LOAD CAPACITY OF GROUTED ROCK BOLTS DUE TO DEGRADATION

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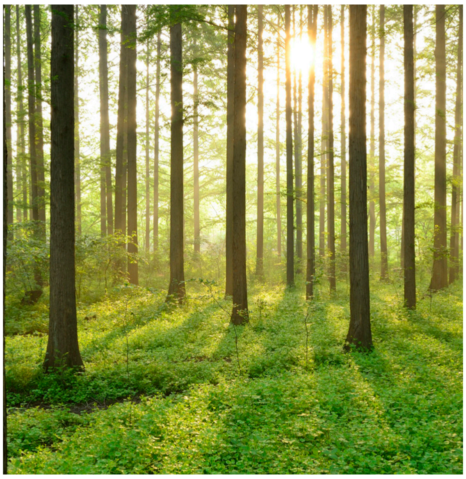

Energiforsk

Load capacity of grouted rock bolts due to degradation

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Foreword

Projekt har bedrivits inom Svenskt vattenkraftcentrum, SVC som är ett centrum för utbildning och forskning inom vattenkraft och gruvdammar. Svenskt vattenkraftcentrum är etablerat av Energimyndigheten, Energiforsk och Svenska Kraftnät tillsammans med Luleå tekniska universitet, Kungliga tekniska högskolan, Chalmers tekniska högskola och Uppsala universitet.

Projektet genomfördes som ett samarbete mellan SVCs verksamhetsområden Geoteknik och bergmekanik (grundläggning) samt Konstruktionsteknik. Huvudsakliga utförare och författare till rapporten är seniorforskarna Richard Malm och Fredrik Johansson, båda verksamma vid KTH i Stockholm. I projektet arbetade även Rikard Hellgren och Fransisco Rios Bayona, båda doktorander vid KTH.

Medverkande organisationer är: Andritz Hydro, Falu Energi & Vatten, Fortum Generation, GE Renewable Energy, Holmen Energi, Jämtkraft, Jönköping Energi, Karlstads Energi, Mälarenergi, Norconsult, Rainpower, Skellefteå Kraft, Sollefteåforsens, Statkraft Sverige, Sweco Energuide, Sweco Infrastructure, SveMin, Umeå Energi, Uniper, Vattenfall Research and Development, Vattenfall Vattenkraft, VG Power, WSP Samhällsbyggnad och ÅF Industry.

Sammanfattning

I föreliggande rapport har bärförmågan hos slaka bergsförankringar (bergbultar) studerats med hänsyn till inverkan av nedbrytning. Detta projekt har som första steg genomförts som en litteraturstudie för att bestämma lämplig spridning på korrosionshastigheten, men även för att hitta information från genomförda inspektioner och statusbedömningar. Samtliga fall som har hittats i litteraturen visar att bergbultar för förankring av betongdammar varit i mycket god kondition med endast begränsad ytlig korrosion, trots att bultarna kan ha varit i drift i mer än 50 år.

I projektet har en metodik utvecklats där nedbrytningen av bergbultar kan inkluderas vid stabilitetsberäkningar av betongdammar. I denna metod inkluderas samtliga potentiella brottmoder, där den med lägst bärförmåga blir styrande. Eventuell reduktion av bärförmågan sker genom att nedbrytningen skattas baserat på vattnets kemiska innehåll i enlighet med en tysk DIN norm.

De flesta fältprovningar av bergbultar redovisar endast lastkapaciteten d.v.s. själva deformationsförloppet vid provdragning saknas. Detta är dock mycket viktig input till att kunna verifiera numeriska och analytiska beräkningar. Inom projektet har därför även egna fältprovningar genomförts av bergbultar som varit i drift i ca 50 år. En vidareutveckling av den testrigg som använts vid tidigare försök genomfördes för att kunna registrera både belastning och deformation. Testriggen består av en hydrauliskkolv som trycker en cylinder mot berget som omsluter bergbulten. På grund av testriggens utformning går det ej att fånga ett eventuellt konbrott i berget. Resultaten visade dock att denna metod kan överskatta bärförmågan i de fall då brottet sker i bruket, p.g.a. inspänningseffekter från testutrustningen.

En fallstudie presenteras i rapporten avseende stabilitetsberäkningar med den analytiska metodiken, finita elementanalyser samt probabilistiska analyser. De numeriska analyserna visade, som väntat, att bultarna belastas av kombinationer av drag- och skjuvkrafter. Resultaten visade även att skjuvkrafterna konstant var högre än normalkrafterna i bulten, och att dessa motsvarande ca 10 % av totala skjuvmotståndet mot glidning vid normala lastnivåer. Detta illustrerar vikten av att inkludera ett brottvillkor där kombinationer av drag och skjuvkrafter kan beaktas.

De probabilistiska analyserna visade att sannolikheten för stjälpning är extremt låg, därmed bör denna knappast anses vara en rimlig brottmod. Från de numeriska analyserna framgick att deformationen ofta startar enligt en stjälpningsbrottmod där uppströmstån förlorar kontakten, detta leder dock till att skjuvkraften måste bäras av en mindre yta vilket initierar ett glidbrott. Detta indikerar att det vore mer lämpligt att definiera en brottmod som avser begränsning av dragspänningar i uppströmstån för bruks¬gräns¬tillstånd, istället för en stjälpningsbrottmod.

Summary

In this report, the influence of degradation on the strength of rock bolts has been studied. A literature study has been performed in order to determine the degradation rate and to present observations and conclusions from available assessments of rock bolts. All cases found in the literature have shown that the rock bolts on concrete dams are in good condition with only minor superficial corrosion even after 50 years of service.

In the project, a methodology to account for the degradation mechanism in evaluations of dam safety is presented, where all possible failure modes of rock bolts are considered. The contribution of the rock bolts to the dam stability is based on the failure mode with the lowest strength. The degradation has been taken into account based on a German DIN standard based on the chemical content of the water.

Most available field tests have only measured the load capacity of bolts, where the deformation is typically not recorded. The relationship between forces and deformations is however important input to verify numerical and analytical analyses. Therefore, field tests have been performed on rock bolts that been in service for 50 years. A previously developed test rig had been modified in this project to register both load and deflection of the pull-out test. The test rig consists of a hydraulic jack that presses a cylinder towards the rock surrounding the bolt. Due to this configuration of the test equipment, a rock cone failure cannot be captured. The results showed that the test rig may influence the obtained load capacity if the failure occurs in the grout.

A case study is presented where analytical, probabilistic and finite element analyses were performed to assess the dam safety. Based on the numerical analyses, it was possible to study the development of forces in the rock bolts due to successively increasing loads. The numerical analyses showed (as expected) that the rock bolts are subjected to both shear and tensile forces at the same time. In addition, the shear force was constantly higher than the tensile forces and that the shear forces were about 10% of the total shear resistance for normal loads. This implies that it is important to use a failure criterion for the rock bolts that considers combinations of tensile and shear forces.

Besides this, the probabilistic analyses showed that pure overturning failure is extremely unlikely and cannot be considered as a relevant failure mode. The numerical analyses showed that the deformation start as for overturning failure resulting in that parts of the contact surface (on the upstream side) lose its contact. Thereby, the shear forces have to be transmitted over a reduced area which initiates the sliding failure. This implies that it is more suitable to define a criterion that limits the tensile forces in the upstream toe from serviceability loads, rather than having an overturning failure criterion.

List of content

1 Introduction

In this chapter, a brief background of the performed study regarding the load capacity of passive rock bolts is given. In addition, the objectives for the study are presented along with a description of how passive rock bolts could be considered in design guidelines for hydropower and dams.

1.1 BACKGROUND

Due to the inaccessible placement of passive rock bolts at the interface between rock and concrete, it is often not possible to verify their structural status in an aging hydraulic structure. During condition assessment and stability evaluation of older dams, the contribution from rock bolts may be significant but their current strength is however considered to be associated with large uncertainties.

In the Swedish power companies' guideline for dam safety (RIDAS), concrete dams are divided into three main categories depending on the consequence of a dam failure (RIDAS, 2011). According to RIDAS (2011), it is not allowed to include the strength of passive rock bolts in the stability assessments of new dams. However, for existing dams it has been shown that it may be difficult to satisfy the current safety criteria's. Therefore, in some cases, it is possible to consider the rock bolts in a highly conservative manner. For dams in the highest consequence class, the contribution from rock bolts should be neglected in all stability calculations. For dams in the lower consequence classes, the contribution of rock bolts may be included where maximum allowed reinforcement stress is limited to 140 MPa for steel with characteristic yield strength of at least 370 MPa (RIDAS, 2011). However, the basis for this design value is unknown, and thus it is important to try to explain the origin of this value.

In Sweden, most of the existing 10 000 dams were built before 1970. Issues concerning structural health, remaining capacity and degradation is therefore of great interest. Many of the dams were anchored to the ground with a significant amount of passive rock bolts, but according to the guidelines these may no longer be considered in stability calculations. Therefore, in cases where existing dams cannot satisfy the current stability criteria, then it is common to further anchor the concrete dams with pre-stressed rock bolts. A lot of money and effort could in several cases be saved if the actual strength of the rock bolts could be assessed and regarded in stability calculations. In order to include the contribution of rock bolts in stability analyses, a method is required where it is be possible to assess the strength of the bolts.

1.2 OBJECTIVE

The aim of the project is to develop a calculation method that takes into account how deterioration affects the load bearing capacity of rock bolts. The project will analyse the load capacity for grouted rock bolts as a function of its age, depending on the surrounding environment.

The purpose of this report is to highlight the influence of passive rock bolts and show how these may be assessed and included in stability calculations. As discussed in Section 1.3, passive rock bolts are considered as an extra safety that should not be included in stability calculations according to RIDAS for dams in the highest consequence class. This leads to that the actual safety factor of the dam is unknown, except that the safety of the dam is higher than assumed in the stability calculations.

1.3 CONSIDERATION OF ROCK BOLTS ACCORDING TO RIDAS

In this section, it is described how rock bolts should be considered in design according to RIDAS (2011).

1.3.1 General

For some certain type of hydraulic structures, for instance spillways with low crests or low dams subjected to ice loads, it may be difficult to satisfy the stability criteria's without considering the capacity of passive rock bolts.

However, if the rock bolts are included in the analyses and thereby retain sliding or overturning failures of a part of the dam, then the consequences of this failure should be limited to only this specific part of the dam and should not lead to a complete dam failure.

In cases where only sliding failure are to be prevented, stainless reinforcement steel may be used as force transfer along joints according to the Swedish design code BBK 04 (2004), chapter 3.11.

1.3.2 New dams

In design of new dams, the strength of passive rock bolts should not be considered in stability analyses. It is however recommended that rock bolts of diameter φ25 – 30 are used for additional safety.[1]

In design of new dam parts, subjected to a low water level, rock bolts may be considered in the stability analyses as long as they satisfy the criteria in Section 1.3.3 for "*Low dam parts*".

1.3.3 Existing dams

Low dam parts (new and existing)

In cases where passive rock bolts are considered in the stability analyses, the uplift pressure should be calculated based on conservative assumptions. The maximum allowed stress in the rock bolts is limited to 140 MPa, and it is required that the characteristic yield strength of the bolts should be higher than 370 MPa.

Passive rock bolts may only be considered in cross-sections where the resulting force (calculated without consideration of the rock bolts) is within the structural

[1] This has been common practice on the older dams, where these typically were installed more than one every square meter of the foundation area.

base area. In addition, the rock bolts may only be considered for sliding failure if the friction angle at failure is not exceeded without considering the rock bolts.

For low dams (with a water level less than 5 m) where it is difficult and very uneconomical to achieve stability only through gravity loads, rock bolts may be considered to carry the ice load. One condition is however that the dam is not classified in consequence class 1+ or 1. For cases with low crests, that are subjected to large horizontal forces and uplift pressures, the rock bolts should be designed to carry both uplift and sliding.

Existing dams in consequence class 2 (or lower)

For existing dams in consequence class 2, installed rock bolts may be considered in stability analyses as long as the failure ratios for sliding and overturning are not reached for a case without consideration of the rock bolts, and that it is judged that corrosion will influence the capacity of the bolts.

Strengthening of existing dams

For existing dams that do not satisfy the stability requirements, the dam may be strengthened with post-tensioned anchors (tendons). These tendons should be installed in a manner that allows for regular control and re-tensioning.

In some cases where the dam body consists of poor or unreinforced concrete it could be more beneficial to install several passive rock bolts or post-tensioned anchors with low utilization, i.e. low level of pre-stress. The calculations should be performed based on conservative assumptions. Rock bolts with a diameter of φ25 or higher, should be used to limit the influence of corrosion. The required amount of rock bolts should be increased with +25 % as an additional safety margin. Post-tensioned anchors and passive anchors may not be simultaneously considered in stability analyses, due to their different mode of action.

The results in this report present how assessments of degradation can be performed and probability functions based on these are given. This also constitutes important input to an on-going research project within Energiforsk regarding probabilistic design of the stability of concrete dams, Westberg and Johansson (2016).

2 Rock bolts

Rock bolts are commonly installed in concrete dams in the concrete-rock interface giving a contribution against sliding and overturning stability. This chapter presents the different failure modes of rock bolts, their stability contribution on concrete dams and the degradation processes with focus on corrosion.

In Figure 2-1, an illustration of a rock bolt installed in the rock mass is shown. The rock bolt consists of a reinforcement steel bar, which is drilled into the rock mass. Bond between the reinforcement bar and the rock mass is ensured with the use of grout. The upper part of the rock bolt is then embedded in concrete as the concrete dam or hydraulic structure is cast.

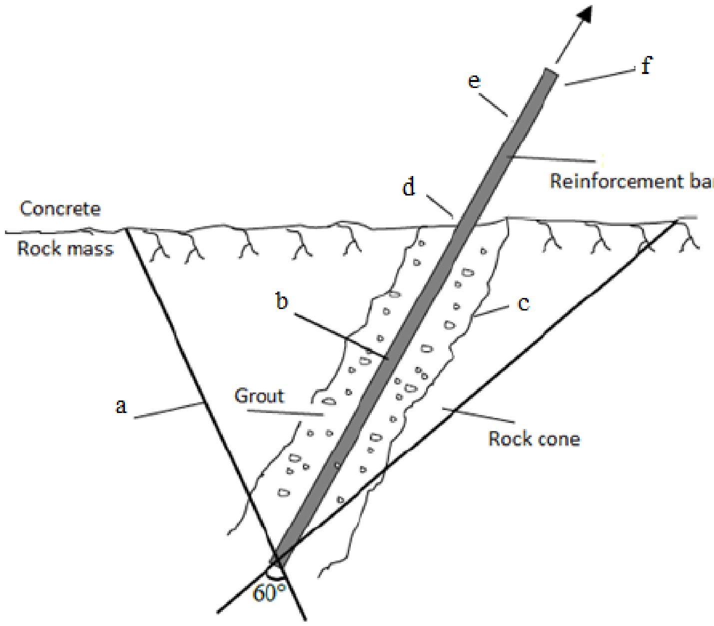


Figure 2-1 Representation of possible failure modes of a rock bolt (Berzell, 2014), a) Rock failure. b) Adhesive failure; steel and grout. c) Adhesive failure; rock and grout. d) Shear failure. e) Adhesive failure; concrete and steel. f) Tensile failure; steel bar.

The capacity of a rock bolt is assumed to behave as a weakest link system, which means that the rock bolt will fail when the lowest value of the failure modes described in Figure 2-1 is reached.

$$R_{rock\ bolt} = \min(R_{rock}, R_{rock-grout}, R_{steel-grout}, R_{concrete-steel}, R_{steel}) \qquad \textbf{(2.1)}$$

In the following sections, methods to assess the strength of each possible failure mode are given.

2.1 FAILURE MECHANISM OF A ROCK BOLT

Ekström et al. (2013) describes possible failure modes for rock bolts. The following section is based on the conclusion from this reference. The methodology and assumptions used in this document concerning the behaviour of a rock bolt in the different failure modes of a concrete dam, as well as the different failure criteria are presented in this chapter.

2.1.1 Failure in the rock

Failure in the rock occurs when a rock volume (theoretically shaped as a cone) fails before the steel capacity is reached. The weight of the rock cone can be estimated according to eq. (2.2), assuming a cone angle of 60° and that both cohesion and friction in the rock interface are neglected, see Figure 2-2 (Berzell, 2014).

$$R_{rock} = V_{cone} \cdot \gamma_{rock} = \frac{\pi r^2 h}{3} \cdot \gamma_{rock} \qquad \textbf{(2.2)}$$

where $R_{rock}$is the rock bolt load that mobilises a rock volume V_{cone} assuming an angle of 60°, γ_{rock} is the rock density and h and r are the geometrical quantities defined in Figure 2-2.

In this report, it is assumed that this failure mechanism is plausible for a concrete dam when the overturning failure mode is analysed. For the sliding failure mode, this failure mechanism has not been taken into account.

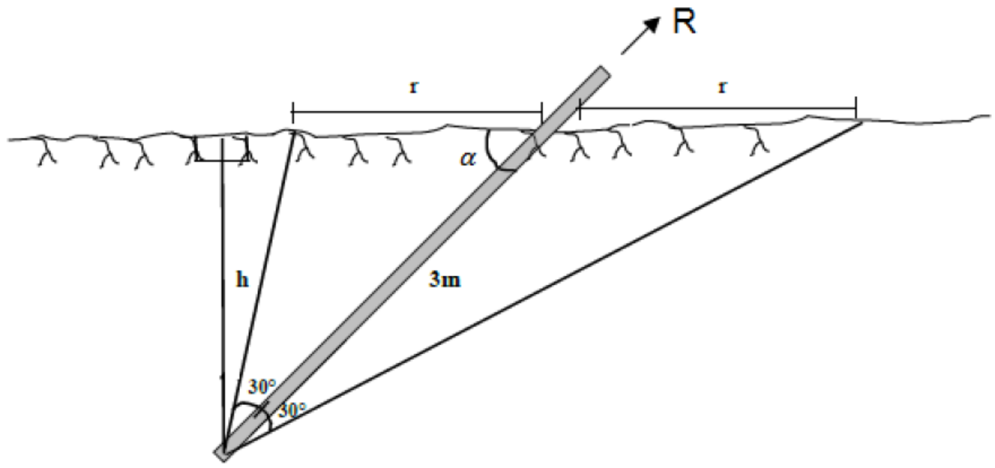


Figure 2-2 Rock cone failure (Berzell, 2014).

According to the Norwegian guidelines for dam safety, the unit weight of rock should be chosen depending on the rock type according to the value in Table 2-1 (NVE, 2005).

Table 2-1 Unit weight for different rock types according to the Norwegian guidelines for dam safety (NVE, 2005).

Rock type	Unit weight [kN/m³]
Granite	26
Gabbro	28
Gneiss	26
Sandstone	27
Limestone	24
Shale	28
Quartzite	25

2.1.2 Adhesive failure; rock and grout

The adhesive failure in the rock-grout interface can be defined as follows:

$$R_{rock-grout} = A_{surface} \cdot c_{rock-grout} = \pi \cdot \emptyset_{bore\ hole} \cdot L_{rock} \cdot c_{rock-grout} \quad \textbf{(2.3)}$$

where $R_{rock-grout}$ is the resistance of the contact interface between rock and grout, $\emptyset_{bore\ hole}$ is the diameter of the bore hole, L_{rock} is the embedded length of the rock bolt in the rock and $c_{grout-rock}$ is the adhesive strength in the contact interface between rock and grout. This adhesive failure depends on the rock mass properties, especially the number of cracks and fractures (Berzell, 2014).

According to the Norwegian guidelines for dam safety, the value of the adhesive strength between the rock and grout depends on the rock type and should be chosen according to Table 2-2 (NVE, 2005). Variations in adhesive strength between grout and granite, sandstone and limestone are also included in the table based on Avén (1984). The lower values are to be used for weathered rock and if the rock is highly fractured.

Table 2-2 Adhesive strength between rock and grout according to the Norwegian guidelines for dam safety (NVE, 2005).

Rock type	NVE (2005) [MPa]	Avén (1984) [MPa]
Granite	2.0	2.0 - 5.0
Gabbro	2.5	-
Gneiss	1.5	-
Sandstone	1.2	0.5 - 2.0
Limestone	2.0	1.0 - 3.0
Shale	0.5	-
Quartzite	2.5	-

2.1.3 Adhesive failure; steel and grout

Similarly to failure in the grout-rock interface, adhesive failure between steel and grout is defined in equation (2.4):

$$R_{steel-grout} = A_{surface} \cdot c_{steel-grout} = \pi \cdot \emptyset_{steel} \cdot L_{rock} \cdot c_{steel-grout} \tag{2.4}$$

where $R_{steel-grout}$ is the resistance of the contact interface between steel and grout, $\emptyset_{steel}$ is the diameter of the steel bar, L_{rock} is the embedded length of the rock bolt in the rock and $c_{steel-grout}$ is the adhesive strength in the contact interface between steel and grout.

According to the Norwegian guidelines for dam safety, the characteristic value of the adhesive strength between steel and grout is 3.0 MPa and should be used with a material safety factor of 2.0 (NVE, 2005). Therefore, the design value is 1.5 MPa. According to Avén (1984), the adhesive strength between steel and grout is proportional to the compressive strength of the grout. For grout with a compressive strength in the interval 20 to 40 MPa, the adhesive strength between steel and grout varies between 1.2 MPa 1.9 MPa for smooth bars and between 1.7 and 2.6 for ribbed bars according to Avén (1984).

2.1.4 Adhesive failure; concrete and steel

The adhesive failure between steel and concrete is according to BBK 04 (2004) as follows:

$$\begin{aligned} R_{concrete-steel} &= A_{surface} \cdot f_b \\ &= \pi \cdot \emptyset_{steel} \cdot L_{concrete} \cdot (\mu_1 \cdot \mu_2 \cdot \mu_3 \cdot \mu_4 \cdot f_{ctd} + f_{tranverse}) \end{aligned} \tag{2.5}$$

where $R_{concrete-steel}$ is the resistance of the contact interface between concrete and steel, $L_{concrete}$ is the embedded length of the rock bolt in the concrete, μ_x are constants reflecting position and type of reinforcement specified in BBK 04 (2004), f_b is the adhesive strength steel and concrete, f_{ctd} is the design value of the tensile strength of the concrete and $f_{tranverse}$ is the strength of transverse reinforcement.

If the characteristic compressive strength (f_{ck}) is known, the mean value of the tensile strength, f_{ctm}, and the design value of tensile strength, f_{ctd}, can according to Eurocode be calculated as

$$f_{ctm} = 0.3 f_{ck}^{\frac{2}{3}} \tag{2.6a}$$

$$f_{ctd} = \frac{f_{ctk}}{\gamma_c} = \frac{0.7 f_{ctm}}{\gamma_c} \tag{2.6b}$$

where γ_c is the partial coefficient of concrete which is equal to 1.5 for permanent and temporary loads.

The most commonly used rock bolts are of the type ribbed reinforcement bars. Assuming un-cracked concrete the values presented in Table 2-3 are obtained for the constants.

Table 2-3 Table for rebar bond constants on concrete

Constant	Value	Assumption
μ_1	1.4	ribbed rebar
μ_2	0.8	Uncracked concrete
μ_3	1.0	No bundling of bars
μ_4	1.0 (or calculated)	Conservative value (the calculated value depending on edge and internal distance between bolts)

2.1.5 Steel failure

Failure in steel is one of the failure mechanisms that occur in a rock bolt when its bearing capacity is reached.

The tensile capacity is equal to the cross section area multiplied by the maximum allowed tensile stress (see eq. (2.7)) and the shear capacity is equal to the cross section area multiplied by the maximum allowed shear stress (see eq. (2.8)). The maximum allowed shear stress is often related to the maximum tensile stress through the von Mises yield criterion as shown in eq. (2.8).

$$N_{steel} = A_s f_s \tag{2.7}$$

and

$$V_{steel} = \tau_{max} A_s = \frac{1}{\sqrt{3}} A_s f_s \approx 0.5 A_s f_s \tag{2.8}$$

where, N_{steel} and V_{steel} is the resistance of the steel bolt due to normal and shear forces respectively, A_s is the cross-sectional area of the bolt and f_s is the yield strength of the steel.

Several studies on the behaviour of rock bolts in rock engineering exist (Holmberg, 1992; Spang & Egger, 1990; Stille, 1992). Rock bolts in concrete dams are normally subjected to both tensile and shear loads. Some factors such as the quality of the rock and the inclination of the bolt influence the resistance and deformability of the steel under the combination of the external loads mentioned previously.

Failure criterion of the steel

Bolts that are subjected to tensile loads already when shearing starts or vice versa, and can only withstand minor deformations according to Stille (1992). The reason to this is that the failure condition is linked to the forces acting on a bolt. In a bolt subjected to both tensile T_t and shear T_s forces, failure can be estimated to occur when the following condition is fulfilled (Stille, 1992):

$$\left(\frac{T_t}{T_{ty}}\right)^2 + \left(2 \cdot \frac{T_s}{T_{ty}}\right)^2 = 1 \tag{2.9}$$

where T_{ty} is the load corresponding to the maximum tensile strength. Figure 2-3 shows the failure criteria for a steel bolt with $\emptyset 25\ mm$ and $f_{yk} = 500\ MPa$.

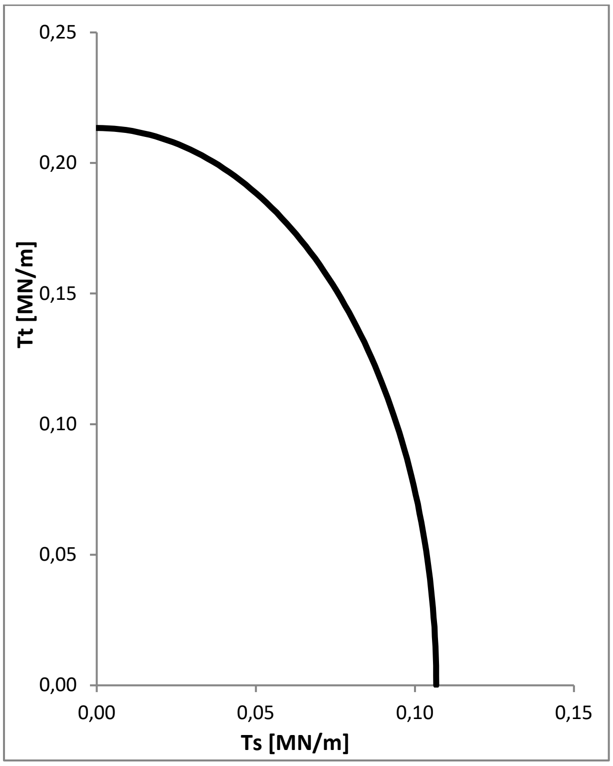


Figure 2-3-Limit curve for yield force in a bolt subjected to both tensile and shear force ($\emptyset 25\ mm$ and $f_{yk} = 500\ MPa$).

2.2 BOLT CONTRIBUTION TO DAM STABILITY

Installed rock bolts may give rise to both vertical and horizontal forces in the contact zone between the concrete dam and the rock foundation. The vertical forces will contribute to prevent an overturning failure while the horizontal forces (or more correctly, the forces in the direction of the sliding plane) will contribute to prevent a sliding failure.

The load contribution from the rock bolts has to be adjusted for if their placement is not perpendicular to the contact zone as illustrated in Figure 2-4. In the figure, the angle β describes its inclination compared to the sliding plane.

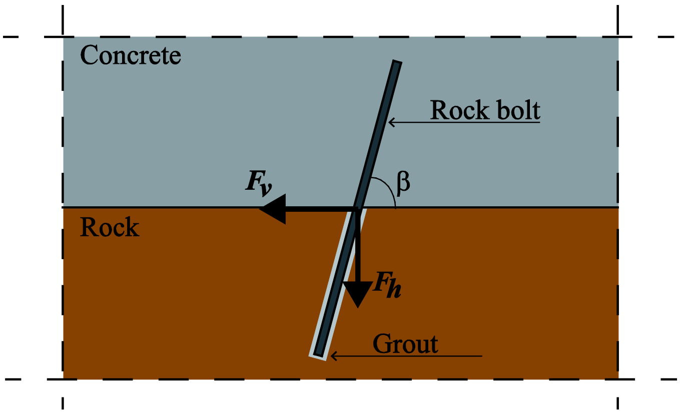


Figure 2-4 Illustration of an inclined rock bolt.

The horizontal and vertical force components of the rock bolt can be calculated as described in eq. (2.10) and (2.11).

$$F_v = R_{rock\ bolt} \cdot sin(\beta) \tag{2.10}$$

$$F_h = R_{rock\ bolt} \cdot cos(\beta) \tag{2.11}$$

where,

$R_{rock\ bolt}$ is the lowest strength of all failure modes as previously described in Section 2.1.

The rock bolts will contribute with additional resistance, $R_{rock\ bolt}$, for the two global failure modes of the dam (overturning or sliding) where the failure mode of the rock bolt with the lowest strength will govern its total resistance. The failure modes that may occur in the rock bolt during an overturning failure are

- Failure in the rock (Section 2.1.1)
- Adhesive failure, rock and grout (Section 2.1.2)
- Adhesive failure, steel and grout (Section 2.1.3)
- Adhesive failure, concrete and steel (Section 2.1.4)
- Steel failure (Section 2.1.5)

For a sliding failure, the rock cone failure is not considered to influence the result while all other modes could contribute to resist a sliding failure.

In most cases, the sliding plane is assumed to be horizontal and it is most common that the rock bolts are installed vertically, i.e. β = 90° which results in that $F_v = R_{rock\ bolt}$ and $F_h = 0$. In this case, the pull-out strength of the bolt will only contribute to resistance against an overturning failure. For a sliding failure with perpendicular rock bolts, it is only the load capacity due to shear deformations of the bolt that will contribute to the resistance against sliding failure.

The resistance of the rock bolt to withstand shear deformations depends on the stiffness and compressive strength of the surrounding materials, i.e. grout, rock and concrete.

If these materials have high stiffness, then no bending of the bolt will occur. Instead localized shear deformation occurs in the contact zone. This localized shear deformation is called the dowel effect and is illustrated in the left sketch in Figure 2-5. In the localized shear case, the load capacity can be defined equal to half the yield strength according to eq. (2.9) in Section 2.1.5.

If the stiffness and compressive strength of the surrounding materials are low then the bolt will gradually deform and crush the surrounding materials and a tensile load component will develop forming a crank shape (Stille, 1992). In that case, the bolt will fail due to the fact that the tensile strength is exceeded giving a high deformation at failure. This deformation is usually in the order of the bolt diameter, which means that full strength is usually not mobilized until deformations around 20 to 30 mm has occurred. At these deformations, the load acting on the dam represented by the uplift and water level might redistribute.

In addition, the contact zone between concrete and rock is not a smooth surface in reality. The roughness of the contact zone will give rise to a dilatation which will lead to tensile stresses in the bolt.

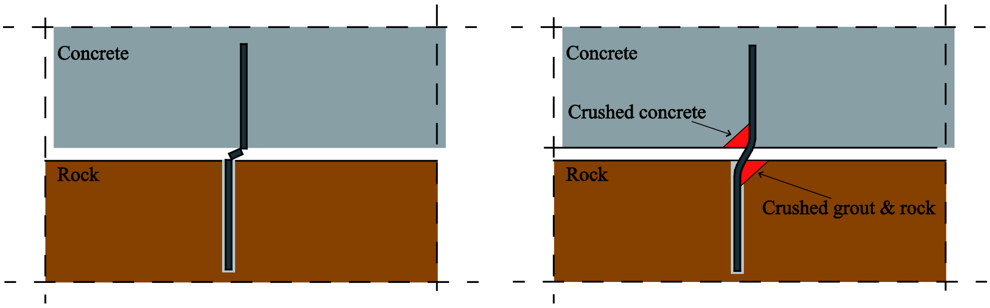


Figure 2-5 Illustration of the dowel effect (left) and crushing of surrounding material (right).

The influence of shear deformations and possible deformations of the surrounding material is difficult to consider in analytical stability calculations. In these analytical calculations this is normally considered as either tensile failure of the bolt or as a dowel effect as described in Section 2.2.1. A more refined approach would be to use the failure criterion given in Section 2.1.5, however, this is normally not considered in analytical calculations.

In harder rock, the mobilized shear stress becomes high enough to cut the bolt at low degrees of deformation before any significant tensile stresses have been developed. According to Bjurström (1973) this deformation could be as small as 1 – 3 mm. It is important that these limitations are considered when calculations are performed and acceptance criteria are chosen. Observed bolt behaviour in different in-situ and laboratory tests are presented in Section 3.3.

In numerical (finite element) analyses, it is possible to also include these effects and thereby obtain more realistic failure behaviour of the rock bolts. This is further discussed in Chapter 8.

2.2.1 Analytical calculation of sliding failure

For a rough rock joint or a concrete rock interface, without any contribution from bolts, the shear strength in a sliding failure mode could be described according to eq. (2.12).

$$T = N' \cdot tan(\emptyset_b + i) \qquad \textbf{(2.12)}$$

where,
T = Shear strength of the joint [N]
N' = Effective normal load [N]
φ_b = Basic friction angle for a macroscopic smooth surface [°]
i = Contribution from surface roughness of the joint [°]

If the rock joint or the concrete rock interface is intersected by rock bolts, they will result in additional shear strength. In conventional design, the rock bolts are however designed to result in a steel failure, due to its ductile behaviour. Therefore, many designers neglect other failure modes and only consider the tensile failure of the steel bolts yielding, i.e. as described by the equation below.

When the sliding plane is subjected to a shear deformation, the roughness will give rise to a dilatation which leads to tensile stresses in the bolt. The vertical component of the tensile stress in the bolt gives rise to a corresponding additional normal stress on the weakness plane, which leads to an increased shear strength for the weakness plane. In order to consider this effect, the equations given in e.g. BBK 04 (2004) for transferring shear forces in a joint are normally used by dam engineers in Sweden

$$f_f = \mu(\rho \cdot f_s + \sigma_{fc}) \quad \textbf{(2.13)}$$

where,
f_f = shear strength of the joint [N/m²]
μ = friction coefficient of the joint [-]
ρ = amount of bolt steel [-]
f_s = yield stress of the bolt steel [N/m²]
σ_{fc} = compressive stress in the joint [N/m²]

If the bolt is installed with an angle β against the shear force direction according to Figure 2-6, ρ in eq. (2.13) should be multiplied with the following expression.

$$\sin(\beta) + \frac{cos(\beta)}{\mu} \quad \textbf{(2.14)}$$

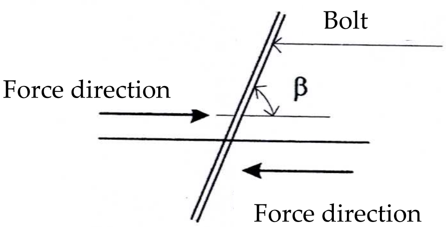


Figure 2-6 Joint with intersecting bolt (From BBK 04, 2004).

If the shear strength of the weakness plane in eq. (2.13) is expressed in terms of forces instead of stresses, the friction coefficient is expressed as the sum of the basic friction angle and the dilation angle according to eq. (2.12). The inclination of the bolt according to eq. (2.14), the total shear strength of the joint with bolts could be expressed as:

$$T = \left(N' + \left(\sin(\beta) + \frac{cos(\beta)}{tan(\emptyset_b + i)}\right) \cdot A_s f_s\right) tan(\emptyset_b + i) \quad \textbf{(2.15)}$$

where,
A_s = area of the steel bolts intersecting the joint/crack [m²]

However, is should be observed that Eq. (2.15) presumes that only tensile stresses develop in the bolt. As shown by eq. (2.9) in Section 2.1.5, simultaneous normal and shear stresses in the steel bolt will decrease the yield strength of the steel. In the case when only pure shear stresses occur for bolts perpendicular to the weakness plane, where the bolt acts as a dowel, T in eq. (2.15) is reduced to:

$$T = N' tan(\emptyset_b + i) + 0.5 A_s f_s \tag{2.16}$$

Since the degree of mobilized tensile and shear stresses are often unknown in analytical calculations, it is generally assumed that the bolt either works as a dowel according to eq. (2.16) or that only tensile stresses are developed according to eq. (2.15).

The contribution from the rock bolts to the sliding stability may also be considered in two different ways depending on if it is considered to be subjected to tensile or shear forces.

The safety for sliding should be calculated as the ratio of forces parallel (i.e. often horizontal forces) to the sliding surface divided by the forces perpendicular to the sliding surface (i.e. often vertical forces)

$$\mu = \frac{\sum H}{\sum V} \tag{2.17}$$

where, μ is the coefficient of friction, $\sum H$ is the sum of all horizontal forces (i.e. parallel to the sliding plane) and $\sum V$ are the sum of all vertical forces.

In the case of eq. (2.15) where the rock bolts are subjected to tensile forces, these are considered to increase the contact pressure in the sliding plane and hence should be considered as a vertical force (see eq. (2.18)). In the case of eq. (2.16), however, the dowel force is a shear force and should be considered as a horizontal force (see eq. (2.19)).

If $\sum H$ and $\sum V$ are the sum of all forces (except for the force from the bolts) acting on the dam in the horizontal and vertical direction respectively, then the coefficient of friction could be calculated as follows

As a tensile force in the rock bolts

$$\mu = \frac{\sum H}{\sum V + \sum R_{rock\_bolt\_tensile}} \tag{2.18}$$

Or as dowels, resulting in a shear force in the rock bolts

$$\mu = \frac{\sum H - \sum R_{rock\_bolt\_shear}}{\sum V} \tag{2.19}$$

where, $\sum R_{rock\_bolt\_tensile}$ or $\sum R_{rock\_bolt\_shear}$ is the contribution from all rock bolts crossing the sliding plane.

2.2.2 Analytical calculation of overturning failure

In an overturning failure mode, analytical calculations are performed where it typically is assumed that the dam will rotate around the downstream toe. A moment balance equation is defined for the stabilizing and overturning moments.

The rock bolts will introduce additional resistance to the stabilizing moment. As described earlier, the load capacity of the rock bolts can be defined as the minimum of all potential failure modes. In conventional design, the rock bolts are however designed to result in a steel failure, due to its ductile behaviour. Therefore, many designers neglect other failure modes and only consider the tensile failure of the steel bolts yielding, i.e. as described by the equation below.

$$F_{v\ (steel\ yielding)} = A_s \cdot f_s \cdot sin\beta \qquad \textbf{(2.20)}$$

In this report, also the influence from degradation will be considered. The different materials will be affected by degradation mechanism differently, see Section 2.3. Therefore, a more correct approach is instead to use eq. (2.10) where each failure mode and its influence from degradation mechanism is considered separately.

2.3 DEGRADATION

Rock bolts are exposed to different types of degradation processes (Berzell, 2014). These processes are normally a combination of natural and external factors such as aging or corrosion among others. The resistance of the materials against these degradation processes is of high interest since it will define the life span of each element in particular and of the whole structure in general. However, uncertainties exist regarding these degradation processes since embedded rock bolts in concrete dams are not easily accessible.

Degradation of steel bolts or by surrounding cement grout may potentially reduce the load bearing capacity of grouted rock bolts. The difficulty lies, however, in estimating the degree of degradation and especially its influence on the load bearing capacity. One additional difficulty is that one degradation process might result in initiation of another degradation mechanism or results in synergy effects between degradation mechanisms.

One example of degradation of concrete (or grout) and a reinforcement bar (or rock bolt) is illustrated in Figure 2-7, where carbonation of concrete results in reduced pH in the concrete and thereby results in reduced protection for steel corrosion. Another effect could be that the corrosion may reduce the bond between grout and the bolt.

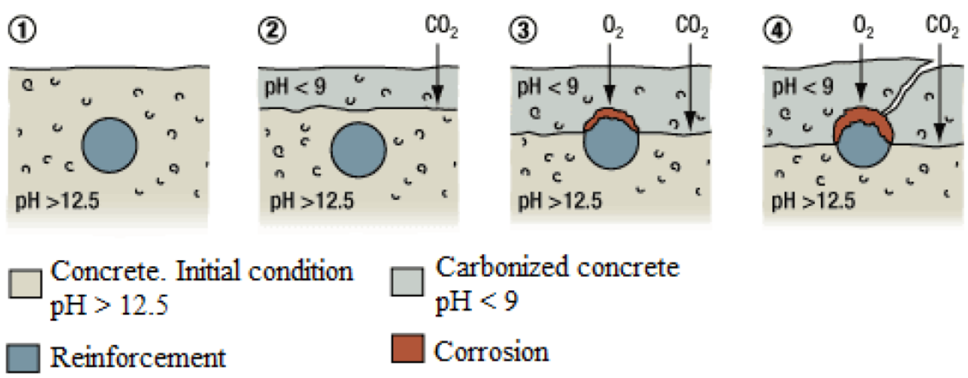


Figure 2-7 Illustration of degradation of steel embedded in concrete due to carbonation and corrosion.

There has been significant amount of research on steel corrosion and to some extent the beneficial impact from the alkaline environment of the cement to reduce the corrosion rate. The alkaline environment of concrete (typically pH 12 to 13) provides corrosion protection/reduction of steel. A thin oxide layer forms on the steel in this high pH environment, which prevents metal atoms from dissolving. This oxide layer results in significantly reduced corrosion rate, typically 0.1 μm per year, compared to normal corrosion which may have rates at least 1 000 times higher. (ACI 222)

Research regarding degradation of cement grout is rather limited compared to the performed research on corrosion. However, some conclusions can be drawn regarding the degradation mechanisms of other cement-based materials such as concrete or shotcrete.

A summary of degradation of steel and cement grout with a focus on embedded rock bolts is presented in the following subsections.

2.3.1 Rock bolts

Corrosion of steel reinforcement is a fully natural electromechanical process which is induced either by carbonation or by chlorides. The aggressiveness of this process depends not only on the concrete quality but also on the surrounding environment.

Corrosion in water solutions may initiate in steel as

1. An electro-chemical process, electrolyte, with a passive and active cell
2. A result of the aggressiveness of the water

In normal ground water, dissolved oxygen is needed in order for any noticeable corrosion to take place in carbon steel, according to Windelhed et al. (2002). In addition, according to Linder (1984), significant corrosion rates are not obtained until the pH-value has reduced to about four.

Based on this, Windelhed et al. (2002) concludes that for rock bolts completely covered by cement grout, no corrosion occurs. However, if the rock bolt has insufficient grout, corrosion may occur on the exposed parts of the rock bolts. A short length of the exposed part of the bolt results in higher corrosion rate. The electro-chemical process leads, however, to a relatively low corrosion rate as shown in Figure 2-8, where the maximum corrosion rate is about 16 μm. This corrosion rate is lower than what may occur due to the aggressiveness of the water.

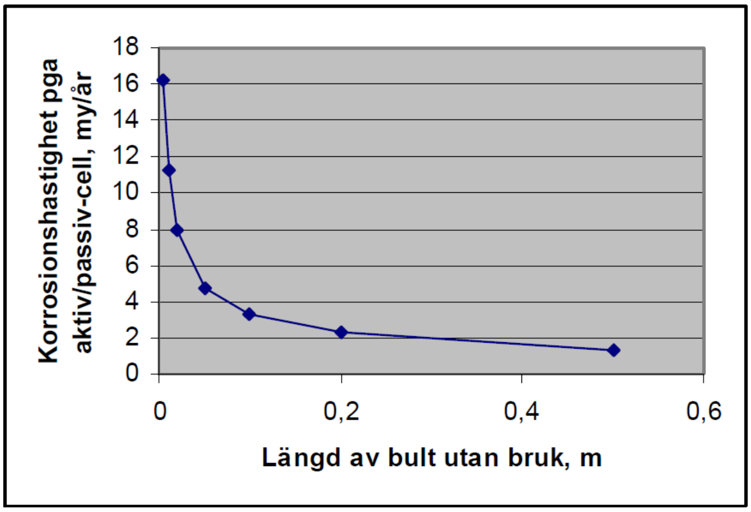


Figure 2-8 Corrosion rate due to passive and active cell as a function of the length of the exposed bar.

The corrosion rate varies strongly with the compound of the water. Corrosion may occur due to general corrosion or pit corrosion. For load bearing rock bolts, the general corrosion rate is a more representative measure according to Windelhed et al. (2002). The corrosion rate for steel in fresh water can in general be estimated to be within 10 to 70 µm/year, and less than 50 µm/year for Swedish rivers and about 30 µm/year according to Vattenfalls estimations. (Windelhed et al. 2002)

In paper by Hellgren et al. (2017), 30 000 samples from 2003 stations in Swedish rivers have been used to calculate the corrosion rate according to the German standard DIN 50929 (part 3). This approach is described further below in this section. The preliminary conclusion from Hellgren et al. (2017) is that the corrosion rate may be assumed having a log-normal distribution, with a mean value corresponding to 20 µm/year and a standard deviation of 5 µm/year.

In conclusion, all these cases indicate that the corrosion rate due to the aggressiveness of the water may also be relatively low for Swedish rivers. This have also been confirmed by cases where old, previously installed, rock bolts have been examined in connection with demolition or structural rehabilitation, Larsson (2007), Ljungberg (2016). In Ljungberg (2016) and Hellgren et al. (2017), 50 year old bolts where examined after demolition. Only some minor superficial corrosion could be detected, which most likely already was present directly after installation.

Figure 2-9 Example of superficial corrosion of a 50 year bolt.

According to Bogdanoff (2013), 199 out of 1400 rock bolts showed evidence of corrosion in the rehabilitation project of a road tunnel called Vindötunneln. In the most cases, the corrosion was found in the bolt at the location of rock surface which is expected since this section is subjected to aerated water. In addition, this section is also subjected to exhaust emissions, moisture and chlorides from the road.

The foundation rock or soil in a dam where the rock bolts are partially embedded is occupied by permeating water through fractures or pores. This water depending on its chemical content can degrade the grout and steel of the rock bolts in a higher or lower rate. Water aggressiveness can be classified according to different standards such as the German standard DIN 50929 part 3, Trafikverket and the Swedish standard SS-EN 12502-5: 2005 among others.

The German standard DIN 50929 part 3, maybe the most used method, assesses the corrosive index (W_0) of water taking into account different chemical factors such as the pH-value, alkalinity, calcium content (Ca^{2+}), chloride content (C^{-}) and sulfate content (SO_4^{2-}), see Table 2-4.

The corrosive index is obtained as follows:

$$W_0 = N_1 + N_2 + N_3 + N_4 + N_5 + N_6 + N_3/N_4 \qquad \textbf{(2.21)}$$

where the values for N_x are obtained from Table 2-4.

Table 2-4 Parameters for assessing the risk of corrosion for carbon steel (DIN 50 929).

WATER PARAMETERS	VALUE
Water type	**N1**
Flowing	0
Stationary	-1
Lake	-3
Anaerobic marsh, sea	-5
LOCATION OF CURRENT CONSTRUCTION	**N2**
Under water	0
Water interface/ air	+1
Splash zone	0, 3
$c(Cl^-) + 2c(SO_4^{2-})$ (mol/m³)	**N3**
<1	0
1-5	-2
>5-25	-4
>25-100	-6
>100-300	-7
>300	-8
ACID NEUTRALIZING CAPACITY UNTILL pH 4,3 (mol/m³)	**N4**
<1	+1
1-2	+2
>2-4	+3
>4-6	+4
<6	+5
$c(Ca^{2+})$ (mol/m³)	**N5**
<0.5	-1
0.5-2	0
>2-8	+1
>8	+2
pH-value	**N6**
<5.5	-3
5.5-6.5	-2
>6.5-7.0	-1
>7.0-7.5	0
>7.5	+1

Table 2-5 Estimation of even corrosion rate based on the assessment sum W_0 (DIN 50 929).

W_0	Average corrosion rate (µm/year)	Maximum corrosion pit rate (µm /year)
≥0	10	50
-1 to -4	20	100
-5 to -8	50	200
< -8	100	500

In order to estimate a corrosion rate for the case study presented in this document, the chemical content of two water samples from a river in the north of Sweden has been analysed, see Table 2-6.

Table 2-6 Chemical content from a river in the north of Sweden.

Parameter	Unit	Sample 1	Sample 2	Average
Calcium (Ca^{2+})	mg/l	3.41	3.26	3.34
pH-value	-	7.1	7.1	7.1
Alkalinity (HCO_3^-)	mg/l	9.50	9.60	9.55
Chloride (Cl^-)	mg/l	1.69	1.44	1.57
Sulphate (SO_4^{2-})	mg/l	2.78	3.48	3.13

	Calcium (Ca^{2+})	Alkalinity (HCO_3^-)	Chloride (Cl^-)	Sulphate (SO_4^{2-})
c [mg/l]	3.34	9.55	1.57	3.13
Mol. Weight	40	60	35.5	96
c [mol/m3]	8.35E-02	1.59E-01	4.42E-02	3.26E-02

The water aggressiveness according to DIN 50 929 and the chemical content in Table 2-6 is as indicated in Table 2-7.

Table 2-7 Estimation of the corrosion index W_0.

WATER PARAMETERS	VALUE
Water type	**N1**
Flowing	0
LOCATION OF CURRENT CONSTRUCTION	**N2**
Under water	0
$c(Cl^-) + 2c(SO_4^{2-})$ (mol/m³)	**N3**
<1	0
ACID NEUTRALIZING CAPACITY UNTILL pH 4,3 (mol/m³)	**N4**
<1	+1
$c(Ca^{2+})$ (mol/m³)	**N5**
<0.5	-1
pH-value	**N6**
>7.0-7.5	0
$\boldsymbol{W_0 = N_1 + N_2 + N_3 + N_4 + N_5 + N_6 + N_3/N_4}$	**0**

The corrosion rate corresponding to a corrosion index $W_0 = 0$ has according to Table 2-5 an average and maximum value of 10 µm/year and 50 µm/year respectively.

According to the results obtained from water samples in a river from the north of Sweden and the literature study presented in this chapter, the corrosion rate is in the following analyses in this report assumed to have a lognormal distribution with mean value $\mu = 30$ µm/year and standard deviation $\sigma = 10$ µm/year, see Figure 2-10. This distribution is chosen as even more conservative compared to the more detailed evaluation presented in Hellgren et al. (2017).

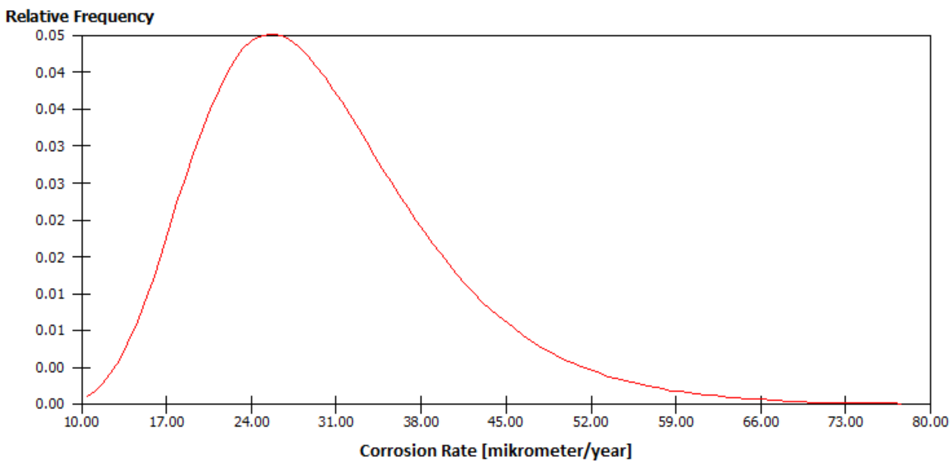


Figure 2-10-Probability density function for a lognormal distribution with mean value of 30 µm/year and log standard deviation of 10 µm/year.

Attention should be given to the fact that for embedded rock bolts, the corrosion rate obtained from DIN 50 929 is higher than the expected since alkalinity from the grout and concrete is not taken into account.

2.3.2 Grout and other cement based materials

The purpose of the cement grout is to anchor the bolt to the rock and to protect the steel from corrosion. Possible degradation may result in loss of both of these functions. Unfortunately, there is fairly limited amount of references that has investigated the degradation of grout.

The intrusion of water in concrete and cement based materials provides a path for the penetration of deleterious materials such as chloride and sulphate ions. The primary transport mechanism for chloride and sulphate ions is diffusion and capillary action. The diffusion alone is typically a very slow process so the main transport mechanism is the capillary transport, especially near an unsaturated concrete surface, according to Martys and Ferraris (1997). Water transport may also occur in cement based materials due to a pressure gradient which is called permeability. The permeability of cement based materials depends on the density or the cementitious content. The dense matrix of concrete (even for moderate quality mixes) provides an extremely difficult environment to push water through even under high pressures.

According to Windelhed et al. (2002), only limited water transport occurs in crack free cement grout with a water to cement ratio less than 0.6. In general, cracks with a crack width that exceeds 0.2 mm are considered to transport water; however, this is significantly dependent on the size of the water pressure. In order for the cracks to transport water, these have to be continuous or form a continuous network of cracks.

Degradation mechanisms in cement based materials is described in more detail in the literature such as Windelhed et al. (2002) and are summarized in the following section. The different degradation mechanism of cement grout can be classified depending on the principal material behaviour these give rise to

- Material loss due to water transport (separation, erosion and/or leaching)
- Shrinkage (plastic, chemical, drying and/or dehydration)
- Swelling (ASR, chemical, frost and/or sulphate attacks)

Swelling of cement-based grout around an embedded rock anchoring bolt is expected to result in that existing cracks are closed or that the cement paste is densified due to the confinement. All mechanisms that result in this behaviour are therefore considered to be beneficial for grouted rock bolts according to Windelhed et al. (2002). It is only the grout closest to the entrance of the borehole that may exhibit negative effects since it may be free to expand out from the borehole.

Shrinkage of the cement paste may have a negative effect of its load bearing capacity and its durability. Shrinkage may occur from different sources (see the list above) but the main reason is due to drying and to some extent chemical (autogenous) shrinkage if water is not added during hardening. The decrease in volume due to shrinkage is larger for cement based grout compared to ordinary concrete, since the aggregates in concrete gives an internal stability, Windelhed et al. (2002). The use of materials with low water to cement ratio results in reduced drying shrinkage since the cement particles will act as aggregates. However, the chemical shrinkage is significantly higher for materials with lower water to cement ratio.

Material loss due to transport of water may occur due to water transport in the interface between rock and concrete in the foundation of for instance a concrete dam, or due to water transport within cracks (in rock or in concrete). Rock bolts are normally not installed in boreholes with inflowing water. Leaching penetration has the potential to cause significant degradation of the cement grout, since leaching results in dissolution of the cement paste, which thereafter may erode away due to flowing water.

Leaching penetration is governed by the hydraulic gradient, the porosity of the material and cracks/fractures according to Lagerblad (2007). An illustration of how leaching may result in erosion where cement particles are eroded is shown in Figure 2-11. This may occur if a leakage occurs from adjacent (ungrouted) cracks crossing the grouted borehole. Cracks planes crossing the borehole may not be grouted if the crack width is too small (typically < 80 µm according to Eriksson and Stille (2005)) but this can also occur for wider cracks if the cement particles form an arch and thereby blockage.

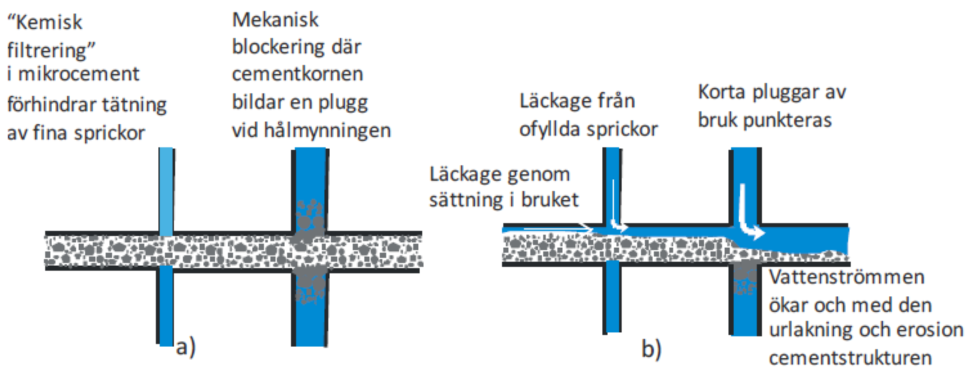


Figure 2-11 Illustration of how leaching may result to erosion, from Bogdanoff (2013).

Erosion of grout due to leaching is expected to result in weakening but only to a limited degree, and according to Li and Lindblad (1995) and Bogdanoff (2013) it is therefore considered unlikely that this could result in complete loss of bond for the rock bolt.

2.3.3 Rock and rock mass

The information regarding degradation of rock is limited where no direct results have been found that could be used to assess the degradation. However, the degradation of rock will not likely to have a significant influence of the service life of grouted rock bolts. In general, the rock foundation of Swedish concrete dams can be assumed to be of fairly good quality. Therefore, degradation of rock has not been considered in this report.

3 Results from previous investigations of rock bolts

There are a few cases in the literature where rock bolts have been tested with the purpose to determine their load capacity and failure modes. In this section, the cases found in the literature are summarized. In addition, within Energiforsk (former Elforsk) another research project has also been performed where different methods for condition assessment of rock bolts were studied.

3.1 TEST OF 50 YEARS OLD BOLTS IN HOTAGEN

Hotagen regulating dam is located in the river Hårkan, a tributary to the Indalsälven, about 80 kilometres northwest of Östersund. The old dam, built in the 1960s had been severely damaged due to degradation, and in 2008 a new dam was built and the old was demolished. The old dam consisted of; an earth fill dam, a spillway with a radial gate and a spillway section, consisting of six wooden gates (stop logs) divided by five concrete monoliths. During the demolition of the old dam, measurements were performed to find and evaluate the status of the grouted rock bolts in the dam foundation under the five spillway monoliths. The full documentation of this test can be found in the report by Larsson (2007).

The five monoliths were founded on rock and were anchored with grouted rock bolts according to Figure 3-1. The rock bolts were reinforcement bars with the steel quality ks40 and the diameter 25 mm. The bolts were embedded with a depth of 0.5 m in the concrete and three meters in the rock mass. The bolts were installed in three meter deep, 55 mm diameter drilled holes.

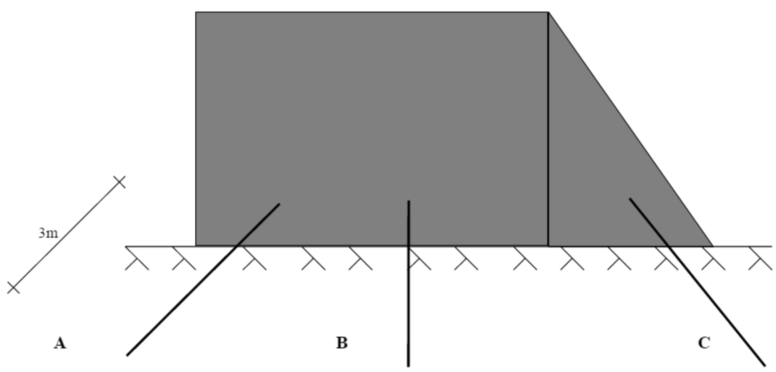


Figure 3-1 Illustration of the Hotagen dam monoliths and a principal sketch of the rock bolts.

Under the five monoliths, in total 21 rock bolts were found after demolition. None of these bolts showed any visible signs of corrosion or degradation. Out of the 21 bolts, a number of bolts where chosen for pull-out test. Due to the bad rock quality, causing the testing equipment to sink, only tests of three bolts were feasible.

The bolts were pulled until one of the five failure modes could be observed. For all three tests, the failure mechanism was governed by yielding of the reinforcement bar. In the first test, the pull-out test was continued after visual signs of yielding and the bar was pulled until rupture, as seen in Figure 3-2.

Figure 3-2 Photo of one of the rock bolts that was loaded until rupture.

For the other two tests, the test was aborted when yielding of the steel was observed. The obtained yielding loads for the bars are presented in Table 3-1 together with the corresponding calculated stress.

Table 3-1 Summary of obtained failure loads at Hotagen, from Larsson (2007).

	Monolith	Inclination	Force (kN)	Stress (MPa)
Bolt 1	3	Vertical	261	*532*
Bolt 2	4	Vertical	331	*674*
Bolt 3	4	Vertical	320	*652*
Theoretical without corrosion			*191*	390

Attempts were made to extract two of the bolts with surrounding rock mass and grouting, with core drilling technique, for further studies in a laboratory. The attempts failed since the bars had been bent during assembly, which caused the drilling equipment to cut through the bars at a depth of approximately one meter. Therefore, only a visual inspection of the bar and grout could be performed in the laboratory. The inspection in the laboratory confirmed the visual inspections at the test site. Despite that the rock bolts had been in use about 50 years, only a small streak of corrosion at the interface between rock and concrete could be found.

3.2 PULL-OUT TEST OF 68 NEWLY INSTALLED ROCK BOLTS

Two larger test programs performed in Norway in the last years were Neby (2011) and Lepine (2012), which tested 18 respectively 50 newly installed rock bolts.

Normally, when performing pull-out tests, it is not possible to capture the rock cone failure due to the design of the testing equipment, where the pull-out force is transferred into the rock surrounding the rock bolt. The tests performed by Neby (2011) and Lepine (2012) was performed with an excavating machine which allowed for performing tests without disturbing the rock closest to the installed rock bolt, see Figure 3-3. This enabled a study of failure occurring in the rock mass, in contrast to most others that only can capture the failure in the steel bolt. Due to this, it was also possible to detect under which geological conditions the rock mass failure occurred. In addition, when pull-out tests are conducted it is usually only the pull-out force that is measured and not the displacement. However, in their tests both pull-out force and displacements where recorded.

a) b)

Figure 3-3 Photo of the excavating machine used to perform the pull-out test, from Lepine (2012).

In the study performed by Neby (2011), 18 bolts where tested having different grouting length, ranging from 0.1 to 1.0 m, in a fractured rock mass (RMR<40). The installed bolts had a yield strength of 500 MPa and a diameter of Ø25 mm resulting in a tensile capacity of the bolt steel of 245 kN. The results from these tests are presented in Table 3-2.

Table 3-2 Results from 18 pull-out tests ϕ=25 mm, from Neby (2011).

Bolt no	Length bolt in rock (m)	Joint distance (mm)	Load (kN)
B1	0.40	100	160
B2	0.39	50	220
B3	0.54	100	200
B4	0.58	0-100	60*
B5	0.50	0-140	120*
B6	0.62	10	90*
B7	0.45	30	70*
B8	0.41	0-150	10*
B9	0.83	30	30*
B10	0.33	0-40	50*
B11	0.51	30-100	130*
B12	0.33	50	20*
B13	0.96	10	150*
B14	0.99	50	170
B15	0.10	20	40*
B16	0.13	100	120*
B17	0.85	0-40	170
B18	0.99	0-100 (knust)	60

*) Represents failure in the rock mass

Neby (2011) concluded that the capacity of the grouted bolts to a large extent depends of the degree of fracturing of the rock mass. In rock masses without joints the capacity will reach 800 - 1 000 kN/m. Therefore, for bolts installed in rock masses with good quality they will reach their yield stress if they are grouted at least 0.3 m, even though the length for practical reason should be longer. With increased degree of jointing in the rock mass, the capacity will be reduced correspondingly. According to Neby (2011), to only consider a cone of rock volume has no relevance to the actual capacity in fractured rock masses.

In order to continue with the work performed by Neby (2011), additional tests were conducted in the follow-up research project performed by Lepine (2012). In the work by Lepine (2012), 50 full-scale pull-out tests of grouted passive rock bolts were performed at the same location as Neby (2011). The tests were performed in Verdalskalk (Verdal).

The rock found in that area corresponds to a limestone of diverse quality. Laboratory tests revealed a relatively high uniaxial compressive strength of the intact rock (σ_c = 90 MPa). The bolts were installed in two types of rock masses; one with a good rock mass quality with a RMR up to 90 and one with poor rock mass quality with an RMR up to 40. The length of the bolts grouted in the rock mass varied approximately between 0.2 to 0.5 m. Testing was performed in pure tension with an excavator with capacity up to 260 kN. Same as Neby (2011), the installed bolts had a yield strength of 500 MPa and a diameter of Ø25 mm resulting in a tensile capacity of the bolt steel of 245 kN. The results from the test performed by Lepine (2012) are summarized in Table 3-3.

Table 3-3 Results from pull-out tests by Lepine (2012)

Failure mode	Number of bolts	Comments
No break	18	Between 19 and 26 tons
Break of the rock mass	21	7 above 19 tons 10 between 8,95 and 18 tons 4 under 4,85 tons (extremely poor rock condition, crushed or loose block)
Break of grout	11	9 between 4,85 and 11 tons 2 equal to 17 tons

In the results, it could be seen that break of grout was observed in 11 of the bolts. This was believed to be due to the short curing time of 13 days together with unfavourable climate conditions. In the 21 bolts that registered a break in the rock mass, the registered pull-out force varied from very low (<4.85 tons) in very poor rock mass quality to relatively high (> 19 tons).

Due to the short length of the bolts installed in the rock mass it is difficult to draw conclusions how representative these tests are compared to longer bolts usually installed under concrete dams.

3.3 LABORATORY TEST RESULTS OF FULLY-GROUTED BOLTS IN ROCK

One of the first who performed systematic research on rock bolts were Bjurström (1973). He performed 46 shear tests on fully cement-bonded rock bolts embedded in blocks of granite. Three different aspects of the bolt effect were considered; tension force in the bolt, friction at the shear surface as a consequence of increased normal stress and the dowel effect of the bolt.

The results from these tests show that the angle of the bolt with respect to the weakness plane influences the failure mechanism of the bolt. At small angles between bolt and weakness plane ($\alpha<40^\circ$) the failure often occurs as a pure tensile failure. However, for cases with angles larger than this, the failure was often a combined tensile-shear failure with yielding of the steel on both sides of the plane.

The results from Bjurström (1973) also shows that almost horizontal ($\alpha<40^\circ$) or steep inclined bolts ($\alpha>75^\circ$) requires significant relative shear displacement of the order 20 to 30 mm to impose failure (with d=12 mm). For bolts with intermediate angles ($40^\circ<\alpha<75^\circ$) only half of that deformation is sufficient to impose failure. Pure dowel action could not be observed, most likely due to the softer grout (compared to the intact granite) that the bolts were embedded in. Measured deformations at failure for different bolt inclinations are presented in Figure 3-4.

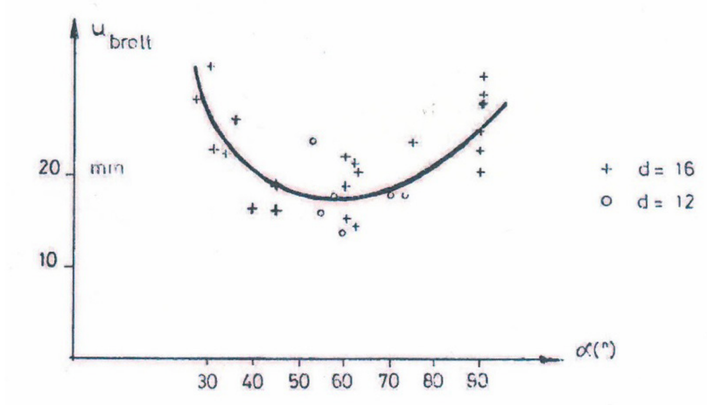


Figure 3-4 Shear deformation at failure for bolts with different inclinations (From Bjurström 1973).

Bjurström (1973) conclude that with increasing shear displacements, tensile stresses gradually increasing in the bolt until the yield stress is reached. Comparisons between calculated and measured values show that a reasonable approximation for the shear strength is obtained, if it is assumed that tensile stresses equal to the yield stress has been developed in the bolt.

A similar, but even more comprehensive, study was performed by Spang and Egger (1990). They studied the behaviour and the mode of action of fully-bonded rock bolts, to evaluate the effect of different parameters on the load capacity of the bolts. In total, 60 laboratory tests with Ø8 mm steel bolts were performed in addition to a series of large-scale field tests with Ø40 mm steel bolts.

The influence of bolt diameter in weak sandstone (uniaxial compressive strength 10 MPa) is presented in Figure 3-5. The results show that the deformation at maximum bolt load is proportional to the bolt diameter and that the failure occurs at large deformations equal to 3-4 bolt diameters. The results also show that large bolt diameters reduce the shear displacements required for a given shear force.

In Figure 3-6 the influence of rock type is analysed. The results show that softer rocks obtain higher capacity due to higher tensile stresses developed in the bolt. In the stronger and stiffer rock such as granite, shear stresses are developed together with tensile stresses which results in lower capacity of the bolt. However, the maximum capacity still occurs at a shear displacement of approximately one bolt diameter.

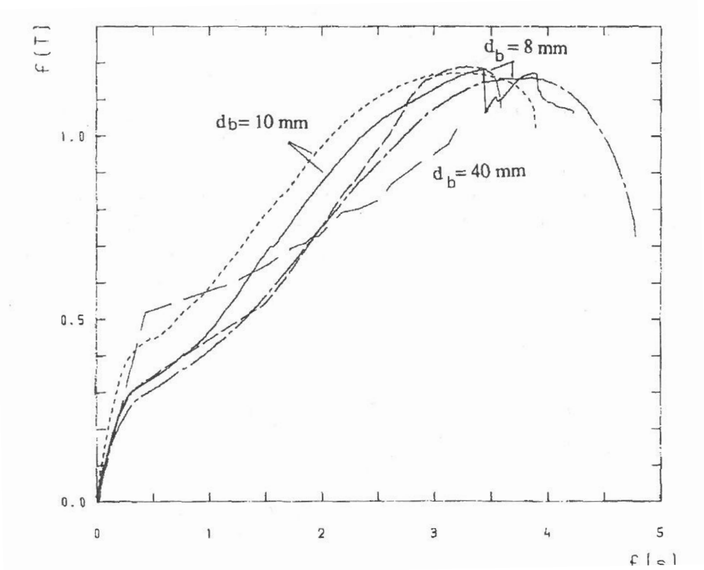


Figure 3-5 Influence of bolt diameter in weak rock (90˚ against joint). Normalized deformation with bolt diameter on the horizontal axis and normalized bolt strength on the vertical axis (From Span and Egger 1990).

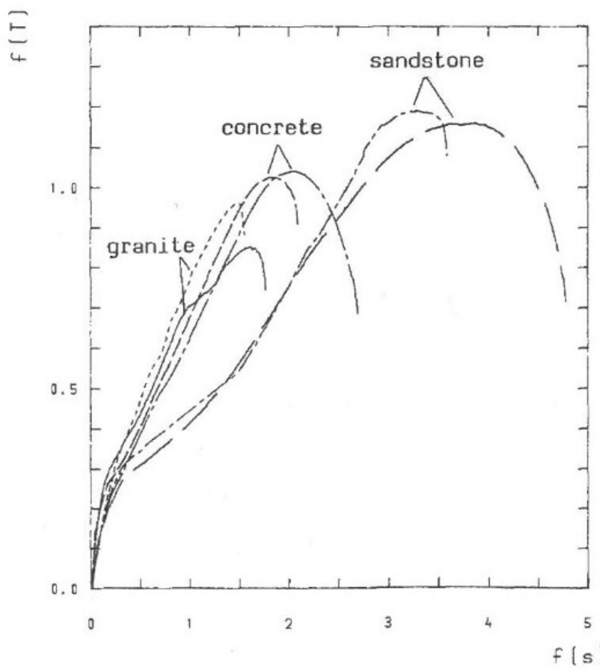


Figure 3-6 Influence of rock type (stiffness and compressive strength) (From Span and Egger 1990)

3.4 METHODS TO ASSESS THE CONDITION OF ROCK BOLTS

Ekström et.al (2013) studied whether it is possible to measure or assess the condition and service life of existing rock bolts. Their focus was on non-prestressed bolts within the field of concrete dams. In the report, work in adjacent fields and possible measuring methods are also included. The following methods to assess the status of the existing bolts were studied:

- Assessment of the status of the steel bars with ultrasound
- Assessment of the anchorage status by analyses of the dam crest displacements
- Assessment of the status of the steel by X-ray
- Assessment of the rocks status through drilling
- Assessment of the rocks status using seismic
- Assessment of the grouts status with ultrasound

For existing inaccessible rock bolts, no method to unambiguously assess the condition and load capacity was found. For accessible rock bolts, destructive testing can be performed where a pull-out test is performed at the rock bolt. Further, if the bolt is accessible and its length is known, it is possible to determine major defects through measurement techniques. These defects can be listed as:

- Anchor rod fracture
- Progressed anchor rod corrosion
- Adhesion loss between rod/grout/rock
- Grout cavities

In addition, methods to monitor the status and/or prepare for condition assessment of new installations of rock bolts were also studied.

- Assessment of the steel's status using probe
- Assessment of the steel's status using resistance measurement
- Assessment of the steel's status using coaxial cable
- Assessment of the steel's status using ultrasound

The work shows that there are various possibilities to monitor newly installed anchor rods – if measures are taken during the installation. Information obtained by such monitoring could be useful for developing analysis methods for existing rock bolts.

Ekström et.al (2013) proposed that a data acquisition program for rock bolts should be launched. To develop new analysis methods, advanced calculations should be combined with field and laboratory testing. Therefore, the program should include testing of existing rock bolts (when available) and instrumentation of newly installed rock bolts. They also proposed that measures must be taken to close the existing gap in knowledge about rock bolts, their degradation and failure mechanisms.

4 Summary of related MSc projects

In connection to this project, several MSc thesis projects have been performed. These MSc projects were performed to examine different aspects that are of interest to this report. These topics are; influence of corrosion rate, the influence of bolts on a sliding failure and an evaluation of the test method used for pull-out tests of bolts.

4.1 CARL BERZELL (2014)

Berzell (2014) studied the influence of rock bolt degradation for stability calculations of concrete dams. The main purpose was to study the justification of the limitations for rock bolts imposed in RIDAS. The study was performed through a literature review and analyses of a case study. The literature review included failure modes for grouted rock bolts as well as possible degradation processes for bolt and grout.

By studying the work previously carried out on corrosion of reinforcing steel in concrete, a suggested corrosion rate of rock bolts was proposed. The corrosion rate for reinforcement bars in concrete depends strongly on the relative humidity in concrete. Berzell (2014) deems the relative humidity in hydraulic concrete structures to be approximately 95 %, giving an expected interval for the corrosion rate of 40 - 180 μm/year for the rock bolts. In order to include the variation of degradation that may occur between individual bolts, the corrosion rate is assumed to be lognormally distributed. In this way, each grouted rock bolts were assigned a separate corrosion rate from a lognormal distributed random variable. The proposed corrosion rate was defined as lognormal with a mean value 60 μm/year.

The proposed method was then applied on Swedish buttress dam in a case study. The buttress dam is anchored with a row of inclined rock bolts under the front plate. Analytical as well as numerical analyses was performed to study the contribution of rock bolts to the stability of concrete dams. The analytical calculations show increased safety against sliding when utilizing the capacity of the rock bolts in the stability calculation. No comparison was made for the failure mode overturning.

The numerical analyses shows the opposite result, where the bolts decrease the safety for the dam. Berzell (2014) believes this is due to inadequate modelling of the interface between the dam body and foundation. One plausible reason is that the actual force in the rock bolts are not considered in the force balance when calculating the factor of safety, which would explain the unlikely results.

Despite this, the report presents interesting results regarding the behaviour during a failure of a dam with rock bolts. The analyses show that the rock bolts are activated only after that the external forces are increased. Secondly, the results indicate that the bolts fail in tension.

The conclusion regarding the contribution of rock bolts to small dams is that the restriction to 140 MPa stated by RIDAS (2011) appears to be too conservative and that the value should likely be higher than the current limitation.

4.2 MARTIN CARLSSON (2015)

Carlsson (2015) studied how the calculation method for un-tensioned rock bolts in hard rock developed by Holmberg (1992) can be applied for concrete dams. The study focuses on the rock bolts' contribution to sliding along fractures in rock. The Holmberg method describes how rock bolts fails, as well as their contribution to the load capacity of rock fractures. Result obtained by the method is compared with previous tests of rock bolts performed by Spang and Egger (1990) and Bjurström (1973).

In comparison between the analytical calculations and the experimental tests performed by Spang and Egger (1990), the following conclusions were drawn

- Good predictions can be made regarding the load capacity of rock bolts for cases where the bolts are installed orthogonal to the rock fault plane.
- The contribution of rock bolts during small deformations seems to be overestimated.
- In hard rock, the load capacity of rock bolts' is overestimated.
- If the bolt is installed at an angle of 60° relative to fracture sliding direction, the analytical calculations seem to provide a good estimation of the shear load capacity but underestimate the deformations.

Comparison between the analytical calculations and the experimental tests performed by Bjurström (1973), shows that the analytical calculations underestimate the displacements of the bolts. The calculations also give a shear failure of the bolt at small deformations which deviates from the experimental tests, where all bolts failed in tension.

A good agreement with experimental tests was achieved for soft rock. For harder rock conditions, the agreement was not as good. Carlsson (2015) proposed that this could be due to the assumption that the bolt rest against the rock. In the test, the deformation and crushing of the grout had a great influence, which the method does not take into account. When the analytical calculations were modified to include a bed module to account for the deformations in the grout, better agreement was obtained both for the deformations and the load capacity. With this change, good agreement between theory and practice was obtained for a bolts installed orthogonally to the fracture. While the modified method still underestimates the deformations for bolts installed at an angle of 60°, providing a shear failure, which did not occur in the tests.

Carlsson (2015) concludes that it has been shown, with some exceptions, that the method proposed by Holmberg (1992) has good compliance with experimentally performed shear test of rock bolts. However, the theory is sensitive and can underestimate the deformations in the bolts. This leads to an incorrect failure mechanism and an inaccurate estimation of the failure load.

The theory was then applied at a case study of a 30 m high spillway monolith with a horizontal fracture in the rock foundation. The rock bolts theoretical contribution to the stability of the concrete monolith was included, with some modification by including the plasticity of the grout. In the calculations, only the installed orthogonal bolts in relation to the fault plane were included since the theory for tilted bolts was deemed to be inaccurate. According to the calculations, the load capacity of the dam monolith is increased about 40 % when assuming a block size of one meter and about 15% when assuming a block size of three meters in the fracture plane. The rock bolts load capacity represents a non-negligible part of the total load capacity, but in relation to the capacity of the sliding plane it presents a small portion of the overall stability.

4.3 JAKOB LJUNGBERG (2016)

In the MSc project by Ljungberg (2016), the test results performed at Lima hydropower station (see Section 5) were evaluated. In these tests, a test rig consisting of a hydraulic piston causing a cylinder pressing down on the rock around the rock bolt was used. The downside with this test method is that it is not possible to capture a rock cone failure, compared to for instance the test method used by Lepine (2012). The focus of the work presented in Ljungberg (2016) was to study if the test rig could potentially influence the obtained test result in any way.

In order to evaluate this, numerical analyses were performed where two types of pull-out tests were simulated, one which agrees with the used test rig where the pull-out force is transmitted as pressure in the rock compared to a pure pull-out test where the pull-out force was simulated using a predefined displacement.

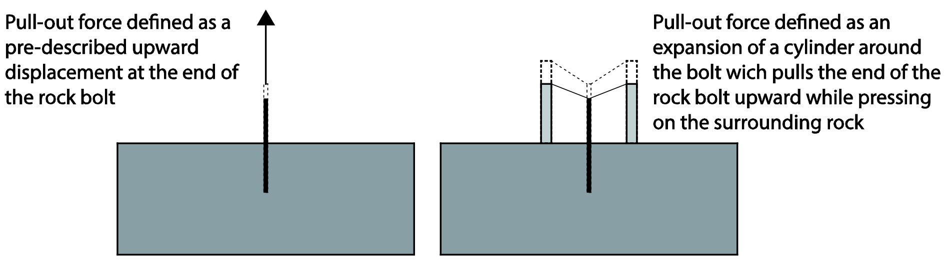


Figure 4-1 Illustration of the two studied methods to perform a pull-out test.

The analyses showed that the configuration of the test rig had no direct effect on the results if the failure mode was yielding of the rock bolt. However, if the failure was governed by failure in the grout, then a significant influence of the test rig could be detected, see Figure 4-2. The results obtained with the studied test rig leads to overestimation of the load bearing capacity. This is believed to be caused by the poisson's effect where high vertical stresses are introduced in the rock results in expansion in the horizontal direction. This horizontal expansion results in that the rock bolt is pinched due to confinement and thereby allows for higher pull-out force.

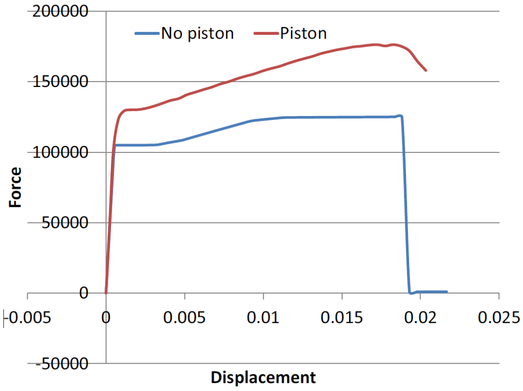


Figure 4-2 Load-displacement curve for two studied methods to perform a pull-out test

5 Pull-out test of 50 year old bolts at Lima hydropower station

The opportunity for further tests of existing rock bolts was given in 2015. To better understand the failure process, the existing testing equipment used by Larsson (2007) was modified in order for the force-displacement relationship to be measured during the test. In this chapter, the field testing is summarized.

Lima hydropower station is located in Västerdalälven in Dalarna, about 20 km northeast of Malung. In 2015, strengthening and reconstruction of the dam was performed to increase the discharge capacity and in conjunction with this, the existing log chute spillway was replaced with a new spillway and a new spillway channel. In connection to the demolition of the old log chute spillway, a number of rock anchors were uncovered and the opportunity to test their residual carrying capacity arose. The test equipment used in the Elforsk projects at Hotagen (described in Chapter 3 (Larsson, 2007)) was available and after a few modifications; it could be used for the tests.

5.1 METHOD

The test equipment consists of a compressor which drives a piston. The piston is threaded on the bolt and is then fasten to the bolt with two wedges. The piston is extended between the rock and the wedge, pulling the bolt upwards. Both force and displacement are recorded during the test. The force is measured by logging the test device signal while the displacements are measured by attaching a measuring device to a plate between the wedge and piston. Full test setup is illustrated in Figure 5-1.

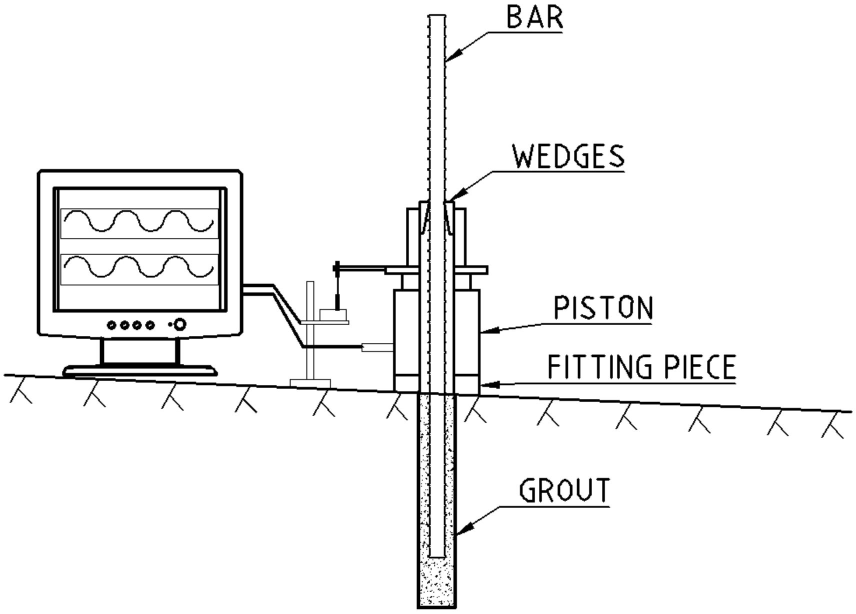


Figure 5-1 Illustration of the test setup.

Before the installation of the testing equipment, the bolts were straightened and the contact surface around the bolts was cleaned. After the testing device and measuring device were installed, the bolt was loaded with a successively increasing load. The load was increased until the maximum capacity of the testing device or a failure occurred.

After the demolition of the log chute spillway, four reinforcement bars, bolts, were identified that had not been destroyed during the demolition process. The bolts were located in two groups at two different locations. The two bolts in Group 1 were positioned just downstream of the dam just to the right of the spillway monolith. Group 2 was located about 50 meters downstream of the dam along the draft tube channel

The two bolts in Group 1 were located just downstream of the dam body. These bolts were ribbed reinforcement bars with a diameter of 19 mm and chamber diameter of 20 mm. According to the technical description from the construction time, the bolts should be drilled 1.5 meters into the rock and 0.7 meters in the concrete. The protruding part for bolt 1 and 2 was 60 and 70 centimetres respectively.

Figure 5-2 Rock bolt 1 and 2 before testing

The second group of bolts were located about 50 meters downstream of the dam body along the outlet channel. The bolts are of the type ribbed rebar with a diameter of 17 mm and placed in boreholes with a diameter of 18 mm. These bolts were significantly deformed as seen in Figure 5-3.

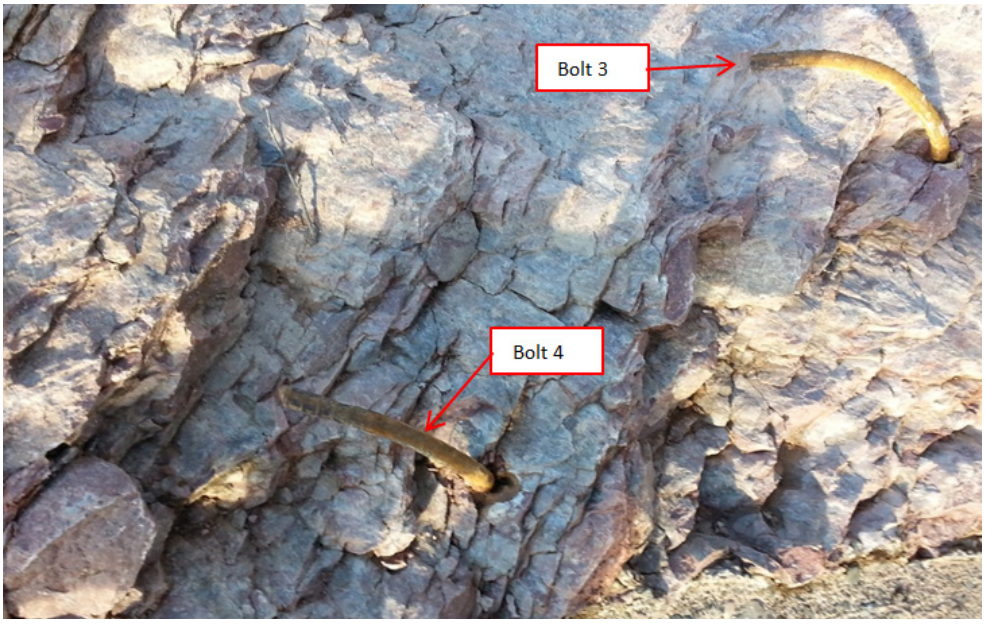


Figure 5-3 Rock bolt 3 and 4 before testing

Before the testing could be performed, efforts had to be made to straighten the bars. However, bolt no 4 was so damaged that the bar could not be straightened and it was excluded from the tests. For bolt no 3, the testing procedure was slightly more complicated than for the bars in the previous group. Only about 30 cm of the bolt protruded from the rock. Due to the design of the test equipment, a minimum of 40 cm protruding bolt is needed to be able to mount the testing equipment. As seen in Figure 5-3, the rock around the bar was much foliated and it was difficult to create a smooth surface to set the test equipment on. To enable testing, a piece of rock was removed and a smooth surface was created.

5.2 RESULTS

For the testing of the two reinforcement bolts in group 1, no failure of the bolt or grout could be obtained. Both reinforcement bars were loaded well beyond their yield stress when the rock under the machine was crushed and thereby made it impossible to reach higher loads that could cause failure of the bolt or the grout. In Figure 5-4, the crushed rock around Bolt no 2 after the testing can be seen.

Figure 5-4 Crushed rock around Bolt 2 after testing

In Figure 5-5, the recorded forces and displacements are presented along with the estimated nominal stress levels are presented. Notably, the obtained forces in the bar were significantly higher than the permissible level according RIDAS.

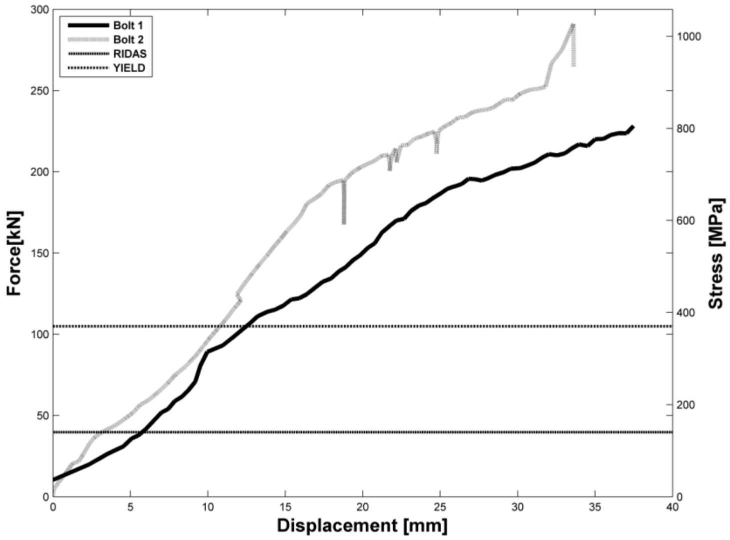


Figure 5-5 Load and deflection curves obtained from the tests of bolt no 1 and 2.

Also for bolt 3, no failure of the bolt or grout could be obtained. As it can be seen in Figure 5-6, the rock at the test site of bolt 3 was significantly crushed. Since the rock was in poor condition prior to the test, it was already weakened and the crushing of the rock under the test rig was more severe than for the bolts in group 1. Therefore, it was not possible to subject the bar to load levels of the same magnitude as the bolts in group 1 before the test rig lost its support. Also in this case with the poor support conditions for the test rig, the obtained forces in the bar were above the permissible level according to RIDAS. In Figure 5-7, measured forces and displacements for bolt 3 are shown along with the estimated nominal stress levels.

Figure 5-6 Bolt 3 after testing

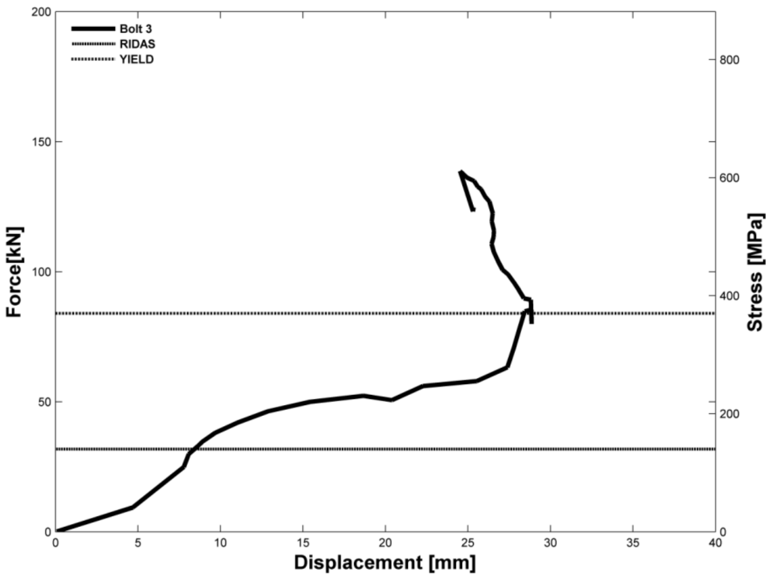


Figure 5-7 Load and deflection curve obtained from the test of bolt no 3.

In all performed tests, the tested bolts have shown high load carrying capacity. All the tested bolts have, with margin, withstood stresses that exceed the maximum allowed stresses according to RIDAS. During visual inspection, not any of the bolts showed any direct signs of degradation and all investigated bolts were deemed to be in very good condition.

6 Case study

To study the effect of rock bolts – and thus the benefits of a refined calculation methodology regarding their contributions to dam safety – analytical, probabilistic and finite element analyses were performed with and without rock bolts for a typical small Swedish monolith.

The studied dam is a small buttress monolith with a total of 20 rock bolts. The monolith has a height of 8 m where the water level is 0.5 m below the dam crest. The front plate has a width of 8 m and a constant thickness of 1.2 m, with a 1 m wide dam toe. The buttress has a constant thickness of 3.0 m. The front-plate and buttress are rigidly connected. A principal sketch of the dam with dimensions is shown in Figure 6-1.

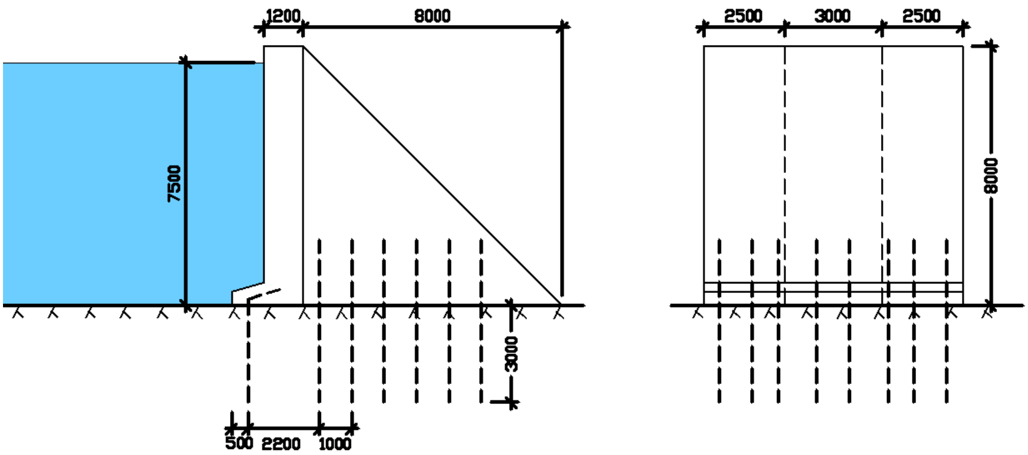


Figure 6-1 Section and elevation of the studied monolith

The monolith is anchored with 20 rock bolts, drilled 3 m into the rock and embedded 2 meters in the concrete. The bolts are ribbed steel bars with a diameter of 25 mm which are placed in boreholes with a diameter of 55 mm. Eight of the bolts are located in the upstream toe while the remaining 12 bolts are placed in two rows under the buttress. In this study, a simplified load case has been analysed, including dead load, hydrostatic pressure and uplift pressure.

To study how different assumptions regarding the rock bolts affect the theoretical dam safety, three cases were studied.

- Case 1: Without bolts according to RIDAS for high consequence dams
- Case 2: Bolts with a maximum stress level of 140 MPa according to RIDAS for low consequence dams
- Case 3a: Bolts with a characteristic yield stress level of 370 MPa (without considering corrosion)
- Case 3b: Bolts with a characteristic yield stress level of 370 MPa (also considering corrosion)

Case 1 serves as a reference case which makes it possible to estimate the influence of rock bolts. This case also corresponds to how the dam should be analysed according to the Swedish standard where rock bolts should be neglected in stability calculations for dams in higher consequences classes. In Case 2, the bolts are included according to the stress limitation in the Swedish standard for dams in lower consequence classes. In Case 3a, the characteristic strength of the steel is used. In order to study effect of degradation of rock bolts on the dam safety, Case 3b is studied. In Case 3b, degradation of the rock bolts are included in the analyses by successively reducing the rock bolt diameter, corresponding to a yearly degradation rate of the steel calculated according to Section 2.3.

6.1 MATERIAL

The material properties used in this study are given in Table 6-1. The values and probability density functions proposed in Table 6-1 are based on (Westberg & Johansson, 2016) and (RIDAS, 2011).

In the probabilistic analyses, see Chapter 7, probabilistic density function, distribution type, mean value, standard derivation and coefficient of variation are necessary inputs. Material properties used only in the analytical and numerical analysis, see Chapter 8, are presented as deterministic and in these cases, only the mean value are given. In the table, the following letters are used to describe if the assumed distribution is normal (N), lognormal (L) or deterministic (D).

In the comment column in Table 6-1 it is specified what or which analyses a material property is used in, analytic (A), probabilistic (P) or Finite element (FE).

Table 6-1 Material properties used in the analysis.

Random Variables	Unit	Notation	Distribution	Mean value	Standard dev.	COV	Comments
Unit weight concrete	$\mathrm{kN/m^3}$	ρ_c	N	23.5	0.94	0.04	A,P,FE
Unit weight rock mass	$\mathrm{kN/m^3}$	ρ_m	N	26.5	0.15	0.01	P
Elastic modulus, concrete	GPa	E_c	D	30	-	-	FE
Possion's ratio, concrete	[-]	ν_c	D	0.2	-	-	FE
Basic friction angle, concrete-rock	°	$\emptyset_b$	N	45	2.25	0.05	P, FE
Concrete Aging Parameter	–	$\beta(t)$	L	1.27	0.08	0.06	P
Concrete Compressive Strength	MPa	f_{cm}	L	43	5	0.12	P
Concrete Tensile Strength	MPa	f_{ctm}	D	3.7	-	-	FE
Corrosion Rate	µm/year	cr	L	30	10	0.33	A,P,FE
Adhesion grout-rock	Mpa	c_{g-r}	L	2.0	0.6	0.30	P,FE
Adhesion grout-steel	MPa	c_{g-s}	L	1.2	0.36	0.30	P,FE
Steel Capacity	MPa	f_{ym}	L	417	29	0.07	P
Elastic modulus, Steel	GPa	E_s	D	200	-	-	A,FE
Ultimate strain, Steel	[-]	$\varepsilon_{s.u}$	D	0.15	-	-	FE
Poisson's ratio, Steel	[-]	ν_s	D	0.3	-	-	FE
Steel, Yield strength, KS40	MPa	$f_{yk.KS40}$	D	370	-	-	A,FE
Steel, Ultimate strength, KS40	MPa	$f_{uk.KS40}$	D	600	-	-	FE
Steel, Yield strength, RIDAS	MPa	$f_{yk.RIDAS}$	D	140	-	-	A,FE
Steel, Ultimate strength, RIDAS	MPa	$f_{uk.RIDAS}$	D	140	-	-	FE

In this study, linear material properties were assumed for both concrete and rock. Two different non-linear material models are used for the rock bolts for case 2 and case 3 respectively. One where the steel is of the Swedish type KS40s, with material properties obtained from (Ljungkrantz, Möller, & Petersons, 1994)) and one where a yield level of 140 MPa is used according to the Swedish Hydropower companies guidelines for dam safety (RIDAS, 2011). The uniaxial stress-strain relationship for the two steel material models is shown in Figure 6-2.

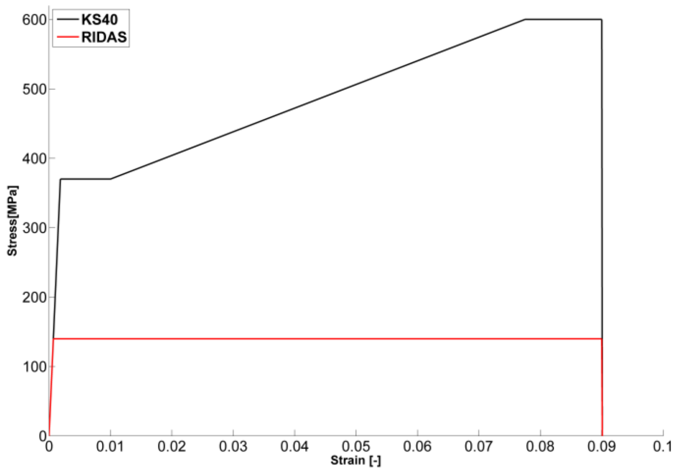


Figure 6-2 Uni-axial behaviour used for the reinforcement

6.2 LOADS

6.2.1 Hydrostatic pressure

Horizontal water pressure and uplift both depend on the upstream water level of the dam. As explained in the introduction of this chapter the upstream water level remains constant and 0.5 m below the dam crest.

6.2.2 Ice load

The magnitude of the ice load assumed in the analyses performed in this document remains constant and equal to 100 kN/m.

6.2.3 Uplift pressure

The uplift pressure is considered to be acting on the front plate and monolith with a linear variation from full water head to zero. In this project, the width of the buttress is 3 m, thus uplift pressure on the buttress was included (RIDAS, 2011). There is no tail water surrounding the monolith on the downstream side, the uplift pressure goes down to 0 kPa downstream of the front plate.

For the probabilistic analysis presented in this document the uplift is modelled as follows according to (Westberg & Johansson, 2016):

$$U = U_d \cdot C \tag{22}$$

where U is the uplift force, U_d is the uplift force obtained in the deterministic analysis (which is the result of a homogeneous foundation) and C is a random variable with mean value and standard deviation. The results indicate that $C \sim Beta(r, t, a, b)$, where r, t, a, b are parameters (Westberg & Johansson, 2016).

Similarly, the moment due to uplift pressure is modelled as follows (Westberg & Johansson, 2016):

$$U_m = U_{dm} \cdot C_m \tag{23}$$

where U_m is the moment of uplift pressure, is U_{dm} is the uplift moment in the deterministic analysis and C_m is the uplift parameter. $C_m \sim Beta\ (r, t, a, b)$.

$$\begin{gathered} C \sim Beta\ (1.96; 1.95; 0.08; 1.9) \\ C_m \sim Beta\ (2.22; 1.33; 0.11; a1) \end{gathered} \tag{24}$$

where $a1 = 1.49$ for gravity type structures. The parameter $a1$ for buttress type structures is obtained from Figure 6-3 (Westberg & Johansson, 2016).

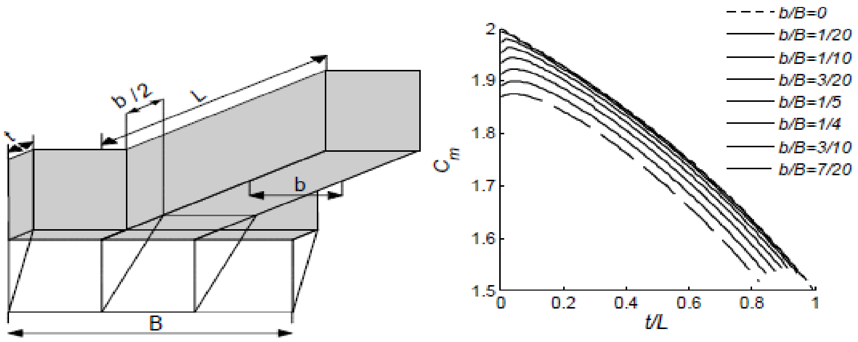


Figure 6-3 a) Uplift distribution assumed for a buttress dam b) Limits of Cm for varying t and b from Westberg (2007).

6.3 CAPACITY OF ONE ROCK BOLT

In Table 6-2, the load capacity of the rock bolts studied in the case study are summarized for the possible failure modes are presented. As it can be seen from the table, the load capacity of the steel bolt is lower than all other failure mechanisms. The tensile failure will mainly occur in an overturning failure while the shear failure will occur in a sliding failure mode.

Table 6-2 Load capacity for a single rock bolt for all failure modes

Failure mode	Load capacity (kN)
Rock cone failure (eq. (2-2))	250
Adhesive failure; rock and grout (eq. (2-3))	1037
Adhesive failure; steel and grout (eq. (2-4))	283
Adhesive failure; concrete and steel (eq. (2-5))	650
Tensile failure; steel bar (eq. (2-7))	182
Shear failure; steel bar (eq. (2-7))	91

6.4 ANALYTIC STABILITY CALCULATIONS (DETERMINISTIC CALCULATIONS)

Stability calculations were performed on the monolith, where the safety factor was calculated for the failure modes sliding and overturning respectively. For the sliding failure safety factor was defined as the sum of the vertical forces divided by the magnitude of the horizontal forces. The safety factor for overturning was defined as the ratio between the stabilizing moment and the destabilizing moments. Stability calculations were performed on the monolith, where the safety factor was calculated for the failure modes sliding and overturning respectively. The safety factor for the sliding and overturning failure modes was calculated for the following cases.

- Case 1: Without bolts
- Case 2: Bolts with a maximum stress level of 140 MPa according to RIDAS
- Case 3a: Bolts with a yield stress level of 370 MPa.

The rock bolts contribution in the analytic stability calculations was included in two ways depending on the studied failure mode. For the sliding failure mode, the contribution from bolts was calculated according to eq. (2.15) and for an overturning failure mode according to eq. (2.20). For comparison, an additional calculation for Case 2 and 3a regarding sliding failure was also calculated based on eq. (2.16) under the assumption that the bolts acts as dowels.

In this chapter, one load case was analysed where dead load, hydrostatic pressure, uplift pressure and ice load are included. Loads and lever arms are presented in Figure 6-4.

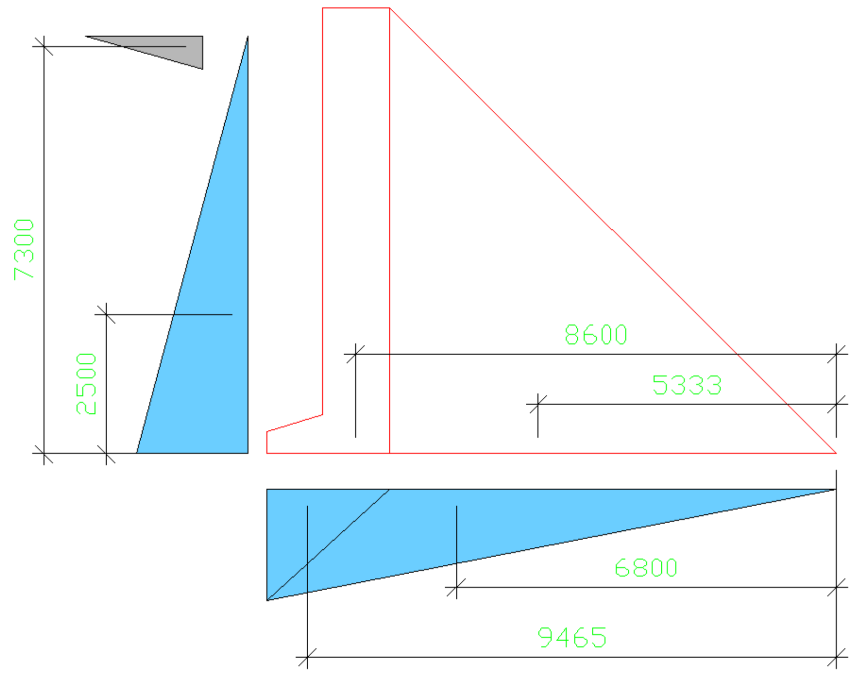


Figure 6-4 Loads and lever arms.

Table 6-3 presents the safety factors obtained from the analytical calculations.

Table 6-3 Safety factors, analytic stability calculations

	Sliding	Sliding, dowel effect	Overturning
	M		Sf
Case 1: Without bolts	0.97	0.97	1.43
Case 2: Bolts according to RIDAS	0.67	0.75	1.87
Case 3: KS40 Bolts	0.45	0.39	2.54

The choice of calculation method for the sliding analysis has a significant influence on the results. For dams that are rather close to the safety margin, how to consider the bolts may result in a difference between a stable and unstable dam. Notable is that the method giving the highest safety factor varies between Case 2 and 3. This is due to the mathematical function of the safety factor, where the contribution of the bolts is included either in the numerator or denominator, as shown in eq. (2.18) and eq. (2.19).

Calculations were carried out with decreasing radius to stimulate the reduced cross-sectional area over time due to corrosion. The diameter of a bolt at a certain age was calculated as

$$d_{model} = d_{initial} - (Construction\ age * 2 * degradation\ rate) \qquad (6.25)$$

where $d_{initial}$ is the initial bolt diameter, d_{model} is the used bolt diameter for the current model and $Construction\ age$ is the dams' age in years. The mean value of the $degradation\ rate$ proposed in Chapter 2 is used with a yearly degradation mean value of $\mu = 30\ \mu m/year$. The safety factor and friction coefficient were calculated at different ages for the dam and presented in Table 6-4. For sliding, the tensile capacity of the bolts is utilized according to eq. (2.17).

Table 6-4 Safety factor for different age of the dam.

Age	Sliding M	Overturning Sf
0	0.45	2.54
25	0.48	2.42
50	0.51	2.30
75	0.54	2.19
100	0.58	2,09
150	0,66	1,91
200	0.74	1.76
250	0.82	1.64

If only considerations are taken to the rock bolts, the dam will remain safe in 200 years. It is interesting to notice that the safety factors for Case 2, with the stress limitation according to RIDAS are similar to the safety factors obtained for the dam after about 150 years.

7 Probabilistic analyses

In order to take into account the uncertainties of the parameters defined in Chapter 6 and their influence on the dam safety, a probabilistic analysis has been performed. The aim of this analysis was to study how the reliability of a dam monolith varies with and without rock bolts and to investigate the influence of bolt degradation on the dam stability.

7.1 METHODOLOGY

Probabilistic analysis has been performed for the case of study presented in Chapter 6 for the failure modes of sliding and overturning. These reliability calculations were carried out using the commercial software Comrel, version 8.1. The main probabilistic analysis method utilized was the "First Order Reliability Method" or "FORM". For further information regarding probabilistic analyses, see for instance Westberg and Johansson (2016).

The results presented in this chapter correspond to the safety index (β) and the probability of failure (p_f) for the following cases:

- Case 1: The dam is analysed for the failure modes of sliding and overturning without rock bolts.
- Case 3a: In this case rock bolts with a characteristic yield stress of 370 MPa are included in the probabilistic analysis without considering corrosion($t = 0\ years$).
- Case 3b: Finally, a corresponding analysis as in case 3a is performed but also considering the corrosion of the rock bolts for different ages ($0 < \mathrm{t} < 120\ \mathrm{years}$) and its influence on the dam stability.

The contribution from the rock bolts in the probabilistic calculation was included as described in Chapter 6.4. For the sliding failure mode, two calculations were performed; the first according to eq. (2.15) with the assumption that tensile stresses develop in the bolt and the second according to eq. (2.16) where the bolt works as a dowel. For the overturning failure mode, the contribution from the bolt was calculated according to eq. (2.20). The failure modes (equations 2.2-2.5) for the rock bolts described in chapter 2.1 are included in the probabilistic calculations as a weakest link system (eq. 2.1). However, the results show that it is the steel capacity that is the governing failure mode, and for this reason the steel capacity is included in equations (2.15), (2.16) and (2.20). The random variables used for this dam are shown in Chapter 6. All of these variables are assumed to be uncorrelated.

In the probabilistic analyses, every single rock bolt is considered as a series system where failure occurs when a failure mode of a component is fulfilled (see Chapter 2.1). In the analyses, the results showed that steel failure was the weakest of all the studied failure modes.

However, the rock bolts installed in the dam are modelled as a parallel system. In contrast with a series system, a parallel system shows a load redistribution capacity after the failure of some of the components. Thus the failure of the entire system requires the failure of more than one element.

In order to simplify the calculation of the probability of failure of a parallel system with correlated members, it has been assumed that all the rock bolts are perfectly correlated. This assumption means that all the rock bolts present the same degradation and hence they will fail at the same time giving a "worst case scenario".

7.2 RESULTS WITHOUT CONSIDERATION OF CORROSION

Table 7-1 and Table 7-2 displayed below show the safety index(β), the probability of failure (pf) and the sensitivity values (α) obtained in the probabilistic analyses for Case 1 and Case 3a.The results shows that an overturning failure is not a feasible failure mode in this case when rock bolts are considered, i.e. Case 3a. The sensitivity values show the importance of different parameters on the final result, where an absolute value close to one indicates high importance.

Table 7-1 Results from probabilistic analysis (FORM) for Case 1 and Case 3a.

	Sliding		**Overturning**	
Cases	β	pf	β	pf
Case 1	0.24	4.10E-01	3.45	2.82E-04
Case 3a (Tensile force)	8.69	1.84E-18	17.93	3.81E-72
Case 3a (Dowel)	4.96	3.47E-07	17.93	3.81E-72

Table 7-2 Results from sensitivity values (α) for random variables for Case 1 and Case 3a.

	Case 1		**Case 3a**	
	Sliding	**Overturning**	**Sliding**	**Overturning**
Concrete Density	0.21	0.80	0.62	0.77
Uplift	-0.96	-0.60	-0.54	-0.16
Tan(ϕ_b)	0.19	0.00	0.28	0.00
Steel Resistance	0.00	0.00	0.48	0.61
Sum of α^2	**1**	**1**	**1**	**1**

The results from the probabilistic analysis displayed in Table 7-1 show that the calculated dam safety is remarkably improved when rock bolts are considered for both sliding and overturning failure modes. At the same time, the sensitivity values displayed in Table 7-2 show that the steel resistance is one of the random variables that most contributes to variation of the limit state function when rock bolts are considered. The other variables that give significant contribution to the variation of the limit state function are the friction angle in the concrete-rock interface (for sliding), the concrete density and the uplift. However, the results obtained when the rock bolts are considered may not be representative since in most cases in Sweden; dams are between 40 and 60 years old. In these cases, the assumption of intact embedded rock bolts may be non-conservative. However, as presented in Section 2.3, the degradation is expected to be small and thereby this assumption is likely to have little impact on the results.

7.3 RESULTS WITH CONSIDERATION OF CORROSION

The analysis carried out in Case 3b is intended as a representation of the evolution of the dam reliability along its life span. The results displayed in this chapter correspond to the safety index(β), the probability of failure (pf) and the sensitivity values of the different random variables considered for the dam monolith for a varying age of 10 to 120 years.

Figure 7-1 and Figure 7-2 show the results from the safety index (β) and the probability of failure (pf) for Case 3b in the dam monolith when degradation in the rock bolts is taken into consideration. Note that for the sliding failure mode rock bolts are considered with two different approaches according to Chapter 6.4.

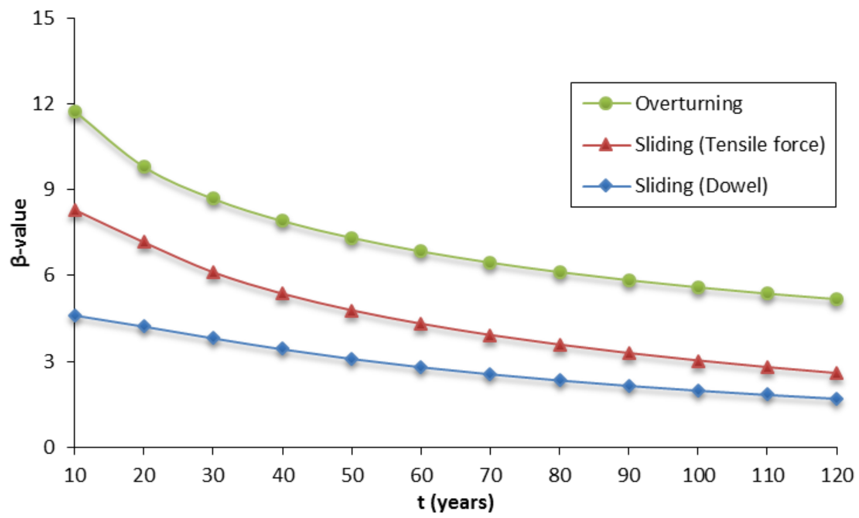


Figure 7-1- Results from safety index (β) for Case 3b.

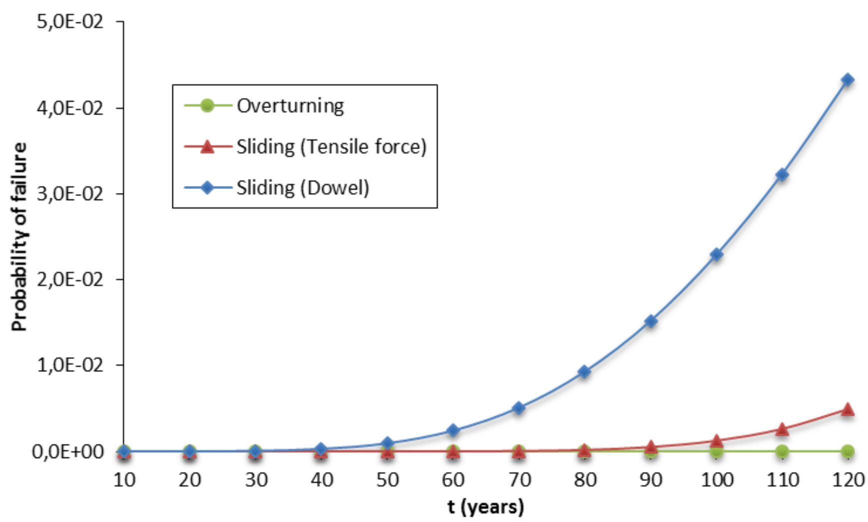


Figure 7-2- Results from probability of failure (pf) for Case 3b.

Figure 7-3 and Figure 7-4 show the results from the sensitivity values (α) for the random variables in Case 3b for both the sliding and overturning failure modes.

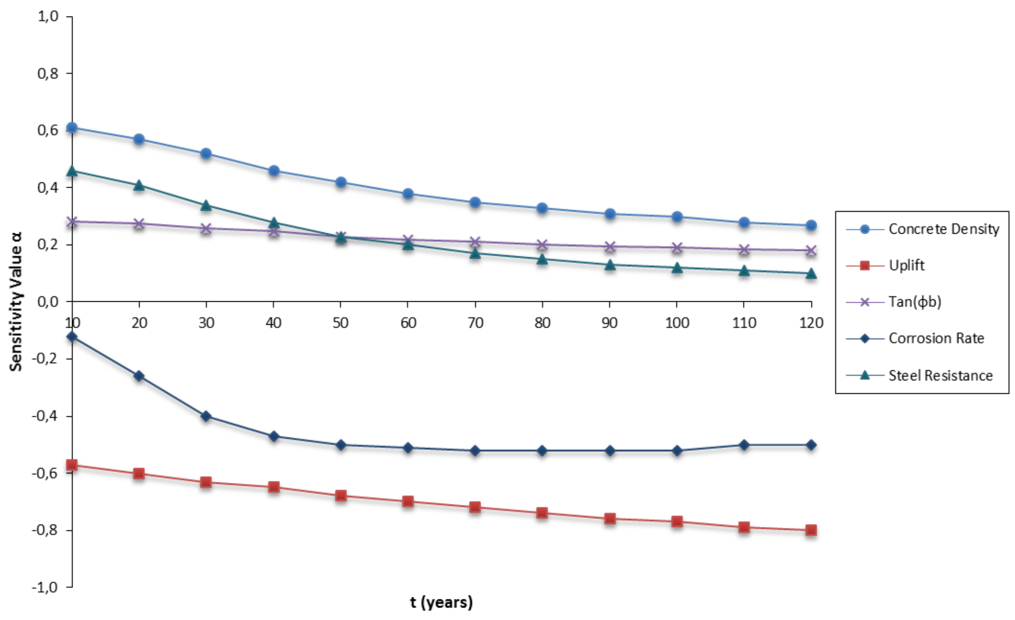


Figure 7-3- Results from sensitivity values (α) for random variables for Case 3b and sliding failure mode.

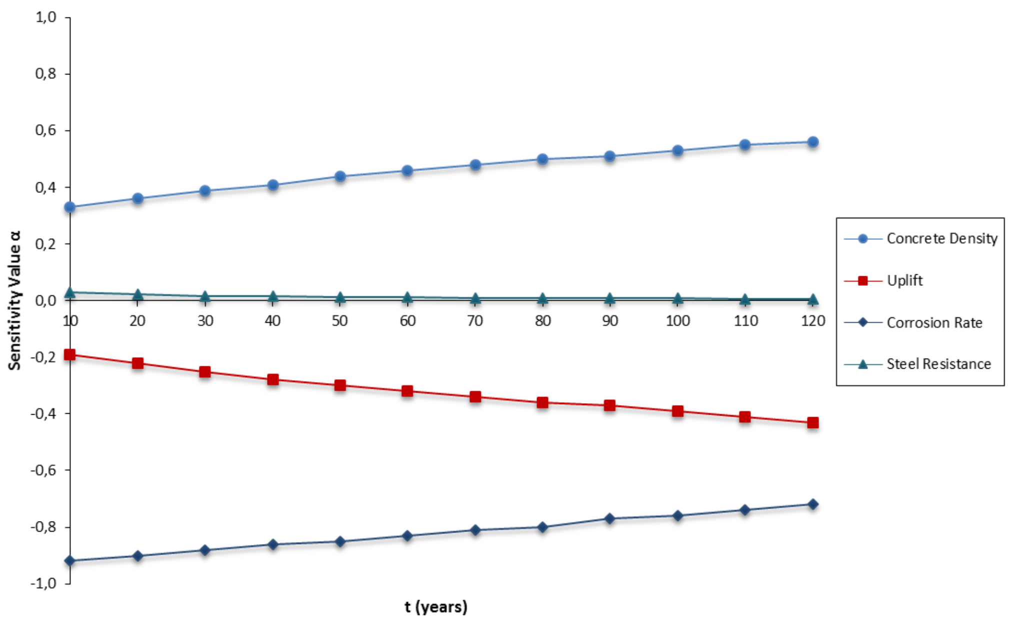


Figure 7-4- Results from sensitivity values (α) for random variables for Case 3b and overturning failure mode.

The results from probabilistic analysis for Case 3b reveal that degradation in the rock bolts, represented in the probabilistic analysis in terms of corrosion rate, affects the reliability of the dam monolith with increasing age. Figure 7-1 and Figure 7-2 shows that the safety index (β) decreases, in the interval between 10 and 120 years, from 4,6 to 1,7 (63% less) and from 8,3 to 2,6 (66% less) for the sliding failure mode taking into account tensile force respectively dowel effect in the rock bolts. The results for the overturning failure mode show that the safety index (β) decreases from 11.8 to 5.2 (56% less) in the interval between 10 and 120 years.

However, the influence of rock bolts with an age of 120 years in dam stability is still positive when comparing to the results obtained in Case 1 for the safety index (β), where rock bolts are not included in the probabilistic analysis.

Figure 7-3 and Figure 7-4 represent the evolution of the sensitivity values for random variables for the failure modes of sliding and overturning. For sliding failure mode, the corrosion rate has an increasing influence on the dam stability for high ages, while for the same age interval the steel resistance has a decreasing influence in the dam stability. For overturning failure mode, the corrosion rate has a decreasing influence in the dam stability for a period between 10 and 120 years. For the same period both the uplift and the concrete density, which represents the self-weight in the dam monolith, have an increasing influence on the dam stability.

7.4 TARGET RELIABILITY

In order to determine if the calculated safety index and probability of failure are acceptable, i.e. if the dam monolith is safe enough, comparisons to a target reliability is necessary.

Table 7-3 displays the recommended target reliability in terms of minimum safety index (β) presented in the Probabilistic Model Code for Concrete Dams (Westberg & Johansson, 2016).

Table 7-3 Minimum values for β in ultimate limit states and a reference period of 1 year (Westberg & Johansson, 2016).

Consequence class	Minimum β-value
A	5.2
B	4.8
C	4.2
U	3.8

The comparison between the recommended values for the safety index (β) Table 7-3 and the results obtained in the probabilistic analysis in Table 7-1 reveals a significant contribution from the rock bolts to dam stability. According to Table 7-1, the safety index (β) is increased to acceptable levels when the rock bolts are included. However, the safety index (β) changes along the life span of the dam monolith due to the degradation process in the rock bolts.

8 Numerical analyses

Numerical analyses have been performed to study the effect of rock bolts and the influence of degradation of rock bolts on the dam safety. The aim was to study how the failure mode and structural safety varies for a dam monolith with and without bolts and to investigate the influence of ageing rock bolts.

8.1 METHODOLOGY

Numerical, progressive failure, analyses have been performed on the monolith for the three cases (without bolts or including bolts) specified in Chapter 6.

- Case 1: Without bolts
- Case 2: Bolts with a maximum stress level of 140 MPa according to RIDAS
- Case 3a: Bolts with a yield stress level of 370 MPa (without considering corrosion)
- Case 3b: Bolts with a yield stress level of 370 MPa (also considering corrosion)

The analyses were first performed for design loads and in the subsequent step all destabilizing loads were increased until failure occurred. The applied loads were dead load, hydrostatic pressure, uplift pressure and ice load. All numerical analyses were performed in the commercial software Abaqus, version 6.14 with the standard implicit solver. The numerical model included both the dam and part of the surrounding rock, see Figure 8-1. The dam and the rock were modelled with 8-node linear brick elements with reduced integration and hourglass control (C3D8R in Abaqus). The average element size was 0.2 meter for the dam and 0.25 m for the rock, meaning that a total of 32640 elements were used for the dam and 40960 elements for the rock.

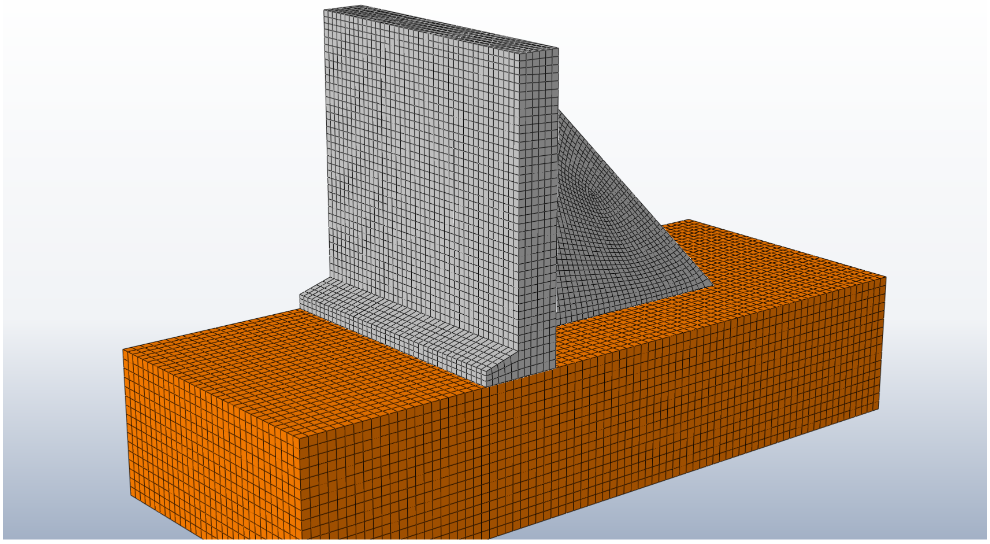


Figure 8-1 Mesh of dam and rock.

The boundary condition were applied at the rock by prohibiting displacements perpendicular to each side at all outer boundaries of the rock, except at the top surface, as seen in Figure 8-2. The dam is held in place by frictional forces at the interface between the dam and the foundation. This is done by introducing a general contact property in Abaqus, where the contact in the normal direction is modelled as ”hard contact” for compressive loads, but allows for separation due to tensile loads. This means that the nodes of the dam are not allowed to penetrate into the foundation but if the resulting nodal force is directed upward then the dam will separate from the ground. In the tangential direction, a friction contact model is introduced. The friction coefficient is set to 1.0 according to (RIDAS, 2011).

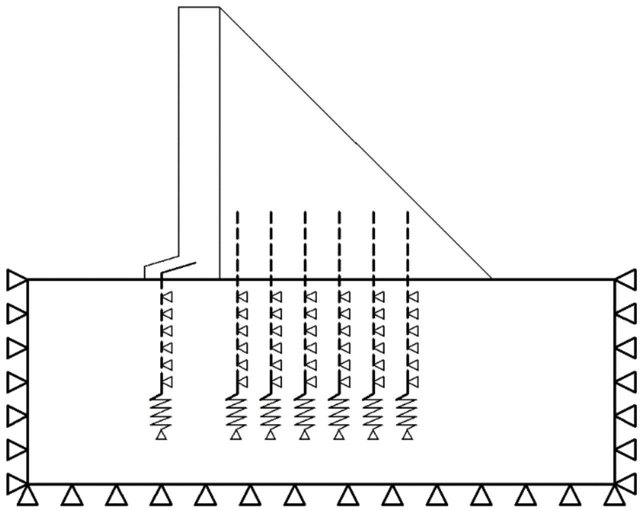


Figure 8-2 Static system with boundary conditions used in the numerical calculations and applied loads.

The bolts were attached to the ground in the axis direction with a non-linear spring (Spring 1 in Abaqus). The spring was defined to fail for the lowest calculated value for the failure modes for the attachment of the bolts, defined in Chapter 2 and with values according to Table 6-2 in Chapter 6. The bolts are meshed with a 3D, 2-node linear Timoshenko beam element (B31 in Abaqus), with the element size 0.035 m. The attachment was assumed as very stiff, allowing for an axial displacement of the spring with 0.1 mm before failure. Notable is that the steel failure was not included as a failure mode for the spring since the bars are included in the finite element model.

Two different non-linear material models were used for the rock bolts for Case 2 and Case 3 respectively. One where the steel is of the Swedish type KS40s, with material properties obtained from Ljungkrantz et al. (1994) and one where a yield level of 140 MPa is used according to RIDAS (2011). The stress-strain relationship for the bolts is presented in Chapter 6 in Figure 6-2.

For the numerical progressive failure analyses, the method of increased density was used. The hydrostatic pressure and uplift pressure were applied with a load-controlled loading system, magnifying the design pressure using a load factor λ

until failure occurs, see Figure 8-3. Failure was assumed to occur when the ultimate load for a monolith was reached.

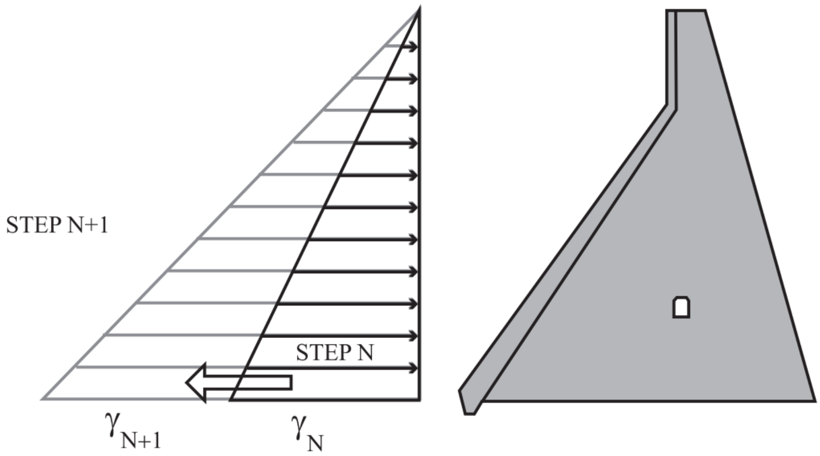


Figure 8-3 Illustration of the method of increased density numerical progressive failure analyses, from (Nordström, Malm, Johansson, Ligier, & Øyvind, 2015).

8.2 PROGRESSIVE FAILURE ANALYSIS

The obtained safety factors from the numerical analysis are presented in Table 8-1 together with the safety factors from the analytical calculations.

Table 8-1 Safety factors, numerical progressive failure analysis and analytic stability calculations (see Section 6.4).

	Numerical	Analytic	
Cases	**Safety factor**	**Sliding**	**Overturning**
Case 1: Without bolts	1.00	1.03	1.43
Case 2: Bolts according to RIDAS	1.98	1.49	2.02
Case 3: KS40 Bolts	3.00	2.22	2.54

Figure 8-4 presents the crest displacement for the progressive failure analyses. For the case that not considers the rock bolts, the crest displacements increases linearly due to increased loading until the external forces reaches the ultimate load capacity of the dam. At this point, the dam starts to slide and the sliding displacement increases towards infinity for a constant load. One interesting result is that the safety factor for the FE model is lower than the corresponding safety factors for sliding and overturning based on the analytical calculations. The crest displacements are identical for the cases up to the load level where the model without bolts fails. As the dam starts to slide, the bolts are displaced from their original location activating forces in the bolts.

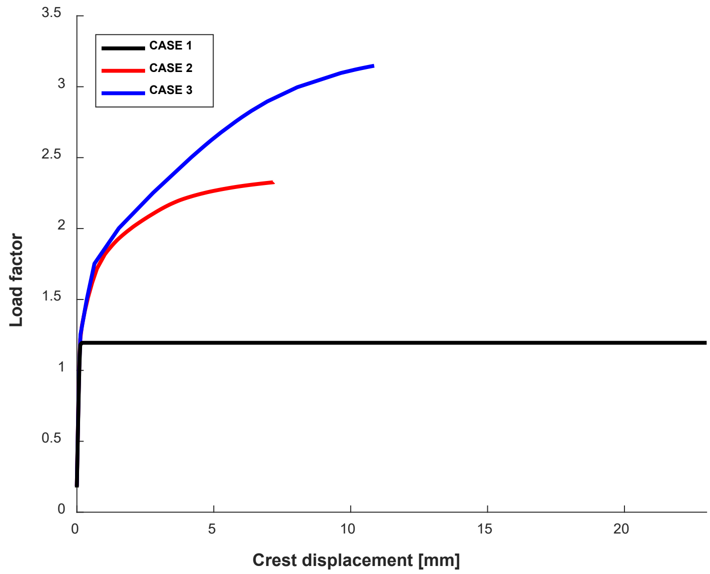
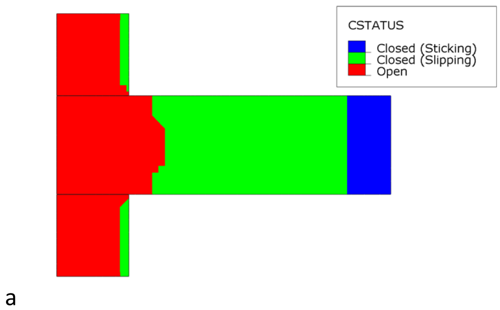


Figure 8-4 Crest displacements for increasing load during the progressive failure analyses.

The failure mode obtained in the FE analyses is a combination of overturning and sliding. First the upstream toe of the dam is lifted of the ground, reducing the contact area between the dam and the foundation. The total shear force must then be mobilized over all smaller areas, causing higher shear stresses and thereby results in a lower safety factor. In Figure 8-5 the status of the contact between the dam and the foundation is plotted during the analyses of the real failure. The status can be either open, closed sticking, or closed slipping. Open and closed refers to if there is a physical gap between the concrete and rock surface for the different points in contact. Closed sticking means that no sliding occurs at this point, while closed slipping means that this point is subjected to sliding deformation. As can be seen in Figure 8-5, the contact under the front plate is open and a part of the buttress has started to slip and the shear forces at the dam-rock interface is then mobilized at a smaller area.



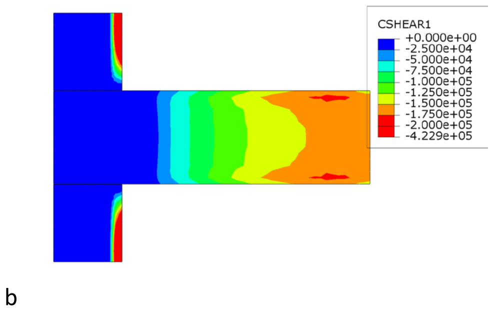


Figure 8-5 Output at the rock-concrete interface at load factor λ=1.0 for Case 1 a) contact status b) shear force.

In Figure 8-6 are the total vertical (normal force) and horizontal (shear force) reaction forces plotted for Case 1 and 2. For Case 1, the reaction forces are equal to the contact forces in the surface between rock and dam and for Case 3 and equal to the sum of the contact forces in the rock-concrete interface in addition to the sum of the reaction force for all bolts.

When the load factor is equal to zero, i.e. only the dead load is applied, only the normal contact force is present. The normal force then decrease linearly as the uplift pressure is increased. The horizontal (shear) forces are initiated and increased as the hydrostatic pressure and ice load is amplified. In Case 1 without bolts the shear force is increased during loading until the shear force is equal to the normal force. At that point, the shear force cannot be increased further since μ = 1.0, and thereby the dam starts to slide. Up to that point, the interface forces are identical for the two cases with and without rock bolts. As the dam starts to slide, the bolts are displaced from their initial position and thereby activated and starts carrying loads. Normally without rock bolts, the failure criterion for sliding is that the horizontal forces must be less than the vertical. However, when bolts are considered the total shear (horizontal) forces greatly exceeds the normal (vertical) forces.

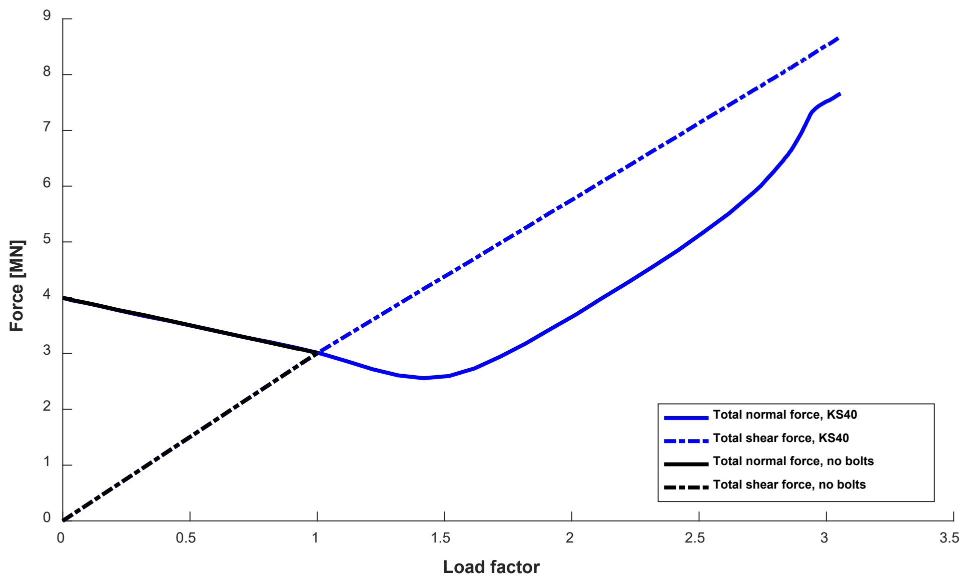


Figure 8-6 Vertical and horizontal reaction force for Case 1 and Case 3.

Initially, the increase of the total horizontal force and decrease of the vertical force continues as the load factor is increased. When the load factor reaches 1.5, the decreasing trend for the total vertical force is broken and the vertical forces starts to increase. The total vertical and horizontal forces thereafter increase linearly at approximately the same rate until the failure occurs. To show the force distribution in detail, the total reaction force for Case 3 is dived into interface force and forces in the bolts which is illustrated in Figure 8-7. It can be seen that a part of the horizontal force is carried by shear forces in the bolts even for load factors below the failure load factor without bolts. It can also be seen, that at the load factor equal to 1.0, i.e. design levels of the loads, the tensile force in the rock bolts is marginal or almost non-existent while the shear force is significant and corresponds to about 10 % of the total shear force resistance of the dam.

The main contribution to the increased load capacity comes from shear forces in the bolts. Shear forces are present at much smaller movements and is during the whole loading process significantly higher than the normal force. The normal force in the bolts contribution seems to be to keep the dam from lifting, enabling a maintained and even increased restraining friction force in the concrete-rock interface. As the friction coefficient of 1.0 is used, the maximum shear force is equal to the normal force. When the shear force reached the value of the normal force they are thereafter equal throughout the calculation. It means that as much of the loads as possible are carried by shear forces in the interface, and the reaming load must be carried by the bolts.

The results also show that the bolts are as stated in Section 2.1.5 carrying a combination of tensile and shear forces. For high load factors, close to the failure, the load carried as normal and shear force are about 4 and 5 MN respectively. This implies that analytical calculation should preferably be based on a combined load carrying capacity of normal and shear forces as described in Section 2.1.5, rather than observing these as separate cases.

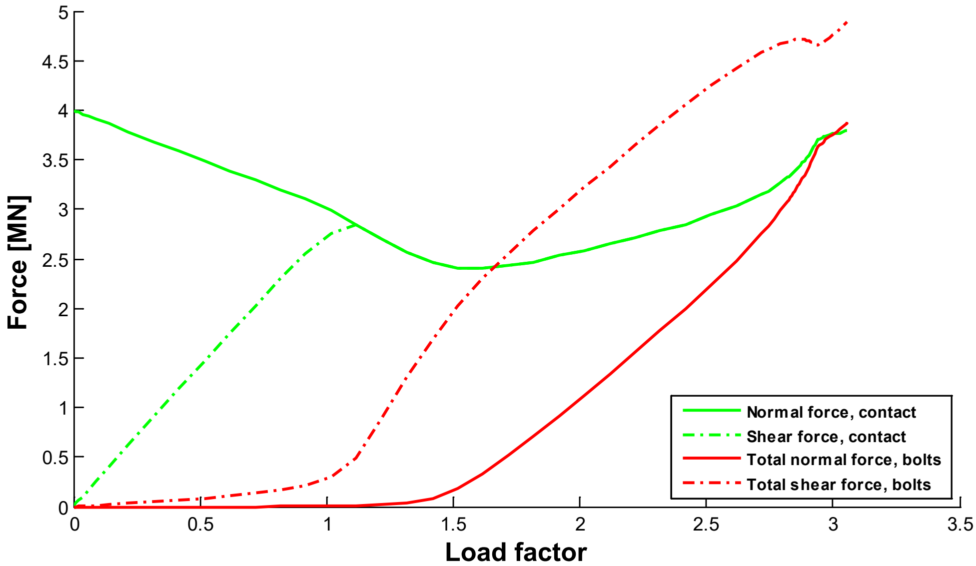


Figure 8-7 Vertical and horizontal forces in rock-dam interface and bolts for Case 3.

8.3 GRADUAL WEAKENING OF THE BOLTS

Furthermore, analyses were performed to study the influence of gradual weakening of the bolts on the dam's safety factor. Starting with the Case 3 model, with bolts having a strength corresponding to material type KS40, the bolts are then gradually weakened row by row from upstream to downstream. The weakening was performed in two ways, by changing material model for the bolts in the current row from KS40 to RIDAS and by completely removing the bolts in the current row. The degradation was assumed to propagate from upstream to downstream, therefore have the bolts under the front-plate been weakened first and thereafter row by row under the monolith. In Figure 8-8 the safety factor versus degradation distribution and the safety factor versus the number of weakened bolts are plotted.

When KS40 bolts replaced with RIDAS bolts, a large drop in safety factor occurs as the first line is weaken. The decrease in safety factor then levels out for the reduction of further bolts placed downstream. This shows that safety factor is not really dependent on the actual the strength of the bolts close to the downstream toe, at least as long as their strength is within the interval of 140 to 370 MPa. If the bolts instead are gradually removed, then the loss in safety factor is similar for the first three rows in this case. However, as the gradually removing of the bolt continues, a significant reduction of the safety factor continues as a result when the bolts are removed under the monolith.

This indicates that the strength of the bolts under the front plate are important, and stronger bolts results in a larger part the dam base in contact with the ground. This results in a larger contact surface, which provides a higher reaction force in the rock-dam interface. For the bolts under the monolith, their presence seems more important than their actual strength. One explanation for this could be the way the dam behaves during the progressive failure analysis, were the upstream part of the dam at first is lifted and thereafter the dam fails when the ultimate shear strength of the bolts is reached. This means that the bolts are activated late in the failure process and that they are loaded rapidly. Notable is that when the bolts in row four is removed, the safety factor is slightly increased. This is due to a change in failure mode occurring at this point, instead of sliding failure an overturning failure occurs.

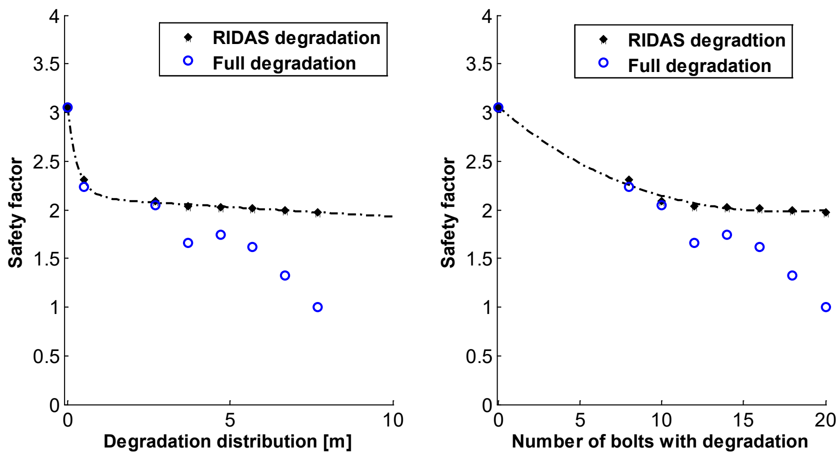


Figure 8-8 Safety factor versus degradation distribution and the safety factor versus the number of weakened bolts.

8.4 ASSESSMENT OF THE SAFETY FACTOR WITH AGE

For Case 3b, calculations were carried out with decreasing radius to simulate the reduced cross-sectional area over time due to corrosion. Failure analyses were performed for Case 3b, with bolts with the diameter of 25, 22.5, 20, 17.5, 15, 10 and 5 mm. The safety factor for different bolt diameters is plotted in Figure 8-9.

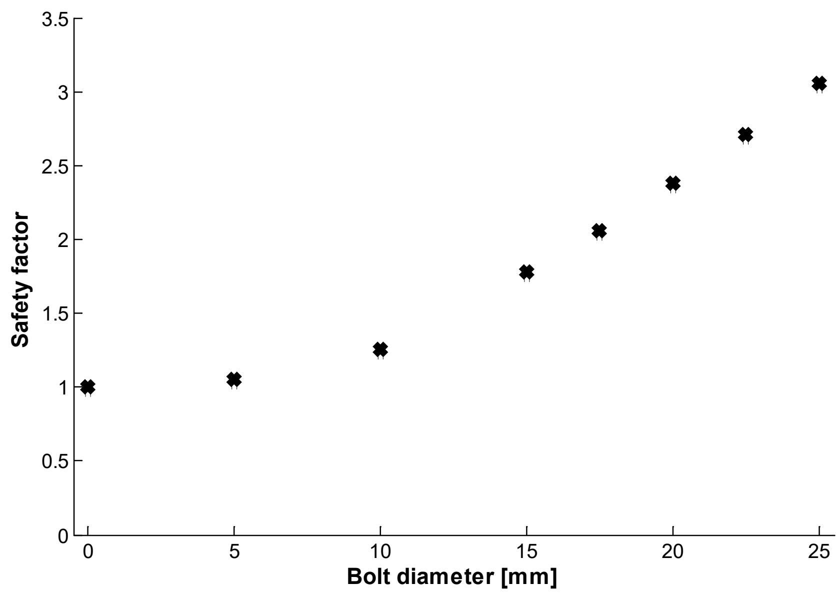


Figure 8-9 Safety factor for different bolt diameter.

The different bolt diameters and corresponding safety factors are then coupled to a construction age by assuming a constant degradation rate.

$$Construction\ age = \frac{d_{initial} - d_{model}}{2 * degradation\ rate} \tag{8.1}$$

where $d_{initial}$ is the initial bolt diameter, $d_{model}$is the used bolt diameter for the current model and $Construction\ age$ is the dam's age in years. The distribution for the $degradation\ rate$ proposed in Chapter 2 is used whit degradation with a lognormal distribution with mean value $\mu = 30\ \mu m/year$ and standard deviation $\sigma = 10\ \mu m/year$. The average corrosion after 100 years is 3 mm and thereby for a bolt with the initial diameter of 25 mm, the diameter after 100 years is 19 mm.

In Figure 8-10 the decreasing safety factor with construction age is plotted. An expected interval is plotted where the limits are the 1st and 10th percentile, corresponding to a degradation rate of 18.8 $\mu m/year$ and 43.2 $\mu m/year$. At a certain age, the dam's safety factor lies within the grey area with a probability of 80 %. For a dam subjected to the mean degradation of the bolts, the safety factor will decreases from Case 3a to Case 2 after 280 years. For this case study, the RIDAS stress limit corresponds to the average structural condition for a dam at the age of 275 years.

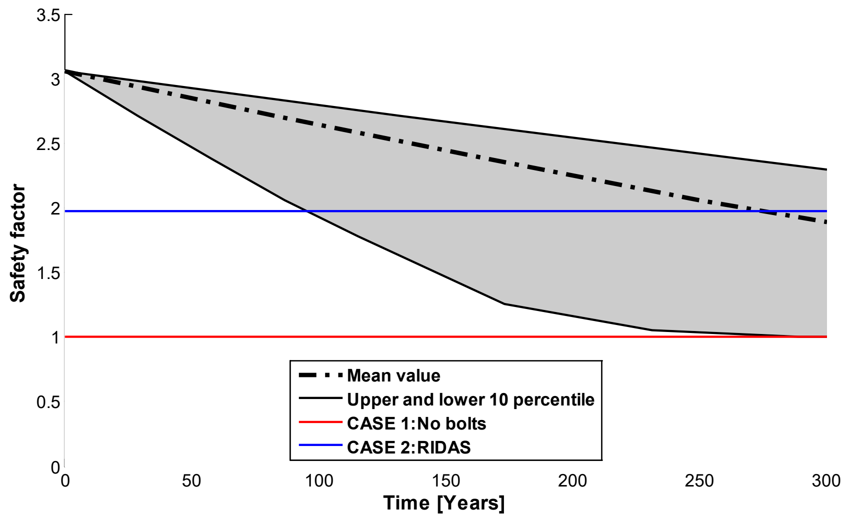


Figure 8-10 Decreasing safety factor for construction age 0-350 years.

In Figure 8-11, the same results as presented in Figure 8-10 is shown but zoomed for the first 100 years. If a corrosion rate corresponding to the 10th percentile is considered, it takes just under 100 years to reach the safety factor of Case 2. At the age of 100 years, the safety factor with respect to the degradation of the bolts, are with 90% probability higher than the safety factor obtained by the use of RIDAS restrictions.

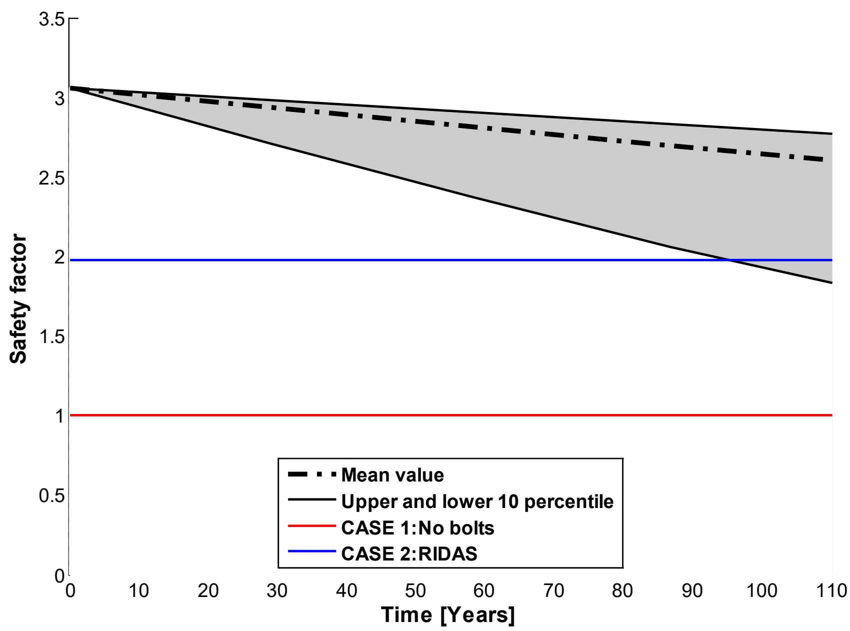


Figure 8-11 Decreasing safety factor for construction age 0-110 years

The numerical analyses show that the bolts have significant impact on the concrete dam stability, where the bolts increases the safety factor between 40 to 200 % for case 2 based on the RIDAS stress limit and between 20 to 170% for Case 3 were the bolts are defined with properties corresponding to KS40 steel.

9 Discussion and conclusions

In this study, the influence of degradation on the strength of rock bolts has been studied. Methods to determine the degradation rate of rock bolts is presented together with a summary of conclusions and gained experience from previous observations and status estimations of rock bolts. From the presented methods, a methodology to account for degradation mechanisms in evaluation of dam safety is presented.

In field tests of 50 year old bolts at a Swedish dam, the extent of corrosion was visually inspected and destructive pull-out tests were performed to determine the remaining strength of these bolts. A case study is presented where analytical, probabilistic and finite element analyses were performed to assess the influence from rock bolts and possible of degradation on the dam safety.

9.1 THE INFLUENCE OF DEGRADATION

Based on the information found in the literature, degradation of rock and grout is not considered to be a major issue. For the grout, most of the possible degradation mechanism will not have a negative influence of the load capacity of a rock bolt. The extent of available research is however rather limited regarding degradation of grout and more research is needed in this field. For instance, it is expected that corrosion may influence the bond between the bolt and the grout. However, within the scope of this project it has not been possible to find sufficient information regarding this to include this effect in the analyses. The results presented in Section 6.3 showed however that the load capacity regarding the bond between bolt and grout is typically significantly higher than the steel failure. In addition, no information regarding degradation of rock could be found in the literature which could serve as input to this project.

The literature regarding degradation due to corrosion, is however much more extensive and a suitable method to estimate the corrosion rate of rock bolts is presented. The approach is based on estimation of the corrosion rate from the chemical composition of the water. Previous implementations of this approach show that the estimated corrosion rate gives roughly the same results as the observed values.

The studies performed on the chemical content of Swedish rivers are rather limited in this report. However, the extensive analyses made by Hellgren et al. (2017), consisting of 30 000 samples from 2003 stations in Swedish rivers show that the corrosion rate is even lower than what has been considered in this report.

Based on available information, our conclusion is that grouted rock bolts under dams are a robust system and that the degradation is expected to be small. Previous studies show that properly installed rock bolts have approximately the same strength after many years of service as when installed. This conclusion is based on the assumption that the rock bolt is installed correctly and that it is not exposed to road salts.

9.2 DESIGN AND FAILURE MODES OF ROCK BOLTS

In the report, a methodology is presented where all possible failure modes of rock bolts are considered and where the contribution of the rock bolts to the dam stability is based on the failure mode with the lowest strength.

Normally, in design of rock bolts these are assumed to either carry the load as a normal force or due to the dowel effect, i.e. shear forces. Depending on which of these assumptions is used, different results may be obtained. As seen in Section 6.4, even though a significant difference in results between the two approaches can be found, it is mainly for dams close to the stability limit where this choice may have a large impact. It could lead to that the same dam satisfies the stability criterion or not, depending on which approach that is used. In addition, it is not possible to say that one of the two approaches is more conservative than the other, since it depends on the studied case.

The numerical analyses showed, as expected, that the rock bolts are subjected to both shear and tensile forces at the same time. The case presented in Chapter 8 showed that the shear force was constantly higher than the tensile forces and that the shear forces was about 10 % of the total shear resistance for normal loads. This depends of course on the stiffness of the rock and the grout, where weaker materials will result in that the bolt is more likely to transfer the loads as normal forces. The case study is based on a horizontal sliding surface, which is normally considered in design calculations. However, in reality the contact surface between the concrete dam and the rock is irregular. For a case with irregular sliding surface, it is likely that the bolts are subjected to higher normal forces compared to a smooth surface due to the dilatation that occurs during shear displacements of the irregular surface.

Regardless of this, it is apparent that the failure criterion of the steel failure in rock bolts is of importance and that analytical calculations should be based on that normal and shear forces act simultaneously in the rock bolts, as described in Section 2.1.5.

The numerical analyses in Chapter 8 also showed that a more realistic failure mechanism is based on the combination of an overturning failure and a sliding failure. In the analyses, deformations starts as for overturning failure resulting in that parts of the dam (on the upstream side) loses its contact with the rock. Thereby, the shear forces have to be transmitted over a smaller area which initiates the sliding failure.

9.3 SUGGESTIONS FOR DEVELOPMENT OF CURRENT DESIGN METHODS

The analyses presented in this report showed that rock bolts may have significant impact on the resistance of concrete dams, especially for dams of medium and low height. For high dams, the influence of rock bolts is only marginal as showed by Berzell (2014).

According to the Swedish guidelines for dam safety, rock bolts should not be considered in stability analyses, except for dams in lower consequence classes and/or low dams that satisfy certain criteria. There are most likely many dams that have been strengthened due to not satisfying current stability criteria even though they would be considered safe if the capacity of the rock bolts were considered.

In addition, for those cases with low consequence dams where bolts may be taken into account in the stability analyses the strength is limited to 140 MPa. The reason for this reduction is unclear. Some possible reasons may be to consider loss of strength due to degradation, due to uncertainties regarding failure mode of the steel bolt, due to uncertainties in the strength of the rock or presence of rock fractures, etc. It should be noted that similar reduction is also made in Norway where 180 MPa is the maximum allowed stress in passive rock bolts (NVE, 2005). In NVE (2005), the reason for this reduction appears to be related to uncertainties in the rock cone failure.

It is our opinion that it is preferable that the safety factor is calculated with the contribution of the rock bolts included. The numerical analyses showed that the influence from the assumed strength of the rock bolts (140 MPa or 370 MPa) did not have as significant influence as if they are considered or not. It is therefore in the authors' opinion preferable that rock bolts may be considered in stability analyses for all dams. As the bolts requires displacement before activation, the criterion that the dam should be stable (Sf>1) without bolts for normal load casers should be kept. The current approach with reduced strength may still be applied.

Thereby, the calculation would give more reasonable indications if the dam needs strengthening or not. One could argue that this would result in lowering the requirements on the dam and thereby its safety. This could be prevented if for instance higher requirements (values) of the factor of safety are chosen.

In addition to this, the probabilistic analyses in Chapter 7 showed that pure overturning failure is extremely unlikely. The probability of failure is so low that it cannot be considered as a relevant failure mode. This has also been shown in the investigations made by Westberg and Johansson (2016). An alternative failure mode should maybe therefore be defined instead of an overturning failure. This could for instance be that tensile forces should be prevented in the upstream toe for loads in the serviceability state.

In a future update of the design code, some requirements regarding the anchoring of the rock bolts should be added. Today, RIDAS does not provide any guidance regarding maximum strength of the grout-rock or grout-steel interface or how the capacity of the rock cone failure should be determined. In future design codes, the strength reduction via a stress limitation should be replaced with partial factors based on Eurocode.

9.4 FURTHER DEVELOPMENT OF TEST RIG FOR PULL-OUT TEST

The conclusions presented above are based on a limited amount of field investigations. Ekström et.al (2013) called for joint initiative where every opportunity given should be captured to gather experiences about rock bolts. Four years later, the tests and condition assessments at Lima Hydropower presented in this report is the only new addition. We encourages dam owners to at least notice the status of old rock bolts as they become available during for example decommission or reconstruction.

In the performed field tests of 50 years old rock bolts, a previously developed test rig was used but with some modifications made to it. The development of the test rig included measurement of the displacement and logging of the pull-out force and displacement. Some restrictions regarding the used test method was known in advance, such as it is not possible to capture a rock cone failure and that a length of at least 50 cm of the exposed rock bolt is needed.

The results presented in Section 4.3 and Chapter 5 showed that the test rig also may influence the obtained load capacity if the failure occurs in the grout. It is likely that the load transferring surface at the rock interface is too small and placed too close to the bar. This results in that the bolt is clamped and therefore increasing the capacity for the failure modes including bond failure between grout and rock or grout and steel. For mounting of the test equipment, the rock surface around the bolts must be smooth. This resulted in that the rock had to be smoothed and thus damaged even before the testing begun. In future tests, the test equipment should be modified further, preferably by adding a tripod with adjustable legs that transfers the load to the rock at a certain distance from the bolt. This distance may however be significant if the rock cone initiates from the end of the rock bolt and with a typical embedment length of 3 m the length of the legs in the tripod needs to be more than 2.5 m away from the rock bolt. A better approach may thereby be to use an excavating machine to perform the pull-out test as in the experiments performed by Lepine (2012), see Section 3.2.

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LOAD CAPACITY OF GROUTED ROCK BOLTS DUE TO DEGRADATION

Här har bärförmågan hos slaka bergsförankringar, det vill säga bergbultar, studerats med hänsyn hur de påverkas av nedbrytning. Ett antal metodiker har tagits fram både för sannolikhetsbaserade analyser och för numerisk FE modellering.

En liten betonglamelldamm har valts som beräkningsexempel för att analysera inverkan av bergbultar med hänsyn till nedbrytning. Här finns också en sammanställning av samtliga kända fältprovningar av bergbultar samt viktiga resultat och observationer som framkom från dessa försök.

Resultaten visar att i samtliga tillgängliga fältprovningar redovisas endast lastkapaciteten, vilket betyder att deformationsförloppet vid provdragning saknas. Detta är ett mycket viktigt bidrag för att kunna genomföra de numeriska och analytiska beräkningarna. Därför har även egna fältprovningar genomförts i samband med rivning av en ledmur vid kraftverket i Lima.