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Design of shotcrete rock reinforcement in hard rock according to Eurocode

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Design of shotcrete rock reinforcement in hard rock according to Eurocode

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ABSTRACT: Shotcrete was invented in the 1900's and first used in rock spaces in the 1930's. The design of shotcrete is currently (2011) mainly based on empirical rules. This paper explores the feasibility of designing shotcrete structures according to the methods allowed in the Eurocodes. The shotcrete material parameters are derived according to the Eurocode and presented for shotcrete grades up to C50/60. The EFNARC panel test (EN 14488-5) and ASTM round panel test (C1550) are used in conjunction with the Yield Line Theory (YLT) to produce the cracking limit of the bending resistance. The shotcrete failure process and failure modes are described. Linear elastic theory is used to calculate the crack initiation capacity. The means on how to calculate the shotcrete capacity for the most common failure modes are presented. Finally, notes on how to apply these results into practical design work are given.

1. Introduction

For an elaborate introduction of development of shotcrete technology in general, see Barrett and McCreath 1995. In the Scandinavian hard rock countries (Finland, Sweden and Norway) the usage of wet-mix shotcrete as rock reinforcement grew rapidly during the 1980's. The research on how to design shotcrete reinforcement was advancing quickly during the 1990's. The EN standards for shotcrete were released on 2006. The usage of Eurocodes was allowed in 2007 and as of 2010 many European countries have withdrawn their national codes. Currently (2011) most shotcrete structures for hard rock reinforcement are still being designed using either thumb rules or empirical rules. No formal guidance on how to design shotcrete reinforcement exists.

Rockplan has committed itself into researching the possibility of utilization of the Eurocodes in the rock engineering field. This research began with two masters theses (Siren 2008 and Ström 2009). The most recent published advances in this field are the First Order Reliability Method (FORM) stochastic analysis performed by Siren and Uotinen in 2009 (reported in Siren et al. 2009 and Uotinen et al. 2009). In Sweden the, IEG (Implementeringskommission för Europastandarder inom Geoteknik) has performed similar research (IEG 6:2006, IEG 3:2008 and 5:2010) on the application of Eurocodes. The most recent publication on the implementation of Eurocodes in Hard Rock Engineering in Finland is Uotinen 2011.

A comprehensive description of the problems related to the design of shotcrete support can be found from Stille and Franzen 1993. The scope of the study is to study the mechanical behaviour of the shotcrete as a supporting structure in hard rock tunnel engineering. Only unreinforced shotcrete and steel fibre reinforced shotcrete will be studied. Steel or polymer meshes or polymer fibres will not be studied further. For polymer fibres see the master's thesis of Olli Salo in 2010. The

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goal of this study is to establish the basic guidelines on how to design shotcrete structures while adhering to Eurocodes. The long term goal is to improve the accuracy and the economy of the design by employing advanced methods allowed by the Eurocodes. The paper explores the feasibility of applying the design methods presented in the Eurocodes into design of shotcrete rock reinforcement. The paper was a part of the author's post-graduate studies and was the end report of Rak-50.3146 Seminar on Geoengineering on spring 2011.

2. Materials and methods

To derive the necessary structural mechanic parameters, the common conventions, analytical relations and constitutive models were used. The methods were adapted from structural engineering to best fit in rock engineering. For determination of the cross sectional stresses the linear elastic theory was used.

For modelling of shotcrete slab behaviour, the Yield-Line Theory (YLT) was used, as previously done by Holmgren 1993. Since experimental data was limited and expensive to produce, a literature survey in conjunction with analytical and numerical modelling was used instead. Table 2-1 lists the material surveyed in the literature search.

Table 2-1. Materials used in the literature survey

Year	Title	Description
1993	Sprayed concrete – Modern use of wet mix sprayed concrete for underground support	Proceedings of a seminar held in Fagernes, Norway on 18-21 th October 1993, 40 articles, 464 p.
2001	Shotcrete: Engineering Developments	Proceedings of a seminar held in Hobart, Tasmania, Australia on 2-4 th April 2001, 33 articles, 255 p.
2004	Shotcrete: More Engineering Developments	Proceedings of a seminar held in Cairns, Queensland, Australia on 4 th October 2004, 31 articles, 291 p.
2010	Shotcrete: Elements of a System	Proceedings of a seminar held in Queenstown, New Zealand, 15-17 th March 2010, 33 articles, 299 p.

Due to time and resource limitations a lot of published work could not be reviewed. To ensure that no widely noted or widely referenced article would go unnoticed, a number of Google and Google Scholar web searches were conducted using a wide variation of search parameters. The most important articles in the scope of this work are listed in table 2-2. Articles that were not peer-reviewed were disqualified from the survey.

Year	Authors	Title
1993	J. Holmgren	The use of yield-line theory in the design of steel-fibre reinforced concrete slabs (4 references according to Google Scholar)
1993	H. Stille and T. Franzen	Design of shotcrete support from the Rock mechanical viewpoint (26 references according to Google Scholar)
1995	S.V.L. Barrett and D.R. McCreath	Shotcrete Support Design in Blocky Ground: Towards A Deterministic Approach (26 references according to Google Scholar)

Additionally, the standards currently regulating the use and design of shotcrete were taken in account. These include the EN standard series for shotcrete and fibres as well as the Eurocodes regulating the design process of all structures. The most important standards affecting the design of shotcrete are stated in table 2-3.

Name and year	Title
EN 14487-1:2005	Sprayed concrete. Definitions, specifications and conformity
EN 14487-2:2006	Sprayed concrete. Execution
EN 14488-1:2005	Testing sprayed concrete. Sampling fresh and hardened concrete
EN 14488-2:2006	Testing sprayed concrete. Compressive strength of young sprayed concrete
EN 14488-3:2006	Testing sprayed concrete. Flexural strengths (first peak, ultimate and residual) of fibre reinforced beam specimens
EN 14488-4:2005 +A1:2008	Testing sprayed concrete. Bond strength of cores by direct tension
EN 14488-5:2006	Testing sprayed concrete. Determination of energy absorption capacity of fibre reinforced slab specimens
EN 14488-6:2006	Testing sprayed concrete. Thickness of concrete on a substrate
EN 14488-7:2006	Testing sprayed concrete. Fibre content of fibre reinforced concrete
EN 14889-1:2006	Fibres for concrete. Steel fibres. Definitions, specifications and conformity
EN 14889-2:2006	Fibres for concrete. Polymer fibres. Definitions, specifications and conformity
EN 197-1:2000	Cement. Composition, specifications and conformity criteria for low heat common cements
EN 206-1:2000	Concrete. Specification, performance, production and conformity
EN 1990:2002	Basis of Structural Design
EN 1992-1-1:2004	Eurocode 2. Design of concrete structures. General rules and rules for buildings
EN 1997-1:2004	Eurocode 7. Geotechnical design. General rules

Table 2-3. The most important standards affecting the design of shotcrete

3. Theory and Model

3.1 Material parameters for shotcrete

It's widely accepted that the same material parameters can be used for both shotcrete and concrete. The only exception is that any requirement for air entrance will not be applied (EN 14487-1). The compressive strength of concrete can be listed using notation "C35/45" where the first number is the cylindrical compressive strength in MPa at 28 days age and the latter number is the cubic compressive strength in MPa. All the other material parameters can be derived from the cylindrical strength (the first number) up to a maximum strength of C50/60.

Fibre reinforced shotcrete

The design compressive strength of fibre reinforced shotcrete is

$$f_{cd} = \frac{\alpha_{cc} \cdot f_{ck}}{\gamma_c}$$
, (EN 1992-1-1 s. 3.1.6) (EC2 3.15) (1)

where α_{cc} is the factor for long term effect and loading effect (nationally defined: 0.80...1.00)* f_{ck} is the cylindrical compressive strength of concrete

 γ_c is the partial safety factor for concrete (EN 1992-1-1 s. 2.4.2.4).

*) in the Finnish National Annex Document this is set to 0.85

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The design tensile strength of fibre reinforced shotcrete is

$$f_{ctd} = \frac{\alpha_{ct} \cdot f_{ctk,0.05}}{\gamma_c} = \frac{\alpha_{ct} \cdot 0.70 \cdot 0.30 \cdot (f_{ck})^{\frac{2}{3}}}{\gamma_c},$$
 (EN 1992-1-1 s. 3.1.6) (EC2 3.16) (2)

where α_{ct} is the factor for long term effect and loading effect (nationally defined, default: 1.00)* $f_{ctk,0.05}$ is nominal value of concrete tensile strength, 5 % fractile

 f_{ck}^{ck} is the cylindrical compressive strength of concrete γ_c is the partial safety factor for concrete (EN 1992-1-1 s. 2.4.2.4).

*) in the Finnish National Annex Document this is set to 1.00

The design shear strength of fibre reinforced shotcrete is

$$f_{cvd} = \sqrt{f_{ctd}^2 + \sigma_{cp} \cdot f_{ctd}},$$
 (EN 1992-1-1 s. 12.6.3) (EC2 12.5) (3)

where f_{ctd} is the design tensile strength σ_{cp} is the compressive force acting on the region (for conservative analysis: $\sigma_{cp} = 0MPa$).

The design bending tensile strength of fibre reinforced shotcrete under pure bending is

$$f_{ctd,fl} = max \begin{cases} \left(1.6 - \frac{h}{1000}\right) \cdot f_{ctd}, \\ f_{ctd} \end{cases}$$
 (EN 1992-1-1 s. 3.1.8) (EC2 3.23) (4)

where h is the height (thickness) of the shotcrete layer.

Unreinforced shotcrete

The corresponding formulas may also be presented for the unreinforced shotcrete (EN 1992-1-1 c. 12). The design compressive strength of unreinforced shotcrete is

 $f_{cd,pl} = \frac{\alpha_{cc,pl} \cdot f_{ck}}{\gamma_c},$ (EN 1992-1-1 s. 12.3.1) (EC2 3.15) (5)

where $\alpha_{cc,pl}$ is the factor for taking account the lack of ductility (nationally defined, default: 0.80)* is the cylindrical compressive strength of concrete f_{ck}

is the partial safety factor for concrete (EN 1992-1-1 s. 2.4.2.4). γ_c

*) in the Finnish National Annex Document this is set to 0.80 x 0.85 = 0.68

The design tensile strength of unreinforced shotcrete is

$$f_{ctd,pl} = \frac{\alpha_{ct,pl} \cdot f_{ctk,0.05}}{\gamma_c} = \frac{\alpha_{ct,pl} \cdot 0.70 \cdot 0.30 \cdot (f_{ck})^{\frac{2}{3}}}{\gamma_c},$$
 (EN 1992-1-1 s. 12.3.1) (EC2 3.16) (6)

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where $\alpha_{ct,pl}$ is the factor for taking account the lack of ductility (nationally defined, default: 0.80)* $f_{ctk,0.05}$ is nominal value of concrete tensile strength, 5 % fractile

 f_{ck} is the cylindrical compressive strength of concrete

 γ_c is the partial safety factor for concrete (EN 1992-1-1 s. 2.4.2.4).

*) in the Finnish National Annex Document this is set to 0.60 x 1.00 = 0.60

The design shear strength of unreinforced shotcrete is

$$f_{cvd,pl} = \sqrt{f_{ctd,pl}^2 + \sigma_{cp} \cdot f_{ctd,pl}},$$
 (EN 1992-1-1 s. 12.6.3) (EC2 12.5) (7)

where $f_{ctd,pl}$ is the design tensile strength (of unreinforced shotcrete) σ_{cp} is the compressive force acting on the region (for conservative analysis: 0 MPa).

The design bending tensile strength of unreinforced shotcrete under pure bending is

$$f_{ctd,fl} = max \begin{cases} \left(1.6 - \frac{h}{1000}\right) \cdot f_{ctd,pl}, \\ f_{ctd,pl} \end{cases}$$
 (EN 1992-1-1 s. 3.1.8) (EC2 3.23) (8)

where h is the height (thickness) of the shotcrete layer.

3.2 Yield-Line Theory

The Yield-Line Theory (YLT) is a widely used method in order to determine the capacity of reinforced concrete slabs (Holmgren 1993). It's an upper limit method to determine the plastic moment capacity using the concept of virtual work. The plate is divided into yielding regions and the virtual work is calculated for both internal and external actions. The sum of these virtual works must be zero, assuming the plate does not violate equilibrium. When the yield lines are fully developed, the plate becomes unstable and fails. YLT can be used for two purposes in the design of shotcrete support: the determination of suitable demands for the used shotcrete or in the design of adequate shotcrete layer thickness against a bending failure.

The dimensions of the EFNARC square plate test for shotcrete (SFS-EN 14488-5:2006) are defined in Figure 3-1 and the corresponding Yield Line Theory model is shown in Figure 3-2.



Figure 3-1. The dimensions of the EFNARC square plate



Figure 3-2. The dimensions and notations of the YLT square plate

The entire inner square is assumed to virtually move "1" downwards ($\delta = 1$). The internal work generated by the movement of the mechanism is

$$W_{int} = -\sum m_p L'\omega,\tag{9}$$

where m_p is the plastic moment capacity (isotropic)

- L'^{P} is the projection of the yield line to the rotation axis
 - ω is the rotation (angle as radians).

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For the EFNARC panel this equates to

$$W_{int} = -4 \cdot m_p \cdot L \cdot \frac{\delta}{\frac{L-B}{2}} = -4 \cdot m_p \cdot L \cdot \frac{2}{L-B} = m_p \cdot \frac{8L}{L-B}.$$
(10)

The external work generated as the external force F moves is

 $W_{ext} = F \cdot \delta = F. \tag{11}$

By requiring equilibrium we get

$$W_{ext} = -W_{int},\tag{12}$$

which leads to

$$F = m_p \cdot \frac{8L}{L-B} <=> m_p = F \cdot \frac{L-B}{8 \cdot L}$$
 or $F = 10 \cdot m_p <=> m_p = 0.1 \cdot F.$ (13)

The presented YLT model does not consider the fan mechanisms in the corners and the possible uplift of the corners. To consider the worst case scenario this might cause, another model was also calculated (Figure 3-3).



Figure 3-3. Conservative YLT model for EFNARC panel test

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For this model the internal work (Equation 9) becomes

$$W_{int} = -4 \cdot m_p \cdot \sqrt{2} \cdot \frac{1}{2} \cdot L \cdot \frac{\delta}{\sqrt{2} \cdot \frac{1}{4} \cdot L} = -4 \cdot m_p \cdot \sqrt{2} \cdot \frac{1}{2} \cdot L \cdot \frac{4}{\sqrt{2}L} = 8 \cdot m_p.$$
(14)

The external work generated as the external force F moves is

$$W_{ext} = F \cdot \delta = F. \tag{15}$$

By requiring equilibrium (Equation 12) we get

$$F = 8 \cdot m_p <=> m_p = \frac{1}{8}F.$$
 (16)

This result suggests that neglecting the corner uplift and/or fan mechanisms and/or the effect of the loading plate can generate a maximum analytical error of up to +25 %. To reduce risk of unsafe design, the following factors should be used together cumulatively with the partial safety factors:

i. Point load capacity requirement calculated based on theoretical material parameters should be increased by +25 % (multiplied by 1.25).

ii. Back calculated plastic moment should be reduced by -20 % (multiplied by 0.80) when using the results from the EFNARC panel test.

iii. Alternatively, the conservative YLT model may be used directly.

The problem with the EFNARC panel test is that it's not very reliable when compared against actual tested behaviour. It has multiple possible failure patterns, it suffers from fan effects, it does not restrict the uplift of corners and it has been designed primarily to produce energy absorption results for fallout of softer rocks or for stress induced damage absorption. A more reliable test, which is more suitable for testing of bending capacity of shotcrete, does exist, but the use of it is currently (2011) not permitted by the European Norms regulating the use of shotcrete. The standard in question is the American Society for Testing and Materials (ASTM) C1550:2010a which specifies a circular slab (Figure 3-4). The circular plate is L = 800 mm \pm 10 mm wide (diameter) and h = 75 mm +15/-5 mm thick.



Figure 3-4. YLT model for ASTM C1550 panel test

Here the internal work (Equation 9) becomes

$$W_{int} = -3 \cdot m_p \cdot \sin 60^\circ \cdot L \cdot \frac{\delta}{\frac{L}{2}} = -3 \cdot m_p \cdot \frac{\sqrt{3}}{2} \cdot L \cdot \frac{2}{L} = 3 \cdot \sqrt{3} \cdot m_p.$$
(17)

The external work generated as the external force F moves is

$$W_{ext} = F \cdot \delta = F. \tag{18}$$

By requiring equilibrium (Equation 12) we get

$$F = 3 \cdot \sqrt{3} \cdot m_p <=> m_p = \frac{1}{3 \cdot \sqrt{3}} F.$$
 (19)

3.3 Shotcrete failure modes

Six shotcrete failure modes are described by Barrett & McCreath 1995: a) adhesive, b) flexural, c) shear, d) punching, e) compressive and f) tensile failure (Figure 3-5). They state that falling block

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tests indicate the adhesive (a), shea1r (c) and punching (d) as the most likely modes.



Figure 3-5d. Punching failure Figure 3-5e. Compressive failure Figure 3-5f. Tensile failure (modified from Barrett and McCreath 1995)

Adhesive failure (a) means that the adhesion between the rock surface and the shotcrete is lost (usually because of tension perpendicular to the surface). This mode precedes the flexural (bending) failure (b) where the shotcrete bends so much, that a crack opens up in mid span. Ultimately the crack grows through the shotcrete. This is not enough for a failure, but also negative cracks (crack initiation at the surface between rock and shotcrete) must develop. Together these cracks form a mechanism and the structure fails.

If the loading is a rigid block (e.g. typical Scandinavian key block or wedge), then a direct shear failure (c) is possible. For a large rigid load or loose rock, the rock bolts can punch (d) through the shotcrete layer. Compressive failure (e) could occur in a shaft that's being heated up (for example inlet shaft with preheated air temperature exceeding the rock mass temperature) and tensile failure (f) can occur in a shaft that's being cooled down (for example inlet shaft during winter with no preheating). Typically compressive and tensile failures are observed after building another rock space in the vicinity of the reinforced space or by shotcreting too close to the end of the tunnel which can lead to a compressive failure unless the elastic modulus of shotcrete is suitably low. Other failure modes exist (e.g. combination modes), but the presented modes cover well over 90 % of typical design problems.

3.4 Cross sectional stresses

Two states can be recognized for shotcrete structures in pure bending: the uncracked intact state (Figure 3-6a) and the cracked residual state (Figure 3-6b). The uncracked state is linear and symmetric until the bending tensile strength is exceeded. After a crack develops, an unreinforced shotcrete structure becomes unstable and will progressively fail. Fibre reinforced structure may have a stable residual state (depending on the fibre type and dosage). If such a state exists, it consists of an elastic part and a plastic part (the fibres). Eventually the elongation of the fibres becomes too much and they will start failing. The compression side is not expected to develop

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plasticity until a very late state in the failure progress.



The cracking moment of unreinforced shotcrete can be calculated using elastic theory as

$$m_p = \frac{\sigma_t h^2}{6},$$

(20)

where σ_t is the tensile strength

h is the thickness of the plate.

The cracking event always precedes the failure or the residual state of the structure. The cracking strength can be used in design, when brittle behaviour is expected (unreinforced shotcrete) or when no stable residual state is expected (typical or low dosages). When a high dosage is used (or when high energy class e.g. E1000 is designated) the residual state may have to be calculated to further optimize the design. This also applies for very thick shotcrete layers (120 mm or more). The residual state calculations can be done iteratively using computer aided design. It should be possible to plot the residual state capacity as a function of displacement and compare the results to the actual measured force vs. displacement diagrams from the standardized tests.

Holmgren (1993) recommends that ideally elastoplastic or ideally plastic models should be not be used. The author wishes to confirm that no evidence was found during this study that would suggest ideally elastoplastic or ideally plastic behaviour. It should be noted however, that suitably thick shotcrete layers with mesh or rebar reinforcement should exhibit this type of behaviour.

4. Results

4.1 Material parameters for shotcrete

The material parameters for shotcrete according to Eurocode can be calculated as shown in chapter 3.1. The Table 4-1 summarizes the results calculated for this work using the nationally determined parameters of the Finnish NADs (National Annex Documents). The parameters are given as characteristic values and they can be converted into design values by dividing them with the partial safety factor of 1.50 (standard structures, structural class II), 1.35 (high end structures, structural class I) or 1.20 (accident cases) according to the Finnish NAD. As an approximation, the tensile strength for pure bending failure can be multiplied by a factor of 1.50 (which is the exact solution for h = 100 mm, see EN 1992-1-1 s. 3.1.8 for details).

Class (<i>f_{ck}/f_{ck,cube}</i>) [MPa]	Unreinforced compressive strength $(f_{c,pl})$	Unreinforced tensile (or shear*) strength $(f_{ct,pl}/f_{cv,pl})$	Fibre reinforced compressive strength (f_c)	Fibre reinforced tensile (or shear*) strength (f_{ct}/f_{cv})	Elastic modulus (Young's modulus) (E_{cm})
C8/10	5.4	0.50	6.8	0.84	25.3
C12/15	8.2	0.66	10.2	1.10	27.1
C16/20	10.9	0.80	13.6	1.33	28.6
C20/25	13.6	0.93	17.0	1.55	30.0
C25/30	17.0	1.08	21.3	1.80	31.5
C28/35**	19.0	1.16	23.8	1.94	32.3
C30/37	20.4	1.22	25.5	2.03	32.8
C32/40**	21.8	1.27	27.2	2.12	33.3
C35/45	23.8	1.35	29.8	2.25	34.1
C40/50	27.2	1.47	34.0	2.46	35.2
C45/55	30.6	1.59	28.3	2.66	36.3
C50/60	34.0	1.71	42.5	2.85	37.3

Table 4-1. Material parameters for unreinforced and fibre reinforced shotcrete

*) Using the conservative assumption that $\sigma_{cp} = 0$

**) Midlevel of NAD to SFS-EN 206-1 presented in the Finnish Building Code part B4:2005

4.2 Yield-Line Theory models

Using the Yield-Line Theory and the described testing specifications for the EFNARC panel test and the ASTM circular plate test it was possible to derive the relationship between the testing force and the plastic moment capacity of the shotcrete (see 3.2). The results calculated in this work are presented in Table 4-2.

	Table 4-2.	Results	from	Yield-Line	Theory	/ models
--	------------	---------	------	------------	--------	----------

Model	Loading force (F)	Plastic moment capacity (m_p)
EFNARC upper bound	$10 \cdot m_p$	$0.1 \cdot F$
EFNARC conservative	$8 \cdot m_p$	$\frac{1}{8}F$
ASTM C1550	$3 \cdot \sqrt{3} \cdot m_p$	$\frac{1}{3 \cdot \sqrt{3}}F$

The values presented in Table 4-2 are characteristic values and should not be used directly in design. When back calculating the plastic moment capacities to be used in design, the values obtained should be divided with the safety factor for concrete (EN 1992-1-1 s. 2.4.2.4) to obtain design values.

The nominal tensile strength of fibre reinforced C35/45 shotcrete is $f_{ct} = 2.25MPa$ (Table 4-1). The height of the EFNARC panel is h = 100 mm, so the multiplier from equation EC2 3.23 becomes 1.5x (pure bending characteristic tensile strength $f_{ct,fl} = 3.38MPa$). Using the elastic cross sectional cracking capacity (Equation 20) the loading force (Table 4-2) for the upper bound EFNARC panel (Figure 3-2) this becomes

$$F_{EFNARC,upper} = 10 \cdot \frac{3.38 \frac{N}{mm^2} (100mm)^2}{6} = 56.3 \, kN \,. \tag{21}$$

For the conservative EFNARC model (Figure 3-3), the loading force becomes

$$F_{EFNARC,conservative} = 8 \cdot \frac{3.38 \frac{N}{mm^2} (100mm)^2}{6} = 45.1 \, kN \,.$$
⁽²²⁾

For the round panel test (Figure 3-4), the height is different and the height corrected bending tensile strength is $f_{ct,fl} = 3.43MPa$ which results in a loading force of

$$F_{C1550} = 3 \cdot \sqrt{3} \cdot \frac{3.43 \frac{N}{mm^2} (75mm)^2}{6} = 16.7 \ kN \ . \tag{23}$$

Since the standards use fixed dimensions without significant tolerances, one can derive the conversion factors between the three presented YLT models:

F _{EFNARC,upper}	= 1.25 x F _{EFNARC,conservative}	= 3.37 x F _{C1550}	(24)
F _{EFNARC,conservative}	= 0.80 x F _{EFNARC,upper}	= 2.70 x F _{C1550}	(25)
F _{C1550}	= 0.30 x F _{EFNARC,upper}	= 0.37 x F _{EFNARC,conservative}	(26)

4.3 Shotcrete failure modes

a) Adhesion capacity

To acquire adhesion capacity, Barrett and McCreath 1995 have back calculated the results of Hahn and Holmgren 1979 and Fernandez-Delgado et al. 1981. The required adhesion strength should be specified as a design requirement (a typical requirement for high grade shotcrete in hard rock could be 0.50 MPa). If the adhesion strength is unknown, the value 0.40 MPa may be used as a conservative approximation. The measured test results are between 0.00 MPa ... 2.00 MPa. Based on Barrett and McCreath 1995, a conservative bond width of 30 mm may be used in design. Design adhesion capacity may be calculated as

$$R_{ad} = \frac{f_{ak} \cdot s \cdot b}{\gamma_c},\tag{27}$$

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where f_{ak} is the adhesion strength (bond strength)

- s is the perimeter of the load to be supported
- *b* is width of the adhesion area (if unknown, 30 mm may be used)
- γ_c is the partial safety factor for concrete (EN 1992-1-1 s. 2.4.2.4).

b) Bending capacity

The author suggests using the Yield Line Theory (YLT) to acquire the bending capacity. YLT requires the plastic moment capacity (or the crack initiation capacity) which can be acquired from the shotcrete loading tests (see 4.2) or calculated theoretically (see 3.3). The YLT produces results that are similar to actual test results by cracking pattern and by load intensity (see 5.2)

Barrett and McCreath (1995) suggest using the Direct Design Method for flexural capacity of unreinforced shotcrete structures. According to them the DDM the design moment is given by

 $M_0 = \frac{1}{8} \cdot w \cdot s \cdot (s - c)^2,$

(Barrett and McCreath 1995, Eq. 4) (28)

where w is the magnitude of the uniformly distributed load

- *s* is the bolt spacing
- *c* is width of the faceplates

and the DDM design capacity for unreinforced shotcrete is given by

$$C_{flex} = \sigma_{flex} \cdot \frac{t^2}{6} \cdot \frac{s}{2},$$
 (Barrett and McCreath 1995, Eq. 5) (29)

where σ_{flex} is the pure bending tensile strength (see Eq. EC2 3.23)

- *t* is shotcrete thickness
- *s* is the bolt spacing.

Attention! The author believes may be a typing error in the Barrett and McCreath formulas for the design moment and for the flexural capacity. The corrected formulas without the mute parameter of width are

 $M_{0,corr} = \frac{1}{8} \cdot w \cdot (s-c)^2,$ $C_{flex,corr} = \sigma_{flex} \cdot \frac{t^2}{6},$ (30)
(31)

(modified from Barret and McCreath 1995)

Holmgren (1993) has developed a multiplier to take in account the residual capacity of highly reinforced shotcrete. Obviously, if the residuals are less than 100 %, then it makes little point of using the formula. The Holmgren multiplier for bending strength is

$$H = 0.90 \cdot \frac{R_{10/5} + R_{30/10}}{200},\tag{32}$$

where $R_{10/5}$ is a residual stress factor according to ASTM C1018

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 $R_{30/10}$ is a residual stress factor according to ASTM C1018. (modified from Holmgren 1993)

The EN 14488-5 test does not provide the necessary residual factors to be used in conjunction with the Holmgren multiplier. Instead, the author suggests using the 5 % quartile of the maximum load and back calculating the residual capacity using the YLT method (see 4.2). The 5 % quartile can be obtained from the test results using the normal distribution presented in this work

$$F_{max,0.05} = \mu_{F_{max}} - 1.645 \cdot \frac{\sigma_{F_{max}}}{\sqrt{n}},$$

where $\mu_{F_{max}}$ is the mean of all maximum loads for the same shotcrete type

$$\sigma_{F_{max}}$$
 is the standard deviation of all the maximum loads for the shotcrete type

n is the number of experiments for the same shotcrete type.

c) Shear failure

The calculation of the shear failure is similar to the calculation of adhesion capacity. The shear strength is a parameter that can be calculated from the shotcrete class (Table 4-1) and the thickness of the shotcrete layer will be specified in the designs. Shear failure capacity may be calculated as

$$R_{vd} = \frac{f_{cv} \cdot s \cdot t}{\gamma_c},\tag{34}$$

where f_{cv} is the shear strength (see Table 4-1)

- *s* is the perimeter of the load to be supported
- t is thickness of the shotcrete layer
- γ_c is the partial safety factor for concrete (EN 1992-1-1 s. 2.4.2.4).

d) Punching failure

Barrett and McCreath 1995 suggest using the Canadian Reinforced Concrete Design Code (CSA Standard CAN3-A23.3-M84). For Eurocode based design, the SFS 1992-1-1 section 6.4 for punching calculation may be followed directly. The calculation sequence is rather long and will not be presented in the scope of this work. In Finland the section 6.4 is currently (2011) not allowed to be used and the Finnish building regulations (Finnish Building Code part B4:2005) for punching must be used instead.

e) Compressive and f) Tensile failure

These failure modes will not be studied further in the scope of this work. One approach is to model the problem using numerical methods and query the theoretical stresses. These stresses must be less than the compressive and tensile strengths given in Table 4-1 divided by the safety factor (see EN 1992-1-1 s. 2.4.2.4). The elastic modulus (E_{cm}) may be taken from Table 4-1 and the Poisson's value (ν) can be taken as 0.20 for uncracked concrete and as 0.00 for cracked concrete (EN 1992-1-1 s. 3.1.3). For time dependent behaviour see EN 1992-1-1 section 3.

(33)

5. Discussion

5.1 Material parameters for shotcrete

Table 4-1 presents the most relevant material parameters for shotcrete excluding the properties of the fibres. Steel fibres are mainly used to distribute the initial (mostly drying induced) cracking. They also act as an expansion limiting reinforcement and will hold small (e.g. fist-sized) chunks of broken shotcrete. If enough steel fibres are applied, then the residual (cracked) state may be used in calculations either using the theoretical approach (see 3.4) or the back-calculated approach (see 4.3b).

The results presented should be directly applicable to design work. A lot of the parameters have already been experimentally validated and the usage of shotcrete requires on site testing which will allow the designer to ensure that proper mix is used. There are a lot of factors that will be ignored to simplify the analysis (e.g. the bolt installation tolerance, the excavation tolerance, spraying tolerance, the jaggedness of the blasted rock...). Any risk these factors may be generate will be assumed to be handled by the partial safety factor approach of Eurocodes, which generates a total factor of safety of $1.35 \cdot 1.50 \approx 2$ for conventional shotcreted structures.

The author sees little point in demanding energy classes (E500, E700, E1000) for the fibres of shotcrete in hard rocks. Energy class requirement does not guarantee ductile behaviour after cracking. Instead, for load capacity the ultimate load should be specified in kN or for ductile behaviour it should be specified that the ultimate load must be at least 1.20 times the cracking load. However, in weak rock, earth like conditions or in energy absorption cases (weapons effect, blasting damage or stress induced damage) the energy classes should be used.

5.2 Yield-Line Theory models

The author has had the opportunity to first calculate and then witness the testing of the accuracy of a YLT model in person at the laboratories of the Helsinki University of Technology (now Aalto University). The YLT is very convenient and allows for easy hand calculation. This work presents two models for the EFNARC EN 14488-5 square, continuously supported panel test, because the EFNARC method is prone for errors. Olli Salo (2010) has pointed out in his master's thesis that the ASTM C1550 round panel is much more robust experiment and the three-point support ensures consistent performance. Figure 5-1a and Figure 5-1b show actual test results of two EFNARC panels from Statens Vegverket report 2534:2009. Figure 5-1c shows actual test result from the ASTM C1550 panel from Bernard 2003.



Figure 5-1a. EFNARC, upper bound



Figure 5-1b. EFNARC, conservative



Figure 5-1c. ASTM C1550 round panel

The EN 14488-5 specifies that the test report shall include "scetch or photograph showing the number and location of the cracks". This is very convenient, because using the true crack pattern, it is possible to back calculate the effect of any testing errors. EN 14488-5 does not require the cracking load to be announced, so if the designer is interested in that parameter he or she will have to announce it in the design papers or back calculate it from the load-deflection graph.

5.3 Shotcrete failure modes

In practical design of shotcrete for rock reinforcement in hard rocks, the most convenient approach is to calculate the shear and adhesion capacities. Bending capacity does not need to be checked, when the adhesion capacity is high enough. However, for certain cases the bending capacity should always be checked (e.g. large slickensided, planar surfaces).

Design of shotcrete for hard rocks differs from the design of shotcrete for soft rock and soil mainly because the loading is different. For soft rock and soil, the bending capacity should always be calculated, because adhesion has little or no physical meaning and shear failure is unlikely. Additionally punching failure is always a possibility in softer rocks. A good description of the design philosophy behind shotcrete design is presented by Stille and Franzén 1993.

The compressive and tensile failures are well suited for computer aided design. The modelling parameters are presented in the work and the analysis is versatile given the high calculation power of modern computers.

5.4 Cross sectional stresses

In this work the pure bending mode was assumed. It is likely that the true mode contains either compression or tension as well. This may change the behaviour of the shotcrete layer (compression makes it stronger for bending and for shear while tension weakens it in both). When shotcreting is done reasonable distance away from the excavation end then the linear elastic assumption is reasonable. The jaggedness of the rock surface causes local concentration points for stresses which may generate localized failures. If the jaggedness is random, this effect should be negated by the low probability of them occurring in a large enough area. However, they may cause small scale damage to the shotcrete layer.

6. References

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