*Table 2. Application rules of Geotechnical Classification for tunnels [7]*

Geotechnical Class GC3	Geotechnical Class GC2	Geotechnical Class GC1
<p>Structures, which fall outside the limits of GC2 and GC1.</p> <p><i>(note) Examples:</i></p> <ul style="list-style-type: none"> <li>- very large or unusual structures;</li> <li>- structures involving abnormal risks, or unusual or exceptionally difficult ground or loading conditions;</li> <li>- structures in highly seismic areas;</li> <li>- structures in areas of probable site instability or persistent ground movements that require separate investigation or special measures.</li> </ul>	<p>Conventional types of structure and foundation with no exceptional risk or difficult soil or loading conditions</p> <p><i>(note) Tunnels in hard, non-fractured rock and not subjected to special water tightness or other requirements.</i></p>	<p>Only small and relatively simple structures:</p> <ul style="list-style-type: none"> <li>- for which it is possible to ensure that the fundamental requirements will be satisfied on the basis of experience and qualitative geotechnical investigations;</li> <li>- with negligible risk.</li> </ul> <p>Only if there is no excavation below the water table or if comparable local experience indicates that a proposed excavation below the water table will be straightforward.</p>

## 2.2 Current changes (2010–2011)

As of 2010, many countries have withdrawn their national codes and the Eurocodes are required to be used. Rock structures are required to last longer than has been previously required, which sets high demands on material durability and/or maintenance. Currently (2010), a common specification is 100 years. Many countries lack a NAD (National Annex Document) suitable for rock engineering, and companies are doing their own research on what should be done and how. Research is being carried out on how to fulfil the design working life requirement without additional maintenance.

## 2.3 Future changes

The current design based on partial safety factors is a Semi-Probabilistic Method (Level I, [3] s. C4 (4)). Eurocode 0 does allow the use of First Order Reliability Method (FORM, Level II) or Full Probabilistic Method (FPM, Level III). The increased accuracy requires more accurate initial data which sets high requirements for geological initial data, site investigation, and data interpretation. Suitable software for stochastic rock engineering is often lacking, and calculations must be done partially with customised programs or with the aid of stochastic software. Here a semi-deterministic approach, such as the Latin Hypercube Sampling, can be of great help in order to control the complexity of the multivariable probability equations.

Advanced FORM analysis has already been carried out by Rockplan in two projects as reported in 2009 by Siren [11] and Uotinen [12]. When initial data of sufficiently high quality is available, the stochastic design approach can reduce the amount of reinforcement without compromising the required level of safety. Rockplan has made enquiries, and at least one of the major companies providing solutions for rock engineering is implementing the requirements of FORM in their software. The practical difference is that in partial safety factor approach, the resistance is compared against the action ( $R_d > E_d$ ), while when using probabilistic approach, the target probability is compared against the failure probability ( $P_0 > P_f$ ).

In stochastic approaches, a target probability (or target reliability) must be declared. In Table 3 you can find the required reliability index and the corresponding probability if normal distribution is assumed as a function of reliability classes (RC, [3] s. B3.2) and design working life ([3] s. 2.3). For

buildings the target level of failure is 1 : 15 000 (during 50-year lifetime, using RC2 and the normal distribution). For tunnels the target level of failure is 1 : 100 000 (during 100-year lifetime, using RC3 and the normal distribution). Here “failure” means any damage above the defined threshold including everything from localised damage to massive failure.

*Table 3. Required reliability indexes in relation to reliability classes and design working life [3]*

	1 a	25 a	50 a	100 a	200 a
RC3	$P_f = 1.0E-7$ $\beta = 5.2$	$P_f = 2.5E-6$ $\beta = 4.6$	$P_f = 5.0E-6$ $\beta = 4.4$	$P_f = 1.0E-5$ $\beta = 4.3$	$P_f = 2.0E-5$ $\beta = 4.1$
RC2	$P_f = 1.3E-6$ $\beta = 4.7$	$P_f = 3.3E-5$ $\beta = 4.0$	$P_f = 6.5E-5$ $\beta = 3.8$	$P_f = 1.3E-4$ $\beta = 3.7$	$P_f = 2.6E-4$ $\beta = 3.5$
RC1	$P_f = 1.3E-5$ $\beta = 4.2$	$P_f = 3.3E-4$ $\beta = 3.4$	$P_f = 6.7E-4$ $\beta = 3.2$	$P_f = 1.3E-3$ $\beta = 3.0$	$P_f = 2.7E-3$ $\beta = 2.8$

### 3. Design of rock spaces

#### 3.1 Eurocode based design

The Eurocode allows for multiple approaches to design methodology. The design method presented here has been developed by Rockplan during 2007–2010. It meets all the requirements set in the Eurocodes and it is consistent with the pre-existing design methodology. For a more detailed description and more case examples please refer to [1] and [2].

A simplified description of the design methodology is as follows:

1. Site investigation and initial data accumulation with field and laboratory research combined with database searches and reference site research;
2. Building a geological model and a model describing constructability;
3. Rock mechanical analyses; “Limit state method” [7] s. 2.4
4. Rock excavation and reinforcement plans;
5. Control measurements during construction and use; “Observational method” [7] s. 2.7
6. Post analyses.

(Modified from [1] p. 66)

#### 3.2 Displacement analysis in the serviceability limit state (SLS)

According to EN1997-2 ([13] s. 1.6), the characteristic parameters needed for displacement analysis can be derived from statistical data as follows. First, a set of laboratory testing reports for rock samples are needed. From the loading history it is possible to determine an intact rock sample elastic modulus for each of the samples. Next, a correlation between the intact elastic modulus and rock mass elastic modulus can be made using the equation

$$E_{rm,i} = E_i \times \left( 0.02 + \frac{1 - \frac{D}{2}}{1 + \exp\left(\frac{60 + 15D - GSI}{11}\right)} \right), \quad (1)$$

where  $E_i$  is the intact rock sample elastic modulus for sample  $i$  [MPa]  
 $D$  is the disturbance factor ( $D = 0$  for excellent quality controlled blasting)  
 $GSI$  is the Geological Strength Index or  
 $9 \ln Q + 44$  if Q-method is used instead  
 $RMR_{76}$  if RMR-method is used instead  
 $RMR_{89} - 5$  if RMR-method is used instead

[14]

This generates a set of rock mass elastic moduli. Characteristic value for displacement analysis should be derived in such a way that the calculated probability of a worse value governing the occurrence of the limit state under consideration is not greater than 5 % ([7] s. 2.4.5.2 (11)).

Following equation may be used to obtain a characteristic value

$$E_{rm.k} = \mu_{Erm} - 1.645 \times \frac{\sigma_{Erm}}{\sqrt{n}}, \quad (2)$$

where  $\mu_{Erm}$  is the mean of the rock mass elastic modulus [MPa]  
 $\sigma_{Erm}$  is the standard deviation of rock mass elastic modulus [MPa]  
 $n$  is the number of samples  
 (Modified from [15] p. 41)

If intact rock sample elastic modulus is not available, the simplified Hoek and Diederichs equation may be used in preliminary analysis

$$E_{rm.k} = 100\,000 \text{ MPa} \times \left( \frac{1 - \frac{D}{2}}{1 + \exp\left(\frac{75 + 25D - GSI}{11}\right)} \right) \quad (3)$$

[14]

If Drill-and-Blast method is used and the blasting quality is not excellent, a rough estimate may be obtained using the formula presented in this paper

$$D = \frac{(L_{tot} - L_{bot} - L_{geo}) - L_{vis}}{L_{tot} - L_{bot} - L_{geo}}, \quad (4)$$

where  $L_{vis}$  is length of visible contour hole half pipes (half casts)  
 $L_{tot}$  is the total theoretical length of contour hole half pipes in walls and roof arch  
 $L_{bot}$  is length of half pipes lost due to the bottom charge area  
 $L_{geo}$  is length of half pipes lost due to geological rock overbreak.

This gives  $D = 0.0$  for sections with all of the border hole half pipes fully visible and  $D = 0.5$  for areas with half of the pipe lengths visible and  $D = 1.0$  when no half pipes are visible. It should be noted that using a larger value than 0 will increase the predicted displacements, but will not affect the stresses. Other estimates are presented in [14] for tunnels, open pits, TBMs, and other applications.

The same procedure is repeated to obtain the other parameters. For Poisson's value ( $\nu$ ) in elastic analysis, the upper 95 % confidence limit should be used, because it gives a larger displacement magnitude and larger stresses. The following equation may be used

$$\nu_k = \mu_\nu + 1.645 \times \frac{\sigma_\nu}{\sqrt{n}} \quad (5)$$

where  $\mu_\nu$  is the mean of the Poisson value  
 $\sigma_\nu$  is the standard deviation of the Poisson value  
 $n$  is the number of samples  
 (Modified from [15] p. 41)

If Poisson's value from laboratory testing is not available a literature quote may be used instead. If a range is given, it is recommended to use a value from the upper end of the range.

The resulting values may be used in numerical analysis to obtain displacements and/or stresses in the Serviceability Limit State (SLS). For complex geometries it may be necessary to create separate cases for each lower and upper parameter combination to find out the conservative combinations.

### 3.3 Displacement analysis in the ultimate limit state (ULS)

The most significant displacements in Finland occur in thin rock roofs in the immediately vicinity of failure zones or as a shear failure of local weakness zones. In weak rocks, the failure of the rock (yielding) is a significant factor influencing the displacements. A lot of the necessary parameters (e.g. the partial safety factors for rock failure) are missing and further research is needed before Eurocodes can be fully implemented here. Displacement analysis in the ultimate limit state is not discussed in the scope of this paper.

## 4. Case example with comparison

### 4.1 Wedge analysis

Eurocode based analysis is fairly straightforward when using wedge analysis software capable of calculating Factor of Safety (FOS). For Ultimate Limit State (ULS) analysis, the rock mass design density can be calculated with the following equation presented in this paper

$$\rho_d = \gamma_{Gj,sup} \times K_{FI} \times \rho_k \quad (6)$$

where  $\gamma_{Gj,sup}$  is the partial safety factor for permanent loads (1.35)  
 $K_{FI}$  is the factor for reliability differentiation: RC1 = 0.9, RC2 = 1.0, RC3 = 1.1  
 $\rho_k$  is the characteristic value for density  $\rho_k = \mu_\rho + 1.645 \times \sigma_\rho / \sqrt{n}$ .

For steel bolt capacity the following equations presented in this paper may be used for rebar bolt tensile strength ( $R_{td}$ ) and shear strength ( $R_{vd}$ )

$$R_{td} = \frac{f_{yk} \times \varphi}{\gamma_s} \quad (7)$$

$$R_{vd} = \frac{f_{yk} \times \varphi}{\sqrt{3} \times \gamma_s} \quad (8)$$

where  $f_{yk}$  is the yield strength of the steel ( $f_{0.2k}$ ) [MPa]  
 $\varphi$  is bolt nominal diameter [mm<sup>2</sup>]  
 $\gamma_s$  is the partial safety factor for structural steel (normally 1.15).

For bond strength (between bolt and grout)

$$R_{bd} = 2.25 \times \eta_1 \times \eta_2 \times f_{ctd} \times s \quad (9)$$

where  $\eta_1$  is the bonding conditions factor (0.7)  
 $\eta_2$  is bolt nominal diameter factor (1.0 up to 32 mm)  
 $f_{ctd}$  is the design tensile strength of the grout [MPa]  
 $s$  is bolt nominal circumference [mm].

([5] s. 8.4.2)

For shotcrete and grout shear strength (and tensile strength)

$$f_{ctd} = \frac{0.7 \times 0.3 \times f_{ck}^{\frac{2}{3}}}{\gamma_c} \quad (10)$$

where  $f_{ck}$  is the cylinder compressive strength of the concrete [MPa]  
 $\gamma_c$  is the partial safety factor for concrete (normally 1.50).

([5] t. 3.1)

For shotcrete without fibres (or an internal mesh) the shear strength is reduced by 20 % ( $\alpha_{ct,pl} =$

0.80) ([5] s. 12.3.1). In Finland this reduction is set to 40 % ( $\alpha_{ct,pl} = 0.60$ ) in the National Annex Document. ([16] s. 12.3.1)

Using the above values, the design is acceptable when Factor of Safety is 1.00 or more. Utility Ratio (UR) may be calculated as the inverse of the FOS. The design is acceptable when the Utility Ratio is less than 100 %.

## 4.2 Comparison

This comparison shows how the Eurocodes will affect the design of a single grouted rebar rock bolt. The initial data used is shown in Table 4.

Table 4. Initial data used in the case example

Parameter	Symbol	Value
Bolt diameter	$\phi$	25 mm ( $\approx 1''$ )
Steel yield strength	$f_{vk}$	500 MPa ( $\approx 72.5$ kpsi)
Grout cylinder strength	$f_{ck}$	35 MPa ( $\approx 5.1$ kpsi)

According to Särkkä and Johansson ([17] pp. 121–122) the bond length (between grout and rock) must be at least 30 times the nominal diameter. This equates to 750 mm. The bond strength (between bolt and grout) according to Eurocodes (equation 9) is 185.3 kN/m and corresponding bonding length 1325 mm (+ 76 %). According to pull out tests (for settled grouted rebar) in Sweden performed in 2009 even 300 mm is sufficient to reach maximum load capacity of 25 mm bolts. [18] This suggests that the value  $\eta_1 = 1.0$  should be used for grouted bolts. Additionally, a reduced safety factor ( $\gamma_c = 1.35$ ) could be used when the grouting can be performed as a high quality work. The combined effect of these modifications would mean bond strength of 294.1 kN/m and corresponding bonding length 830 mm (still exceeding recommendation in [17] by + 11 %).

The total safety factor should be at least 1.5 and preferably 2.0 if the bolts are not grouted or the wedge is located in a difficult location ([17] p. 122). In Eurocode based analysis the total safety factor is simply  $1.15 * 1.5 = 1.725$  (+ 15 %). The Finnish National Annex Document allows to use partial safety factor of  $\gamma_{s,red1} = 1.10$  for steel and  $\gamma_{c,red1} = 1.35$  for concrete when the structure can be performed as a high quality work ([16] s. A.2.1). This leads to a total safety factor of 1.485 (- 1 %) which is still very reasonable. The safety generated by the Eurocode approach is very comparable to what we have previously used in Finland.

## 4.3 Discussion

Some parameters are still missing (e.g. the partial safety factors for rock mass compressive, shear and tensile strength for ULS analysis). National Annex Document for rock engineering or a guidance document for rock engineering would clarify the situation and set common ground rules. Currently the Eurocode approach allows a variety of analysis methods. This means that the same initial data could generate a wide variety of rock engineering solutions.

The probability approach in Eurocodes holds great potential to aid in risk analysis (risk equals to probability multiplied with the consequence). Currently the software solutions available are not capable of probability based design methods and third party solutions (such as calculation sheets or customized programs) must be used. When suitable software emerges, the initial data quantity and precision must be suitable for probability analysis. It may lead to large improvement in precision which may in turn yield savings in the form of a more economical rock engineering solutions.

## 4.4 Conclusions

The Eurocode methodology works fairly well for rock bolts and shotcrete. Using the design methodology given in this paper it is also possible to model displacements and stresses according to Eurocodes. The design process remains the same, but the parameters are now derived differently and some formulas have changed. The Eurocode analysis can be more precise if

suitable initial data exists. Comparison to design guidance from almost 30 years ago shows, that the changes introduced in the Eurocode are reasonable. For geotechnical classification purposes we suggest that the FISE table is used (Table 1), because the clarifying note given in Eurocode 7 is unreasonably restrictive. During the next revision of Eurocode 7 this note should be corrected in order to avoid confusion and conflicting interpretations.

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