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STIFTELSEN BERGTEKNISK FORSKNING
ROCK ENGINEERING RESEARCH FOUNDATION

ANALYSES OF SHOTCRETE STRESS STATES DUE TO VARYING LINING THICKNESS AND IRREGULAR ROCK SURFACES

Andreas Sjölander

BeFo Report 173

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Analyser av spänningstillstånd i sprutbetong med hänsyn till varierande tjocklek och bergets oregelbundna yta

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Preface

Rock support for tunnels in hard rock normally consists of unreinforced shotcrete or fibre reinforced shotcrete in combination with rock bolts. The design of such rock support is a complex problem where both the external load and the development of stresses in the structure depend on the interaction between shotcrete, rock and rock bolts. Rock support design is based on empirical relations, numerical simulations or in some cases, a combination of them. The problem in the design of the rock support is what loads shall be considered and how variations in important parameters such as thickness of the shotcrete and bond strength should be assessed. By increasing this knowledge, a more efficient use of shotcrete is possible.

This report presents the first part of a Ph.D. project with the long term goal of improving the design methods for shotcrete when used as rock support. A numerical model capable of describing varying thickness and the non-linear behaviour of shotcrete is presented. The model also considers bond failure between shotcrete and rock and the structural effects of drying shrinkage. Numerical examples are presented with focus on how the irregular rock surface and the varying shotcrete thickness affect the development of stresses and cracking of the lining. Two important load cases were studied: restrained movements due to shrinkage of the shotcrete and the gravity load from a single block.

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Stockholm, 2017

Per Tengborg

Förord

Tunnlar i hårt berg är vanligtvis förstärkta med oarmerad sprutbetong eller fiberarmerad sprutbetong i kombination med bergbultar. Dimensioneringen av en sådan bergförstärkning är ett komplicerat problem där både den externa lasten och spänningsuppbyggnaden i förstärkningen beror på interaktionen mellan sprutbetong, berg och bultar. Bergförstärkningars dimensionering baseras på empiri, numeriska analyser eller i vissa fall en kombination av dessa. Problemet med dimensioneringen är vilka laster som ska antas verka samt hur variationen av viktiga parametrar såsom tjocklek hos sprutbetongen och vidhäftningshållfastheten ska beaktas. Genom att öka denna kunskap kan sprutbetong användas mer effektivt.

Den här rapporten presenterar den första delen av ett doktorandprojekt med det långsiktiga målet att förbättra dimensioneringsmetoden för sprutbetong vid användning som bergsförstärkning. En numerisk modell som kan beskriva variationerna i tjocklek och det icke-linjära beteendet av sprutbetongen presenteras. Modellen beaktar även vidhäftningsbrott mellan sprutbetong och berg samt det strukturella beteendet orsakat av uttorkningskrympning. Numeriska exempel som fokuserar på hur den oregelbundna bergytan samt den varierande tjockleken av sprutbetong påverkar spänningsuppbyggnaden och sprickbildningen i förstärkningen presenteras. Två viktiga lastfall har studerats: tvångsrörelser i sprutbetongen orsakat av krympning samt egentyngden av ett enstaka block.

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Abstract

Shotcrete is sprayed concrete applied pneumatically under high pressure and was invented in the beginning of the 1900's. Soon after, attempts were made to use shotcrete to support rock tunnels. This new technique decreased the construction time and when steel fibres were introduced in the shotcrete during the 1970's, the heavy labour work of placing ordinary mesh reinforcement could be reduced. Since then, shotcrete has been the primary support method for tunnels, especially in hard rock where thin layers of shotcrete can be used as the only reinforcement.

Tunnels are normally excavated using the drill and blast method which creates a highly irregular rock surface on which shotcrete is applied. While spraying, the actual applied thickness is hard to determine and the final shotcrete lining will therefore have a varying thickness. Depending on in situ conditions, unreinforced or fibre reinforced shotcrete in combination with rock bolts can be used as rock support. The structural behaviour as well as the loads acting on the shotcrete lining depends on the interaction between the shotcrete, rock and rock bolts. There are several parameters influencing this interaction, e.g. bond strength, the stiffness of the rock and thickness of the shotcrete. All of these parameters are difficult to predict accurately which makes the structural design of the lining to a complex problem.

This thesis present the first part of a research project with the long-term goal to improve the understanding of the structural behaviour of the shotcrete lining. To achieve this, numerical modelling have been used to study the build up of stresses and cracking of shotcrete when subjected to restrained loading caused by e.g. temperature differences and drying shrinkage. The response in the lining when subjected to a gravity load from a block has also been studied. A numerical model for the analysis of shotcrete stresses is presented in which time-dependent material behaviour. Furthermore, the model is capable of describing the non-linear deformation behaviour of both plain and fibre reinforced shotcrete and uses presented in situ variations in thickness to more accurately account for the effects of expected variations in thickness. The thesis discuss and demonstrate the effect of important loads that acts on the shotcrete lining and how the irregular geometry of the rock surface in combination with the varying thickness of the shotcrete affect the development of stresses in the lining. It is also discussed how a full or partial bond failure affect the structural capacity of a shotcrete lining.

Sammanfattning

Sprutbetong är betong som appliceras pneumatiskt under högt tryckt, en metod utvecklad i början av 1900-talet. Kort därefter gjordes de första försöken att använda sprutbetong som bergförstärkning. Den här nya tekniken minskade produktionstiden och när stålfibrer introduceras under 1970-talet kunde det tunga arbetet med att placera armering minimeras. Sedan dess har sprutbetong blivit den preliminära förstärkningsmetoden, särskilt för tunnlar i hårt berg där tunna lager av sprutbetong ibland kan användas som den enda förstärkningsåtgärden.

Tunnlar byggs normalt genom metoden borrning-sprängning-vilket leder till att bergytan där sprutbetongen appliceras får en oregelbunden form. Under sprutning är det svårt att fastställa den exakta tjockleken och sprutbetongen har därmed en oregelbunden tjocklek. Beroende på in situ förhållanden kan oarmerad eller fiberarmerad sprutbetong i kombination med bergbultar användas för att förstärka berget. Det strukturella beteendet och lasterna som påverkar förstärkningen beror på interaktionen mellan sprutbetong, berg och bergbultar. Denna samverkan styrs av flera parametrar som t ex; vidhäftningshållfastheten, bergets styvhet och tjockleken hos sprutbetongen. Dessa parametrar är svåra att förutsäga vilket gör dimensionering av en sprutbetongförstärkningen till ett komplext problem.

Den här uppsatsen presenterar den första delen av ett forskningsprojekt med det långsiktiga målet att öka förståelsen för det strukturella beteendet hos en sprutbetongförstärkning. För att uppnå detta har numerisk modellering använts för att studera spänningsuppbyggnaden och uppsprickningen av sprutbetong som utsätts för förhindrade rörelser orsakade av temperaturförändringar eller uttorkningskrympning. Sprutbetongens beteende när den utsätts för en blocklast har också studerats. En numeriskt modell för att analysera spänningarna i sprutbetong som tar hänsyn till tidsberoende materialegenskaper har använts. Modellen kan beskriva det icke-linjära deformationsbeteendet av oarmerad samt fiberarmerad sprutbetong och använder sig av presenterad fältdata för att beskriva de förväntade tjockleksvariationerna. Uppsatsen disskuterar och demonstrerar effekten av viktiga laster som verkar på sprutbetongförstärkningen och hur bergets oregelbundna yta i kombination med sprutbetongens varierande tjocklek påverkar spänningsuppbyggnaden i förstärkningen. Det diskuteras också hur ett fullständigt eller partiellt vidhäftningsbrott påverkar sprutbetongförstärkningens bärförmåga.

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Chapter 1

Introduction

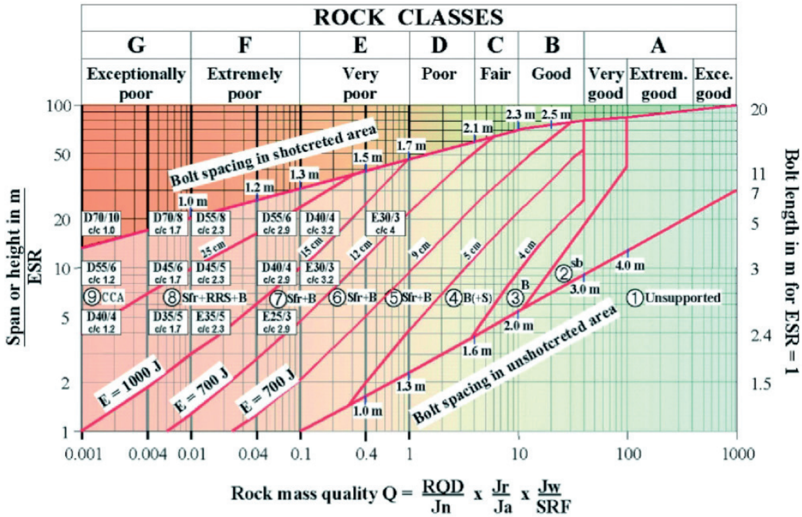
1.1 Background

Shotcrete is sprayed concrete applied pneumatically under high pressure. The technique was originally developed by Carl Akeley at the Field Columbian Museum in the beginning of the 1900's [102]. To create realistic animal models for the museum, Akeley developed a small machine that used compressed air to spray gypsum. At this time, the facades of the museum were in bad condition and Akeley was asked to improve his machine to paint the facades with a layer of gypsum. Akeley and his colleague Clarence L. Dewey then developed the plaster gunthat used compressed air to spray dry gypsum through a hose [102]. Water was added at the nozzle and they successfully sprayed gypsum for an hour before the hose clogged up. His invention was introduced as the "Cement Gun"in 1910 [108]. The first attempts to use shotcrete as underground support in USA were carried out in 1914 [23] while in Europe and Iran, shotcrete in combination with bolts was used for the first time during the 1930's [15]. Until then, shotcrete was applied using the dry mix process, i.e. water was added at the nozzle. The wet mix method was introduced in 1955 [74] allowing ready mixed shotcrete to be sprayed directly. This method has since then gradually taken over the market and is the dominating method in e.g. Sweden and Norway since the 1980's [43, 57]. The advantages compared to the dry mix method is less dust during spraying, lower rebound and that a higher output capacity generally can be achieved [106]. Using shotcrete for construction of tunnels reduces the need of formwork which decreases the production time. When needed, mesh reinforcement is placed at the rock surface before spraying which earlier was a time consuming and heavy labour work. To further reduce part of this work, steel fibres were introduced in the shotcrete mix in the 1970's [43, 74]. Since the start with the cement gun the technique of application has naturally been greatly improved and nowadays, shotcrete can be applied by remotely controlled machines, see Figure 1.1. Shotcrete and fibre reinforced shotcrete (FRS) are widely used for structures where ordinary cast concrete is difficult to use. Examples of such structures are the support of rock tunnels, slopes and overhead repairs of concrete structures.

Figur 1.1: Spraying of shotcrete with a remotely controlled machine. Photo from Sika.

Today, FRS and rock bolts are the two most used materials to reinforce tunnels constructed in hard and jointed rock. Both materials could be used separately but more commonly they are used in combination which creates a complex composite structural system together with the rock. Even though this type of rock support has been used for nearly forty years and the behaviour of each of the materials is well understood there is still a need to improve the understanding about the composite structural system. The design and construction when using shotcrete is as described by Hoek [52] often considered as a craftsman work. Stille and Palmström [99] also point out that the design of tunnels is often based on observations, experience and personal judgement. The design of modern rock tunnels that follow the guidelines given in Eurocode 7 [34] can be based solely on empirical methods such as the widely popular Q-method developed by Barton et al. [18]. The required shotcrete thickness is then a result of the estimated rock quality, a Q-value, in combination with the span of the tunnel, see Figure 1.2.

The method has been developed based on a large number of projects and no estimations or calculations of the loads acting on the tunnel lining or the stresses that develop are required. Reported failures in tunnels while using the Q-method are rare and are often a result of a misjudgement of the rock quality [81]. However, it is possible that the few reported failures are due to an unnecessarily high amount of rock support. By introducing more numerical calculations in the design work for tunnel linings it could be possible to optimize the use of FRS. However, numerical simulations of this type of structures are not an easy task. First of all the type and magnitude of loads acting on the lining must be determined. This depends on the interaction between shotcrete and rock as well as the quality of the rock and must be determined for each tunnel or even for specific parts of the tunnel. Secondly, tunnels are today commonly constructed by drill and blast which inevitably creates an irregular rock surface. The actual thickness of the applied shotcrete is difficult to determine without destructive testing and measured in situ thickness shows great local variations. How this affects the structural capacity of the lining has not been studied to any great extent before.



Figur 1.2: Required shotcrete thickness and bolt distance as a function of the estimated rock quality Q and span or height of the tunnel according to Barton et al. [18].

1.2 Aims and goals

The aim with this thesis is to improve the understanding on how the composite structural system of shotcrete, rock bolts and rock interact and react when subjected to loading. This thesis represents the first part of a research project with the long-term goal to find a suitable numerical model that can describe the behaviour of a shotcrete lining using realistic material and geometrical parameters with the aim of improving and optimizing the use of the material. For the first part of the project, three important research questions have been addressed:

1. Which are the important loads that acts on the shotcrete lining?
2. How does the irregular geometry of the rock surface in combination with a varying thickness of the shotcrete affect the development of stresses in the lining?
3. How does full or partial failure of the bond between shotcrete and rock affect the structural capacity of the lining?

1.3 Limitations

This thesis is limited to the study of large scale use of wet sprayed shotcrete to reinforce hard and jointed rock with a quality similar to that found in e.g. Sweden, Norway and Finland. Shotcrete tunnel linings are in such conditions typically designed as an open ring with a thickness in the range of 50-150 mm. The implemented numerical methods that describe the behaviour of shotcrete are general but the studied applications focus on the support of hard rock.

Only static or quasi-static type of loading has been studied. Studies regarding the effects of dynamic loading and exposure to fire can be found elsewhere; see e.g. Holmgren [54, 56] Ansell [6], Ahmed [3] and Jansson [63]. The thesis is limited to numerical simulations of shotcrete in interaction with rock and rock bolts and focus on the behaviour of the shotcrete. Relevant material data to describe the elastic and non-linear behaviour of shotcrete is taken from the literature. Studies with focus on the behaviour of rock are presented by e.g. Malmgren [69].

1.4 Structure of the thesis

Chapter two gives a background to the construction and the design of tunnels in hard rock. This includes a review regarding design standards used today, historical failures as well as some material characteristics for the shotcrete and important loads that should be considered. Chapter three aims at giving a background to previous studies of shotcrete linings with irregular geometry and/or thickness. Field data are presented to highlight the expected variation in thickness that can be found for tunnels in situ. The Chapter ends with a review of bond strength between shotcrete and rock. Parameters that influence the strength are discussed and test results both from a laboratory environment and in situ are presented. Numerical modelling of shotcrete is then presented in Chapter four. Models for the non-linear behaviour of unreinforced and fibre-reinforced shotcrete are presented together with a multi-physical approach to consider the effects of non-linear shrinkage. Details of how bond failure can be modelled are reviewed and a concept of how the irregular geometry can be modelled using statistical data is finally presented. In Chapter five, numerical examples of shotcrete subjected to load from shrinkage and gravity load from a block are presented. A brief summary of the scientific papers published within this project is given in Chapter six, followed by a chapter of discussion and the conclusions of this thesis together with some interesting topics for further research.

Chapter 2

Construction and design of tunnels

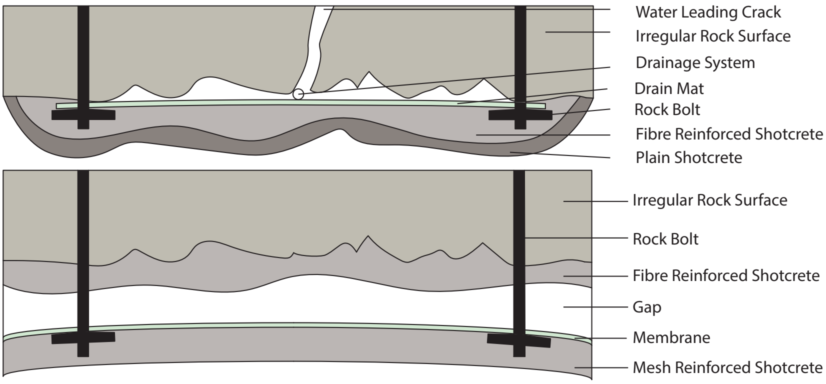
According to Hoek and Brown [51], the main principle in the design of rock tunnels is to enable the rock mass to carry its own weight. Tunnels are therefore usually excavated with an arch shape which, theoretically, enables the rock to carry its own weight. Modern rock tunnels are commonly excavated using the drill and blast method. This will damage the rock mass closest to the excavation. With careful blasting, this damage could be kept to a minimum which could reduce the amount of rock support needed [99]. Blasting that is performed careless or when major differences in rock quality exist will result in geometrical deviations from the ideal arch shape of the tunnel, i.e. too much rock is excavated. This could cause stability problems for the arch, i.e. the shift in centre of gravity of the arch causes rotational forces acting on the blocks which could cause individual blocks to be pushed out. A reduction in horizontal force could also create stability problems for the rock. For a jointed rock mass, the stability will to a great extent depend on the interlocking between individual blocks [99]. To secure the arch shape and to stabilise individual blocks, shotcrete and rock bolts are normally used to reinforce the rock. The high pressure that shotcrete is applied under enables it to penetrate and partly fill up existing joints [99]. A contact area between the blocks is thereby generated which stabilises the arch and prevents rotation of the blocks [23]. This is usually referred to as the mortar effect.

Before an excavation takes place, the rock mass is in equilibrium with an initial state of stress. When excavation takes place, both equilibrium and the stress state in the rock mass are locally disrupted [52]. Movements of the rock occur to find a new state of equilibrium and the magnitude of both movement and stress depends on factors such as, rock quality, in situ stress and time of installation of rock support. Depending on the geometry of the tunnel opening and quality of the rock, different failure mechanisms such as fall out or bursting of rock blocks could occur [51, 99]. To ensure a safe working environment, a temporary rock support could be installed. This could consist of either rock bolts or shotcrete or a combination of both. The choice to install a temporary support is normally the contractor's decision and the amount of support is normally determined based on previous experience [81, 99].

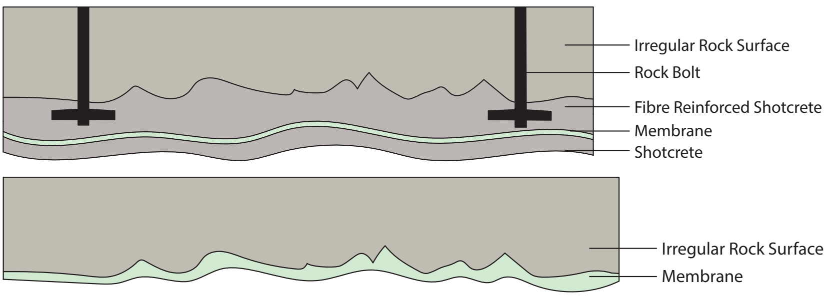
Different techniques to support tunnels in hard rock exist and depend on geotechnical and water conditions in situ. For tunnels in hard rock, typical rock support is unreinforced shotcrete or fibre reinforced shotcrete in combination with systematic rock bolting, see e.g. [81]. For unreinforced shotcrete, the load is transfered through bond to the rock. When bond is insufficient, the structural connection between shotcrete and rock is ensured with rock bolts with washers (steel plates). Any load applied on the shotcrete is assumed to be carried through bending of the shotcrete between the rock bolts so the shotcrete is always reinforced with fibres or mesh reinforcement when used in combination with rock bolts. If good bond between shotcrete and rock is expected, shotcrete could be sprayed directly on the rock surface and rock bolts could be used to fixate individual blocks that are spotted during the excavation.

Infiltrating water can cause problems with deterioration of the shotcrete, cause problems for an electrified railway track or form ice on the roads which reduces the safety. Infiltrating water must therefore be drained and different techniques exist and the choice usually depends on the contractor's and the client's previous experience of a specific solution. During the 1990s, the Swedish Road Administration investigated and evaluated different types of design of drainages for tunnels [57]. The drain concept presented in the report by Fredriksson et al. [44] consisted of a synthetic drain mat placed over sections with infiltrating water. A drainage tube was placed between the rock and drain-mats which were fixed to the rock surface with rock bolts and then covered with shotcrete, see Figure 2.1. At the end of each section the rock was exposed to enable bonding between the shotcrete and rock and thus creating an end-restrained structural system. If infiltrating water is discovered at a later stage, the drainage system could also be placed over a layer of shotcrete. During the construction of the Southern Link motorway tunnel in Stockholm, Sweden a large number of shrinkage cracks were found in the shotcrete [57]. The reason for the cracking was the shrinkage of the shotcrete as presented by Ansell [8]. In two recently constructed motorway and railway tunnels in Sweden a system with an inner lining has been used. According to [78], this system should streamline the construction phase and result in a dry traffic environment in the tunnel. FRS is first sprayed against the rock surface and serves both as a temporary and primary rock rock support. A water proof membrane is then applied over the whole section and kept in place with the help of rock bolts. Mesh reinforcement was then placed before a shotcrete lining with a thickness of 100 mm was sprayed. This creates a free standing arch structural system, see Figure 2.1.

This principle of using a primary lining followed by an inner one has until recently been common practice in road and railway tunnels in Norway. However, a new composite lining system as presented by Holter [59] is being considered to replace this system. For this type of composite lining, FRS is first sprayed directly against the rock. A water proof membrane is then applied to the shotcrete by spraying and then covered by a second layer of FRS, see Figure 2.2. Shotcrete will bond to the membrane and generate a composite structure, for further details see [59, 60]. Internationally, ongoing research on tunnel linings also includes the use of thin polymer linings. These linings are fast setting and have a thickness of around 2-5 mm [83]. Obvious advantages with such thin linings are the reduced application time and transportation need. Tannant [101] presents some structural principles of thin liners and Ozturk and Tannant [84, 85] present experimental results for the adhesion between the lining and rock surface.



Figur 2.1: Different rock support systems used in Sweden for sections with leaking water, system with synthetic drain-mats (top) and inner lining system (bottom).



Figur 2.2: Rock support systems evaluated internationally showing at top, sprayed waterproof membrane in combination with FRS tested in Norway [59, 60] and bottom, thin spray-on liners used in e.g. Canada [49, 101].

2.1 Design of rock support in hard rock

The design of civil engineering structures within parts of Europe, including Sweden, Norway and Finland, must follow the rules in Eurocode [31]. According to the European standards for design of geotechnical structures such as tunnels [34], the limit states should be verified by any or a combination of the following methods:

- Design calculations
- Adoption of prescriptive measures
- Experimental models and tests
- The observational method

The adoption of prescriptive measures means that the tunnel could be designed solely on empirical methods. This is normally used for tunnels with similar rock conditions and geometry to previous built tunnels. For more complex tunnels, a combination of empirical and numerical methods are normally used. One widley known empirical method is the Q-method which was developed by Barton et al. [18]. It was developed based on a trial and error approach to best fit between dimension of the excavation, required support and rock quality based on field data from over 200 cases [81]. The equation to describe the rock mass quality Q is presented as:

$$Q = \frac{RQD}{J_n} \cdot \frac{J_r}{J_a} \cdot \frac{J_w}{SRF} \tag{2.1}$$

In Eq. (2.1) RQD is the rock quality designation value, the variable$J$ describes the joints where indices n is the set number, r the roughness, a alternation number and w the water reduction factor. The final parameter, SFR, is a stress reduction factor. The three quotients in Eq. (2.1) could according to Barton et al. [18] be a crude measure of block size, shear strength between blocks and active stress, respectively. The Q-system is further explained in various textbooks, see e.g. [51, 99] and will not be explained in any more detail here.

A review of tunnel failures is presented below in Section 2.3 and shows that failures including downfall of blocks or shotcrete is uncommon in Sweden, Norway and Finland. This indicates that the used design methods lead to a reliable design in the ultimate limit state (ULS). However, it could also indicate that more rock support than necessary is used. Gaining a better understanding of actual loads that act on the rock support, the interaction between the support and the rock as well as the resulting stresses in the lining is therefore vital to improve the design. In Sweden, traffic tunnels are separated from other civil engineering tunnels by specific rules [103] and guidelines [104, 105] regarding the design given by the

Swedish Transport Administration. The structural capacity for tunnels must be verified by numerical calculations and some specific load cases are given in [103] as a complement to rules in Eurocode [31]. Norway has a long history of using the Q-system in the design of rock tunnels. Design of traffic tunnels shall be done in accordance with [80] published by the Norwegian Public Road Administration. The only specific rule regarding the design of a shotcrete lining is that a thickness less than 60 mm must be avoided with respect to the lifespan of the tunnel. According to the design guideline by the Norwegian Concrete Association, FRS is used on broken hard rock and minor crushed zones while unreinforced shotcrete is used when no deformations are anticipated [79]. The design of tunnels in Finland should follow the criterion given in Eurocode and no specific rules similar to those in Sweden and Norway have been found in the literature.

2.2 Shotcrete material characteristics

Shotcrete is basically ordinary concrete that is applied to the surface under high velocity caused by high pressure. There are however some characteristic material properties that differentiate shotcrete from ordinary concrete that should be addressed. Upon hitting the surface, the large momentum compacts the shotcrete directly which makes vibrating of the material unnecessary. To make the material stick to the surface and enable spraying on vertical faces, set accelerators are added at the nozzle [90]. This affects the setting time of the shotcrete and will interact with the cement hydration and therefore partly change the structure of the material [66]. Normally more cement is used in shotcrete and a short setting time is usually desirable [90]. Both of these factors will affect the shrinkage which therefore is important to consider for shotcrete. Large aggregates tend to rebound (not stick to the surface), and the maximum aggregate size is therefore usually 8 mm [90]. Due to the differences listed above, the development of material strength and behaviour, such as shrinkage and creep, of shotcrete does not necessarily follow that of ordinary concrete. For the structural analysis of a shotcrete lining, material properties of shotcrete should be used. However, there is a lack of official guidelines, such as Eurocode, describing material strength development and behaviour of shotcrete. Properties of ordinary concrete are therefore normally used in the practical design of shotcrete. Scientific papers devoted to shotcrete are also rare in comparison with ordinary concrete.

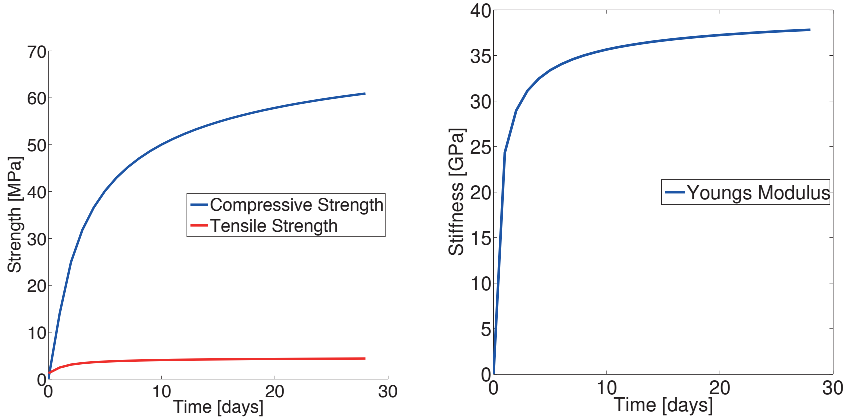
2.2.1 Development of material strength

Extensive testing of the time dependent development of shotcrete strength is presented by Bryne [24]. Here, compressive and flexural strength were tested together with Young's modulus for a time period of 0-112 days. Testing of free shrinkage

Tabell 2.1: Composition of shotcrete used for material testing [27]

Material	Content [kg/m^3]
Cement CEM I 42.5 N-SR 3 MH-LA	495
Densified silica	19.8
Water	220
Superplasticiser	3.5
Glass fibre	0 / 5 / 10
Aggregate 0-2 mm	394
Aggregate 0-8 mm	1183

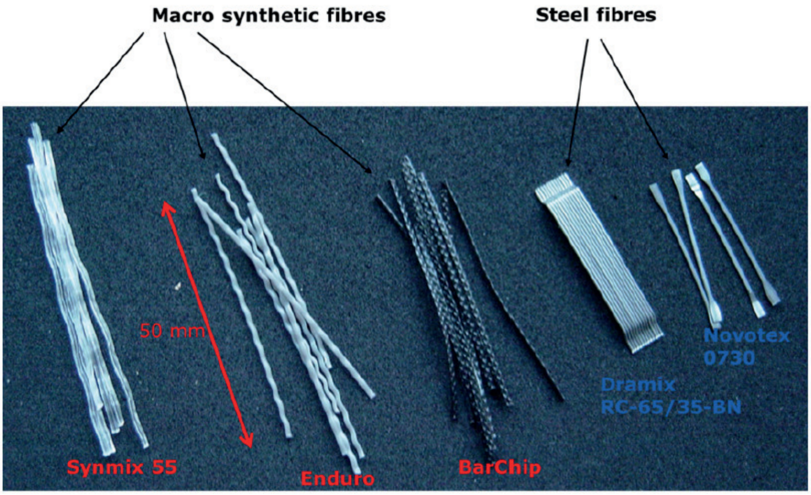
and bond strength was also performed but these results are discussed in Section 2.4.1 and 3.3, respectively. The used shotcrete recipe is presented in Table 2.1 and in Figure 2.3 data from the tests are presented.



Figur 2.3: Development of compressive and tensile strength (left) as well as Young's modulus (right) for shotcrete according to Bryne [24].

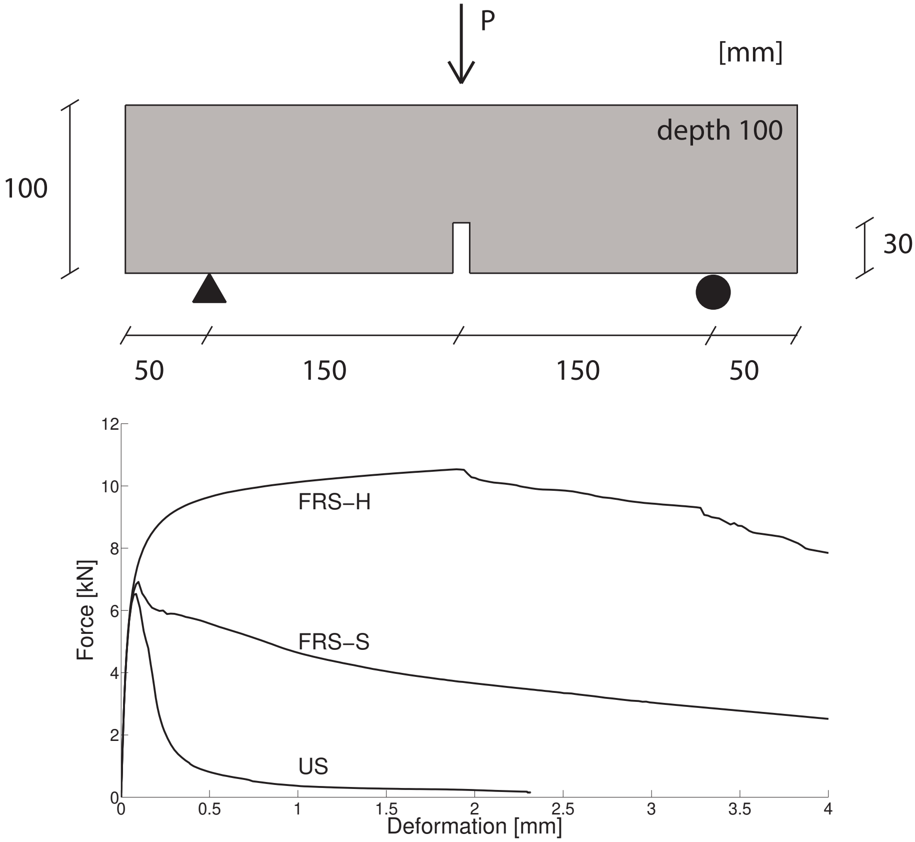
2.2.2 Unreinforced and fibre reinforced shotcrete

It is widely known that unreinforced concrete and shotcrete have a good compressive strength but are weak in tension. Furthermore, the failure in tension is brittle which is undesirable in any structure. Still, as mentioned above unreinforced shotcrete is sometimes used in Norwegian tunnels [79] and also elsewhere. In such cases, the expected tensile stresses are small or a good bond between shotcrete and rock can be achieved. For a continuous bond of good quality, the rock acts as reinforcement and distributes the cracks into a fine pattern just as reinforcement. This was shown in Paper I. However, more commonly FRS is used as rock reinforcement. There are two types of fibres, micro and macro. Micro fibres typically have a diameter of less than 0.3 mm [12, 90] and are mainly used to increase the fire resistance [90] and the cohesion of the shotcrete mix [21]. Its ability to reduce shrinkage cracking has also been presented by Banthia et al. [12] and Bryne et al. [24, 27]. Macro fibres have typically a diameter between 0.3 and3.0 mm and a length of 25-60 mm [12, 26] and are used to increase the structural capacity of the shotcrete. In Figure 2.4 some examples of macro fibres are shown.



Figur 2.4: Different macro fibres, from Bryne [26]

It has been shown that the addition of macro fibres has a negligible effect on the compressive and tensile strength as well as the Young's modulus, see e.g. Banthia and Sheng [11] and Yoo et al. [110]. The benefit of adding fibres to the shotcrete comes in the increased ductility [17] which is due to the fibres ability to bridge the crack and transfer stresses [97]. To clarify the difference in post-cracking behaviour between unreinforced and fibre reinforced shotcrete, a notched three-point bending test as shown in Figure 2.5 is used. Here, the crack will initiate and propagate at the location of the notch and such a test is commonly used to measure the response of FRS. In Figure 2.5, Force-Displacement curves are plotted for unreinforced shotcrete (US), fibre reinforced shotcrete with strain softening and hardening behaviour (FRS-S and FRS-H). For unreinforced shotcrete, the formation of the crack will be followed by a brittle failure. When some fibres are added, the behaviour will change to strain-softening. Once the crack is formed, the external force drops but the fibres bridging the crack increase the ductility and possible deformation of the shotcrete before failure occurs. Adding more fibres, the resistance and fibre bridging effect increases and the external load can be further increased after cracking.



Figur 2.5: Top, test set-up for four-point bending test and bottom, response for unreinforced shotcrete (US), FRS strain-softening (FRS-S) and FRS strain-hardening (FRS-H).

Since the post-cracking behaviour of FRS depends on the fibres ability to transfer stresses over the crack it is easy to understand the the ductility depends on the number and orientation of fibres bridging the crack as well as their ability to anchor in the shotcrete [2]. Fibres are added in the batching of the shotcrete and their orientation in the structure will be random. A scatter in test results is therefore expected when testing FRS [97]. According to Banthia [10], the governing failure mechanism for FRS is pull-out failure of the fibres. The full tensile capacity of the fibre can therefore not be utilized and the anchorage length and bonding to the shotcrete are two important parameters determining the efficiency of the fibres. Different fibre types exist on the market where either end-hooks are used to increase anchorage or the surface of the fibre is rough to increase its bond to the shotcrete. However, in the choice of fibres to reinforce shotcrete one must consider the fibres ability to distribute evenly in the batching process and to keep the shotcrete sprayable [21, 90]. A commonly used fibre type in Swedish tunnels is a macro steel fibre with two end-hooks. For this type of fibres a strain-hardening behaviour could be expected for a fibre content of around 80 kg/m^3 (1 %), see results presented by Barros [16], Yoo et al. [110]. However, it is according to Holmgren [55] hard to spray shotcrete with more than 80 kg/m^3 of fibres and more commonly 60 kg/m^3 are used which results in a strain-softening behaviour.

2.3 Reported failure and cracking of tunnels

Studying past failures and problems related to cracking in the shotcrete is a good way to increase the knowledge about the structural capacity and loads that could act on a tunnel lining. Information regarding failures in Scandinavian hard rock tunnels that involve downfall of shotcrete and/or rock is, however, hard to find in the literature. Malmgren [71] reports results from failure mapping from the Kiirunavaara mine in Northern Sweden. A total length of 7 km was investigated and 80 % of the failure consisted of small fall-outs of shotcrete. In more than 90 % of the failures, the thickness of the shotcrete was less than 20 mm and suspected causes of failure were shrinkage, rock deformation and dynamic loading due to blasting in combination with low bond strength [71]. In a report by Perez [86], monitoring of deformations in three different hard rock tunnels in Sweden is presented. The purpose of this report was to collect and present deformation data that later could be used to develop a failure criterion based on monitored deformation in a tunnel. In one of the monitored tunnels a failure including fallout of blocks occurred during the monitoring period. In the second tunnel, a controlled failure occurred, i.e. the purpose was to study the failure of the rock mass when subjected to excavation and thermal induced stresses. In the third case no failure occurred. The presented data could be an interesting project for future studies. During the construction of the Southern link motor way tunnel in Stockholm, Sweden, a large number of cracks where found in the shotcrete before the opening. An investigation was conducted and results are presented by Ansell [8]. Cracks were located in the shotcrete covering the soft drain-mats, see Figure 2.1, and the governing failure mechanism was considered to be shrinkage of the end-restrained shotcrete. The newly sprayed shotcrete was normally watered during the first two to three days and the average temperature was 10° C and the relative humidity (RH) was 75 %. Motivated by this a research project was conducted at KTH Royal Institute of Technology with the focus on time-dependent material properties and shrinkage of shotcrete. The results of this project were presented by Bryne [24].

Some larger failures during construction and operation of tunnels in Norway are presented in the report issued by the Hong Kong government [30]. One of the more notable failures is that of the Hanekleiv highway tunnel in Vestfold, Norway. The tunnel was opened in 1996 and the majority of the rock support was FRS in combination with rock bolts [72]. In 2006 a major cave-in occurred when 250 m^3 fell into the tunnel. The collapse occurred when there was no traffic in the tunnel. The Hanekleiv tunnel was designed according to the Q-system and the cause of failure was a severe overestimation of the rock quality [81]. The tunnel could after repair work be re-opened in 2007 and for further information see e.g. Nilsen [76] and Mao et al. [72].

2.4 Important load cases

One of the most fundamental aspects in the design of any structure is the knowledge of what loads to consider. For the majority of civil engineering structures, loads to consider for different limit states are well defined in different design codes, see e.g. Eurocode [32]. Loads acting on geotechnical structures such as a rock tunnel are not as well defined and must normally be determined from case to case. The magnitude of the loads acting on the rock support depends on the interaction between the support system and the rock and on the time when it is installed. According to Holter [59], few cases with detailed monitoring of loads acting on the rock support are presented in the literature. Furthermore, the monitored loads presented in [59] show that low stresses occur in the lining. Below, there is a discussion regarding important load cases for the design of a shotcrete lining in hard and jointed rock. The discussion is focused on the physical aspects and mechanisms causing the load together with experimental results from the literature. The implementation of the loads in the numerical model is presented in Chapter 4.

2.4.1 Restrained shrinkage

Shrinkage of a restrained concrete or shotcrete structure is a risk factor for cracking, see e.g. Banthia [13], Leung et al. [68] and Bryne et al. [26] who all present novel test techniques for the investigation of shrinkage induced cracking. Shrinkage depends on the volumetric contraction of the cement paste. This is internally restrained by the aggregates and the amount of shrinkage will therefore, naturally, depend on cement content, volumetric relation between cement and aggregate and stiffness of the aggregates. The maximum size and the grading of the aggregates will have an indirect influence on the magnitude of shrinkage. According to Neville and Brooks [75], a more lean mix can be used for larger aggregates which results in lower shrinkage. The shrinkage of shotcrete is therefore typically larger than ordinary concrete since the cement content usually is higher and the maximum size of the aggregate is lower. As pointed out in [75], the coarsest particles, i.e. larger aggregates, are prone to rebound from the surface during spraying of shotcrete. This will change the relation between aggregate and cement in the applied shotcrete lining and the magnitude of shrinkage will therefore to some extent also depend on the skill of the operator.

Shrinkage could be divided into early and long term. Early shrinkage will be driven by the hydration process, see e.g. [61] which causes changes in internal temperature and relative humidity. Long term shrinkage will be driven by chemical reactions or loss of free capillary water in the shotcrete. This thesis has focused on the effects of drying shrinkage. Attempts have been made to include the effects of early shrinkage caused by autogenous and possible thermal and plastic shrinkage in a simplified way, see Section 4.3.

Stiffening of young shotcrete

The early behaviour differs between shotcrete and concrete due to use of specially developed set accelerators for shotcrete [66]. During the first period after spraying, cement reactions will be slow. This period is called the dormant period. After some time the acceleration period will start where major cement reactions occur that result in stiffening and hardening of the material. According to Bryne [26], this period starts after 10-12 hours. However, the onset of the acceleration period depends on the type of cement and temperature so the usually cold rock surface could according to Lagerblad et al. [66] delay this. The aluminates in the set accelerators react with the gypsum in the cement to form ettringite [66] which generates the early development of strength that shotcrete needs. Autogenous and drying shrinkage thus occur in a stiff structure which could alter and increase the porosity of shotcrete and result in larger shrinkage [66].

Early shrinkage

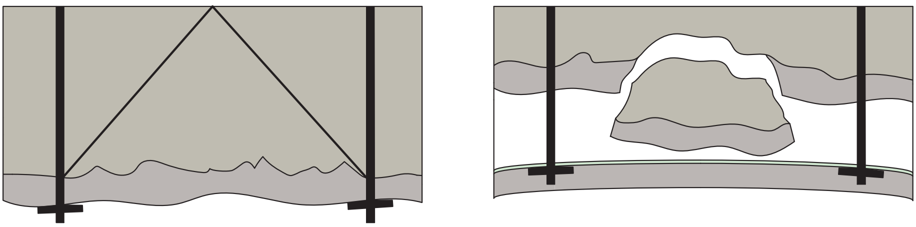
When the cement paste goes from a fluid to a rigid state water is lost due to evaporation from the surfaces or suction by other materials [75]. This leads to plastic shrinkage which could be reduced by adding water to the surface. If this for various reasons is not possible, studies by Branch et al. [22] show that the crack widths caused by plastic shrinkage could be reduced by introducing micro-fibres. The hydration process that takes place, i.e. the chemical reaction between cement paste, water and aggregate, is an exothermal process. The increased temperature leads to an expansion of the shotcrete. Depending on the development of bond and mechanical strength this expansion and the following decrease and contraction in volume could lead to compressive and tensile stresses. As discussed in Gasch [47], the hydration process consumes water which also leads to autogenous shrinkage. Furthermore, the hydrated particle of cement and water occupies less space than each of them separately which leads to chemical shrinkage.

Drying shrinkage

Drying shrinkage is the volume reduction of the cement paste due to loss of free water from the pore system of the shotcrete. Water is transported both in liquid and vapour phase and migrates towards regions where the water content is lower. The process is complicated and depends on several factors such as pore structure and degree of saturation in the material [47]. The aim here is not to give a full description of all the governing mechanisms behind the drying but rather a brief overview. For a more detailed explanation see e.g. Indiart [61] or Gasch [47]. In short; drying shrinkage is the result of an excessive use of water that increases the workability. Free capillary water in the pores migrates towards areas with a lower degree of saturation, i.e. normally the surface. At the surface, free water evaporates to the ambient air. When drying out, the capillary pores will contract which leads to shrinkage of the shotcrete [66].

2.4.2 Block load

A typical design load for the shotcrete in the ultimate limit state (ULS) is the gravity load from a block. The design load from the block could be assumed based on distance between rock bolts and arching effect between the bolts, see e.g [51, 99]. For traffic tunnels in Sweden specific rules exist regarding the specific load of the block [103]. Furthermore, an accidental load case exists when the load from a falling block should be considered for linings installed with a distance from the rock, see Figure 2.1. Both cases are illustrated in Figure 2.6. The design load from the block can be assumed to be distributed uniformly or triangularly. Two major types of failure modes could occur depending on the relationship between bond, flexural and shear strength. For high bond strength between the rock and shotcrete, a shear failure could occur. If a bond failure develops, flexural stresses develop in the shotcrete which then likely fails due to bending.



Figur 2.6: Load scenarios for deadweight of block with triangular shape (left) and falling block (right).

To investigate the load carrying capacity large scale testing of FRS subjected to a quasi-static load from a pushing block is presented in [53, 55]. Andersson [5] presents results from mesh reinforced shotcrete subjected to a dynamic load from a falling block. In the above mentioned tests the thickness of the shotcrete was practically uniform. For a case with an inner lining, such as the case studied by Andersson [5], this is a rather good approximation but for the case studied by Holmgren [53, 55] the thickness found in situ will be highly irregular. However, the presented results in [53] are very interesting in terms of failure mode. The tests was deformation based and in all tests, failure was initiated by bond failure between shotcrete and rock. The resulting load then dropped to around one third and was almost constant as the adhesion crack propagated. The final failure was then a flexural failure in the shotcrete. In the work with Paper IV the irregular thickness of the shotcrete was included to study its effect with respect to the load carrying capacity for a block. Results were compared with the capacity for a lining with uniform thickness and are summarised in Chapter 5.

2.4.3 Other loads

Restrained shrinkage and block load have been the focus when studying load effects in this thesis. There are, however, other loads that will affect the shotcrete in various ways during the lifespan of the tunnels. These are briefly mentioned and discussed below.

If the drainage is insufficient or not installed, infiltrating water will freeze during the winter and cause an ice pressure against the shotcrete. The load is caused by the expansion of water and the restrained movement of the shotcrete. The formation of cracks and the resulting reduced stiffness of the shotcrete allow a larger expansion of the ice and stresses decreases. Restrained movement caused by temperature is another such load where stresses gradually decrease due to the effect of cracking. The temperature in the rock is rather constant and the variation in temperature for a traffic tunnel is caused by heating from cars and the ventilation. By studying the effects of shrinkage which gives a similar load effect as described above, i.e. stresses are induced by a restrained movement and cracking decreases the stresses, the effects of these loads could be partly understood.

Shotcrete linings on hard rock usually end at the walls of the tunnel and this type of lining is described as an open ring. Compared to a close ring, the stiffness of the tunnel is not significantly changed by the addition of the shotcrete. Deformation of the rock results mainly in an increased stress in the rock and this load case has not been considered as important for the shotcrete. If shotcrete is installed at or close to the tunnel face it must resist the impact loading caused by the blasting when excavation continues. The early strength and resistance of the shotcrete are therefore important to study since they will be decisive when excavation can continue. This has been studied and presented by both Ansell [6] and Ahmed [3]. Another interesting dynamic load case is the air pressure caused by a passing train. This is especially interesting since the development and construction of a high-speed train-line is ongoing in Sweden. The stresses in the shotcrete sprayed on soft drains due to passing of trains are presented by Holmgren and Ansell [58].

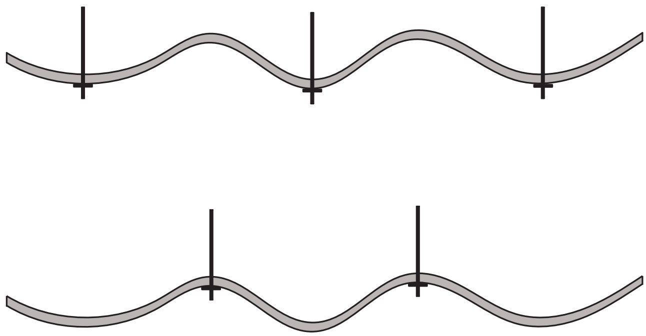
Chapter 3

In situ geometry and bond strength of shotcrete

This chapter will start by presenting a review on previous research regarding the irregular rock surface and the variation in shotcrete thickness found in situ. A literature review considering modelling of irregular shotcrete linings is also presented by Ansell [7] as part of a preparatory study to this project. Furthermore, the development of bond strength between shotcrete and various rock types is discussed and results from measured bond strength are presented.

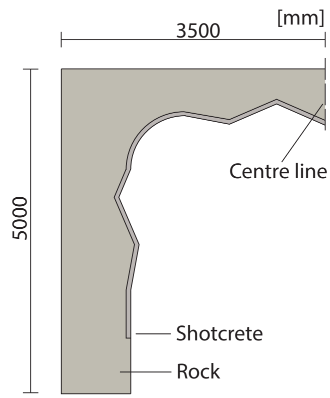
3.1 Research on shotcrete linings with varying thickness

Research on shotcrete linings and their structural behaviour when used as rock support is uncommon in the literature. In presented studies, the effects of an irregular shotcrete thickness are seldom included. Nilsson [77] devoted part of his doctoral thesis to investigate the structural behaviour of a bolted shotcrete lining, see Figure 3.1. Sine-waves in two directions were used to give the lining a harmonic shape and the resulting stress with respect to the position of the bolts was studied using numerical simulations. The thickness of the shotcrete was uniform and was modeled using shell elements. Point loads were placed over the whole lining to investigate the difference in stiffness and peak load. The thickness of the shotcrete was either 40 or 80 mm and the amplitude of the sine-waves varied between 0 and 400 mm. The results indicated that the structural capacity of the lining increases when bolts are placed on peaks rather than crests. According to [77], this is due to the arching effect between the bolts.



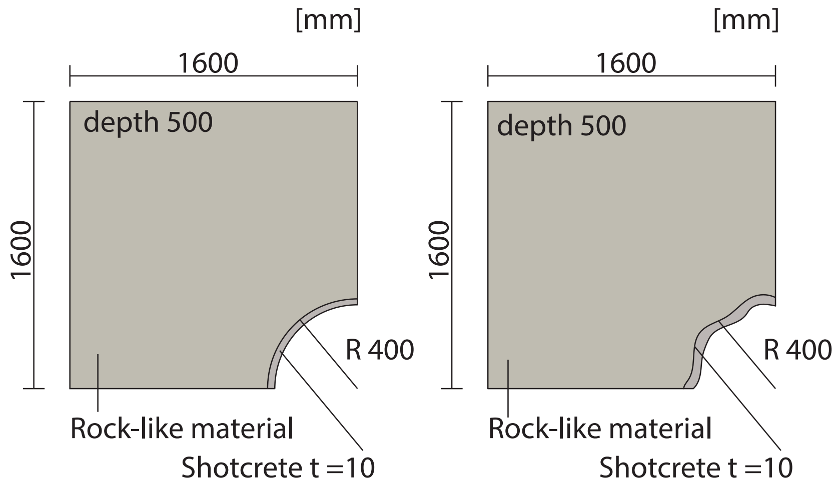
Figur 3.1: Modelling of irregular shotcrete linings and placement of rock bolts by Nilsson [77].

A parametric study of the interaction between shotcrete, rock bolts and rock is presented by Malmgren et al. [70]. The surface of the rock was assumed to either follow the ideal tunnel profile or have a saw-tooth profile with an amplitude of 80 / 150 / 300 mm, see Figure 3.2. The thickness of the shotcrete was 70 mm and loads were applied along all the boundaries of the 2D-FE model. The presented results show that the mode of failure differs for the different tunnel profiles. For the saw-tooth profile, a majority of the failures occurred in the lining which is likely due to the development of stress-concentrations caused by the irregular geometry. For the ideal tunnel profile, the relation between the horizontal and vertical load was decisive whether the failure occurred in the lining or at the interface, i.e. bond failure.



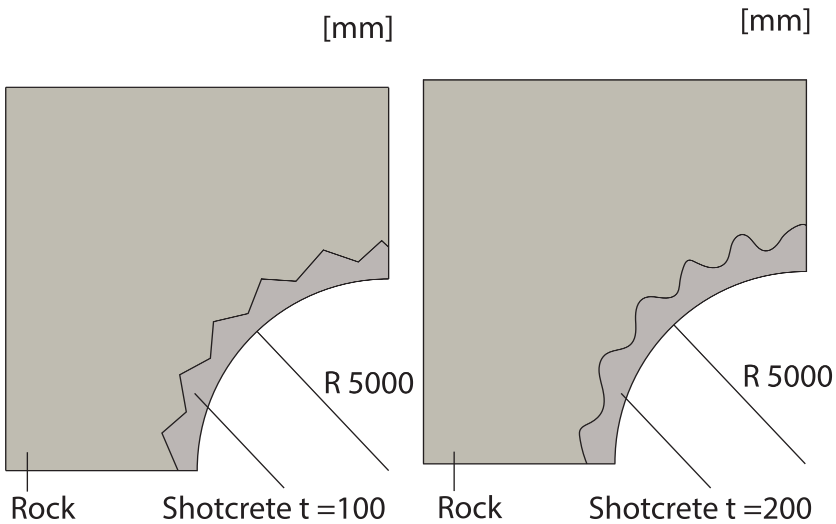
Figur 3.2: Modelling of irregular shotcrete linings by Malmgren et al. [70].

No more studies including an irregular rock surface or a varying shotcrete thickness have been found for hard rock. Some additional studies regarding the support of weak rock were therefore added to the review. Large scale testing of irregular shotcrete linings is presented by Chang [36]. In the laboratory a $1.6{*}1.6{*}0.5\ m^3$ (length $*$ with $*$ thickness) mould was used to cast a concrete-like material with properties corresponding to weak rock. The cast rock represented a quarter of a tunnel span with a radius of 0.4 m. The load capacities for three linings were investigated; uniform, simply waved uneven surface and doubled waved uneven surface, see Figure 3.3. Load was applied along both sides until failure occurred. The conclusion drawn by Chang was that the support effect of a double waved lining was better than a single waved lining while the strength and stiffness of a double waved was similar to that of a regular lining [36]. However, it must be mentioned that only one test was performed for each lining and no quantitative conclusions could therefore be drawn. The experimental result presented by Chang [36] was used for validation of numerical simulations presented by Zhang [111].



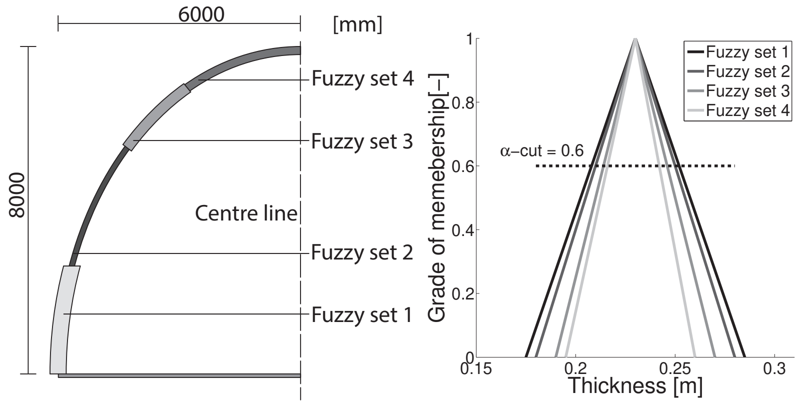
Figur 3.3: Rock support studied by Chang [36] uniform lining (left) and simply waved lining (right).

Numerical studies on shotcrete linings designed as a closed ring are presented by e.g Son and Cording [98] and by Lee [67], see Figure 3.4. In both studies, a 2D FE-model was used to model the shotcrete and the rock. The rock surface was considered irregular and the shotcrete surface towards the tunnel was uniform. Load was applied along the boundaries of the model. Different relationships between the vertical and horizontal load as well as a different stiffness ratio between the rock and shotcrete were studied in both cases. The irregular rock surface was in [98] modelled with a saw-tooth profile with a height of 50 / 150 / 300 mm which implies that the mean thickness of the lining changed. The presented results show a correlation between an increase in the amplitude of the saw-tooth shape and the normal force. In the model, a larger amplitude resulted in an increased stiffness as expected. The relationship between normal force in the lining and difference in stiffness was also investigated by changing the relationship between Young's modulus of the shotcrete and the rock. As expected, an increased stiffness of the rock resulted in an decreased normal force in the shotcrete. In the study by Lee [67], the mean thickness of the shotcrete was kept constant at 200 mm while the amplitude of the used sine-wave changed between 0 / 50 / 100 / 150 mm. The results show that the difference in normal force between a regular and an irregular lining is small but the stresses will increase with an increased amplitude. This is an expected result since a larger amplitude introduces larger geometric changes and therefore increases stress concentrations in the lining.



Figur 3.4: Modelling of irregular shotcrete linings by Son and Cording [98] (left) and Lee [67] (right).

An interesting approach to study the influence of an irregular shotcrete thickness is presented by Barpi and Peila [14]. Here, fuzzy logic was used to introduce a random function for the variation in thickness. The basis of fuzzy logic is that the studied parameter is given a grade of membership, i.e. a specific interval for which the parameter can take any value. The shotcrete lining was modelled with eight separate beams structurally connected at each end, see Figure 3.5. Each of the beams was given a membership function of the thickness as shown in Figure 3.5. The thickness of each of the eight beams can take any value given by the membership function, i.e. no probability or spatial correlation is linked to this function. The parameters studied, in this case the displacements and resulting sectional forces as a function of the shotcrete thickness are then calculated for different cuts of the membership function, called α-cut [14]. As an example, an α-cut of 0.6 corresponds to a thickness interval for fuzzy set 4 between approximately 0.20 and 0.25 m, see Figure 3.5. Resulting deformations and sectional forces are calculated by minimizing and maximizing their respective equation. Results are presented for each α-cut and show that the variation in shotcrete thickness affects the sectional forces. One important conclusion drawn by Barpi and Peila is that a thicker shotcrete lining not necessarily is better since it could result in higher stresses which could cause cracking of the lining [14].



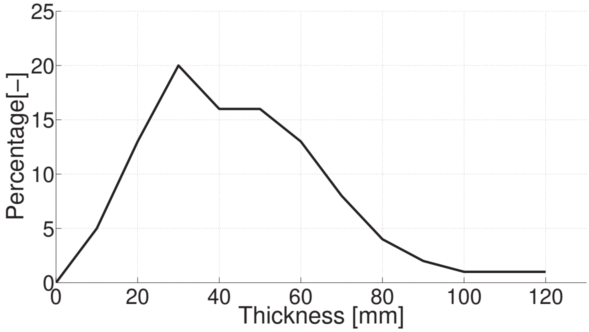
Figur 3.5: Modelling of varying shotcrete thickness using Fuzzy logic according to Barpi and Peila [14] showing to the left, division into Fuzzy sets for half of the tunnel and to the right, example of a triangular shaped membership functions for the shotcrete thickness.

3.2 Varying thickness in situ

As discussed in Section 2.1 the most used construction method for tunnels in hard rock today is the drill and blast method. Deviations from an ideal tunnel profile are therefore inevitable. The magnitude of under- or overbreaks as well as the damage on the remaining rock depend on the used blasting technique [51, 99]. With careful blasting, it is possible to achieve a rather smooth surface but the quality of the rock, or rather the variation in rock quality, as well as existing joint patterns will influence the final shape. Shotcrete linings have normally a visually harmonic surface and the variation in thickness will largely depend on the irregular rock surface. When spraying, several factors influence the thickness of the final lining. First of all, part of the shotcrete will rebound, i.e. not stick to the surface. The amount of rebound depends on several different factors such as the skill of the operator, type and roughness of the surface and size and grading of the aggregate [21]. The volume of shotcrete used for a section is therefore not a clear indication regarding the mean thickness of the lining. Secondly, there are no natural references regarding the thickness of the lining. Different depth gauges could be placed on the rock surface to verify that the required thickness is obtained but will give no further information about the actual thickness. Using such a method, it is rather likely that more shotcrete than necessary is used since the visibility during spraying is limited.

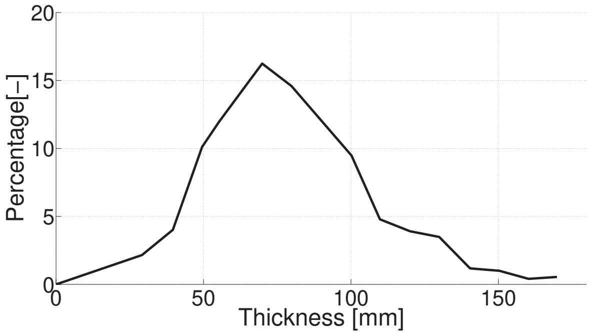
One way or the other, the actual applied thickness of the lining must be controlled after spraying so that the structural capacity can be verified. This can either be done by drilling a number of holes in the hardened shotcrete and measure the thickness and its variation. Different countries and design situations have different requirements regarding deviations from required thickness. Eurocode [35] gives guidelines for how thickness shall be measured but not to which extent testing shall be done. For tunnels in Norway, all joints, cavities and depressions shall first be filled out before an even layer of shotcrete is applied [79]. As a requirement in [79], the average thickness should be specified and the minimum local thickness shall at least be 60 % of the specified value. This can be controlled in three ways; drilling holes in different areas of the support, measure thickness when drilling for rock bolts or by placing steel studs against the surface and later check that shotcrete has covered the studs. According to Norwegian standards, one test for every 250 m^3 of applied shotcrete should be performed [81].

The amount of data presented in the literature regarding the in situ thickness variation for shotcrete linings is limited. Malmgren et al. [71] present measured thickness from the Kiirunaaara mine in northern Sweden. A total of 370 holes was drilled in the shotcrete and the variation in thickness is presented in Figure 3.6. The required thickness varies between 30 and 50 mm and based on the presented data, it can be concluded that a variation up to $\pm$ 50 % could be expected. Furthermore, it was concluded by Malmgren that the targeted thickness was not achieved in approximately 38 % of the cases.



Figur 3.6: Distribution of shotcrete thickness in the Kiirunavaara mine for a required thickness of 30-50 mm [71].

During the construction of the Southern Link motor way tunnel in Stockholm, Sweden, a large number of shrinkage cracks were found. An investigation was conducted and results from this are summarized by Ansell [8]. Results include measured thickness of the shotcrete from a total of 480 drilled holes and are plotted in 3.7. The required thickness was 60 mm and it can be seen that there is a tendency of spraying too much shotcrete. This could be due to the fact that it is more time and cost effective to spray more shotcrete than necessary the first time to avoid relocating the machine at a later stage if the thickness is insufficient. Another reason for the increased use of shotcrete could be to fill out joints and cavities before the actual lining is applied.



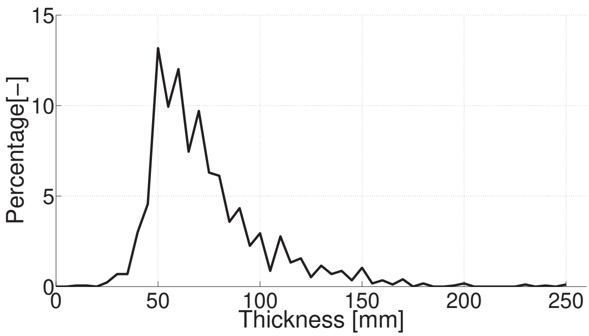
Figur 3.7: Distribution of shotcrete thickness for the Southern Link motorway tunnel in Stockholm, Sweden with a required thickness of 60 mm [8].

The tendency of spraying thicker linings than necessary can be seen from the thickness data collected from the construction of a recent railway tunnel in Stockholm,

Tabell 3.1: In situ measurement of thickness of applied shotcrete

Reference	Thickness [mm]		
	Required	Mean	St.D
Södra Länken [8]	60	72	27
LKAB [71]	30-50	42	23

Sweden. During the installation of rock bolts, the thickness of the shotcrete was measured in the drilled holes. A total of 1731 measurement was made and the results are presented by Sunesson [100]. The data shows a tendency of using too much shotcrete and that a great variation in thickness is obtained. The data are representative for the thickness in one tunnel and gives no information about the spatial correlation. All the data were normalized against the required thickness and the best statistical fit was a log-normal distribution [100]. In Figure 3.8, this data is used to exemplify the expected distribution in thickness when the required thickness is 50 mm. This data were used in the to create a shotcrete lining with a statistically based variation in thickness [96]. The results from the above discussed measurements are summarized in Table 3.1.

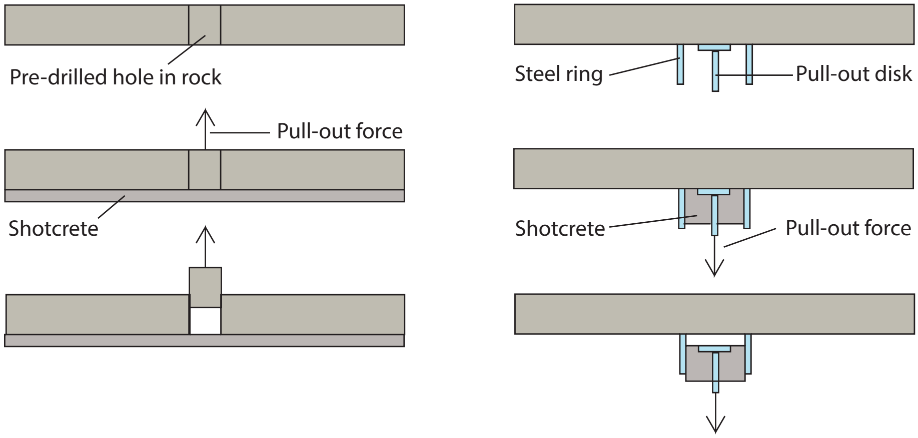


Figur 3.8: Distribution of shotcrete thickness for a railway tunnel in Stockholm, Sweden. All measurments were normalized against the required thickness and are here exemplefied for a required thickness of 50 mm [100].

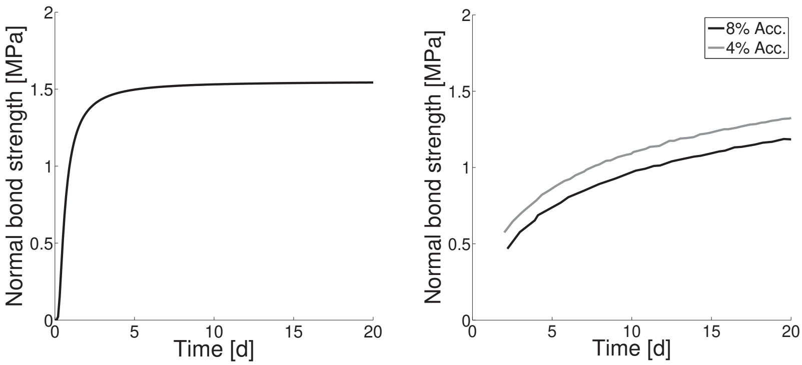
Laser scanning is an interesting technique that obviously increases the knowledge of the applied shotcrete thickness. When used, the rock surface is scanned before and after spraying and the actual thickness can be obtained from the scanning data. Furthermore most of the modern 3D laser scanners can also produce high quality images which allows for permanent documentation and detailed analysis of the rock without being located at the tunnel front [40]. Feng [41] also stressed that too much personal work is included in the in situ data acquisition and that laser scanning should reduce this problem. Furthermore, the safety could be increased since no person has to approach the unsupported rock [41]. Results from direct use of laser scanning during spraying is presented by Wetlesen and Krutrök [107]. A scanning rig was mounted on the shotcrete robot which enables scanning before and directly after spraying. The accuracy of the scanning is $\pm$ 10 mm per m^2 and areas on the rock with insufficient shotcrete thickness could be highlighted by a laser grid. Using this technique, the LKAB mines in Northern Sweden could reduce their annual use of shotcrete with 20 %. Nowadays contractors are usually paid by shotcrete volume [107] and to increase the usage of the scanning an incitement for the contractor to save material must be introduced.

3.3 Development of bond strength

The bond strength between two adhering surfaces depends on the quality of the two contact faces. The condition of the rock surface is therefore of great importance to achieve a good bond strength [71, 90]. The bond between shotcrete and rock is important to consider for the structural behaviour of the shotcrete lining. If a continuous bond of good quality can be achieved, the rock will act as reinforcement for the shotcrete. When subjected to restrained forces such as shrinkage and temperature, a fine crack pattern can develop even for unreinforced shotcrete or concrete, see e.g Malmgren[71], Carlswärd [29] and Groth [48]. In the study by [29], concrete beams were cast on a concrete slab and subjected to drying. If no debonding occurred, the resulting crack widths were of similar size for unreinforced, steel fibre reinforced and steel bar reinforced concrete. However, larger cracks formed in the unreinforced concrete if the bond was partly lost. Studies presented by Holmgren [53] show that the load from a pushing block is transferred through bond over a small area. High local stresses can therefore develop in the bond and failure might lead to a flexural failure in the lining itself. If shotcrete is installed close to the tunnel front, the development of bond strength is important for the continuation of the excavation. Testing bond strength at an early age can be difficult due to low strength and stiffness of the shotcrete. Two new test methods have been developed by Bryne et al. [24, 25] and Bernard [20]. The method by [25] is intended for laboratory testing and is based on pull-out of shotcrete cores in the reverse direction while Bernard [20] used a steel pull-out disk in combination with a spiral ring used to confine the shotcrete. Both methods are shown in Figure 3.9. Different rock types were used and a bond strength up to 0.2 MPa was reported in [20] and 1 MPa in [25] after 24 hours. Early development of bond strength between shotcrete and rock or concrete are presented in Figure 3.10.



Figur 3.9: Different test methods for early bond strength. To the left, pull-out in the reversed direction [24] and to the right, pull-out using steel pull-out disk and spiral ring [20].



Figur 3.10: Development of bond strength to the left for shotcrete and granite from Bryne et al. [25] and to the right for shotcrete and concrete with various amount of accelerator from Malmgren et al. [71].

Two of the most influential factors for the development of bond strength are the type of rock mineral and the cleanness of the surface. The difference in rock mineral is likely to have the highest influence and was together with the surface roughness extensively studied by Hahn [50]. Important results from this study is reproduced in Table 3.2 and additional results of bond strength in the normal and shear direction are presented in Table 3.3.

Tabell 3.2: Important test results of normal bond strength between shotcrete and various rock types reproduced from Hahn [50].

Rocktype	Grain size	Bond strength [MPa]	
		Smooth surface	Rough Surface
Shale	Very fine grained	0.24	0.28
Lime stone-marlstone	Middle grained	1.49	1.89
Marble	Fine grained	1.38	1.52
Granite	Middle grained	1.04	1.40
Granite	Fine-Middle grained	1.48	1.71

Tabell 3.3: Compilation of test result for normal and shear bond strength. σ and τ is the normal and shear bond strength, respectively. From [93].

Reference	Interface	Test Conditions	σ [MPa]	τ [MPa]
Bernard [20]	Various / Shotcrete	In situ	0.2	-
Silfwerbrand [91]	Concrete / Shotcrete	In situ	0.38	2.85
Saiang [88]	Magnetite and Trachyte / Shotcrete	Laboratory	0.56	0.50
Elisson [39]	Granite / Shotcrete	In situ	1.37	-
Bryne et al.[25] [1]	Granite / Shotcrete	Laboratory	1.50	-
Silfwerbrand [91]	Concrete / Shotcrete	Laboratory	1.72	3.35
Moradian [73] [2]	Barre granite / Concrete	Laboratory	-	4.79
Krounis [65] [3]	Granite / Concrete	Laboratory	-	4.18

[1] Results are mean values after three days of curing
[2] Results are mean values for 100 % bonding
[3] Results are mean values with a combined normal stress of 0.8 MPa and 100 % bonding

Malmgren [71] demonstrated the importance of cleanness and preparation of the surface when the bond strength between shotcrete and rock (iron ore) was compared for two different surface preparation methods. In the first method, the surface was prepared by mechanical scaling followed by water cleaning with a pressure of 0.7 MPa. In the second method, water jetting with a pressure of 22 MPa was used. A total of 45 and 24 tests was performed with each method and a clear difference in bond strength and failure modes was reported. In the first case, a majority of the failure occurred in the rock and in the second method at the interface between shotcrete and rock. Presented adhesion strength in [71] is the mean value for all tests with no respect to failure mode. These results probably indicate that water jetting removes more of the damaged rock prior to shotcreting rather than actually increasing the bond strength. Anyhow, the results clearly indicate an increased strength of the interface when water jetting is used.

Chapter 4

Numerical modelling of shotcrete

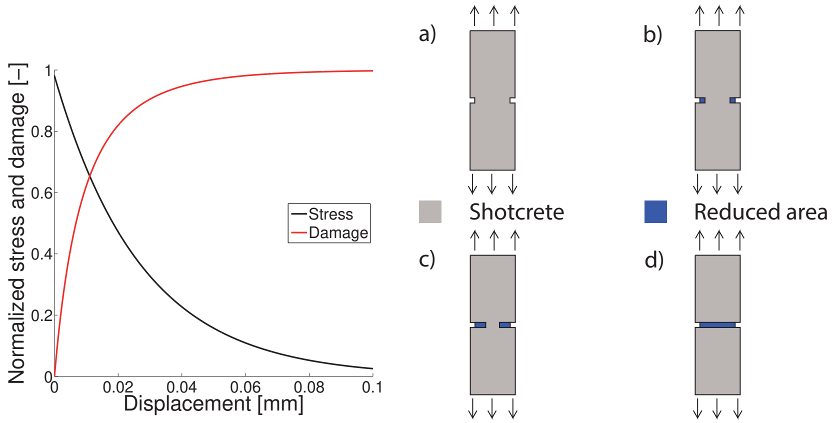
In this chapter, the numerical modelling of shotcrete is described. To accurately describe the behaviour of shotcrete, the model must be able to describe the post-cracking behaviour of both plain and fibre reinforced shotcrete. The interaction, i.e. the bond, between shotcrete and rock is important to accurately describe the composite action of the tunnel lining. The damage and the following breakage of the bond are therefore important to include in the model. A multiphysical approach to model the effects of drying shrinkage is described and finally, a modelling approach to consider the varying thickness based on statistical data is presented.

4.1 Non-linear behaviour

Non-linear behaviour of materials is commonly described with fracture mechanics, damage mechanics or theory of plasticity. In the work by Sjölander and Ansell [93], the concrete damaged plasticity model (CDP) implemented in the FE-software Abaqus [1] was used to describe the non-linear behaviour of unreinforced shotcrete. For the work by Sjölander et al. [94–96], the post-cracking behaviour of shotcrete was described using damage mechanics and this concept is therefore presented in detail below.

4.1.1 Damage model for unreinforced shotcrete

The pioneer work within damage mechanics is commonly acknowledged to Kachanov [64]. The basic concept of damage mechanics is to describe the effects of fracture in the material with a decreased load bearing area which in turn results in a reduced stiffness of the material, see Figure 4.1.



Figur 4.1: Concept of damage mechanics; force-displacement (left) showing a softening response and schematically view of decreased load bearing area a-d (right).

An isotropic damage model based on the work by Oliver et al. [62] has been used to describe the non-linear behaviour of plain and fibre reinforced shotcrete [94–96]. The FE software Comsol Multiphysics has been used and the implementation of the damage model for plain shotcrete is based on the work presented by Gasch et al. [45, 47]. Up to the tensile strength σ_{tf} there is a linear relation between stress σ and strain ε according to Hooke's law. When the strains are increased beyond the elastic limit $\varepsilon_0 = \sigma_{\mathrm{tf}}/E$, damage occurs which is described with a single isotropic damage parameter ω according to:

$$\sigma \;=\; (1-\omega)E\varepsilon \tag{4.1}$$

in which E is the Young's modulus of the undamaged material. The use of a single damage parameter implies that damage is equal in all directions which preserves the isotropic behaviour [62]. In this model, cracking is represented by softening of the whole element. A Rankine strain tensor as described in Eq. (4.2) is used

$$\varepsilon_{\mathrm{eq}} \;=\; \frac{1}{E}\max\langle\sigma_{\mathrm{i}}\rangle, i = 1, 2, 3 \tag{4.2}$$

in which, $\langle\sigma_{\mathrm{i}}\rangle$ is the positive principal stress, i.e. only tension damage is considered in the model. The evolution of damage is controlled by an internal damage variable κ that describes the maximum strain level in the element. To ensure that damage is history dependent, i.e. damage is non-reversible, κ is evaluated as:

$$\kappa \;=\; max(\varepsilon_{\mathrm{eq}}, \kappa_{\mathrm{old}}) \tag{4.3}$$

where κ_{old} is the maximum recorded strain in the element for the previous timestep. To evaluate the damage growth, a Kuhn-Tucker loading/unloading condition is used.

$$f \;=\; \varepsilon - \kappa' \text{ and } f \leq 0,\; \kappa \geq 0,\; \kappa' f \;=\; 0 \tag{4.4}$$

Here, κ' is the derivative with respect to time of the damage variable and therefore, damage is only increasing when the equivalent strain ϵ_{eq} is equal to the previously maximum strain κ in the element. The softening of plain shotcrete is described by an exponential softening function:

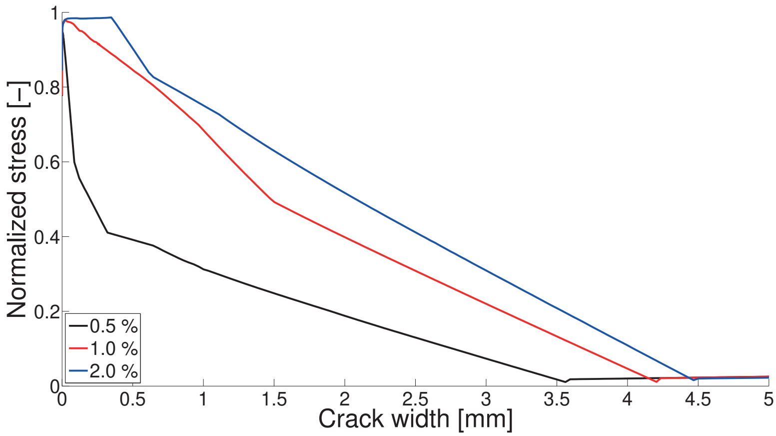
$$\omega(\kappa) = 1 - \frac{\varepsilon_0}{\kappa}\exp - \frac{\kappa - \varepsilon_0}{\varepsilon_{\mathrm{f}} - \kappa} \tag{4.5}$$

where ε_f expresses the ultimate strain and depends on the fracture energy of the shotcrete. An exponential softening function for plain concrete/shotcrete is used by many other researchers, see e.g. [46, 62, 97].

4.1.2 Damage model for fibre reinforced shotcrete

As discussed in Section 2.2, the ductility of the shotcrete increases when fibres are added. In contrast to conventional steel-bar reinforcement, that is systematically placed in a structure, both the orientation and distribution of fibres are random. Since the area of the individual fibre compared to steel bars is considerably smaller, it is obvious that the number of fibres added in the shotcrete mix is significantly higher. Instead of modelling individual fibres, the increased ductility is usually considered by describing the relation between stress crack opening w and σ, i.e. the fracture energy, for different types and amounts of fibres. Many researchers have used bi-linear or tri-linear $\sigma - w$ curves to describe the post-cracking behaviour of fibre reinforced concrete, see e.g. Burrati et al. [28], Olesen [82] and Soetens and Matthys [97]. Guidelines for the design of fibre reinforced concrete are also given by e.g. Rilem [87], ModelCode [42] and SIS, [92]. Test on FRS are normally performed on notched beams subjected to three or four point bending. In such tests, the relation between stress and crack width can be measured. The increased ductility is governed by ability of the fibres to bridge and transfer stresses across the crack. This depends on the amount and orientation of the fibres in relation to the crack. A scatter in test results for FRS is therefore to be expected.

To include the effect of fibres, the softening part of the damage model presented above was further developed. The exponential softening curve used for unreinforced shotcrete was replaced with a tri-linear stress-crack width curve as suggested by e.g. Yoo et al. [110]. The tri-linear curve is flexible and can account for different post-cracking behaviour as visualized in Figure 4.2 where crack width-stress curves for concrete reinforced with 0.5, 1.0 and 2.0 volume % of fibres are plotted, from Yoo et al. [110].



Figur 4.2: Relation between crack width and stress for 0.5, 1.0 and 2.0 volume % of fibres. Results reproduced from Yoo et al [110].

Stress as a function of the crack width $\sigma(w)$ was described with three linear functions, each valid within a specific range of the crack width as:

$$\sigma_i(w) \;=\; \sigma_{\mathrm{fi}} - k_{\mathrm{i}}\kappa \text{ for } w_{\mathrm{i}-1} \leq w < w_{\mathrm{i}} \tag{4.6}$$

in which i defines the intersecting points between the three linear softening curves, i.e. the crack widths $w_1 - w_3$. To simplify the implementation of the stress function, a fictitious stress σ_{fi} was added which represent the stress at zero strain for each linear function, $\sigma_{\mathrm{i}}(w)$. This way, the linear decrease in stresses for each function can be expressed as the derivative of the linear function k_{i} multiplied with the history dependent maximum strain κ as defined in Eq. 4.3.

$$k_i \;=\; \frac{\Delta\sigma}{\Delta\varepsilon} \tag{4.7}$$

To accurately describe the post-cracking behaviour with the presented model, the relation between stress and crack width is required as input. This data is often measured on tests performed on notched beams, see e.g. [37, 89, 109, 110]. Using the crack widths as input instead of strain reduces the mesh dependency. However, both the derivative k_{i} and the resulting stress according to Eq. (4.1) is calculated based on the total strain ε. Crack widths must therefore be converted to strains. This has been done based on the crack width band which has been assumed to be equal to the characteristic element length h_f:

$$\varepsilon_{\mathrm{i}} \;=\; \varepsilon_{\mathrm{t0}} + \frac{w_{\mathrm{i}}}{h_{\mathrm{f}}} \tag{4.8}$$

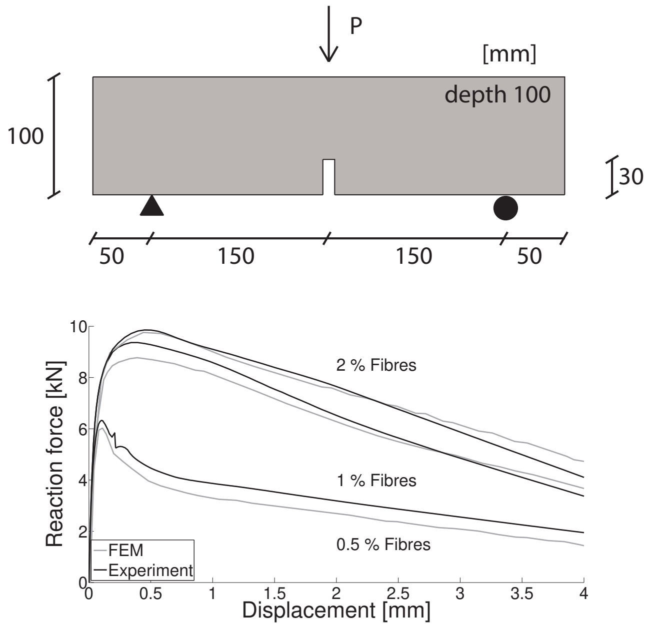
To define the model in terms of total strain, $\varepsilon_{\mathrm{t0}}$ must be added since the the crack width is zero when the tensile strength is reached. The damage function $\omega(\kappa)$ is derived by setting Eqs. (4.1) and (4.6) equal and converting the crack widths to total strain according to Eq. (4.8). This results in three linear damage functions.

$$\omega_1(\kappa) = 1 - \frac{\sigma_{f1} - k_1\kappa}{E \cdot \kappa} \text{ for } \varepsilon_0 < \kappa < \varepsilon_1 \tag{4.9}$$

$$\omega_2(\kappa) = 1 - \frac{\sigma_{f2} - k_2\kappa}{E \cdot \kappa} \text{ for } \varepsilon_1 < \kappa < \varepsilon_2 \tag{4.10}$$

$$\omega_3(\kappa) = 1 - \frac{\sigma_{f3} - k_3\kappa}{E \cdot \kappa} \text{ for } \varepsilon_2 < \kappa < \varepsilon_3 \tag{4.11}$$

The presented damage model for fibres was verified against experimental results from the literature. A test series presented in Yoo et al. [110] was chosen and input for the material models is presented in Table 4.1. A standard three-point bending test of a notched beam was used and force displacement curves for normal concrete reinforced with 0.5, 1.0 and 2.0 volume fraction of fibres are plotted in Figure 4.3.



Figur 4.3: Set-up for experiment (top) and numerical results (bottom) from Yoo et al. [110]

Tabell 4.1: Input for tri-linear damage model for FRS according to [110].

Parameter	Unit	Fibre content		
		0.5 %	1.0 %	2.0 %
σ_{t}	MPa	2.58	3.00	3.15
σ_1	MPa	1.51	2.44	3.15
σ_2	MPa	1.07	1.55	2.66
σ_3	MPa	0.03	0.03	0.03
w_1	mm	0.091	0.649	0.354
w_2	mm	0.32	1.54	0.63
w_3	mm	4.0	4.5	4.5

4.2 Modelling of bond failure

Bond failure between shotcrete and rock was modelled using a cohesive zone model implemented in Comsol 5.2a [38]. The separation between two materials is described with a traction-separation law. Up until failure, the traction σ and separation u of the interface increases linearly with the penalty stiffness K according to:

$$K_{\mathrm{i}} u_{\mathrm{i}} = \sigma_{\mathrm{i}} \tag{4.12}$$

where index i stands for mode I or II displacements, i.e. normal or shear direction. For single mode delamination, failure occurs when either the tensile or shear failure stress is reached. Normally, presented results of the bond strength in the normal direction only contains the failure stress, see e.g [20, 25, 91]. Once the failure stress σ_{fi} is reached, softening occurs until the two interfaces become completely separated. This happens when the failure displacement u_{fi} is reached which is based on the fracture energy of the interface G_{fi} according to:

$$u_{\mathrm{fi}} = \frac{2G_{\mathrm{fi}}}{\sigma_{\mathrm{fi}}} \tag{4.13}$$

The failure displacement in Eq. (4.13) is based on linear softening which has been used for the work with this thesis. Furthermore, the fracture energy was set equal in the normal and shear directions while values for normal and shear bond strength have been taken from Table 3.3. The bond failure is brittle and it has been assumed that the fracture energy of the interface is equal to that of unreinforced shotcrete. Single mode delamination was initially used [93] while mixed mode delamination was considered later [95, 96]. The mixed mode displacement, u_{m}, is calculated based on the mode-I and mode-II displacements $u_{\mathrm{I}}, u_{\mathrm{II}}$ according to:

$$u_{\mathrm{m}} = \sqrt{u_i^2 + u_{II}^2} \tag{4.14}$$

In this thesis a power-law failure criterion has been used and failure occurs when

$$\left(\frac{G_{\mathrm{I}}}{G_{\mathrm{Ic}}}\right)^n + \left(\frac{G_{\mathrm{II}}}{G_{\mathrm{IIc}}}\right)^n = 1 \tag{4.15}$$

with the exponent n set to 2.

4.3 Modelling of drying shrinkage

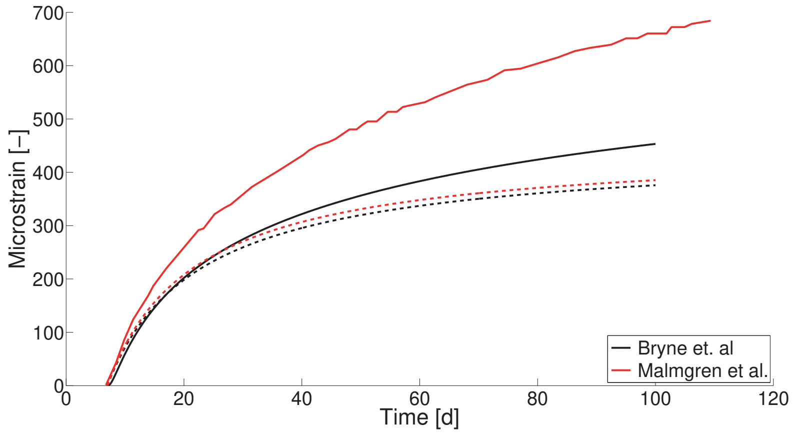
As presented in Section 2.4.1, shrinkage is an important load case to consider since it is a normal cause of cracking in a restrained concrete structure. In this thesis, two different approaches to modelling drying shrinkage have been evaluated and are presented below.

In engineering practise following the guidelines from Eurocode 2 [33] or Model Code [42], shrinkage is considered to be uniformly distributed over the cross-section. The total strain due to shrinkage is calculated based on factors such as type of cement, area exposed to drying and external relative humidity. Shrinkage is here normally divided into autogenous shrinkage and drying shrinkage. Due to the physical mechanism of drying, i.e. moisture transport in the shotcrete is caused by a moisture gradient, drying will always be non-linear. This will cause an internal restraint which is omitted in this simplified approach. Depending on the type of structure these effects may or may not be of importance which was investigated by Sjölander et al. [94, 95]. The predicted amount of shrinkage $\varepsilon_{\mathrm{sh}}$ was then modelled as an equivalent temperature difference δT based on the relation to the thermal coefficient of expansion for the shotcrete α_{t}.

$$\delta T = \frac{\varepsilon_{\mathrm{sh}}}{\alpha_{\mathrm{t}}} \tag{4.16}$$

This model is simple to use and implement in a numerical model but neglects the effects of non-linear drying shrinkage. Furthermore, the predicted amount and rate of shrinkage is not always correctly predicted. This is visualized in Figure (4.4) where results from free shrinkage tests on shotcrete by Bryne et al. [27] and by Malmgren et al. [71] are plotted. The predicted shrinkage according to Eurocode 2 corresponds rather well to experimental results in [27] but underestimates the shrinkage presented by Malmgren [71]. For the later test, no information of cement type is given which could be the source of error for the prediction of shrinkage. To improve the simulation of drying shrinkage and also account for the strain gradient over the thickness, a more detailed modelling approach was used based on multiphysics. This allows that the governing equations for different physical fields such as temperature, moisture transportation and mechanical strain can be solved simultaneously and also have a coupled behaviour. Using this concept, the drying process of shotcrete is described as a pure diffusion process. All transportation of water occurs in the vapour phase and is driven by the gradient in relative humidity $\Delta\phi$. The flux J, i.e. the change in relative humidity over the area, is described by Fick's first law:

$$J = -D_{\mathrm{H}} \cdot \Delta\phi \tag{4.17}$$



Figur 4.4: Free shrinkage based on; experimental results and predictions according to Eurocode 2 [33] plotted with solid and dashed lines, respectively. To the left with T = 20°C and RH = 50 % [27] and to the right, T = 20°C and RH = 65 % [71].

where $D_{\rm H}$ is the diffusivity of the shotcrete which describes the rate of moisture transportation. This is according to Bazant and Najjar [19] described as non-linear function depending on the relative humidity H in the shotcrete.

$$D_{\rm H} \;=\; -D_1[\alpha_0 + \frac{1-\alpha_0}{1+(\frac{1-H}{1-H_c}^n)}] \tag{4.18}$$

When the relative humidity is equal to 1 and 0, i.e. 100 % and 0 %, the diffusivity is described by D_1 and D_0. The factor α_0 is used to describe their relationship as:

$$D_0 \;=\; \alpha D_1 \tag{4.19}$$

When H approaches H_c, the diffusivity suddenly changes several orders of magnitude. The rate of change is described by the parameter n in Eq. (4.18) The change in relative humidity described by Fick's law in Eq. (4.18) is valid for steady state conditions. For a transient event such as drying of shotcrete, Fick's second law must be used.

$$\frac{\partial H}{\partial t} \;=\; D_{\rm H}\nabla\phi \tag{4.20}$$

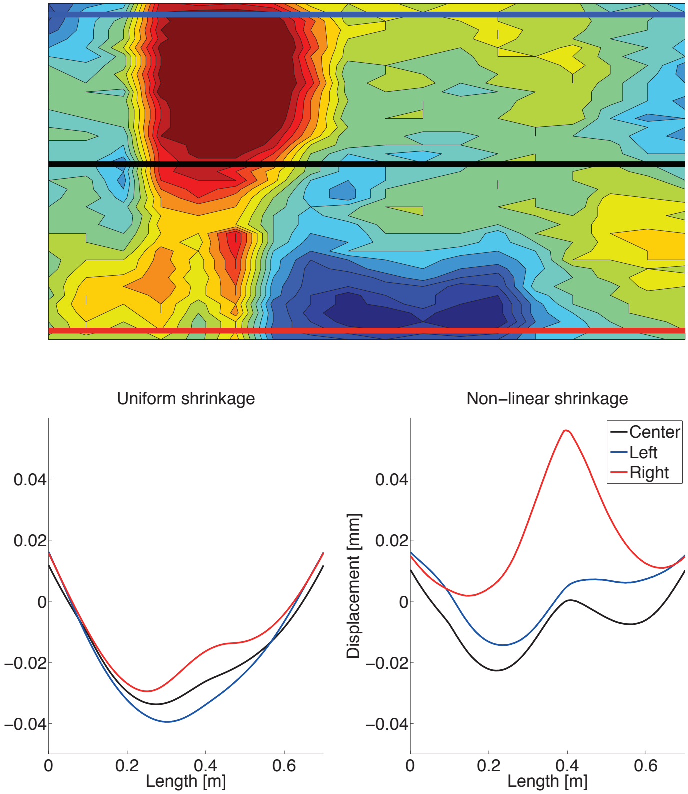
The partial derivative $\partial H/\partial t$ describes the change in relative humidity over time which depends on the second derivative of the relative humidity $\nabla\phi$ multiplied with DH. To complete the moisture model, a boundary condition describing the exchange in relative humidity at the surface and the ambient air is needed. A boundary condition specifies the value of the solution to the differential equations along the boundaries. Different types of boundary conditions and their impact on the solution of the moisture transportation problem are:

- First type boundary condition, Dirichlet boundary condition. This specifies the value of H along the boundary.
- Second type boundary condition, Neumanm boundary condition. This specifies the value of the derivative H' along the boundary.
- Third type boundary condition, Robin boundary condition. This specifies the value along the boundary using a linear combination of H or H'.

Here, a Robin boundary condition has been used. The flux J at the boundary is described by the difference in relative humidity between the surface H and the ambient air H_{env} multiplied by the surface factor β_{h} which describes the rate of exchange in moisture between the surface and the ambient air. Finally N in Eq. (4.21) is the normal to the surface.

$$-J \cdot N = \beta_{\text{h}} \cdot (H - H_{\text{env}}) \tag{4.21}$$

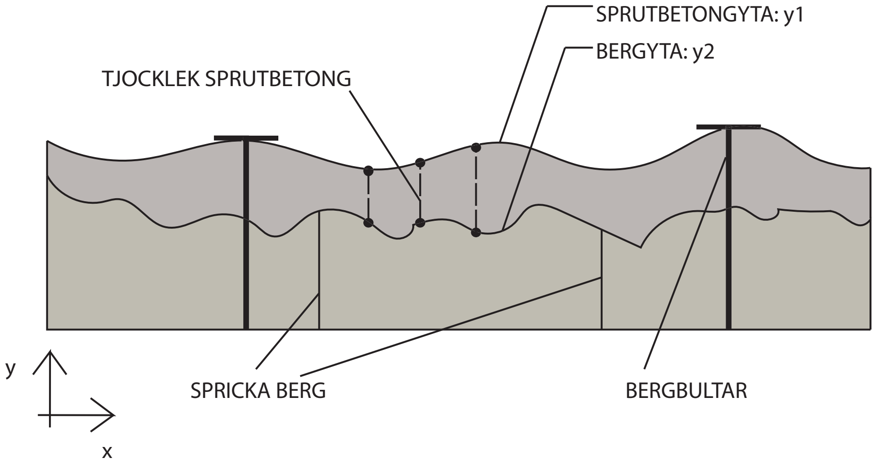
To visualize the difference in structural behaviour between the two presented models, the vertical displacements for a 3D end-restrained slab are plotted along three different lines, shown at top in Figure 4.5. The results for uniform shrinkage which corresponds to the temperature model are plotted at the bottom left in the figure while results for non-linear shrinkage are plotted at the bottom right. As can be seen in the figure, there is a clear difference in vertical displacements between the two models. The internal restraints caused by the non-linear drying affects the displacement and the effects of non-linear shrinkage should be considered to accurately describe the structural behaviour.



Figur 4.5: Top part of the figure shows topography of S5 with variation in thickness from 20 mm (blue) to 58 mm (red) and result lines for vertical deformation. Bottom part of the figure shows vertical deformation for the end-restrained slab subjected to linear and non-linear shrinkage. Positive deformation indicates that the slab is rising.

4.4 Modelling of varying thickness

As shown in Section 3.2, significant variations in shotcrete thickness are to be expected in situ. Previous research focusing on shotcrete thickness has been based on periodic or harmonic functions to describe its variation. This is not very representative to the actual variation. A more suitable method is to use a probabilistic approach, where the thickness of the shotcrete is generated by random numbers that follow the representative distribution in situ. In the work by Sjölander et al. [96], Comsol Multiphysics was used to create a 2D geometric model with three rock blocks, schematically shown in Figure 4.6. The surface of the rock has been modelled with an irregular surface which is representative for tunnels constructed with the drill and blast method. The shotcrete was given a harmonic surface which is commonly found in situ. The variation in thickness is based on statistical data but there is no correlation between the rock surface and the thickness of the shotcrete since no such data is available.



Figur 4.6: Schematic view on the build-up of the irregular thickness

The length of the three blocks is described by the x-coordinate while the y-coordinate represents the height (thickness) of the rock and shotcrete. The shotcrete surface was given a harmonic shape described by:

$$y_1(x) \;=\; h_\mathrm{m} + 0.025 \cdot \sin\frac{x}{0.25} \tag{4.22}$$

In Eq. (4.22), h_m is the selected target thickness of the shotcrete that is given a periodic offset using a sine function. The function in Eq. (4.22) can easily be changed to any other representative expression. The rock surface was assumed to have an irregular surface described by:

$$y_2(x) \;=\; y_1(x) - h_\mathrm{m} R(x) \tag{4.23}$$

The variable $R(x)$ in Eq. (4.23) is a function that generates random numbers to scale the selected target thickness h_m so that a variation in thickness can be achieved. To generate this function, a representative distribution of the thickness such as the one presented in Figure 3.8 [100] is implemented in Comsol. This distribution must be normalized against the in situ target thickness. The thickness specified by Eq. (4.23) is adjusted in a specified number of points along the length x. In order to generate random numbers, Comsol uses pseudorandom numbers which implies that if the analysis is repeated using the same x-vector, the exact random numbers will be reproduced by $R(x)$. Several tricks to overcome this problem exist if a probabilistic analysis shall be performed where new random numbers must be generated for each analysis.

Chapter 5

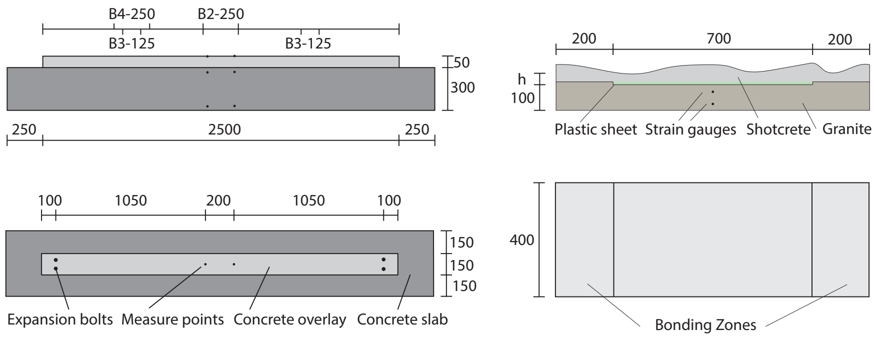
Numerical examples

In this chapter, additional numerical results to Paper I-IV are presented and discussed. The basic concepts of the studied cases and numerical models are explained but for further details, see respectively paper or Chapter 4.

5.1 Uniform shrinkage for continuously restrained shotcrete

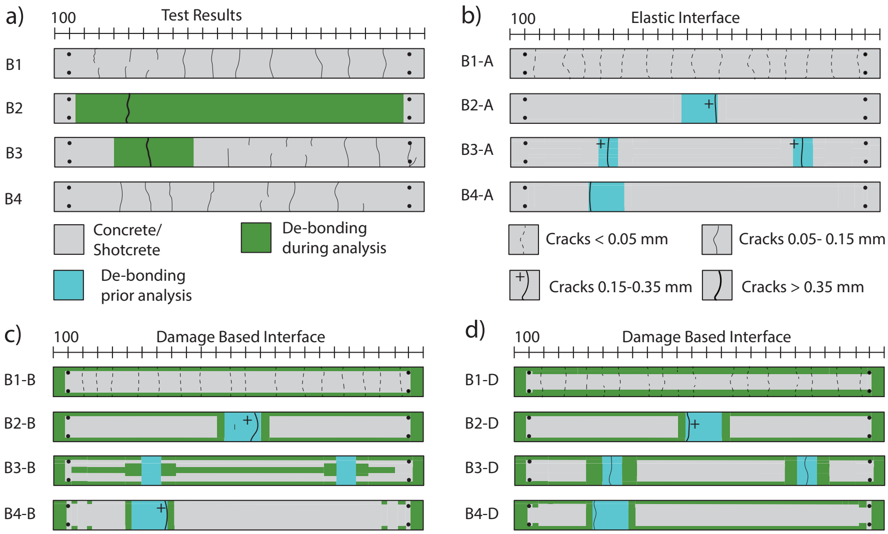
Restrained shrinkage of shotcrete is a common reason for cracking. Experimental results have shown that unreinforced concrete and shotcrete with a continuous bond to a stiff substrate can exhibit a strain hardening structural behaviour, see e.g. Carlswärd [29] and Malmgren [71]. This means that finer and narrow cracks form instead of few and wide cracks which is beneficial from a structural and serviceability point of view. The mentioned studies investigated the response of beams with uniform thickness and for such beams, cracks tend to form with a systematic distance between them. No results were found in the literature where the influence of a varying thickness was studied. The irregular geometry introduces stress concentrations which could affect the number of formed cracks and the distance between them. This was investigated by numerical simulations in Paper I for shotcrete that was either unreinforced or reinforced with micro glass fibres. Four-point bending tests of glass fibre reinforced shotcrete performed by Bryne showed little or no increased ductility compared to unreinforced shotcrete. The reason for adding glass fibres was that this had prevented the formation of cracks due to restrained shrinkage in ring test [9]. The effect of partial debonding with respect to cracking was also investigated.

Material properties for the shotcrete and the bond strength between shotcrete and granite were chosen from [24] and the development of material strength with respect to time was not considered. The structural behaviour of the FE-model was verified against an experimental set-up by Carlswärd [29] in which concrete overlays were cast on top of a concrete slab. The length × width × thickness of the overlays were 2500 × 150 × 50 mm. After casting the concrete was cured under an air tight plastic sheet for five days. The slabs were kept indoors during the test period kept indoors at a rather stable temperature of 18°C and RH = 15-20 % and were monitored for over 100 days. The displacement of the overlays and the slab was measured between glued metal points on the concrete and using a mechanical instrument of type Staeger, giving an accuracy of 1 micrometer [29]. The effect of different surface treatments before casting was investigated as well as the difference in structural behaviour between unreinforced, fibre reinforced and steel bar reinforced concrete overlays. Geometric data from experiments presented by Bryne et al. [27] was used to study the influence of a varying thickness. The set-ups for both experiments are presented in Figure 5.1.

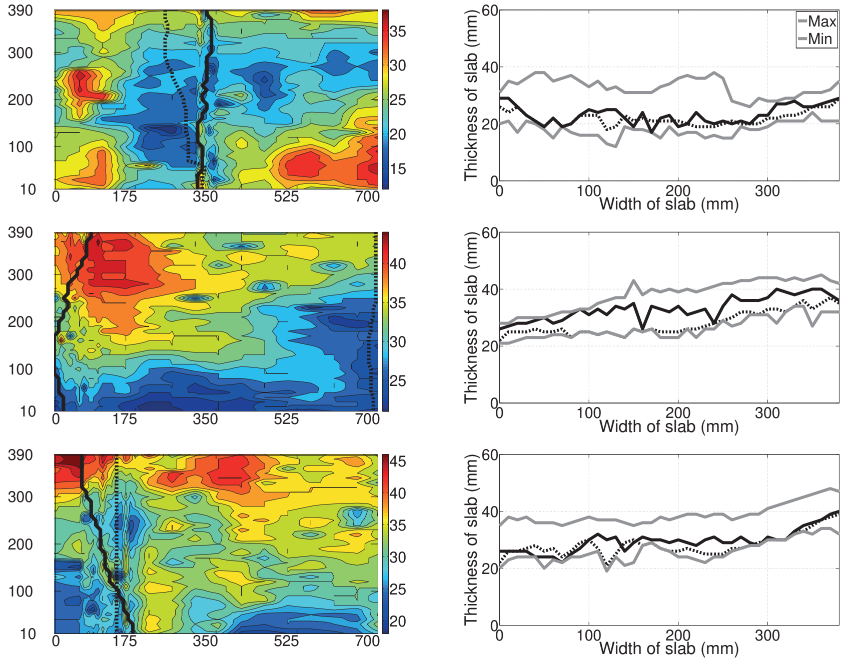


Figur 5.1: Experimental set-up for verification of the FE model from Carlswärd [29] (left) and to study the influence of a varying thickness, from Bryne et al. [27] (right).

Results presented in Figure 5.2 show that the numerical model was able to capture the structural behaviour from the experiment. A systematic pattern of fine cracks developed when the bond was continuous and partial debonding lead to the formation of few and wide cracks. Bond failure only occurred around the perimeter of the overlay and no failure similar to those in the experiments where captured by the model. This could be due to that bond failure was modelled using a single stress failure criterion, i.e. failure either occurs if the tensile or shear strength was exceeded and mixed mode failure was not considered. Furthermore, the bond strength was uniform and no local weak spots were considered. In reality, variation in bond strength exists. This is clearly indicated by test results from [29] which show that the standard deviation in bond strength was between 0.5 and 2.0 MPa for the various surfaces.

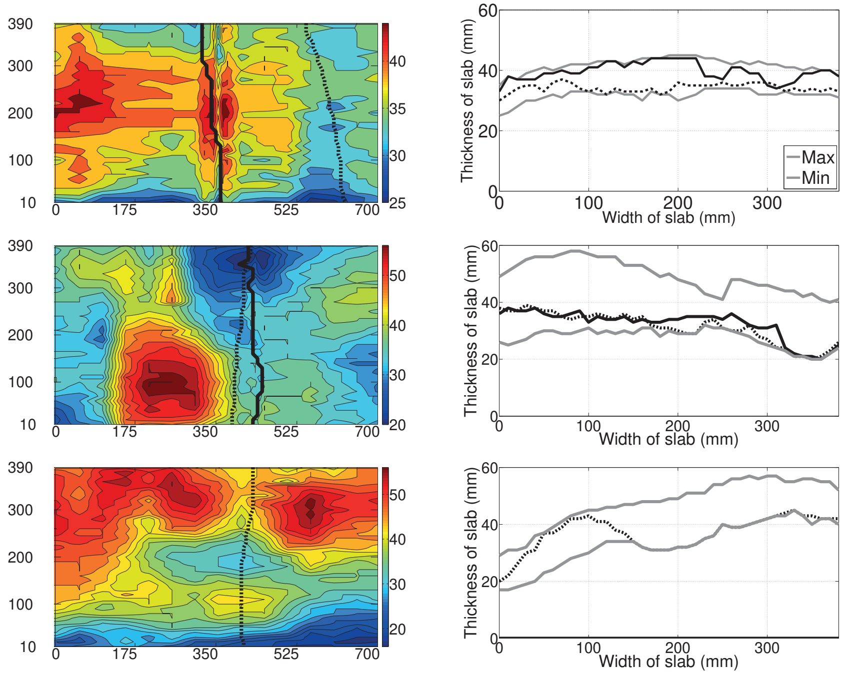


Figur 5.2: Selected results from experiments by Carlswärd [29] (top left) and numerical simulations from Paper I.



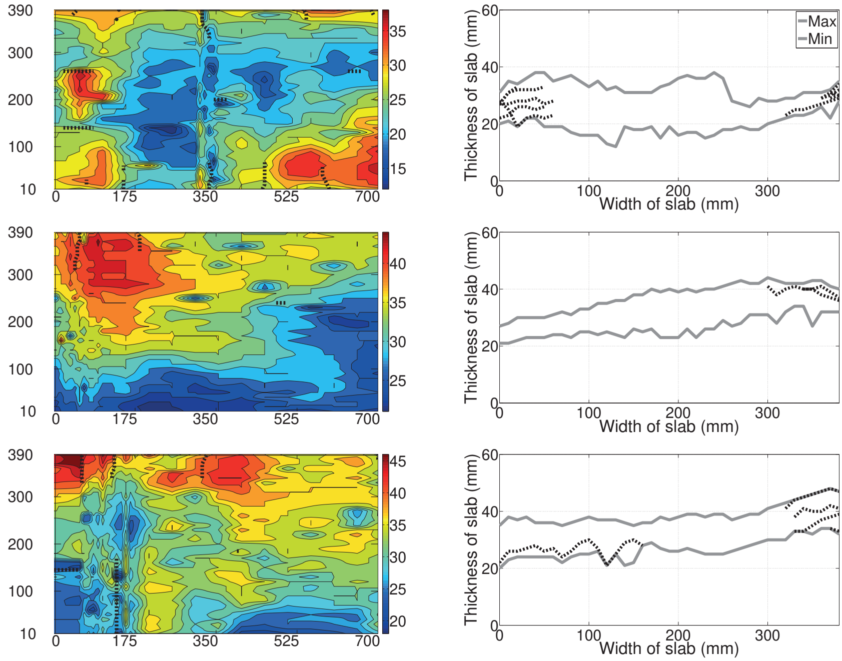
Figur 5.3: Formation of cracks and their location with respect to minimum (blue) and maximum (red) thickness of the slab from Paper I for S1-S3 (top to bottom). Solid and dashed lines show experimental and numerical results, respectively.

In Figure 5.3 and 5.4, results for the end-restrained slabs with varying thickness are presented. A uniform temperature field was applied to the slabs to study the effects of restrained shrinkage. The topography are plotted in left part of the figures which show the variation in thickness and the location of the cracks. The right part of the figures show the maximum and minimum thickness along the width of the slab as well as the thickness of the shotcrete along the cracks. Experimental and simulated results are plotted with solid and dashed lines, respectively. For all slabs, one through crack formed and propagated along the width to finally separated the slab into two parts. After cracking, the slabs were fully unloaded.



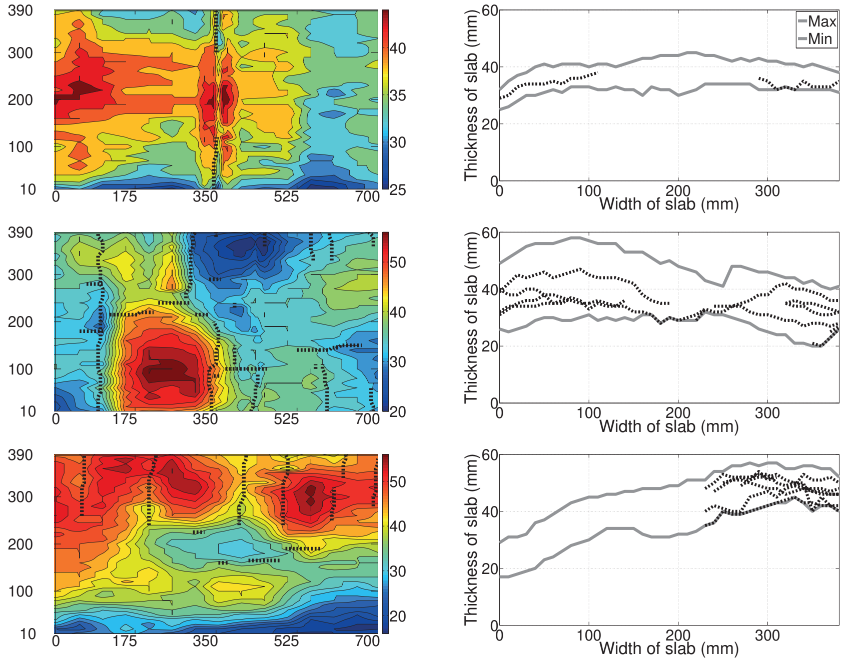
Figur 5.4: Formation of cracks and their location with respect to minimum (blue) and maximum (red) thickness of the slab from Paper I for S4-S6 (top to bottom). Solid and dashed lines show experimental and numerical results, respectively.

The simulated cracks are normally going through sections with low thickness which is partly true for the experimental slabs. The crack patterns for S2 and S3 are both propagating through areas with higher thickness and in both cases, the simulated crack pattern is more reasonable. However, in the numerical model the tensile strength is uniform in the whole slab and no local weak spot exists. In reality the tensile strength is not completely uniform which could explain the deviations in crack patterns. Another explanation discussed by Bryne [24], is that an unfavourable distribution of large aggregates could affect the crack pattern. In this case, the maximum aggregate size was 8 mm and with a slab thickness between 20 and 50 mm, the distribution of large aggregates could have a significant impact on obtained crack patterns.



Figur 5.5: Formation of cracks and their location with respect to minimum (blue) and maximum (red) thickness of the slab from Paper I for S1-S3 (top to bottom). Solid and dashed lines show experimental and numerical results, respectively.

In Figure 5.5 and 5.6, numerical results for the continuously restrained slabs with varying thickness are presented. Results are presented in the same way as above, with the topography and crack patterns in the left part of the figures and the thickness in the right part. Only cracks wider than 0.05 mm are plotted and no damage could occur at the interface, i.e. debonding was not considered.



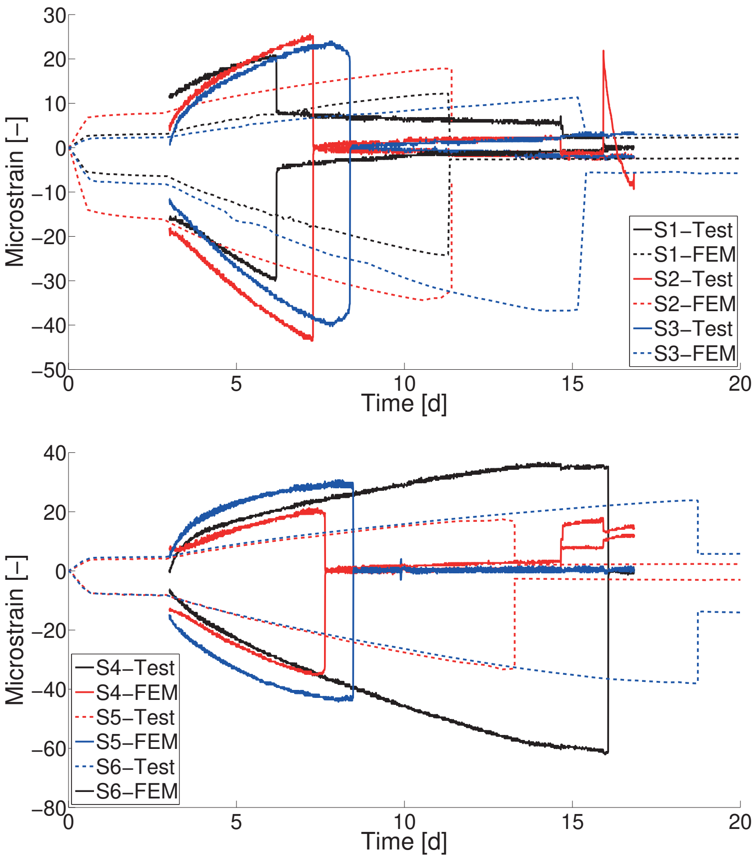
Figur 5.6: Formation of cracks and their location with respect to minimum (blue) and maximum (red) thickness of the slab from Paper I for S4-S6 (top to bottom). Solid and dashed lines show experimental and numerical results, respectively.

For a continously restrained concrete beam, a systematic pattern of fine and narrow cracks could be obtained even for unreinforced concrete, see results from Carlswärd [29] presented in Figure 5.1. The strain-hardening behaviour comes from the reinforcing effect by the substrate and thinner beams will therefore show a finer distribution of cracks according to Groth [48]. Slabs with a varying thickness also shows a strain-hardening behaviour but the crack patterns will not be as systematic as when the thickness is uniform. The influence of the thickness and its variation with respect to crack patterns are visualized by the result in Figure 5.5 and 5.6. If the amplitude variation is moderate, fewer and longer cracks form compared with a case with larger amplitude variations, compare Figure 5.6 top and bottom.

5.2 Drying shrinkage for end-restrained shotcrete

Drying shrinkage, as described in Section 2.4.2, depends on factors such as relative humidity of the ambient air and thickness of the shotcrete. Drying of a shotcrete with varying thickness results in a strain gradient not only through the thickness but also in the plane perpendicular to the thickness. The experimental set-up presented by Bryne et al. [27] shown to the right in Figure 5.1 was again used for comparisons of numerical results. The reason for reusing the experimental set-up is that no other results for an end-restrained shotcrete slab with varying thickness subjected to drying shrinkage was found. The set-up in [27] consisted of a sawn-out granite slab with a flat surface. The length $\times$ width $\times$ thickness of the overlays were 1100 $\times$ 400 $\times$ 100 mm, respectively. A centric area of 700 $\times$ 400 mm was covered with two layers of plastic sheets to simulate the structural effects of drain-mats, i.e. no bonding occurs and the shotcrete can move freely over the mats. After shotcreting, the slabs were stored in a climate chamber with T = 20°C and RH = 50 % for the duration of the test. A total of six slabs were sprayed and the mean thickness of the shotcrete varied between 25 and 41 mm [27]. For the first three days, the slabs were covered with a wet cover to prevent early drying shrinkage. The wet cover was removed after 72 h and strain gauges were placed close to the upper and lower surface at the centre of the granite slab.

In the work with Papers II-III, the capability of describing the structural effects of restrained drying shrinkage was investigated using two different numerical approaches. In the first model (denoted FEM-T), drying shrinkage was considered to be uniform over the cross-section which is a common practical engineering approach to describe drying shrinkage. The amount of shrinkage was based on guidelines for ordinary concrete according to Eurocode 2 [33]. In the second model (denoted FEM-M), the drying process was described using the moisture transportation model presented under Section 4.3. This model was tuned according to test of free shrinkage presented by Bryne et al. [27]. The main difference between the two models is that the first will consider the effects of non-linear shrinkage while for the latter shrinkage is considered to be uniform over the whole thickness. Even though the thickness of the slabs was between 12 and 58 mm, a significant strain gradient could occur for one sided drying condition which was shown in [29] for concrete overlays with a thickness of 50 mm. In both numerical models, cracking and debonding of the shotcrete was modelled according to Section 4.1.1 and 4.2, respectively. The development of tensile strength, Young's modulus and fracture energy was based on their relation to the compressive strength as presented in Eurocode 2 [33] and ModelCode [42] while the development of compressive and bond strength was described according to test results presented in Section 2.2.



Figur 5.7: Experimental and numerical results from [24] and Paper II for slabs 1-3 (top) and slabs 4-6 (bottom). Experimental results are plotted with solid lines, simulations using the moisture transportation model with dashed lines and shrinkage according to Eurocode 2 with dotted lines.

In Figure 5.7, experimental results from Bryne [24] and Paper II are presented. The time of failure for both the simulated cases is rather similar but underestimates the experimental time of failure significantly for all cases except for slab 4. There is a difference in localization of the crack between the two models which together with fibre content, the time of failure and measured thickness is presented in Table 5.1. Here it can be seen that for the moisture transportation model (FEM-M), cracks form between the bonding zones which were also the case for most of the experimental slabs. For the model using uniform shrinkage (FEM-T), cracks normally form close to or at the bonding zones. The difference in crack localizations is believed to be due to a combined effect of a varying thickness and the non-linear

Tabell 5.1: Amount of glass fibre reinforcement, time at failure due to shrinkage and measured thickness for Slab 1-6.

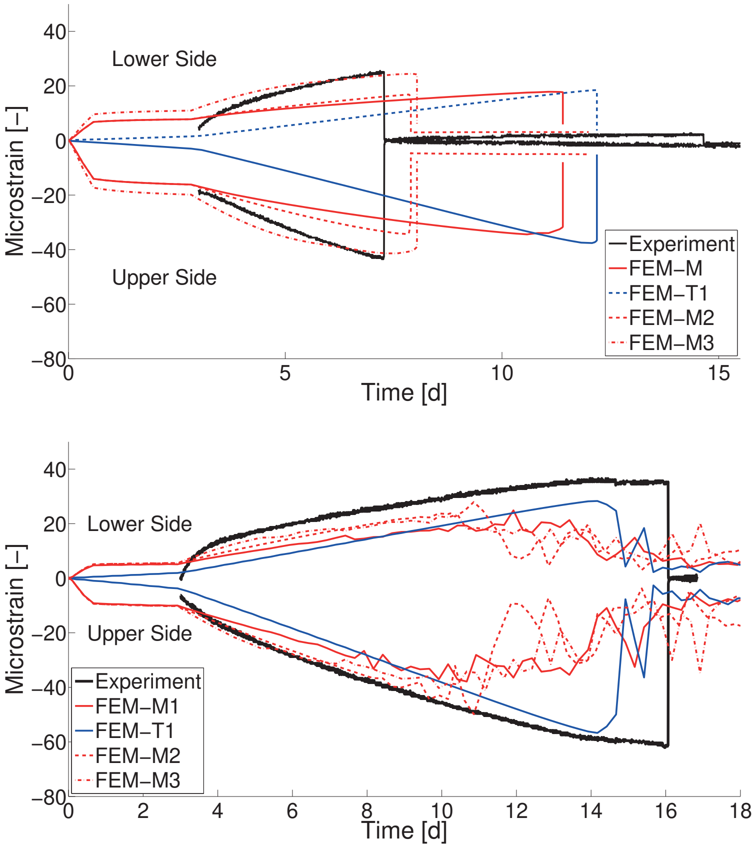
Slab	Fibres	Time of Failure [d]			Thickness [mm]		
	[kg/m^3]	[Test]	[FEM-M]	[FEM-T]	Min / Max	Mean	St.D
S1	0	6-C	11-C	12-CS	12 / 38	25	5
S2	0	7-CS	15-CS	12-CS	21 / 45	33	6
S3	5	6-C	15-CS	13-CS	19 / 48	32	5
S4	5	16-C	16-C	15-CS	25 / 45	37	4
S5	10	7-C	13-C	15-C	20 / 58	37	7
S6	10	6-B	18-C	16-CS	17 / 57	41	9

B = Bond failure, C = Cracking between bond zone, CS = Crack along bond zone

shrinkage. The part of the slab between the bonding zones can move freely in the vertical direction and the effects of non-linear shrinkage introduces a bending moment in the slab thus increasing the tensile stresses between the bonding zones.

As described above the moisture model was tuned according to free shrinkage tests performed using the same shotcrete recipe, see Table 2.1, and the same environmental conditions as the end-restrained tests. Even though, the model was unable to describe the rate of drying correctly and the reason for this could be due to any, or a combination of the following reasons:

- The shotcrete specimens were cured under water and hence fully saturated when the measurements of free shrinkage started. This could affect the initial rate of drying for the free shrinkage tests compared with the end-restrained test where the shotcrete was cured under a wet cover.
- The free shrinkage test gave reliable results first after six full days of curing, while measurements on the end-restrained tests started after three days. During this time the development of material strength and stiffness is significant which could affect the rate of drying or the magnitude of the stress and strain it creates.
- A linear relation between change in relative humidity and development of strain in the shotcrete was used. This relation might be dependent on the relative humidity or the development of strength and stiffness in the shotcrete which would explain the more rapid initial shrinkage.
- The early development of bond strength also implies that the end-restrained shrinkage will occur in a stiff structure and as discussed by Lagerblad et al. [66], this could affect the porosity of the shotcrete which could change the rate of drying when comparing specimens subjected to free and end-restrained shrinkage, respectively.

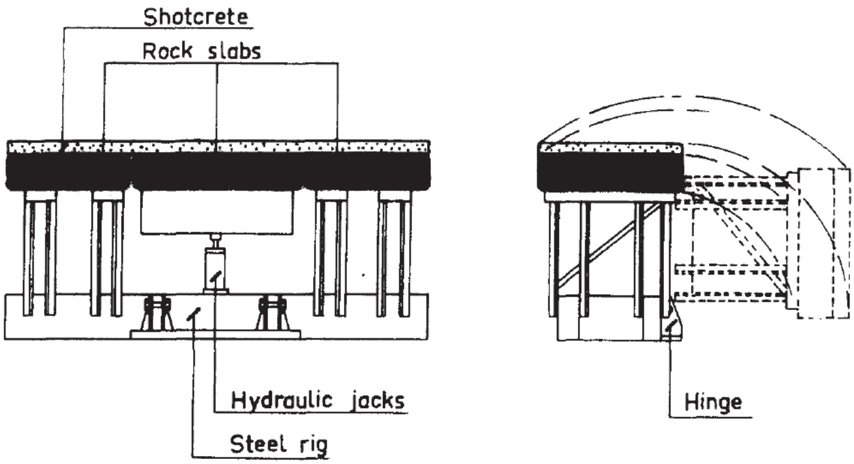


Figur 5.8: Experimental and numerical results from [24] and Paper II for S2 (top) and S4 (bottom).

To gain a better match in the results and to investigate possible explanations to the mismatch in time of failure, two alternative models were tested. In the first model, drying was considered to be two-sided, i.e. drying also occurs towards the side facing the granite. The surface factor which controls the moisture exchange between the surface and the ambient air was for this side set to 50 % of the value towards the free surface. In the second model, the diffusivity was increased with a factor four at the start of the drying $t = 3$ days and exponentially decreasing towards the original value at $t = 6$ days. The surface factor was also increased with a factor two. These results are presented in Figure 5.8. It can be seen that the best fit for the experimental results for S2 is obtained with the non-linear function of the diffusivity.

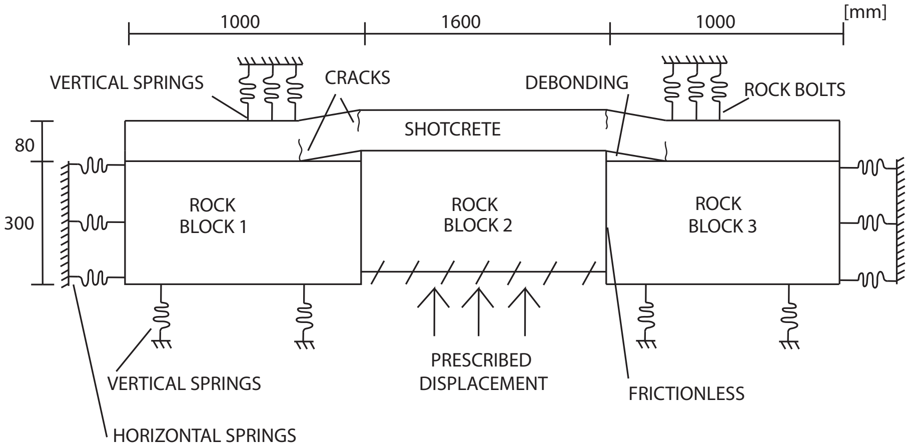
5.3 Gravity load from block

In the work with Paper IV the load carrying capacity influence of an irregular shotcrete lining subjected to load from a pushing block was investigated. The basic concept for the numerical model was taken from an experimental set-up presented by Holmgren [53, 55]. Here, large scale testing of unreinforced, FRS and mesh reinforced shotcrete subjected to a load from a pushing block was simulated using three rock blocks and a hydraulic jack, see Figure 5.9. The aim with the paper was to create a numerical framework suitable for a probabilistic design approach and a 2D numerical model was therefore created. Doing such a discretization leads to a problem regarding the modelling of the rock bolts. The depth of the model was 1200 mm and spherical steel washers with a diameter of 160 mm were used. The bolts are important to include in the model since it will govern the length of possible debonding. In all the experimental results presented in [55], debonding started at the rock joints and propagated towards the location of the washers. Cracking due to bending then followed and the final failure mode was the propagation of major cracks near the rock joints and washers, see Figure 5.10. When a depth of the 2D mode is specified the washers will be extended to cover the whole depth of the model and thus increasing the stiffness significantly. A direct comparison between experimental and numerical results will therefore not add any interesting results and instead the structural behaviour of the model was verified against the experiment.

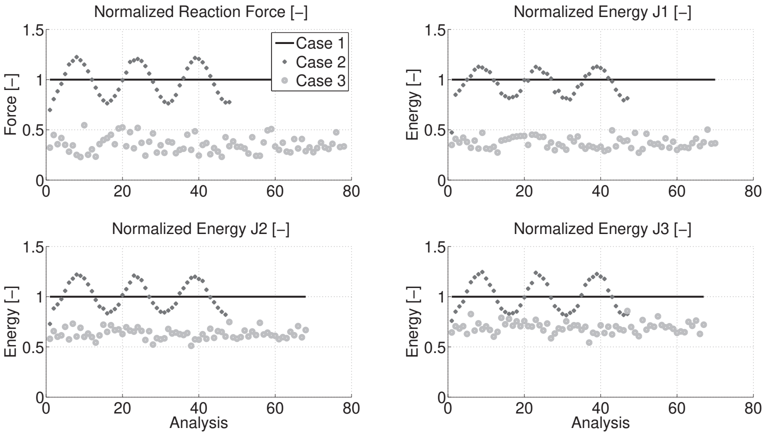


Figur 5.9: Experimental set-up from Holmgren [53, 55].

The non-linear behaviour of FRS was modelled according to Section 4.1.2 and the irregular geometry according to Section 4.4. The amount of fibre reinforcement was set to 80 kg/m^3(1 %) and input for the material model was taken from Yoo et al. [110] as presented in Table 4.1. The variation in shotcrete thickness was assumed to belong to a log-normal distribution as presented by [100] and shown in Figure 3.8. Three linings were compared; uniform thickness, regular rock surface and sine shaped shotcrete surface and irregular rock surface and sine shaped shotcrete surface. The presented results are the peak normal force and energy levels J1-J3. The energy levels represent the area under the force-displacement diagrams for the three displacement limits, 1, 5 and 10 mm. All results are normalized against the results for the lining with uniform thickness, see Figure 5.11.



Figur 5.10: Schematic view of structural behaviour and boundary conditions for the model, from Paper IV.



Figur 5.11: Normalized normal force and energy levels J1-J3 for FRS with 80 kg/m^3(1 %), from Paper IV.

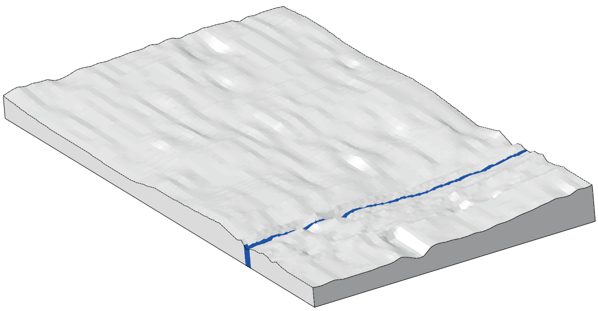
Chapter 6

Summary of scientific papers

6.1 Paper I: Numerical simulations of restrained shrinkage cracking in glass fibre reinforced shotcrete slabs

Andreas Sjölander and Anders Ansell

The focus of Paper I was to establish a method where geometric field data could be used to generate a numerical model. For this purpose, experiment results presented by Bryne et al. [27] were used. Shotcrete was sprayed against a flat granite surface and the thickness was measured in around 700 points. The software HyperMesh [4] was used to generate a 3D solid mesh from the geometric data. The mesh was imported to Abaqus [1] and a uniform temperature field was applied to investigate the difference in structural behaviour between an end-restrained and a continuously restrained shotcrete slab with varying thickness.

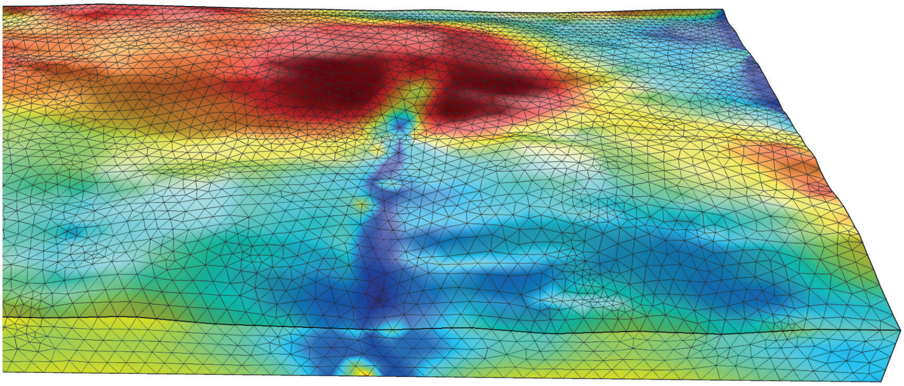


Figur 6.1: Simulated crack pattern for an end-restrained shotcrete slab subjected to uniform shrinkage.

6.2 Paper II: Investigation of non-linear drying shrinkage for end-restrained shotcrete with varying thickness

Andreas Sjölander and Anders Ansell

In Paper III the number of studied cases was limited and the numerical simulations could not accurately predict the time of failure. In Paper II, the parameters controlling the moisture transportation model were revised to increase the accuracy. Debonding between shotcrete and rock as well as an increased diffusivity through cracks were also added to the model. In practical engineering design, shrinkage is normally predicted according to guidelines, e.g. Eurocode 2 [33] or ModelCode [42], and modelled as a uniform temperature load. Therefore, a comparison was made between the predicted time of failure for the moisture transportation model and the temperature model. To accurately capture the rate of early shrinkage, the diffusivity was increased at the start of the drying and the decreasing using a non-linear function.

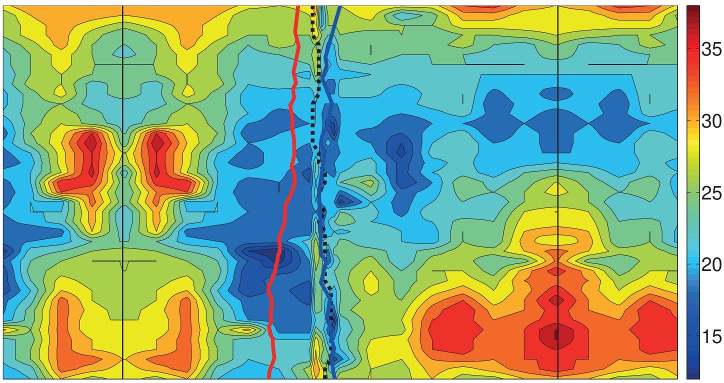


Figur 6.2: Normalised maximum principle stress just at the onset of cracking. Red colour indicates stresses equal to the tensile strength and blue fully unloaded elements.

6.3 Paper III: Shrinkage cracking of thin irregular shotcrete shells using multiphysics models

Andreas Sjölander, Tobias Gasch, Anders Ansell and Richard Malm

In Paper I, shrinkage was modelled as uniformly distributed over the cross-section. As drying shrinkage was identified as the governing mechanism for early cracking of shotcrete, the aim with Paper II was to implement and use a refined numerical model to describe drying shrinkage. Using the software Comsol Multiphysics [38], drying of shotcrete was modelled as a pure diffusion process with the relative humidity as the driving force. Moisture migrates toward areas with lower relative humidity and drying is thus dependent on the thickness. Due to the varying thickness, this results in a strain gradient not only through the thickness but also in the horizontal planes. A linear relation between the reduction in internal humidity and mechanical strain was used and the implementation of the model is based on the work presented by Gasch [47]. The model was used in an attempt to reproduce experimental results from [27] and the development of material strength and stiffness was modelled as time-dependent.

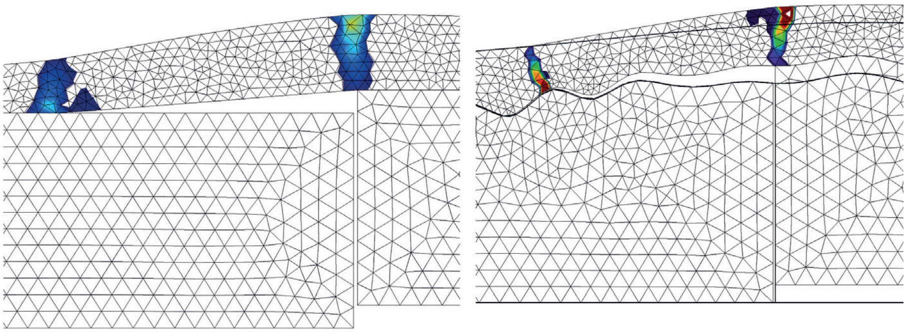


Figur 6.3: Topography showing variation in thickness and simulated crack patterns for an end-restrained shotcrete slab subjected to drying shrinkage.

6.4 Paper IV: On failure probability of thin irregular shotcrete shells

Andreas Sjölander, William Bjureland and Anders Ansell

As shown in Section 3.2, a great variation in shotcrete thickness is to be expected and the distribution could according to Sunesson [100] be described with a log-normal distribution. Considering the in situ variation in not only thickness but also in tensile and bond strength, a probabilistic design seems to be more reasonable than a deterministic one. The aim of Paper IV was therefore to implement a numerical model suitable for a probabilistic design. The isotropic damage model used for unreinforced shotcrete in Papers II-III was further developed to account for the effects of fibres. To model the varying thickness in a realistic way, the shotcrete was given a harmonic outer surface in form of a sin-wave while the rock surface was given an irregular surface. As a case study, the load and energy capacity for three different shotcrete linings were compared. The results showed the capability of the model to be used for a probabilistic design as well as the importance of considering the effects of an irregular geometry when a shotcrete lining is to be designed for the gravity load from a block.



Figur 6.4: Development of cracks for a shotcrete slab with periodic varying thickness (left) and irregular varying thickness (right)

Chapter 7

Discussion and conclusions

This chapter contains a general discussion and conclusions regarding this report. Some interesting further topics to study within this project are discussed and other interesting topics are mentioned.

7.1 Discussion

After the work with this thesis, one difficult aspect still is what loads to consider in the design of a shotcrete lining. There are few reported failures of tunnels in the literature and most of the reported cracking seems to be related to shrinkage. As mentioned by Holter [59], there are few reported measurements of tunnel linings in the literature. Therefore, it is difficult to understand which loads that really are affecting the lining. The presented measurements in [59] also show that the stresses in the shotcrete are low compared to its capacity. To increase the knowledge more field measurement should be performed. To be able to optimize the design of the lining we must first understand what we shall optimize it for. The typical load to consider in the design in the ultimate limit state is the gravity load from a block. The size of the block could be estimated based on existing joint patterns or centre distance between bolts. Shotcrete is assumed to carry the load from the block through bending between the bolts. According to the Swedish standards for design of traffic tunnels [103], this load must be considered. However, it might be more reasonable to decide this from case to case depending on when the shotcrete is installed. A block tends to fall down as soon as it is loose [51] and if the shotcrete is not installed at the tunnel front any potentially loose block should have been falling down either due the effects of blasting or the mechanical scaling that should follow each blast round. If no gravity load from blocks exists, the only loads that affect the shotcrete might be internal loads caused by restrained shrinkage or expansion. However, shotcrete might still play an important part for the rock reinforcement due to the mortar effect and the bond between shotcrete and rock. The high pressure enables shotcrete to penetrate and partly fill out cracks. This creates a contact pressure between the blocks which reduces the possible rotation of the blocks and therefore stabilizes the arch [99]. Shotcrete might be unable to penetrate and fill out fine cracks but and in such case the bond will be important to stabilize the arch.

Presented numerical studies in [93–95] and experiments [27, 70] show that shrinkage causes cracking of a restrained shotcrete structure. It has been shown in [93] that the number and width of the cracks that develop depend on the bond to the substrate. Even though stress concentrations develop in a slab with varying thickness multiple cracks form if the bond to the substrate is continuous. For slabs with uniform thickness, cracks tends to develop with a regular distance, see e.g [29]. This is not always the case for slabs with varying thickness and the crack distance seems to be influenced by the variation in thickness. The results presented by Sjölander and Ansell [93] also confirmed the experimental results presented by Malmgren [70] and Carlswärd [29] that a small area of local debonding leads to formation of large cracks. However, as discussed in Groth [48], it is hard to determine whether cracking follows debonding or vice versa. Nevertheless, these failures seem to be linked and occurrence of cracks with large width should indicate areas of bond failure.

In the work presented by Sjölander et al. [94, 95], an attempt was made to reproduce experimental results presented by Bryne et al. [27], see Section 5.2. End-restrained slabs were sprayed and subjected to drying until cracking occurred. To account for the effects of non-linear drying a more refined model, see Section 4.3, was used to describe the drying shrinkage of the shotcrete. The parameters that control the moisture transportation model were determined to fit experimental data for a free shrinkage test performed using the same shotcrete mixture. Even though a very good fit to this free shrinkage test was achieved the model was unable to describe the drying rate of the end-restrained test. There are some differences between the test set-up and drying conditions that could have affected this. The test specimens for the free shrinkage test were water cured for three full days and were hence fully saturated when the test began while the end-restrained slabs were cured under a wet cloth for three days. Reliable results for the free shrinkage test were obtained after six days. The moisture transportation model was thereby verified for shotcrete that had been cured for six days and then used to describe the moisture transportation for shotcrete that had been cured for only three days. During the first days, a rapid development of material strength and stiffness occurs which affects the composition of the material and could affect the rate of drying. Furthermore, it is possible that plastic shrinkage has occurred for the end-restrained slabs since these were not stored in water. This could lead to formation of micro-cracks which could increase the rate of drying.

7.2 Conclusions

Based on the presented failure review, it can be concluded that failures in tunnels are uncommon but cracking due to restrained shrinkage can often be found. Shrinkage is therefore an important load case to consider in the serviceability limit state. Since shotcrete normally is applied without the use of any joints, cracking caused by shrinkage is inevitable. This is primarily causing a maintenance problem, but since FRS used in tunnels normally has a strain-softening behaviour, pre-existing cracks could affect the structural capacity of the lining. For the design in the ultimate limit state, the theoretically most important load is the gravity load from a block. However, the literature review has shown that failures including a single falling block is rare. The reported failures from Norway were major cave-ins in which a large amount of rock material fell into the tunnel. This was caused by a severe underestimation of the quality of the rock and was not initiated by a single loose block. It is, however, likely that minor accidents with falling blocks are not reported by the scientific community. It must also be mentioned that there is other external loads that are not covered by this thesis. These are mainly load induced by fire or by explosions which normally are treated as accidental loads or caused by the drift of the tunnel.

Field data has been collected to increase the understanding of the variation in shotcrete thickness that could be expected in situ. A numerical framework has been developed which can generate shotcrete linings with varying thickness based on these field data. Numerical analyses have shown that the varying thickness affects crack patterns and also the structural capacity for a lining subjected to gravity load from a block. The presented numerical framework is suitable for probabilistic analysis which seems to be a reasonable approach for the design of shotcrete since many parameters are uncertain.

The influence of bond between shotcrete and rock with respect to development of cracks has been studied. It has been shown numerically that a fine crack pattern develops for unreinforced shotcrete subjected to shrinkage if the bond to the rock is continuous. The varying thickness affects the localization of cracks and compared with a case with uniform thickness, the crack pattern is not always as systematic. For cases with good bond strength, unreinforced shotcrete could be used as the only reinforcement. Theoretically, this type of lining has a strain-hardening behaviour and could transfer the gravity load from a block through bond to the surrounding rock. However, collected field data shows a great variation in bond strength and numerical and experimental results shows that partial debonding leads to formation of wide cracks. Local spots with low bond strength could initiate bond failure but other parameters, such as effects of drying shrinkage and induced stress concentrations caused by the irregular geometry will also affect the bond failure. Thus, a variation in bond strength as well as the effects of the geometry should be considered for the design of a shotcrete lining with bond to the rock.

7.3 Further research

Finally, it can be concluded that the design of a shotcrete lining is a complex problem and further research are needed to increase the understanding of the structural behaviour and the loads that acts on the lining. This thesis has mainly focused on the induced stresses caused by individual load cases, e.g. restrained movement or block load. It has been shown that shrinkage causes problems with early cracking but future studies should include how this affect the load capacity of the lining for other load cases. For the continuation of the project, more field data should be collected to increase the understanding of the spatial correlation of the thickness and to improve the accuracy in the prediction of the thickness. This data should be used to increase the knowledge of the in situ variation in thickness but also to improve the understanding of the in situ development of stresses in the shotcrete. The collection of field data should be complemented by experimental work where the influence of partial debonding and the variation in shotcrete thickness could be investigated in a controlled environment. The numerical method presented here should be further developed and the obtained experimental data shall be used to verify or update the model.

Some other interesting topics for further studies that have been identified during the work with this thesis are listed below:

- Further development of the fibre model. An interesting and possibly more realistic way to include the effects of fibres could be to model the shotcrete and fibres as separate materials.
- The effects of creep of shotcrete must be included to study its time-dependent deformation and behaviour. A numerically efficient method to include its effects should be investigated.
- Further develop the concepts of probability based design. Determine distributions for important parameters such as bond strength and thickness.
- Compare the drying rate of shotcrete between a restrained and a free specimen.
- Force controlled displacement test, i.e. loading until failure. Compare the structural capacity between a lining with uniform and varying thickness experimentally.

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