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Failure Mechanisms of Fibre Reinforced Shotcrete: Numerical Simulations Considering Local Variations in Thickness and Bond Strength



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ABSTRACT

Fibre-reinforced shotcrete is the most common support method for hard rock tunnels in the Nordic countries. The design of shotcrete is often based on empirical methods or simplified analytical equations, which neglect variations in mechanical properties and shotcrete thickness. Data collected from the field shows that significant variations in shotcrete thickness and bond strength should be expected during tunnel construction. However, how this affects the structural behaviour and capacity of the shotcrete lining is unknown. Moreover, the design philosophy for shotcrete assumes that the primary failure modes of shotcrete, i.e. bond and flexural failure, can be treated separately. This was derived based on observations of experiments in a laboratory environment.

Therefore, the focus of a finalized doctoral project was to develop a numerical framework to simulate the structural behaviour of fibre-reinforced shotcrete in interaction with hard rock and rock bolts. The effect of variations in shotcrete thickness and bond strength was studied through numerical simulations to increase the understanding of its effect on the failure load of the lining. The results indicate that the most important parameter is the mean value of the shotcrete thickness and bond strength around a narrow perimeter of the block.'

Key words: fibre-reinforcement, shotcrete, rock support, failure modes and bond strength

1. INTRODUCTION

Tunnels in hard and jointed rock are typically supported with a combination of fibre-reinforced shotcrete (sprayed concrete) and rock bolts. Shotcrete is sprayed pneumatically under high air pressure directly on the rock surface. The air pressure, in combination with the use of set accelerators and the cement paste ability to bond, allows the shotcrete to stick to the rock surface. This creates a composite structure of shotcrete and rock, often complemented with rock bolts. Depending on the quality of the rock mass, the span of the tunnel and the expected behaviour of the rock mass, bolts are either placed systematically or as spot bolts. The design philosophy for shotcrete with spot bolts is that shotcrete should be able to distribute the load from small loose blocks to the surrounding rock mass through its bond to the rock. In areas with systematically placed rock bolts, the shotcrete should be able to support the load from a potential loose block that fits between the group of four rock bolts. Here, it is assumed that the load is carried either by bond, shear or flexural strength of the Fibre-Reinforced Shotcrete (FRS).

The design of a rock tunnel, and its support, is complicated due to the many uncertainties surrounding the rock mass. Before the tunnel is excavated, the mechanical properties of the rock and the expected deformation caused by the excavation of the tunnel can only be estimated. The basis for such estimation is data from expensive field testing and knowledge from previous projects in the same area. Thus the data are in many cases limited. After excavation, a geotechnical investigation of the exposed rock mass could increase the level of knowledge regarding its mechanical properties. However, the rock mass quality estimation is still subjective, and the inspection is limited to joint patterns visible on the rock surface. Moreover, the rock support installation must occur rapidly not to slow down the excavation. For these reasons, the rock support design is often based on a rock mass classification system, such as the Q-system by Barton et al. [1] or the Rock Mass Rating system by Bieniwanski [2]. With these classification systems, a geotechnical evaluation and estimation of the rock mass quality are performed based on a fixed number of parameters. This yields a suggested support system for the rock tunnel, e.g., the shotcrete's thickness and the length and distance between rock bolts. In the Q-system, a required energy absorption for the shotcrete is also suggested. In design today, these classification systems are often used to get a preliminary design and to ensure the overall stability of the tunnel. Thereafter, a common approach is to calculate the structural capacity of the shotcrete lining to ensure that it is sufficient to carry the load from an estimated loose block. Here, simplified analytical equations are commonly used based on a uniform shotcrete thickness and bond strength. However, in-situ investigations by Ansell [3] and Malmgren et al. [4] have shown that significant variations in both parameters are expected in an actual tunnel.

The focus of a recently finalized doctoral project at KTH Royal Institute of Technology was, therefore, to develop a numerical framework capable of simulating the structural behaviour and

failure of a FRS in interaction with hard rock and rock bolts. Moreover, this framework should be used to study how variations in shotcrete thickness and bond strength affect the structural capacity of the lining. The results from this project are reported by Sjölander [5, 6], and in this paper, the most important results will be summarized.

2. IN-SITU DISTRIBUTIONS OF SHOTCRETE PARAMETERS

Bjureland et al. [7] present a unique set of data collected from the quality control program during the construction of the City Tunnel in Stockholm. This included shotcrete thickness from more than 6000 drilled holes together with more than 350 tests on bond strength and almost 700 results concerning compressive strength and residual flexural strength.

2.1 Mechanical strength of shotcrete

Based on the project requirements, the compressive strength f_{ck} and residual flexural strength f_{fl} at 2 mm vertical deflection should qualify for C32/40 and S3D2, i.e. a f_{ck} of 40 MPa and f_{fl} of 3.0 MPa. For the compressive strength, the mean value was 59 MPa but the characteristic, taken as the 5-percentile value of the test series was 45 MPa. Thus, the f_{ck} for the field data was 12.5 % higher than required. The residual flexural strength was evaluated based on standard four-point bending tests [8] which requires a mean value for one test series with three beams that is higher than the required strength. Also, no individual samples within the series should be lower than 80 % of the required value, i.e. the mean value should be 3.0 MPa and the lowest accepted value for an individual test is 2.4 MPa. Based on the tested samples, the mean value was 3.8 MPa and the standard deviation 0.8 MPa. Statistically, this means that approximately 95 % of all tested beams should have f_{fl} greater than 2.2 MPa. Thus, it can be expected that a large majority of test series on beams will fulfil the minimum required f_{fl} of 2.4 MPa and the mean value of 3.0 MPa.

2.2 Shotcrete thickess

The thickness data were sorted and divided into three groups based on the required thickness, which was 50, 75 and 100 mm, respectively. Thereafter, the data were analysed for each group separately. The best fit for the data were a lognormal distribution, and the fitted distribution for a case with a required thickness of 50 mm is shown to the left in Figure 1. The cumulative distribution function for shotcrete with a required thickness of 50, 75 and 100 mm, respectively, is shown to the right. Here, a few interesting things can be noted. First, the probability that the target thickness is not reached is between 13 % and 32 % for the presented cases. Thus, several sections with insufficient thickness are likely to exist. Moreover, the probability that the target thickness is not reached increases with an increased required thickness. According to the standard [9], the shotcrete thickness is verified by drilling five closely spaced holes and measuring the thickness. The mean value of the test must fulfil the required thickness, while the minimum thickness of individual holes must be at least 80 % of the target value. It was shown [6] that between 4 % and 13 % of the sections have insufficient thickness. Moreover, the large variations in sprayed thickness also highlight that the mean value of the thickness, regardless of required thickness, is approximately 25 mm more than the requirements. For more results, see Bjureland et al. [7].

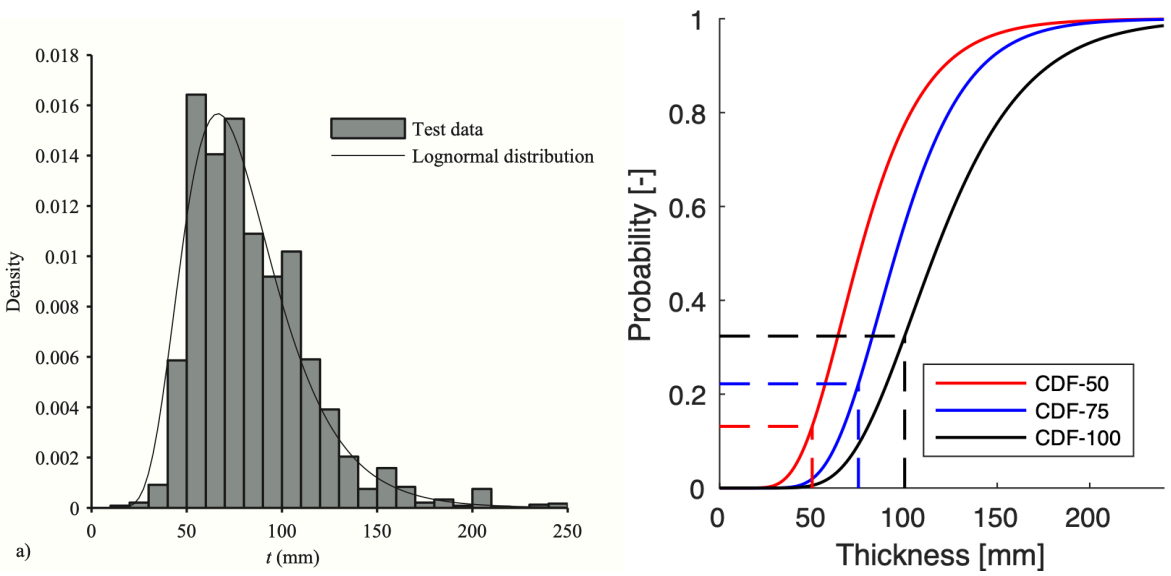


Figure 1 - Probability density functions for shotcrete thickness with a target thickness of 50 mm (left) and the cumulative distribution function for shotcrete thickness with a target of 50, 75 and 100 mm (right). From Bjureland et al. [7] and Sjölander [6].

2.3 Bond strength

The results for the bond strength are shown in Figure 2. To the left, all results are divided into subsets depending on where the failure occurred. For the design, the location for the failure is not important. However, the reason to divide the results in the subgroup is to highlight the importance of proper preparation of the rock surface. Malmgren et al. [4] investigated the influence of water-jetting with respect to bond failure. It was shown that water-jetting significantly improved the bond strength. One of the possible explanations to this is that low quality rock is being teared down during the water-jetting. Thus, it does not necessarily improve the actual bond strength between the shotcrete and rock but it removes rock with low strength that otherwise would fail at stress level under the actual bond strength and in that way reduce the bond strength. Thus, removing low quality rock before spraying shotcrete is vital and will significantly increase the bond strength between the shotcrete and rock. As discussed in [7], the bond strength between shotcrete and rock is sometimes disregarded in the design when the Rock Mass Rating (RMR) is less than 50. To the right in Figure 2, the bond strength is plotted against the RMR value in subsets for granite and gneiss. Clearly, no correlation can be identified between RMR and the bond strength.

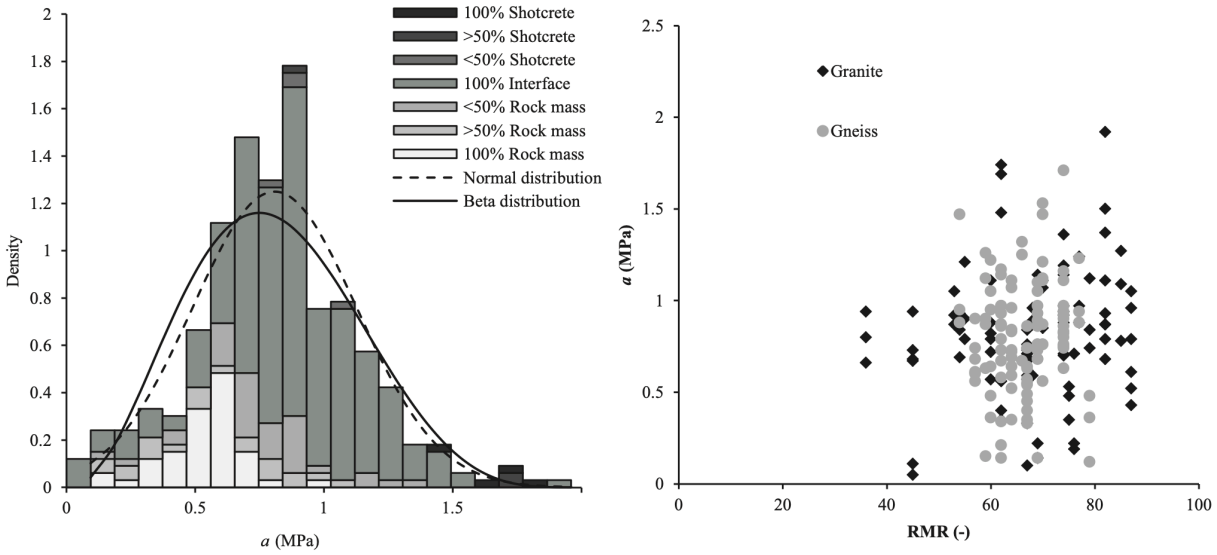


Figure 2 - Bond strength between shotcrete and rock divided into subsets depending on failure surface (left). Correlation between bond strength and Rock Mass Rating (RMR) to the right. Bond strength is here denoted a. From Bjureland et al. [7].

3. FAILURE MODES FOR A BOLT-ANCHORED SHOTCRETE LINING

Most of the theoretical knowledge on the structural capacity and failure modes for a bolt-anchored shotcrete lining has been derived from two large-scale experimental campaigns by Fernandez-Delgado et al. [10] and Holmgren [11] during the 1970s. Based on these experiments, analytical equations and design methodologies for shotcrete linings have been presented by e.g. Barrett and McCreath [12].

3.1 Experimental set-up

The set-ups for the experiments [10, 11] are shown in Figure 3. In the test by Holmgren [11], the connection between the rock bolt and shotcrete, the size and thickness of the washer plates, as well as the reinforcement content in the shotcrete varied. In the tests by Fernandez-Delgado [10], the shotcrete thickness and the distance between the edge of the block and the rock varied. From the experiments, it was concluded that tensile bond failure between shotcrete and rock occurs first. Then, the bond failure propagates to the location of the rock bolts, where it is arrested. At this point, cracks form in the shotcrete at the location of the bolts and joints between the moving blocks. The ultimate failure was reached once the cracks had propagated along the entire width and through most of the shotcrete thickness. Furthermore, it was concluded that shear failure of the shotcrete lining is rare if the thickness is more than 50 mm [11]. Based on these observations, the design of a shotcrete lining is today based on the fact that the governing failure modes, i.e. bond and flexural, can be treated separately. Thus, the load from the block is carried either through the bond or flexural strength. However, this has not been verified with numerical simulations.

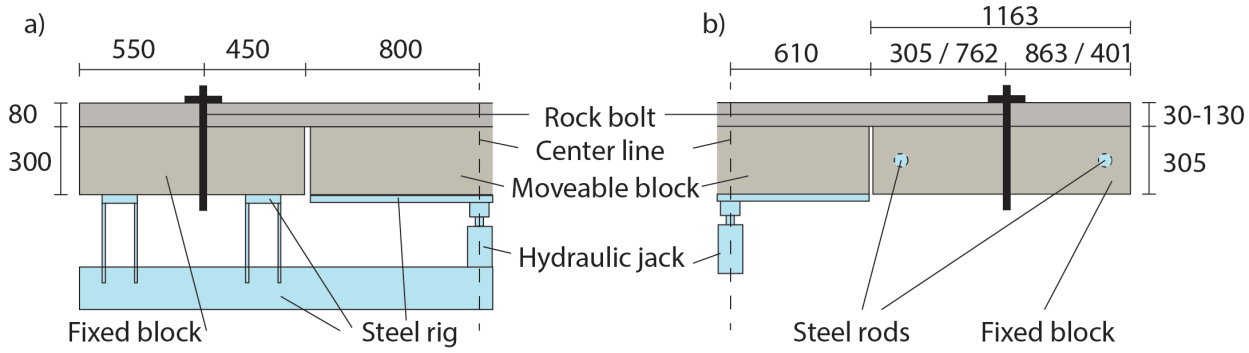


Figure 3 - Experimental set-up for the study of the structural behaviour of bolt-anchored and thin shotcrete linings subjected to the load from a loose block. From Holmgren [11] (left) and from Fernandez-Delgado et al. [10] (right).

3.2 Numerical modelling

To increase the understanding of the failure of a bolt-anchored shotcrete lining and to numerically investigate if any interaction between the different failure mechanisms exists, numerical models for the individual failure mechanisms are presented by Sjölander et al. [13]. Moreover, the capability for each failure mechanism was verified against experiments from the literature. Then, the structural behaviour and failure of the complete model were verified against experimental results by Holmgren [11] and used to study the contribution from each failure mechanism, i.e. bond failure, cracking of FRS and location of rock bolts. The result from this study is presented by Sjölander et al. [14].

The numerical model to describe the non-linear behaviour of FRS and the interaction between shotcrete and rock was based on damage mechanics. An isotropic damage model was used which implicates that damage is equal in all direction. Thus, damage can be described with a scalar, here denoted ω. For a simple 1D case, the relationship between stress σ and strain ε in the shotcrete can be described as:

$$\sigma = E\varepsilon(1-\omega) \tag{1}$$

Here, E is the Young's modulus for the undamaged material and ω ranges from zero for an undamaged material and going towards one for a fully damaged material. In the literature, various methods to numerically account for the effect of fibres exist, from the simplified hinge models by Olesen [15] to more advanced models, including discrete modelling of fibres and their orientation by Soetens and Matthys [16]. Here, the fibres are included in a smeared way, i.e. individual fibres are not modelled, and their effect is considered by increasing the fracture energy of the concrete material. Commonly, unreinforced concrete is modelled using an exponential softening curve, while a bi-linear or tri-linear function often is used to describe the behaviour of fibres. Sjölander et al. [13] combined an exponential curve to describe the plain concrete and a bi-linear curve for the increased fracture energy from the fibres. With this approach, the initial response is governed by the fracture energy of the plain concrete. Thereafter, the fibres are activated at a certain crack width and from this point onwards, a bi-linear curve determines the post-cracking behaviour. The damage model is schematically shown to the left in Figure 4. The interface between shotcrete and rock was also modelled using damage mechanics. The interface was modelled using the penalty model with zero thickness elements as suggested by Camanho and Davila [17]. The basic principle is that the nodes along the interface between two materials are connected using a series of stiff springs. The Mode-I stress σ_I, i.e. tensile stress, is then described as:

$$\sigma_{\mathrm{I}} = K_{\mathrm{I}} u_{\mathrm{I}} (1 - \omega) \tag{2}$$

In Eq. 2, u_{I} is the separation between the two surfaces perpendicular to the surface plane and K_{I} is the corresponding stiffness of the spring in that direction. The model can account for mixed-mode failure of the interface, and a complete definition of the interface should include the spring stiffness and fracture energy for Mode-I and Mode-II as well as mixed-mode failure. However, experimental data with enough data points to describe and simulate the failure of the interface between shotcrete and rock are rare. Sjölander et al. [14a] used experimental data presented by Dong et al. [18] and suggested that an exponential softening curve with a fracture energy much lower than unreinforced concrete was suitable to model Mode-I failure of the interface. A comparison between experimental results [18] and numerical simulations [14] is shown to the right in Figure 4. For a complete description of the damage models for FRS and the shotcrete-rock interface and a suggested non-linear material model to simulate the pull-out of bolts, see [12, 14].

3.3 Numerical simulations of failure mechanism

The experimental set-up from Holmgren [11], shown in Figure 3, was used to investigate the failure mechanism of a bolt-anchored shotcrete lining. First, the numerical model was verified against the experimental results. Thereafter, the influence of each failure mode was investigated numerically. The results are shown in Figure 5, and the result when all failure modes are included is plotted with a solid line. Then, the bond between shotcrete and rock was removed entirely. This resulted in the loss of the initial stiff response of the lining. Hence, the contribution from the bond is the first peak load and high initial stiffness. The lining with no bond has a much softer response and reaches an approximate load level of 30 % of the capacity compared to a model with complete bond. In these simulations, the bond strength was 1 MPa. For the third simulation, the shotcrete was simulated with a linear elastic material model to investigate how the reinforcement of the shotcrete contributes to the load-carrying capacity. Here, the response initially follows the original model with a stiff response up to a first failure load. Thereafter, softening occurs in both models, which indicates that this part of the response is due to failure propagation at the interface between shotcrete and rock. Then, when the load has dropped to approximately 50 % of the peak load, the

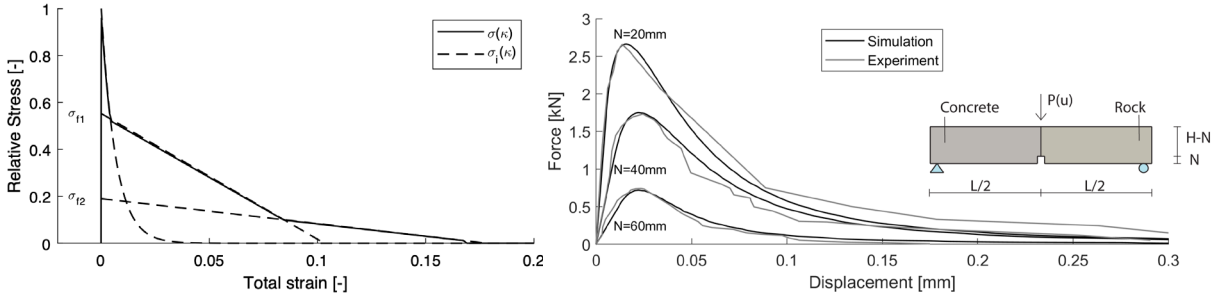


Figure 4 - Schematic view of the damage model for fibre reinforced shotcrete from Sjölander [6] (left). Comparison of force and displacement between experiments from Dong et al.[18] and numerical simulations from Sjölander et al. [14] (right).

results from the latter two simulations diverge. This indicates the point when cracks occur, and the post-cracking performance of the shotcrete will govern the structural behaviour. Thereby, the result from these simulations confirms that the failure modes should be treated separately in the design of shotcrete. Moreover, a design based on the bond strength yields a higher structural capacity and potentially results in a more economical design. However, as shown in Figure 5, bond failure is quasi-brittle and occurs at small displacements. Moreover, the failure of the bond will significantly reduce the structural capacity of the lining. In a tunnel, the bond will restrain the movement of the shotcrete. Hence, bond failure could occur during the hydration process, e.g. due to restrained shrinkage. If the lining should be designed based on bond strength, the designer must ensure that the bond remains intact. Therefore, a design based on the residual flexural strength of the FRS will always be on the conservative side.

4. LOCAL VARIATIONS IN THICKNESS AND BOND STRENGTH

Based on the experiments, it was concluded that the load from a loose block on the shotcrete lining is transferred over a narrow band between shotcrete and rock. Fernandez-Delgado et al. [10] suggested that the stress is triangularly distributed, and the width of the band is between 50 to 100 mm, while Holmgren [11] suggested a rectangular stress distribution with a width of 30 mm. Today, the Swedish guidelines for the design of tunnels [19] suggest that the width of the stress transfer band is linearly correlated to the shotcrete thickness, which is based on results by [11, 20]. This is reasonable since a larger part of the lining should be activated during failure when the thickness increases. However, as shown in [7], local variations in shotcrete thickness and bond strength should be expected for an in-situ lining. How these affect the structural capacity is not well studied, and a numerical study was therefore conducted. The complete results from this study are presented by Sjölander et al. [21] and summarized below.

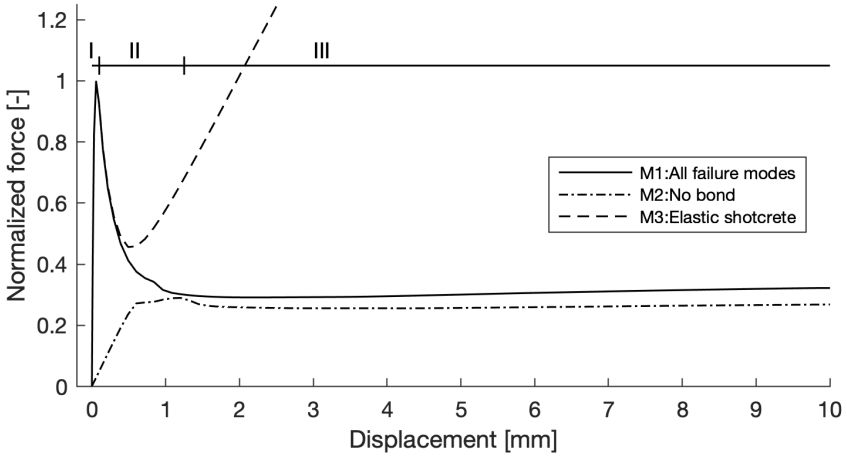


Figure 5 - Results from numerical simulations to study the influence of different failure mechanisms.

4.1 Numerical model

A numerical model based on the set-up from Fernandez-Delgado et al. [10] was used. The purpose of the model was to investigate how local variations in shotcrete thickness and bond strength affect the maximum load capacity for the shotcrete. Since the results from [14] showed that the failure modes can be treated separately and that bond failure occurs first, the rock bolts were not included in the model, and the shotcrete was modelled as linear elastic to simplify the model. The model contained three separate rock blocks covered with a layer of shotcrete. In the first model, the bond strength was kept constant at 1.0 MPa, and the shotcrete thickness varied. In the second, the shotcrete thickness was kept constant at 50 mm, and the bond strength varied. In the simulations, the centre block was pushed through the lining. Here, failure of the lining is defined as the moment when the peak load is reached. To compare the results between the models, a bond failure area (BFA) was defined for each model separately. First, the deformation when the failure load was reached was determined. Then, the Mode-1 bond stress was plotted along a centre line of the model for this specific deformation. At this point, both tensile and compressive stress exists along the interface, and the BFA was determined to be the part subjected to tensile stresses. This procedure is visualized in Figure 6. For both studies, the variations in shotcrete thickness and bond strength were plotted for the BFA, and the load capacity was then compared and plotted against the minimum, maximum and mean value of either shotcrete thickness or bond strength.

4.2 Local variations in shotcrete bond strength

As shown by Bjureland et al. [7], large variations in the bond strength σ_b between shotcrete and rock exist. Since the spatial correlation of the bond strength is not the known, the variations in σ_b along the shotcrete-rock interface plane, here defined as the xy-plane, was modelled using a series of elementary waves as defined in Eqs. (3-4)

$$\sigma_b(x,y) = \sigma_{b,mean} + \sigma_{b,random} \tag{3}$$

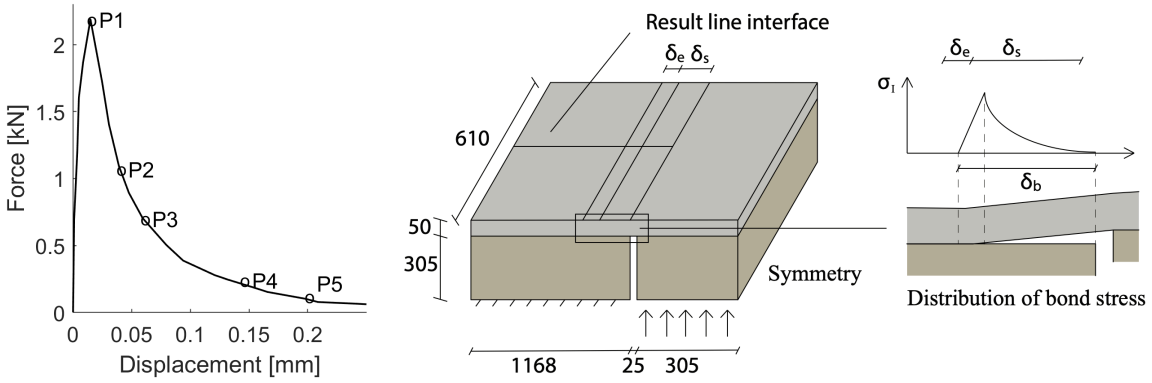


Figure 6 - Force displacement plot to determine peak load (P1) and displacement at failure (left), location of results line where interface stress is plotted (centre) and stress along interface for the displacement at failure (right). From Sjölander et al. [21].

$$\sigma_{\mathrm{b,random}}(x,y) = \sum_{m=-M}^{M} \sum_{n=-N}^{N} a(m,n)\cos(2\pi(mx+ny)+\phi(m,n) \tag{4}$$

This modelling approach was used by Sjödin [22] and enables the user to control the variations by specifying the number of cosine waves, M and N, the amplitude of each wave, $a(m,n)$ as well as the phase angle between each wave defined by $\phi(m,n)$. For a complete description see [22] and for a more thorough description of its implementation to study variations in bond strength see [21]. With this approach, the variations in bond strength will vary smoothly over the surface and clusters with high and low bond strength values will exist. According to [23], this is a likely scenario since the bond strength depends on factors such as cleanliness and the presence of water on the rock surface. This approach generated a variation in shotcrete strength between 0 and 1.7 MPa. This is a reasonable in-situ variation and could be compared to the mean value and standard deviation in [7], which was 0.81 and 0.32 MPa. A total of 20 cases with local variations in bond strength were studied together, with three cases in which the bond strength was uniform. An example of variations in bond strength is plotted for the model R1-N2 to the left in Figure 7. Here, the bond strength along the interface varies between 0.2 and 1.6 MPa and the black solid lines indicate the joints for the loose block that is being pushed through the lining. In all simulations, the centric block was pushed through the lining and the maximum force required was noted. Thereafter, the variations in bond strength in the bond failure area, i.e. the area in which the interface is subjected to tensile stresses, was investigated. To the right in Figure 7, the maximum force, i.e. the failure load, for all the cases is plotted against the mean value of the bond strength in the bond failure area. Clearly, a strong linear correlation exists, and the bond strength's mean value significantly impacts the failure load. By comparing the results between samples with similar mean bond strength, the results highlighted that a small area with low or no bond strength could exist without having any significant effect on the failure load.

4.3 Local variations in shotcrete thickness

LiDAR data were used to model this realistically and with high accuracy to study the influence on local variations in thickness. The data contained a scanning of the rock surface after excavation and of the shotcrete surface after spraying. The geometry for the model was created by taking a

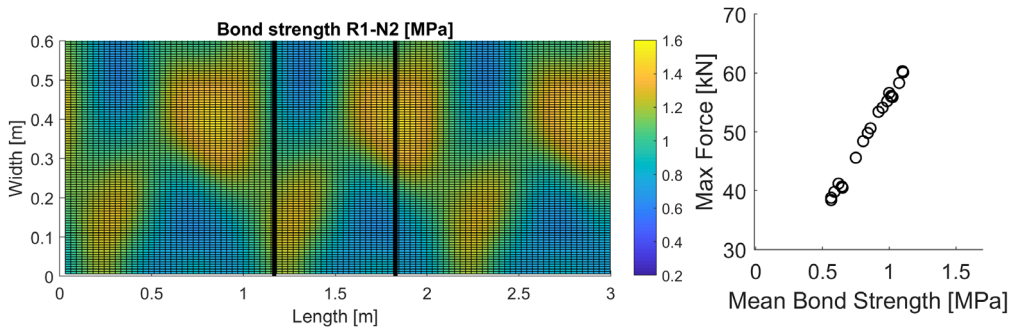


Figure 7 - Example of variations in shotcrete bond strength for one of the studied cases and the correlation between failure load and the mean shotcrete thicknes in the bond failure area. From Sjölander et al. [21].

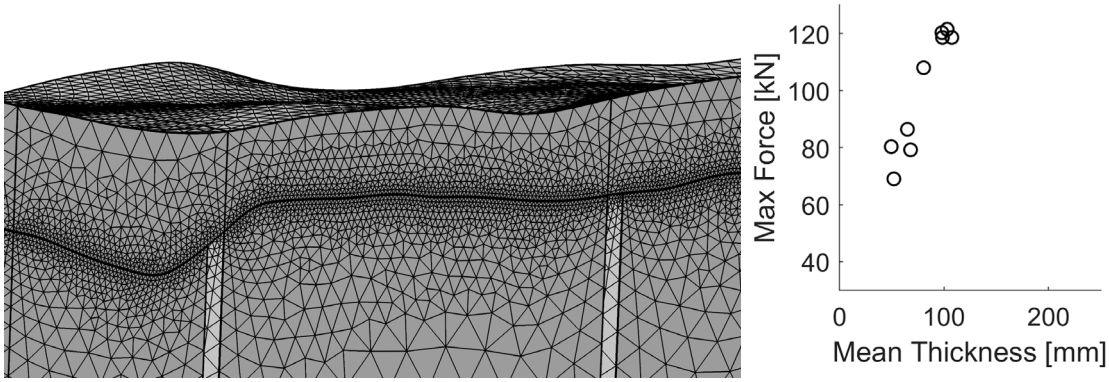


Figure 8 - Example of meshed model with local variations in shotcrete thickness and correlation between mean shotcrete thickness in the bond failure area and the failure load. From Sjölander et al. [21].

section from the tunnel roof with the same length and width as the model used to study local variations in bond strength. A joint was created in the rock to enable the centre block to be pushed out by removing a part of the rock mass. The model was collected from a recently constructed road tunnel in Sweden, and ten models were created from different parts of the tunnel. The simulations was performed in the same way as the model with variations in bond strength, i.e. a centric block was pushed through the lining and the maximum force and variations in shotcrete thickness in the bond failure area was investigated. To the left in Figure 8, an example of a meshed model of the three rock blocks and the shotcrete is shown to highlight the variations in thickness along the interface. To the right in Figure 8, the maximum force, i.e. the failure load, is plotted against the mean shotcrete thickness in the bond failure area. When the shotcrete thickness inreased, the size of the bond failure area, i.e. the load transfer area, increased. As for the case with varying bond strength, there is a strong linear correlation between the mean shotcrete thickness and the failure load. As discussed in [21], one possible explanation for the strong correlation between failure load and the mean value of shotcrete bond and thickness is the combination of a stiff interface and a narrow area over which the bond stress is transferred. Thus, local weak sections may initiate the failure but will not control the failure progression. Due to the stiff interface, bond failure must progress more or less uniformly along each side of the perimeter. This means that local weak sections along one side of the blocks perimeter could be balanced by a section with higher bond strength. However, more cases should be studied to determine how the size and distribution of local weak zones could affect the failure load.

5. CONCLUSIONS AND FURTHER RESEARCH

In this paper, the results from a recently finalized doctoral project have been summarized. The key concepts from the numerical framework presented by Sjölander [6] were briefly presented together with in-situ distributions of shotcrete parameters. The failure modes of shotcrete and how local variations in shotcrete thickness and bond strength affect the structural capacity of fibre-reinforced shotcrete were also shown.

Based on the analysis of a large amount of field data for shotcrete thickness and bond strength, it was shown that there is a tendency to spray more shotcrete than required. The mean value of the shotcrete thickness was around 25 mm higher than the target thickness. However, the measured values and the suggested statistical distribution of the shotcrete thickness also suggest that some

sections will not reach the target thickness. Similar results were observed for the bond strength in which the target bond strength is reached in most of the sections, but local areas with no or insufficient bond strength also exist. Through numerical simulations, it was shown that the force from a loose block is transferred to the shotcrete over a narrow band. Thus, the failure load depends on the size of the band and the bond strength. Moreoever, a strong linear correlation exists between the mean value of the shotcrete thickness and bond strength and the maximum load that the shotcrete lining can carry through bonding. Based on the simulations, it was also shown that local weak sections, i.e. sections with a low thickness or bond strength, could exist around the perimeter of the loose block without significantly affecting the structural capacity. However, these conclusions are based on a limited number of numerical simulations and should be further studied. The failure mechanisms for a bolt-anchored and fibre-reinforced shotcrete lining were also studied with the developed numerical framework. In the design codes, it is common practice that the failure modes can be treated separately. This knowledge has been derived based on observation of experimental tests and has not been verified numerically. Here, it was shown that the failure mechanisms can be treated separately in the design and that a design based on the residual strength of the fibre-reinforced shotcrete is, in most cases, more conservative than a design based on the bond strength.

An interesting topic for further research is to put these results in perspective regarding in-situ quality control of shotcrete. Today, individual samples of shotcrete thickness and bond strength are allowed to be 80 % of the requirements as long as the mean value is sufficient. The importance of the mean value was in this project verified, but it also indicated that the minimum value has a low influence, and perhaps the minimum requirement could be reduced or even removed. However, this might also require more sophisticated methods for measuring the thickness. The quality control of shotcrete thickness today consists typically of five drilled holes for every 500 m^2 of shotcrete. With the large variations in shotcrete thickness, this method gives insufficient data to verify the actual thickness. A better and more reliable way forward is to use LiDAR data. Recently, it has become common practice to scan the rock and shotcrete surfaces. However, the data are not used for quality control of the shotcrete thickness. Therefore, an interesting topic for further research is to investigate how this type of data should be used as a tool for quality control. This should include further numerical studies to understand how local weak sections could be distributed around a perimeter of a block, as well as statistical considerations on how many weak sections that could be allowed.

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