R-08-115

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Rock mechanics and rock support

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Abstract

This report presents the work on rock reinforcement systems in the design step D2 of an underground repository facility at the Forsmark site, Östhammar Municipality. The general objective is to assign rock reinforcement for each functional area of the repository and show that it is feasible from a rock mechanical and operational point of view.

The main findings are that support of the underground openings does not appear to be problematic due to the exceptionally good rock conditions at the chosen site for the deep repository. The major issues to consider are potential spalling-induced failures and the lifetime for the openings. Recommendations for continued studies are, therefore, to focus on spalling stability.

The report presents analytical and numerical calculations of the stress concentrations that occur around the openings in different directions in relation to the in situ stress field. Based on these it is concluded that under the most likely stress situation spalling is not likely to occur in the tunnels or the rock caverns. Due to the uncertainty in the directions and magnitude of the stress field it can however not be excluded that minor areas can be subjected to spalling.

The stress concentrations in the deposition holes are examined using a three-dimensional model, which indicate a peak value in the tangential stress in the upper part of the deposition hole. The maximum value depends on the orientation of the deposition tunnel. Based on the most likely value on stress magnitude an orientation of the deposition tunnels within 25° in relation to the orientation of the stress field is recommended. However, this recommendation do not include uncertainties in stress magnitude, stress field orientation or spalling strength. A deterministic evaluation of the spalling potential in the boundary of circular openings using the Kirsch equations for plane strain gives safety factor of 1.26 for the rock domain RFM029, which predominates the target volume in Forsmark. This verges on a limiting value, indicating that the probability of spalling increases. It is thus suggested that the potential is further evaluated by probabilistic means.

The geological conditions of the rock mass encountered in the repository layout are described as four different ground types (GT). Influencing factors, such as geological discontinuities and hydrological and stress conditions have been evaluated by three categories of ground behaviours (GB), without considering the effect of reinforcement or the benefit of modifications. The interaction between ground types, ground behaviour and assigned support types have been assessed by using the construction experiences from SFR, the empirical Q-system and analytical calculations. The analysis has been carried out both for the most probable and the most unfavourable system behaviour.

As rock support a general minimum support of shotcrete is proposed for all tunnels and caverns excluding the deposition tunnels. The reasons are that this will limit and simplify periodic inspections. Additional to this are five different rock support classes presented for different ground types and functional areas.

All together the amount of necessary support material is found to be less than estimated during design step D1. This is considered an effect of the established good rock conditions and the use of the observational method as design concept.

Sammanfattning

Föreliggande rapport presenterar arbetet med bergförstärkningslösningar inom projekteringsskede D2 för en underjordisk slutförvarsanläggning i Forsmark, Östhammar kommun. Det övergripande syftet är att anvisa bergförstärkning för respektive funktionalitetsområde i förvaret och visa att det är lämpligt ur ett bergmekaniskt och driftsmässigt perspektiv.

De huvudsakliga resultaten är att förstärkning av undermarksutrymmena inte antas vara problematisk på grund av de exceptionellt goda bergförhållandena vid den valda platsen för djupförvaret. De huvudsakliga frågeställningarna är att beakta potentialen för spjälkbrott och utrymmenas livslängd.

Rapporten presenterar analytiska och numeriska beräkningar av spänningskoncentrationer som förväntas uppkomma runt utrymmena med olika riktningar i relation till in situ spänningsfält. Baserat på dessa är slutsatsen att under den mest sannolika spänningssituationen är det inte sannolikt att det kommer att ske någon spjälkning i tunnlar eller berghallar. På grund av osäkerheten i spänningsfältens riktning och magnitud kan det dock inte uteslutas att mindre områden kommer att vara utsatta för spjälkning. Det föreslås därför att framtida studier fokuseras på spjälkning.

Spänningskoncentrationerna i deponeringshålen granskas med en tredimensionell beräkningsmodell, som indikerar ett maximalt värde på den tangentiella spänningen i övre delen av deponeringshålet. Maxvärdet beror på deponeringstunnelns orientering. Baserat på det mest sannolika spänningsmagnituden rekommenderas deponeringstunnelns orientering ligga inom 25° i relation till huvudspänningsriktningen. Denna rekommendation inkluderar dock inte osäkerheter i spänningsmagnitud, spänningsfälts orientering eller styrkan hos spjälkningen. En deterministisk utvärdering av spjälkningspotentialen i övergången till cirkulära öppningar genom användning av Krisch ekvation för plant deformationstillstånd ger en säkerhetsfaktor på 1,26 för bergdomän RFM029, som dominerar förvarsvolymen i Forsmark. Detta är nära den angivna säkerhetsfaktorn, vilket indikerar att sannolikheten för spjälkning ökar. Det föreslås därför att detta analyseras ytterligare i en sannolikhetsbaserad studie.

De geologiska förhållanden som förväntas i förvarslayouten kan beskrivas i termer av fyra olika bergklasser. Påverkande faktorer så som geologiska diskontinuiteter, hydrologiska förhållanden och spänningsförhållanden har utvärderats i tre kategorier av brottmoder, utan att ta hänsyn till förstärkningens effekt eller modifieringsfördelar. Växelverkan mellan bergklasser, brottmoder och fastslagna förstärkningsmetoder har utvärderats baserat på konstruktionserfarenheter från SFR, det empiriska Q-systemet och analytiska beräkningar. Analysen har genomförts både för de mest sannolika och de mest ofördelaktiga systemegenskaperna.

Som bergförstärkning föreslås en generell minimiförstärkning av sprutbetong för alla tunnlar och hallar bortsett från deponeringstunnlarna. Syftet är att begränsa och förenkla periodiska besiktningar. Dessutom föreslås fem olika bergförstärkningsklasser för olika bergklasser och funktionalitetsområden.

Sammanfattningsvis uppskattas behovet av förstärkning till mindre än vad som uppskattats under designsteget D1. Detta anses vara en effekt av de goda bergförhållandena och användningen av observationsmetoden som ett designkoncept.

Contents

1 Introduction

1.1 Background

SKB has been commissioned to deal with the radioactive waste from Swedish nuclear power plants. Spent nuclear fuel is currently transferred successively from Swedish nuclear power plants to a central intermediate repository for spent nuclear fuel (CLAB). SKB is planning to store the spent nuclear fuel in a final repository designed in accordance with the KBS-3 method, in which the spent nuclear fuel is encapsulated in watertight and load-bearing copper canisters. The canisters are deposited in crystalline rock at about 500 m depth, and enclosed in a buffer, which prevents water ingress and protects the canister. When deposition is complete, the tunnels and cavern are sealed.

The current activity involves carrying out design phase D2 for the two selected sites, Forsmark (Östhammar Municipality) and Laxemar (Oskarshamn Municipality). A number of different studies of both sites have been carried out in parallel in order to determine which of the two sites is the more suitable for a final repository.

Rock reinforcement studies are a central part of this design. This involves evaluating and defining the amount and composition of material needed for rock reinforcement in the various parts of the repository in order to ensure that the requirements for functionality and safety are attained (described in more detail in Section 1.3).

1.2 Objective and scope

This document reports the work on rock reinforcement systems in the current design (Stage D2) of an underground repository facility at the Forsmark site, Östhammar Municipality. The general objective is to assign rock reinforcement and show that it is feasible from a rock mechanical and operational point of view.

The work includes demonstrating that the site adaptation for a repository is feasible from a rock mechanic point of view and to assign rock reinforcement based on the particular requirements for each functional area. Also the constructability should be evaluated. Additional objectives are to identify site-specific critical issues and provide feedback concerning technical risks, safety and environmental impact assessment as well as investigation strategies.

The rock reinforcement work is mainly based on an assessment of the distribution of ground types and ground behaviours in each functional area of the repository, as well as a system behaviour analysis, evaluating the interaction between ground behaviour and construction measures. An important aim is to outline the uncertainties identified for stability issues. Also a general quantification of the materials and resources needed for the assigned rock reinforcement is given for each functional area.

1.3 Methodology

To address the uncertainty and variability of the geological conditions and ground structure interaction that may occur during the underground excavations of the final repository facility, SKB directs an approach known as the 'Observational Method'. It is a risk-based approach that employs adaptive management by various monitoring and measurement techniques to substantially reduce costs while protecting investment, human health and the environment. In the work on rock reinforcement systems it is appropriate to apply the method in situations where uncertainties in prediction of the geotechnical behaviour may occur. The focus in design step D2 shall be on the following issues:

- Assessment of acceptable limits of the behaviour.
- Assessment of the range of possible behaviour.
- Outline the content and the parameters for a monitoring plan in line with the proposed design solutions.

Critical observation parameters for the work on the reinforcement systems are block sizes and rock stresses. The technique proposed in this report to determine block sizes is geological mapping. Rock stresses, on the other hand, are adequately monitored by the occurrence of spalling. The suggested methods for this are acoustic measurements and driving pilot tunnels with cross-sections that favour spalling. Another important part in the application of the observational method is the proposed minimum shotcrete reinforcement, which will facilitate the detection of brittle failure.

The guidelines for the work on the rock reinforcement systems for the deep repository are detailed /SKB 2008a/. According to this the work in design step D2 follows a concept where the geological conditions of the rock mass encountered during construction are described in engineering terms as four different ground types (GT). Moreover, it involves evaluations of the potential ground behaviour (GB) considering each ground type, without considering the effect of reinforcement or the benefit of modifications. Three general categories of ground behaviours, as modified from /Palmstrom and Stille 2006/, have been provided for the evaluation of influencing factors, such as geological discontinuities and hydrological and stress conditions. After the ground types and ground behaviour have been determined, appropriate reinforcement types are suggested. The final step in the concept is an assessment of the system behaviour, defined as the interaction between the ground types, ground behaviour and support types.

The system behaviour has been assessed using the construction experiences from SFR, the empirical Q-system and analytical calculations on load bearing capacity. The analysis has been carried out both for the most probable and the most unfavourable system behaviour.

An essential task in this work is to assess the rock mass stability during construction quantitatively. The main findings of design step D1, as concluded in /Martin 2005/, is that gravity-driven, fall-out or wedge stability failures can be adequately handled by standard rock support and hence is not an issue for layout adaptation or for safety case. Therefore, the main attention is given to stress induced failures. The stress analysis aims at describing the stresses around the openings in the central area, main and deposition tunnels, including crossings, in the deposition area and in deposition holes.

1.4 Requirements for the reinforcement system

The design and reinforcement of the repository should be optimised with regard to the rock mechanics of the area. The critical factors are occurrence and orientation of fractures and deformation zones, as well as any risks of stress-induced spalling. This should be considered in:

- Choice of repository depth.
- Orientation of deposition tunnels.
- Discarding of deposition holes due to the risk of spalling.

Chapter 4 in /SKB 2008a/ describes requirements for the reinforcement systems listed briefly below:

- Repository depth should take account of the risk of spalling.
- Orientation should be optimised with respect to the risk of spalling and geological factors (fractures and fracture zones).
- Distance between deposition holes should be adjusted for the risk of spalling.

Material used for rock reinforcement should not create unfavourable chemical conditions, which may affect the barrier function of the repository.

1.5 Controlling documents and guideline instructions

The task was implemented on the basis of the controlling documents, various documents with input data, and guideline instructions. Controlling documents for the work are:

- Underground Design Premises (UDP/D2) /SKB 2008a/.
- Client's environmental programme for final repository /SKB 2007a/.
- Preliminary safety analysis (PSAR) – Requirements and construction premises /SKB 2006/.
- Statement of layout and technical solution /SKB 2007b/.

Design parameters and engineering guidelines for the rock mass are provided in the Site Engineering Report, Forsmark /SKB 2008b/. This document incorporates details regarding rock and fracture domains, as well as hydraulic and in-situ stress conditions (obtained from the Site Descriptive Model), with parameters required to provide a description in terms of ground types (GT). /SKB 2008b/ outlines also the use of previous construction experiences /Carlsson and Christiansson 2007/ as empirical reference. Guidelines regarding the geometrical design of the underground repository are given in Appendix 1 of /SKB 2008a/.

1.6 Reference design and guideline instructions regarding rock reinforcement

In order to meet the requirements of Section 1.4, guidelines are given in /SKB 2008a/, and in certain supplementary reports. A brief summary of the key guidelines used in design work is given below.

In /SKB 2008b/, repository depth is given as 450–500 m. Consequently, the rock reinforcement study carried out is not to determine depth, but only to check whether spalling problems are foreseen.

The orientation of the deposition tunnels has been studied by /Martin 2005/. The conclusion of this work was that the risk of spalling is ‘significantly reduced’ if the tunnel is oriented within 30° of the direction of the maximum horizontal stress. This conclusion was further verified by calculations presented in this report, showing that the risk of spalling is small to undetectable in tunnels aligned close to the direction of the maximum horizontal stress.

For thermal safety reasons, the distance between deposition holes has been set at a minimum of 6.0 m in accordance with /SKB 2008b/.

Rock reinforcement should provide sufficient stability and load-bearing capability to the final repository facility in order to secure operations and the working environment during the design working life. Methods and material used for reinforcement work must not adversely affect the load-bearing functions of the final repository. Apart from the fact that all cement used in underground installations which have to remain after back-filling and sealing should have a pH-value less than 11, SKB’s aim is that conventional methods and materials should be used for all rock reinforcement.

Facility parts with a design working life of at least 100 years (i.e., all sections apart from the deposition tunnels) should have a minimum reinforcement of shotcrete in both the roof and the upper parts of the walls in accordance with the reference design. The entrance to the deposition tunnels should also have this level of minimum reinforcement. The corners should also be chamfered, mainly to restrict stress concentrations around the intersection of the deposition tunnels.

The deposition tunnels should be designed for a working life of at least 5 years. The Reference design quotes mesh and bolts without bolt cement as possible reinforcement material.

There should be no reinforcement of the deposition holes. If there is any indication that reinforcement would be needed in a deposition hole, the hole should be rejected.

1.7 Layout

The final repository facilities are divided in three functional areas (Figure 1-1):

1. Ramp.
2. Central area, including connected ventilation facilities, as well as skip and elevator shafts (Figure 1-2).
3. Deposition area, including deposition tunnels and holes, as well as ventilation shafts SA01 and SA02.

The actual layout work, including the geometries, positions and orientations of all facility parts of the deep repository, has largely been separated from the study the rock reinforcement system. The layout work is presented in /Hansson et al. 2008/ and their work has served as an immutable basis for the rock reinforcement studies presented herein.

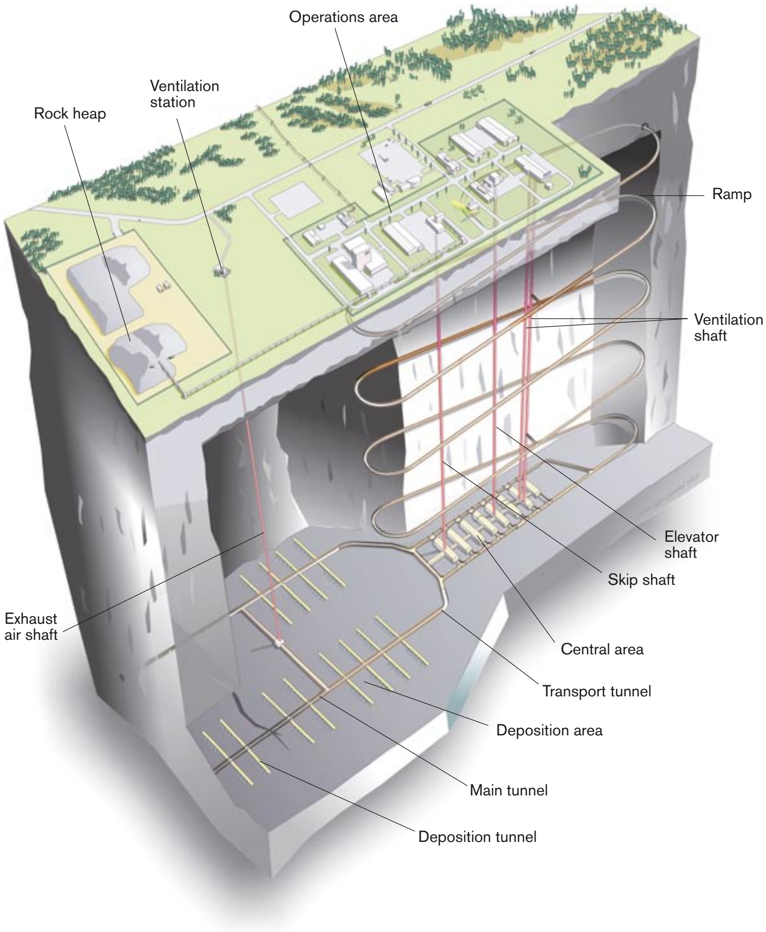


***Figure 1-1.** Layout and functional disposition of final repository facility. From /SKB 2008b/.*

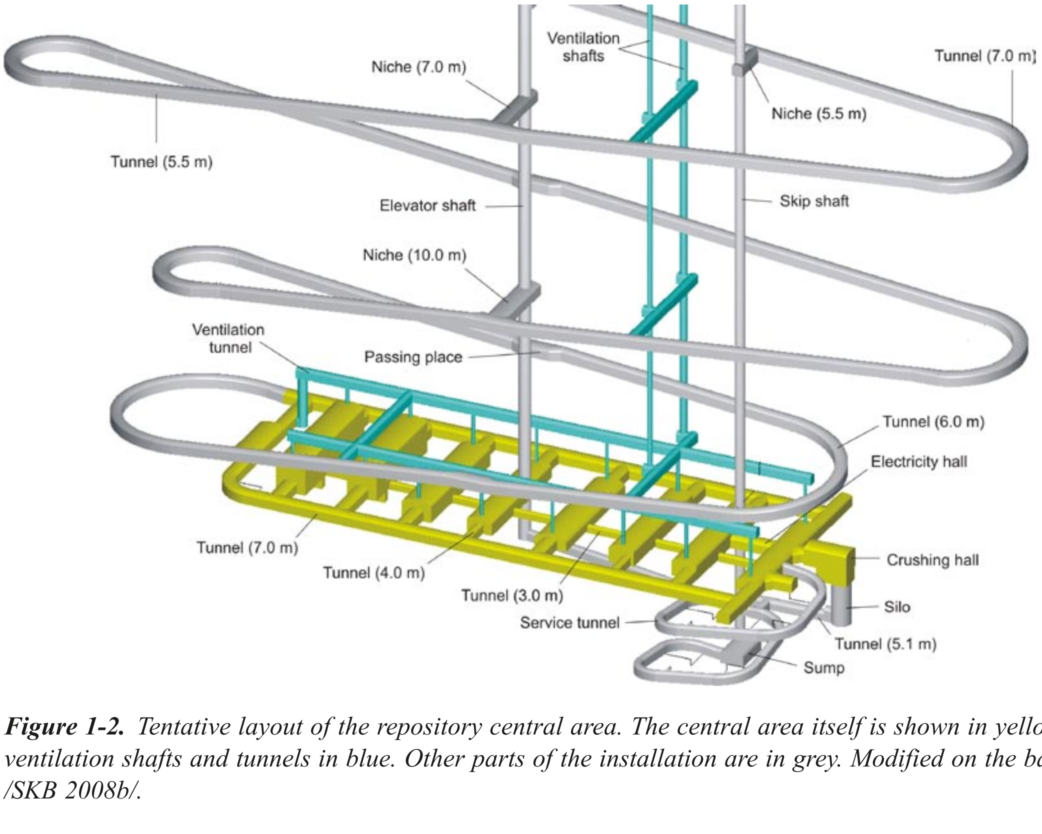


***Figure 1-2.** Tentative layout of the repository central area. The central area itself is shown in yellow, with ventilation shafts and tunnels in blue. Other parts of the installation are in grey. Modified on the basis of /SKB 2008b/.*

2 Design permises and site conditions

2.1 Geological outlines

The following description of the geological conditions in the repository area are based on information given in /SKB 2008b/.

The area is situated in a tectonic lens surrounded by belts of more intense ductile strain. The lens can be described as two rock domains, RFM029 and RFM045, where the latter is located north of RFM029 and has a rod-shaped geometry that plunges moderately to steeply towards southeast. The dominant rock type in RFM029 is a medium-grained metagranite-granodiorite, whereas RFM045 mainly consists of an aplitic metagranite and subordinately the medium-grained metagranite-granodiorite.

Based on the fracture conditions, three separate domains can be recognised in the target volume. Fracture domain FFM02 occupies the superficial part of the target volume. Below that occurs fracture domain FFM01 and FFM06, which correspond to rock domain RFM029 and RFM045, respectively (Figure 2-1). The boundary between FFM02 and the two underlying domains slopes gently to the southeast with a maximum depth of about 150 m.

Fracture domain FFM02 is characterised by a high frequency of hydraulically connective, horizontal to gently dipping fractures. Fracture domain FFM01 and FFM06 are more sparsely fractured, without significant water flows, and predominantly vertical to steeply dipping fractures. Also gently dipping to sub-horizontal fractures occur, but experience at the SFR facility suggests that their distribution is more restricted /SKB 2008b/.

In the deposition area, there is only one brittle deformation zone large enough (> 3 km in length) that it requires a respect distance: ZFMENE060A. This zone is a vertical to steeply dipping structure running east-northeast, which consists particularly of sealed fractures and sealed fracture networks. The hydraulic permeability is therefore generally low.

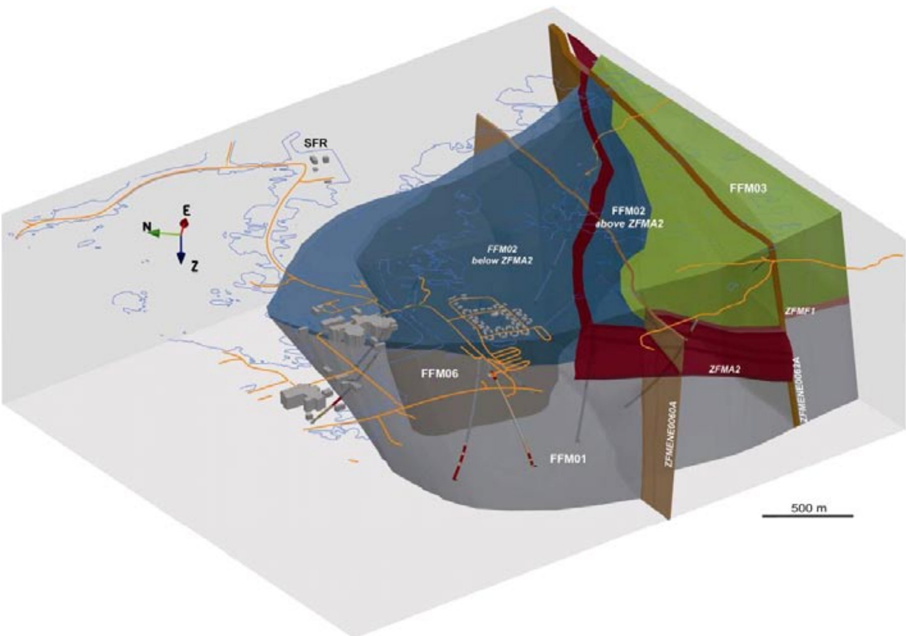


Figure 2-1. Three-dimensional illustration of fracture domain distribution and deterministically modelled deformation zones. Fracture domains FFM01, FFM02, FFM03 and FFM06 are shown in grey, dark grey, blue, and green, respectively. The zones included are ZFMA2, ZFMF1, ZFMENE0060A and ZFMENE62A. They have a trace length at ground surface > 3 km and therefore require respect distance View obliquely upwards to ENE. Modified from /SKB 2008b/.

2.2 Definitions of ground types

In order to describe the geological conditions of the area concerned in engineering terms, four different ground types (GT) have been defined in /SKB 2008b/. A summary of these ground types and the Q-values given in /SKB 2008b/ for each type is listed in Table 2-1.

The application of ground types to various phenomena in the geological model of the Forsmark area are provided in /SKB 2008b/ and a summary is presented in Table 2-2.

2.3 Distribution of ground types in the repository

2.3.1 Ramp and central area

Table 2-3 shows total lengths of various facility parts in both the ramp and central area, as well as their distribution in zones and fracture domains. This includes passing places, niches (mainly sumps) along the ramp and connected ventilation shafts and tunnels (cf. Figure 1-2).

Starting from the springline of the ramp, the part located in the fracture domain FFM02 amounts to about 449 m. The ramp is crosscut by deformation zone ZFMNNW1205, a minor zone that is less than 1 km in length /SKB 2008b/. The linear parts of the ramp pass through the zone at ten locations and each passage is approximately 16 m long. The total ramp length in steep modelled zones (i.e. ZFMNNW1205) amounts to 164 m. Only the lowermost of the six connection drifts are partly intersected by ZFMNNW1205.

The central area borders on only two modelled zones: ZFMENE1061A and ZFMNNW1205. Zone ZFMENE1061A touches the central area at the northeast corner and is consequently restricted to tunnels 7 m wide. Zone ZFMNNW1205 divides the central area in two along the central elevator hall. Both elevator and ventilation shafts have passages of slightly more than 70 m within the zone. In addition, there are several passages in the ventilation tunnels and two passages in the 7 m wide tunnels of the central area.

A classification in terms of ground types for the various facility parts in both the ramp and central area is given in Table 2-4. Classification is based on the distributions established in Table 2-2 and Table 2-3.

Table 2-1. Summary of ground types (GT) /SKB 2008d/.

Ground types	Q-value	Description
GT1	> 100	Sparsely fractured rock with isotropic properties.
GT2	40–100	Blocky rock mass. Individual blocks are intimately interlocked. Water-bearing fractures occur, especially in gently dipping zones.
GT3	10–40	Sealed fracture network, which may result in blocky rock if fractures are reactivated.
GT4	4–20	Major deformation zones that require a respect distance. Water transmission may be significant if fractures are not sealed.

Table 2-2. Estimated distribution of ground types in modelled zones and fracture domains according to /SKB 2008b/.

Zones and fracture domains	GT1 [%]	GT2 [%]	GT3 [%]
Modelled zones			
ZFMENE0060A	20	40	40
Respect distance ZFMENE0060A	80	20	–
Steeply dipping zones (> 70°) < 3 km in length	20	40	40
Gently dipping zones (< 70°): ZFMB7	–	100	–
Fracture domains except for modelled zones and respect distances			
FFM01	95	5	–
FFM02	85	15	–
FFM06	95	5	–

Table 2-3. Length distribution for the ramp and central area, as well as their distribution in modelled zones and fracture domains.

Facility part	Total length [m]	Steep zones (70–90°) [m]	FFM01[2]	FFM02[2]
Ramp				
Tunnel (5.5 m wide)[1]	4,060	156	3,578	326
Tunnel (6.0 m wide)	119	–	119	–
Tunnel (7.0 m wide)	698	–	575	123
Passing places (8.0 m wide)	135	8	127	–
Niche (5.5 m wide)	20	–	20	–
Niche (7.0 m wide)	129	–	129	–
Niche (10.0 m wide with 5x5x16 m below)	38	3[3]	35	–
Ventilation				
Shaft (ø 1.5 m)	258	1	257	–
Shaft (ø 2.5 m)	446	71	332	43
Shaft (ø 3.5 m)	446	73	328	45
Shaft (ø 4.5 m)	25	–	25	–
Tunnel (4.0 m wide)	1,008	86	922	–
Central area				
Skip shaft (ø 5.0 m)	524	2	480	42
Elevator shaft (ø 6.0 m)	494	73	376	45
Silo (ø 9.5 m)	22	–	22	–
Tunnel (3.0 m wide)	134	9	125	–
Tunnel (4.0 m wide)	29	–	29	–
Service tunnel (4.0 m wide)	500	–	500	–
Tunnel (5.1 m wide)	48	–	48	–
Tunnel (7.0 m wide)[4]	912	182	730	–
Caverns (13.0–16.0 m wide)	541	40	501	–
Sump (12.0 m wide)	20	–	20	–
Electricity hall (7.0 m wide)	21	–	21	–
Crushing hall (10.3 m wide)	22	–	22	–

[1] Includes also transitions to wider tunnel sections and passing places.
[2] Modelled zones not included.
[3] Although one of the corners of the pit is crosscut by ZFMNNW1205, it is not included in the table.
[4] The belevel between connecting tunnels are not included.

Table 2-4. Distribution of ground types in various facility parts of the ramp and central area.

Facility part	GT1 [m]	GT2 [m]	GT3 [m]
Ramp			
Tunnel (5.5 m wide)[1]	3,707	290	63
Tunnel (6.0 m wide)	113	6	–
Tunnel (7.0 m wide)	651	47	–
Passing places (8.0 m wide)	123	9	3
Niche (5.5 m wide)	19	1	–
Niche (7.0 m wide)	123	6	–
Niche (10.0 m wide with 5x5x16 m below)	35	2	1
Ventilation			
Shaft (ø 1.5 m)	245	12	1
Shaft (ø 2.5 m)	365	53	28
Shaft (ø 3.5 m)	364	53	29
Shaft (ø 4.5 m)	24	1	–
Tunnel (4.0 m wide)	893	80	35
Central area			
Skip shaft (ø 5.0 m)	492	32	1
Elevator shaft (ø 6.0 m)	411	54	29
Silo (ø 9.5 m)	21	1	–
Tunnel (3.0 m wide)	120	11	3
Tunnel (4.0 m wide)	28	1	–
Service tunnel (4.0 m wide)	475	25	–
Tunnel (5.1 m wide)	46	2	–
Tunnel (7.0 m wide)[2]	729	110	73
Caverns (13.0–16.0 m wide)	479	44	18
Sump (12.0 m wide)	19	1	–
Electricity hall (7.0 m wide)	20	1	–
Crushing hall (10.3 m wide)	21	1	–

[1] Includes also transitions to wider tunnel sections and passing places.
[2] The belevel between connecting tunnels are not included.

2.3.2 Deposition area + ventilation shafts SA01 and SA02

Tunnel lengths in the deposition area and their distribution in zones and fracture domains are presented in Figure 2-2. The deposition area consists in total of 72,291 m of tunnels, of which about 10% is in modelled deformation zones longer than 1 km and barely 3% in modelled deformation zones less than 1 km in length. A total of 26 modelled deformation zones longer than 1 km (Figure 2-2) are involved, including attached branches of ZFMENE0060A, and 18 modelled deformation zones less than 1 km in length. Four passages of transport tunnels through ZFMENE0060A give a total length in the zone of 73 m. The total tunnel length in the respect distance amounts to 1,040 m after lengths in the actual zone and splays (ZFMENE0060B and ZFMENE0060C), as well as zone passages through ZFMNE2282, ZFMNNE2255, ZFMNNE2273, ZFMENE1192 and ZFMNNE0725 are deducted. Apart from an 8 m passage of ZFMB7, all modelled zones that may be expected to intersect with tunnels in the deposition area are vertical or steeply dipping (70–90°).

It should be noted that, in tunnel passages where crossings between two or more deformation zones occur, overlapping parts have excluded out in order to avoid these lengths being calculated twice.

Ventilation shafts SA01 and SA02 have a total length of 266 m in fracture domain FFM02. The only modelled deformation zone that cross cuts the shaft is the gently dipping zone ZFMB7, which passes SA01 along an interval of 30 m.

A classification in terms of ground types for the various tunnel types occurring in the deposition area and ventilation shafts SA01 and SA02 is given in Table 2-6. Classification is based on the distributions established in Table 2-2 and Table 2-5.

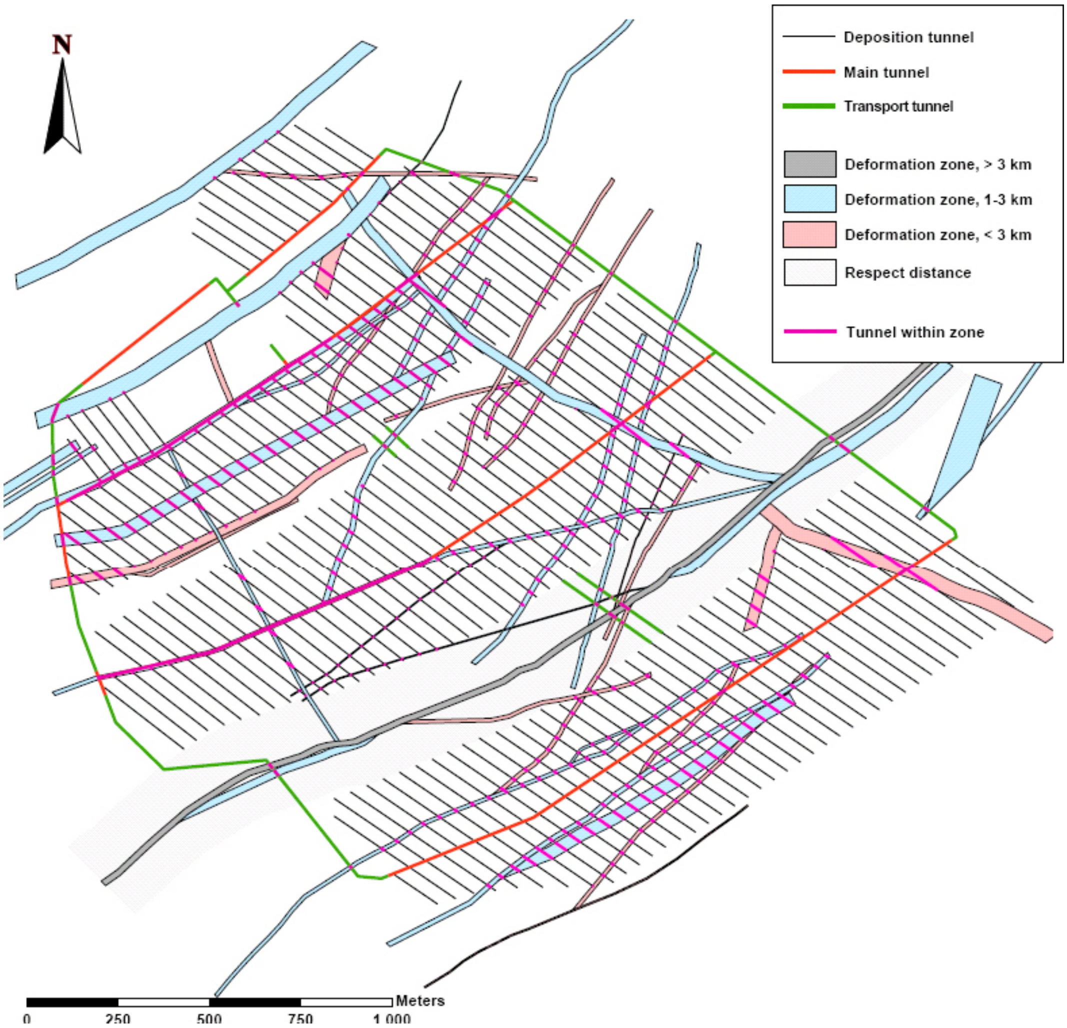


Figure 2-2. *Schematic sketch of the deposition area, showing all modelled deformation zones. Modified after /Hansson et al. 2008/.*

Table 2-5. Tunnel and shaft lengths in the deposition area and their distribution in zones and fracture domains.

Zones and fracture domains	Ventilation shafts SA01 and SA02 [m]	Main tunnel [m]	Transport tunnel [m]	Deposition tunnel [m]
Total length	935	6,473	4,629	61,189
Modelled zones				
ZFMENE0060A	–	–	73	–
Respect distance to ZFMENE0060A[1]	–	–	1,040	–
Gently dipping zones (< 70°): ZFMB7	30	–	8	–
Steep zones (70–90°) < 3 km	–	2,201	483	6,593
Fracture domains[2]				
FFM01 and FFM06	639	4,272	3,025	54,596
FFM02	266	–	–	–

[1] Outside ZFMENE0060A and intersecting deformation zones.
[2] Modelled zones and respect distance not included.

Because two of the four main tunnels in the deposition area to a great extent coincide with east-northeast striking, steep deformation zones (ZFMENE0061A and ZFMENE0159A), the proportion of GT2 and GT3 is significantly higher than in other tunnel types. The proportion of GT2 and GT3 is lowest in the deposition tunnels where they are jointly restricted to about 13%.

Table 2-6. Distribution of ground types in various tunnels and shafts in the deposition area.

Facility part	GT1 [m]	GT2 [m]	GT3 [m]
Ventilation shafts SA01 and SA02 (ø 3.0 m)	833	102	–
Main tunnel (10.0 m wide)	4,497	1,096	880
Transport tunnel (7.0 m wide)	3,876	560	193
Deposition Tunnel (4.2 m wide)	53,185	5,367	2,637

3 Ground behaviour

3.1 Definition of ground behaviour

Three general categories of ground behaviour (GB), which should be used to evaluate the properties in the repository, have been defined in /SKB 2008b/. A summary of ground behaviour types is given in Table 3-1.

The predominant ground behaviour in the area is GB1.

In accordance with /SKB 2008b/ the occurrence of GB3 is expected to be restricted to fracture domain FFM02, i.e. to the upper 50 m of the repository, where the occurrence of open, water-bearing fractures locally may be significant. The occurrence of GB2, which includes all rock with a risk for stress-induced failure, is assumed to be restricted to greater depths, i.e. to FFM01 and FFM06.

Analyses of the stress field in the area shows that the maximum horizontal stresses have a direction of 145° ± 15° at repository depth, while the minimum stresses are vertical. Deviations may occur in connection with deformation zones, but these are thought to be of a magnitude that they do not need to be considered in the current design (Stage D2).

The main problem with the actual state of stress in Forsmark is the risk of spalling. At the expected stress level given in /SKB 2008b/ for Forsmark, calculations in Chapter 4 show that spalling is not occurring if the tunnels are aligned at a low angle to the direction of σ_H. Similar to /Martin 2005/, the calculations in this study show that the risk of spalling is reduced considerably in tunnels where the longitudinal direction deviates less than ± 30° from σ_H.

3.2 Assumptions regarding the ground behaviour

It should be noted that calculations are carried out only for caverns in the central area, deposition tunnels, deposition hole, and crossings between main and deposition tunnels. In this report, however, it is assumed that the results apply generally and are applicable to all tunnel types of the repository. At the expected distribution of ground behaviour, it is therefore assumed that GB2A does not occur at all in the repository, irrespective of depth and orientation of facility parts. At the most unfavourable distribution of ground behaviour, on the other hand, it was assumed that GB2A might occur in parts of the repository located at 400 m depth or deeper in FFM01 and FFM06 and then only in the parts with a longitudinal direction deviating by more than 30° from the direction of σ_H.

Because of the increased frequency of fracture in deformation zones, it is likely that the probability of spalling is lower in these passages. In this work, it is assumed however that any occurrence of spalling is independent of deformation zones.

Table 3-1. Summary of ground behaviour (GB) categories /SKB 2008b/.

Rock class	Description
GB1	Gravity driven, mostly discontinuity controlled failures (block falls). Pre-existing fragments or blocks become released on excavation.
GB2	Stress induced, gravity assisted failures caused when the stresses exceed the local rock strength. These failures may occur in two main types: A) Spalling, buckling or rock burst in brittle rocks. B) Plastic deformation, creep or squeezing in massive, soft/ductile rocks or soils and heavily jointed rocks.
GB3	Water pressure; an important load to consider in heterogeneous rock conditions. A) Fractures initiated by groundwater. May cause flowing ground in particulate materials exposed to large quantities of water and unstable conditions (eg swelling and slaking) in clay bearing material. Water may also dissolve minerals such as calcite. B) Water may also influence rock falls, especially in fractures with soft mineral filling.

In /SKB 2008b/ it is also noted that GB2B may occur in FFM01 and FFM06; mainly in modelled deformation zones. Since the content of mica or other minerals with plastic properties is low in the steeply dipping zones that occur in FFM01 and FFM06, it was assumed that the occurrence of GB2B is marginal and may be neglected when assessing ground behaviour at repository depth. The frequent occurrence of laumontite in the north-easterly oriented deformation zones should however be considered. Laumontite crack during hydration or dehydration and the occurrence of GB3A can therefore not be excluded in the deformation zones with laumontite filled fractures. Since this is not noted in /SKB 2008b/, it is recommended to disregard any occurrence of GB3 in parts other than those located in FFM02.

Regarding the occurrence of GB3 in FFM02, it has been assessed that it exclusively concerns GB3B, because the risk of instability in unconsolidated material due to large quantities of water is thought to be very small. A possible exception may be the horizontal, up to meter-wide fractures, filled with glacial material, that occur down to about 5 m depth in the area, see /Carlsson 1979/. However, it is recommended that GB3A may be ignored when assessing ground behaviour in fracture domain FFM02. A number of the flat-lying, transmissive fractures that occur in fracture domain FFM02 contain friction-reducing minerals such as clays and chlorite, which can increase the risk of block fall. Significant occurrences of fractures with this type of filling or coating generally coincide with rocks classified as GT2. Since an overwhelming part of the open fractures in FFM02 are of this type, the occurrence of GB3B is estimated at about 5–15% in rock volume outside the modelled zones. The expected quantities are recommended to be 10%, while those in the most unfavourable case may be taken as 15%.

3.3 Combinations of ground type and ground behaviour

The various combinations of ground types and ground behaviour expected to occur in the repository are GT1–GB1, GT2–GB1, GT2–GB3B and GT3–GB1. Under the most unfavourable conditions (i.e. at maximum stress level given in /SKB 2008b/) it is assumed that a further three combinations with GB2A may occur, GT1–GB2A, GT2–GB2A, GT3–GB2A. Table 3-2 summarize the properties of the different combinations ground types and ground behaviours, as well as in which parts of the repository volume they are expected to occur.

Table 3-2. Combinations of ground types and ground behaviours.

GT–GB	Description	FFM01 and FFM06	FFM02	MDZ >70°	MDZ <70°
Expected case					
GT1–GB1	Sparsely fractured, isotropic rock with gravity driven, mostly discontinuity controlled failures (block falls).	95	85	20	–
GT2–GB1	Blocky rock mass with gravity driven, mostly discontinuity controlled failures (block falls). Water-bearing fractures occur, especially in MDZ <70°.	5	5	40	100
GT2–GB3B	Blocky rock mass with possible water assisted block falls, especially in fractures with soft mineral filling.	–	10	–	–
GT3–GB1	Sealed fracture network. If reactivated it may result in blocky rock mass with gravity driven, mostly discontinuity controlled failures (block falls).	–	–	40	–
Additional combinations in the most unfavourable case					
GT1–GB2A	Sparsely fractured, isotropic rock with possible spalling.	All facility parts with a longitudinal direction deviating by more than 30° from the direction of σ_H.			
GT2–GB2A	Blocky rock mass with possible spalling.				
GT3–GB2A	Sealed fracture network. If reactivated it may result in blocky rock mass with possible spalling.				

3.4 Distribution of ground behaviour in the repository

3.4.1 Ramp and central area

Table 3-3 gives a distribution of ground behaviour in various facility parts in both the ramp and central area. This includes passing places, niches (mainly sumps) along the ramp and connected ventilation shafts and tunnels.

From the start of the ramp springline, the part located in fracture domain FFM02 amounts to some 326 m. The expected distribution of ground behaviour along this tunnel section is 90% GB1 and 10% GB3B, with a least favourable distribution of 85% GB1 and 15% GB3B.

All parts of the repository located in FFM01 are expected to consist of GB1, irrespective of the occurrence of modelled zones. However, at the maximum stress magnitude (i.e. under the most unfavourable stress conditions), it is assumed that spalling may occur at depths greater than 400 m in the ramp sections with a longitudinal direction deviating more than 30° from the direction of the maximum horizontal stress. In general, the ramp is made up of straight tunnels segments with horizontal projections that strike 50°. These are connected by curved segments at 180°. From the point where the springline of the ramp passes –397 m depth, ground behaviour category GB2A has been recommended for all straight ramp segments, and ⅔ (i.e. 120°) of the curved segments. Total occurrence of category GB2A is 591 m on a stretch of 683 m.

The only parts that belong to the central area, which are designed within fracture domain FFM02, are two ventilation shafts, an elevator shaft, and one skip shaft. The boundary between FFM01 and FFM02 is between –43 and –45 m in the area and the expected quantity of GB3B is 4 m in each shaft, while the least favourable distribution is 6–7 m in each shaft.

All parts of the central area and the ventilation facility located in FFM01 are expected to consist of GB1, irrespective of the occurrence of modelled zones. At maximum stress level it is, however, assumed that spalling may occur in the tunnels and halls where the longitudinal direction deviates more than 30° from σ_H. In general, the central area is made up of linear elements that either are aligned at 50 or 140°. Under the most unfavourable stress conditions, ground behaviour category GB2A has been recommended for all the parts of the central area, which are aligned at 50°. The service tunnel connecting the central area with the sump, at the bottom of the skip shaft, includes eight curved sections at 90°. Two thirds (i.e. 60°) of the curved segments are recommended to belong to category GB2A. The same distribution applies to the 7 m tunnel in the south-westernmost part of the central area that bends through 90°.

3.4.2 Deposition area + ventilation shafts SA01 and SA02

According to /SKB 2008b/, all deposition tunnels should be oriented along the direction of maximum horizontal stress in order to reduce the risk of spalling. In the present design, the vast majority of the deposition tunnels have an orientation of 125–127°. However, in deposition area A (model file: 191BD_00_4202), they are oriented 123° and 152°. Regardless of the uncertainty of ± 15° in the direction of the maximum horizontal stress, a deviation of ± 22° (i.e. 145°–123°) is permitted with the current layout /Hansson et al. 2008/.

If it in the most unfavourable case is assumed that all tunnels that deviate more than ± 22° from the direction of the main stresses fall into category GB2A, it covers 100% of main tunnels and 28% of transport tunnels. If the deviation is increased to ± 30°, the coverage of GB2A falls to 97% of main tunnels and 24% of transport tunnels. A ground behaviour distribution of tunnel types in the deposition area is given in Table 3-4. Tunnels in which the longitudinal direction deviates more than 30° from the direction of the main stresses are assessed as belonging to ground behaviour category GB2A under the most unfavourable stress conditions. The expected distribution is, however, that all tunnels will belong to category GB1 irrespective of the occurrence of modelled deformation zones.

Ventilation shafts SA01 and SA02 are the only parts of the installation that occur in FFM02. The total shaft length in FFM02 is 266 m, and the expected amount of category GB3B is 27 m shaft, while at the most unfavourable distribution it is 40 m.

Table 3-3. Ground behaviour distribution for various facility parts in the ramp and central area.

Facility part	Expected distribution GB1 [m]	GB3B [m]	Most unfavourable distribution GB1 [m]	GB2A [m]	GB3B [m]
Ramp					
Tunnel (5.5 m wide)[1]	4,026	34	3,512	497	51
Tunnel (6.0 m wide)	119	–	40	79	–
Tunnel (7.0 m wide)	686	12	680	–	18
Passing places (8.0 m wide)	135	–	120	15	–
Niche (5.5 m wide)	20	–	20	–	–
Niche (7.0 m wide)	129	–	129	–	–
Niche (10.0 m wide with 5x5x16 m below)	38	–	38	–	–
Ventilation					
Shaft (ø 1.5 m)	258	–	258	–	–
Shaft (ø 2.5 m)	442	4	440	–	6
Shaft (ø 3.5 m)	442	4	439	–	7
Shaft (ø 4.5 m)	25	–	25	–	–
Tunnel (4.0 m wide)	1,008	–	494	514	–
Central area					
Skip shaft (ø 5.0 m)	520	4	518	–	6
Elevator shaft (ø 6.0 m)	490	4	487	–	7
Silo (ø 9.5 m)	22	–	22	–	–
Tunnel (3.0 m wide)	134	–	–	134	–
Tunnel (4.0 m wide)	29	–	29	–	–
Service tunnel (4.0 m wide)	500	–	162	338	–
Tunnel (5.1 m wide)	48	–	–	48	–
Tunnel (7.0 m wide)[2]	912	–	351	561	–
Caverns (13.0–16.0 m wide)	541	–	541	–	–
Sump (12.0 m wide)	20	–	20	–	–
Electricity hall (7.0 m wide)	21	–	–	21	–
Crushing hall (10.3 m wide)	22	–	–	22	–

[1] Includes also transitions to wider tunnel sections and passing places.
[2] The belevel between connecting tunnels are not included.

Table 3-4. Ground behaviour distribution in the deposition area.

Facility part	Expected distribution GB1 [m]	GB3B [m]	Most unfavourable distribution GB1 [m]	GB2A [m]	GB3B [m]
Ventilation shafts SA01 and SA02 (ø 3.0 m)	908	27	895	–	40
Main tunnel (10.0 m wide)	6,473	–	220	6,253	–
Transport tunnel (7.0 m wide)	4,621	–	3,505	1,116	–
Deposition tunnel (4.2 m wide)	61,189	–	61,189	–	–

4 Stress analysis

4.1 Introduction

In the following chapter analyses concerning the rock mechanics are made. The attention is given to stress induced failures (i.e. spalling), due to the elevated stress magnitudes at Forsmark relative to other parts of Sweden.

The shape and orientation of the opening and the depth of the repository can impact on the stress concentrations and potential for spalling. A methodology for assessing the spalling potential in the boundary of circular openings using the Kirsch equations for plane strain are proposed by /Martin 2005/. According to this, the spalling potential should be determined by deterministic analysis initially. If the probability for spalling is judged to be significant, the potential should be evaluated using probabilistic means and three-dimensional elastic stress analysis instead. For non-circular openings numerical two- or three-dimensional methods is required to evaluate the stress situation.

The analyses made in this chapter describe the stresses around the openings in the central area, main and deposition tunnels, including crossings, in the deposition area and in deposition holes.

In tunnels and caverns the attention is given to the compressive stresses in the roofs and springlines (defined as the transition between the arched form of the roof and the flat area of the walls), i.e. where spalling induced failures should be handled for a safe working environment and/or stability. Stress-relieved areas in the walls and spalling in the floor are not discussed. These situations can be handled using standard rock support but should with reference to /Carlsson and Christiansson 2007/ not be expected.

In the deposition holes a detailed study of stresses is made to facilitate evaluation of spalling and spalling depth. This information is valuable for the safety analysis.

4.2 Strength and stress parameters

The strength and stress parameters of the rock are given in /SKB 2008b/. Equations 4-1, 4-2 and 4-3 give the in-situ stress field in the depth range 400–600 m and are used in the calculations.

$$\sigma_H = 29.5+0.023z \quad \pm 15\% \ [MPa] \tag{4-1}$$

$$\sigma_h = 9.2+0.028z \quad \pm 20\% \ [MPa] \tag{4-2}$$

$$\sigma_v = 0.0265z \quad \pm 0.0005 [MPa] \tag{4-3}$$

In /SKB 2008b/ was given that the repository depth range should be 450–500 m. All analyses are made for the greatest depth 500 m, i.e. the conservative case of this depth interval. This gives stresses magnitudes as presented in Table 4-1. In the calculations the expected value on in-situ stresses was used. For the case of the deposition holes study, a “worst case” scenario with elevated stresses according to /SKB 2008b/ as presented in Table 4-2.

Table 4-1. Stress magnitude at the depth of 500m /SKB 2008b/.

Principal stress components	Most likely value [MPa]	Minimum value [MPa]	Maximum value [MPa]
Major principal stress (σ_H)	41	35	47
Minor principal stress (σ_h)	23	20	27
Vertical stress (σ_v)	13	13	13

Table 4-2. Elevated stress magnitude at the depth of 500m /SKB 2008b/.

Principal stress components	Most likely value [MPa]	Minimum value [MPa]	Maximum value [MPa]
Major principal stress (σ_H)	56	50	62
Minor principal stress (σ_h)	35	27	43
Vertical stress (σ_v)	13	13	13

The most likely direction (trend) of major principle stress according to /SKB 2008b/ 145° and correspondingly the minor principal stress is 55°. These are also associated with an uncertainty, estimated as ±15°.

The strength parameters and the elastic properties vary with ground type. In Table 4-3 the values for spalling strength (σ_{sm}), Young's Modulus (E) and Poisson's ratio (ν) estimated for tunnel scale is presented.

4.3 Analysis methods

Two-dimensional stress analyses have been carried out both analytically and numerically using Examine^{2D} /RocScience 2007/, which is a boundary-element program for elastic stress analysis of underground excavations used to calculate stress situations around underground structures. The analytical portion was carried out on the deposition hole and concerns average stress in accordance with the Kirsch equations. Initially, the spalling potential should be determined by deterministic analysis using the Kirsch equations for plane strain, as proposed by /Martin 2005/. If the deterministic factor of safety (FOS), calculated by Equation 4-4, is 1.25 or less the probability for spalling is judged to be significant and the potential should be evaluated further.

$$FOS = CIR \cdot UCS_{mean}/(3\sigma_H - \sigma_h) \qquad (4\text{-}4)$$

where

CIR = crack initiation ratio

UCS_{mean} = mean uniaxial compressive strength

The spalling potential has been evaluated deterministically for the two rock domains denoted RFM029 and RFM045, which make up the entire target volume at repository depth. Values used in the analysis for principal stresses are given in Table 4-1, and for CIR and UCS_{mean} in Table 4-4.

Table 4-3. Strength and elastic properties on tunnel scale for the different ground types. From /SKB 2008b/.

Ground type	Spalling strength (σ_{sm}) [MPa]	Youngs modulus (E) [GPa]	Poissons ratio (v) [–]
GT1a	120	60	0.23
GT1b	170	70	0.23
GT2	120	50	0.3
GT3	80	35	0.3
GT4	80	35	0.3

Table 4-4. Parameters to be used for a deterministic assessment of spalling potential according to /SKB 2008b/.

CIR	UCS_{mean} RFM029	UCS_{mean} RFM045
0.53	230 MPa	310 MPa

The calculated safety factor for RFM045 is 1.69, whereas RFM029 that predominates the target volume in Forsmark has a safety factor of 1.26. The latter verges on the limiting value of 1.25, indicating that the uncertainty if spalling occurs increases. It is therefore suggested that the potential is further evaluated and include probabilistic means.

Numerical two-dimensional stress analyses were carried out on a section of the main tunnels using the Examine^{2D} software to study the effect of a varied cross-section of the tunnel. The analyses aimed to visualize how the stress concentration is influenced by the cross-section and that adjustment of the cross-section can limit the stress concentrations.

In order to obtain a correct picture of the stress situation a three-dimensional analysis is needed for some cross-sections and conditions in the repository. The calculations were carried out using Examine^{3D}, a three-dimensional analysis program for underground structures in rock. This is a boundary-element program designed to perform three-dimensional elastic stress analyses /RocScience 1998/. The following were studied using three-dimensional analysis:

- Crossings between main and deposition tunnels
- Deposition tunnels with deposition holes.

4.4 Central area

The central area consists of nine 13–16 m wide rock caverns of different dimensions, as well as minor caverns, various tunnels, shaft and pits, designed to facilitate various activities. A two-dimensional study of stress concentrations was made similar to /Martin 2005/ to investigate the stress distribution.

In Figure 4-1 the layout of the central area is shown, along with a two-dimensional section, taken perpendicular to the length axes of the caverns. This is used in the calculations in stress analyses.

The stress in the caverns of the central area was analyzed for orientations between 55° and 145°, i.e. orthogonal to parallel in relation to the major principal stress (Table 4-5).

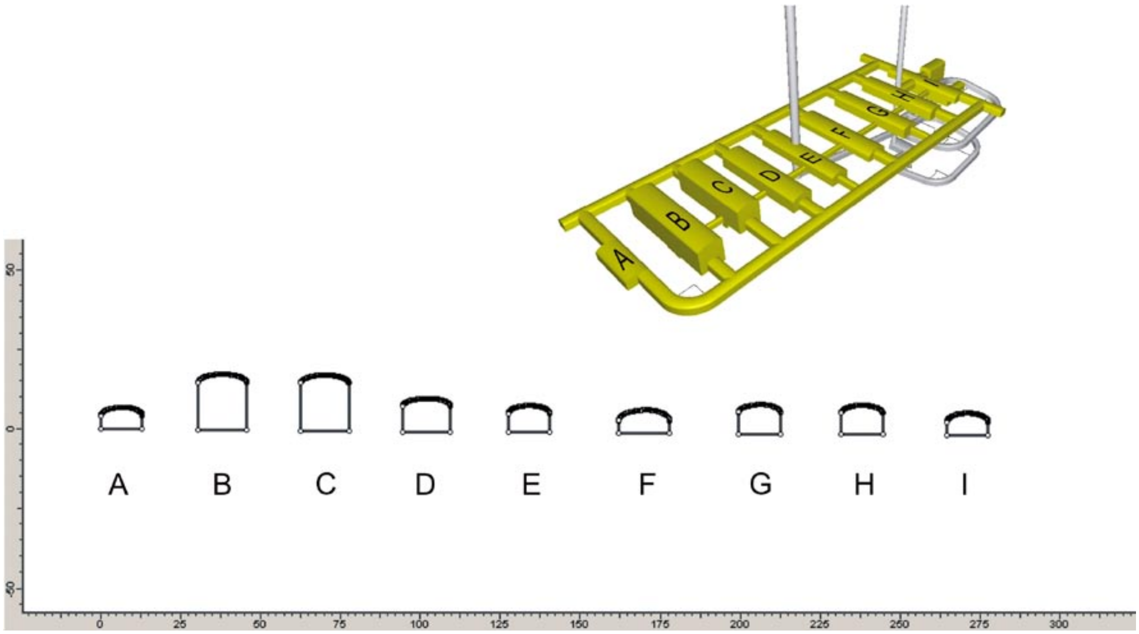


Figure 4-1. Layout of the central area and the two-dimensional section used in the calculations.

Table 4-5. Cases for calculation concerning potential stress problems in the central area.

Case	Orientation of cavern	Description
C1	55°	Orientation 90° towards σ_H
C2	85°	Orientation 60° towards σ_H
C3	115°	Orientation 30° towards σ_H
C4	145°	Orientation 0° towards σ_H

The results of the calculations are presented in Table 4-6 in terms of highest calculated stress in the roof. It is noticed that in all cases the maximum stress is found in the left hand side springline of Cavern B.

The results should be viewed in respect of the geometry of the full central area. Since facility parts other than the caverns have been excluded it is likely that the results for some caverns are conservative, i.e. a lower stress concentration would have been calculated using a tree-dimensional model. However, since Cavern B is relatively far from the ramp and the shafts, the result in this case is found to be representative. The results are therefore considered valuable as a survey of the stress level for different orientations of the central area.

In Figure 4-2 a detailed presentation of the stresses around Cavern B and C is shown for calculation C1. It is seen that only limited areas develop with compressive stresses over 80 MPa, corresponding to the spalling strength in GT3.

4.5 Profile of the main tunnels

The effect of the profile shape of the main tunnels, i.e. the shape of the roof, has been studied. Figure 4-3 shows three different sections that have been analysed where the wall height have been altered. The section to the right corresponds to the Reference layout /SKB 2008a/. The section to the left and the middle section have a higher wall height which gives a smoother roof profile.

Table 4-6. Calculated result on the central area.

Case	Maximum calculated stress [MPa]	Position of maximum stress
C1	85	Left hand side springline on Cavern B
C2	80	Left hand side springline on Cavern B
C3	65	Left hand side springline on Cavern B
C4	47	Left hand side springline on Cavern B

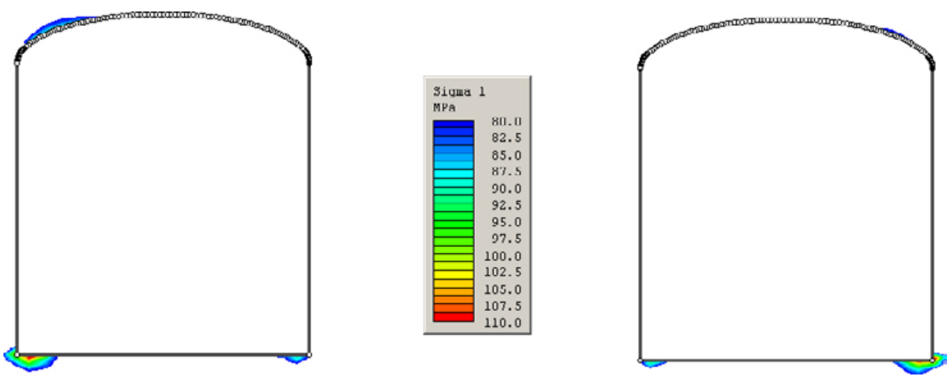


***Figure 4-2.** A detailed presentation of stressed areas (>80 MPa) in Cavern B and C.*

/Martin 1997/ showed that for intermediate stress environments (roughly corresponding to 250–1,500 m depth for virgin stress states similar that at Forsmark), a flat tunnel roof is more stable than an arched roof, with respect to stress-induced brittle failure (spalling). For low-stress environments (<250 m depth), an arched roof is preferable as this results in a small zone of unloading, thus reducing the potential for structurally-controlled failures. For very high stresses, an arched roof is also preferable, as a flat roof then results in a larger volume of failed rock. It was also shown that once failure initiates above a flat roof, the advantages of a flat roof quickly diminishes, and rock bolts also become less efficient in this situation, compared to an arched roof.

For the case of the main tunnels, located at moderate depth, a flatter roof may be slightly more advantageous than an arched roof, provided that very little failure is expected. However, this advantage regards spalling failure and must be set in relation to possible structurally-controlled failures (block fall-outs). Thus, a reasonable comprise may be to retain a slight curvature of the roof, to reduce the unloading zone to some extent (compared to a flat roof), while at the same time providing a reduced potential for spalling failure (compared a strongly arched roof).

Calculations were made to evaluate the relative difference in roof stress. The three profiles were analyzed between 55° and 145°, i.e. orthogonal to parallel in relation to the major principal stress.

The result of the calculations shows that the profile of the roof has an impact on the calculated stresses. The highest stresses in the roof and springline are presented in Table 4-7. It is noticed that the reference case profile gives the lowest stress peaks, hence the stresses are distributed smoothly resulting in lower peak values. All calculations indicate that spalling should not occur in the main tunnels and that the profile according to /SKB 2008b/ gives low stress peaks.

Table 4-7. Results from calculations of maximum stresses in roof and springline on three different roof profiles.

Case	Profile	Maximum calculated stress [MPa]	Position of maximum stress
55°	Wall height +0.4 m	100	Springline
	Wall height +0.2 m	85	Springline
	Reference case	76	Springline
85°	Wall height +0.4 m	97	Springline
	Wall height +0.2 m	84	Springline
	Reference case	74	Springline
115°	Wall height +0.4 m	88	Springline
	Wall height +0.2 m	78	Springline
	Reference case	69	Springline
145°	Wall height +0.4 m	78	Springline
	Wall height +0.2 m	69	Springline
	Reference case	61	Springline

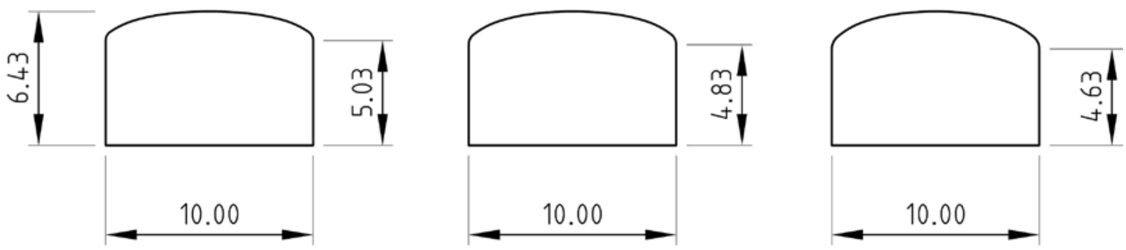


Figure 4-3. *Illustration of the three different sections of the deposition tunnels that have been analyzed. The section to the right corresponds to the reference case section. In the left hand side section the wall height has been raised 0.4 m and in the middle section the wall height has been raised 0.2 m compared to the reference case.*

4.6 Tunnel crossings

The stress concentrations in crossings between the main and the deposition tunnels have been analyzed in a three-dimensional model and using the Examine^{3D} software. Both orthogonal and skewed crossing have been analyzed as illustrated in Figure 4-4. The models consist of 40×40 m main and deposition tunnels with a layout according to /SKB 2008b/.

Crossings have been studied where the deposition tunnel is skewed towards the main tunnel between 0 and 90° in intervals of 15°. In the calculations the stress fields have been set to expected values at the depth levels of 500 m.

The orthogonal crossings have been analysed for five different orientations in relation to the stress field according to Table 4-8 and Figure 4-5. Direction 55° is when the deposition tunnel is orthogonal to the major principal stress (σ_H). The system is rotated in steps of 30° until the deposition tunnel is aligned with the major principal stress that is in the direction 145°. One additional case (K3b) is calculated corresponding to the most likely direction of the stress field including the uncertainty of the direction, i.e. 145°–15°=130°.

With the skewed crossing all calculations were made for two different directions of the main tunnel, 25° and 55°. Based on these directions on the main tunnel, the different models with skewed deposition tunnel are calculated, according to Table 4-9 and illustrated in Figure 4-6. It is noticed by comparing cases K3v-15° and K4v-15° that the result is only insignificantly affected by the angle between the main and the deposition tunnel.

In Table 4-10 the result of the calculations are presented. It is seen that the maximum stress is 102 MPa, found in case K1. It is further seen that the point where the maximum stress is found varies.

Table 4-8. Cases for calculation with orthogonal crossing.

Case	Direction of deposition tunnel	Description
K1	55°	Deposition tunnel 90° towards σ_H
K2	85°	Deposition tunnel 60° towards σ_H
K3	115°	Deposition tunnel 30° towards σ_H
K3b	130°	Deposition tunnel 15° towards σ_H
K4	145°	Deposition tunnel 0° towards σ_H

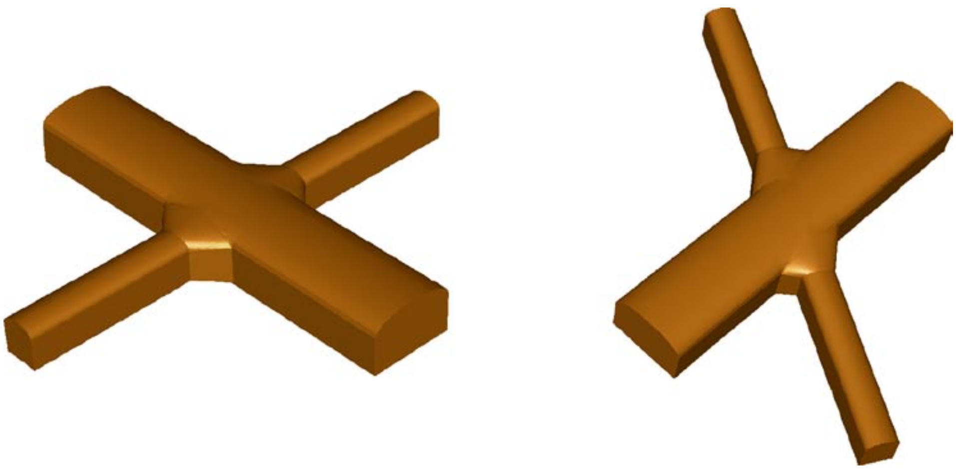


Figure 4-4. Layout for studying of spalling in tunnels and crossings. The orthogonal (90°) crossing is shown to the left and to the right a crossing where the deposition tunnel is skewed 30° towards the main tunnel.

Figure 4-5. *Illustration of different cases with orthogonal crossing.*

Table 4-9. Cases for calculation with skewed crossing.

Case	Direction of main tunnel	Direction of deposition tunnel	Description
K3v-15°	25°	160°	Deposition tunnel 15° from σ_H
K3v-30°	25°	145°	Deposition tunnel 0° from σ_H
K3v-45°	25°	130°	Deposition tunnel 15° from σ_H
K4v-15°	55°	130°	Deposition tunnel 15° from σ_H
K4v-30°	55°	115°	Deposition tunnel 30° from σ_H
K4v-45°	55°	100°	Deposition tunnel 45° from σ_H

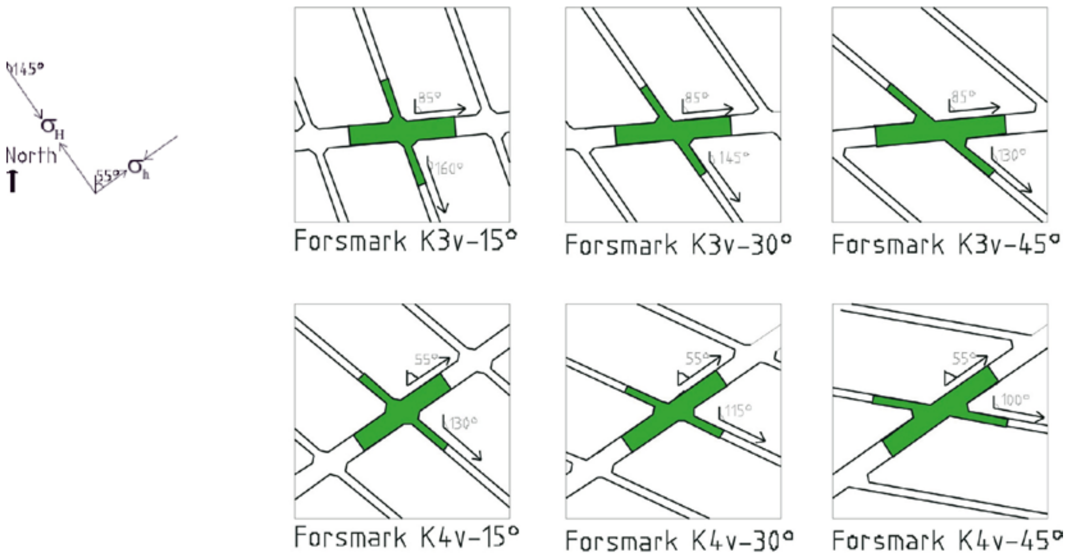


Figure 4-6. *Illustration of different cases with skewed crossing.*

Table 4-10. Result for the different cases.

Case	Maximum calculated stress [MPa]	Position of maximum stress
K1	102	Centre of roof of deposition tunnel
K2	94	Centre of roof of deposition tunnel
K3	80	In the springline in the crossing
K3b	79	In the springline in the crossing
K4	75	In the springline in the main tunnel
K3v-15°	79	In the springline in the crossing, the sharp side
K3v-30°	78	In the springline in the crossing, the sharp side
K3v-45°	77	In the springline in the crossing, the sharp side
K4v-15°	78	Springline of the main tunnel, next to crossing on the blunt side.
K4v-30°	80	Springline of the main tunnel, next to crossing on the blunt side.
K4v-45°	82	Springline of the main tunnel, next to crossing on the blunt side.

4.7 Deposition holes

To analyze the stress concentration in the deposition holes a three-dimensional model was used where a part of the deposition tunnel and five deposition holes were modelled, as shown in Figure 4-7.

The stress concentration in the deposition holes is studied in two cases of in-situ stress level, one according the Most Likely stress model (according to Table 4-1) and one at the Maximum Stress model (according to Table 4-2).

In the elastic three-dimensional study, calculations were made with the deposition tunnel in four different directions according to Table 4-11. The different cases are also illustrated in Figure 4-8. The deposition tunnels are positioned in areas with ground types GT1 according to /SKB 2008b/.

An example of how the stresses are distributed in the deposition holes is shown in Figure 4-9. This shows the stresses along a section 0°, 90° and 180° relative the tunnel length axis and the maximum calculated stress along the hole for Case D1. The results of the calculations are further presented with diagrams in Appendix A.

Table 4-11. Cases for calculation for spalling in deposition holes. D1–D4 are made at the expected stress level and D1E–D4E are made at the elevated stress level.

Case	Orientation of deposition tunnel	Description
D1/D1E	55°	Deposition tunnel 90° towards σ_H
D2/D2E	85°	Deposition tunnel 60° towards σ_H
D3/D3E	115°	Deposition tunnel 30° towards σ_H
D4/D4E	145°	Deposition tunnel 0° towards σ_H

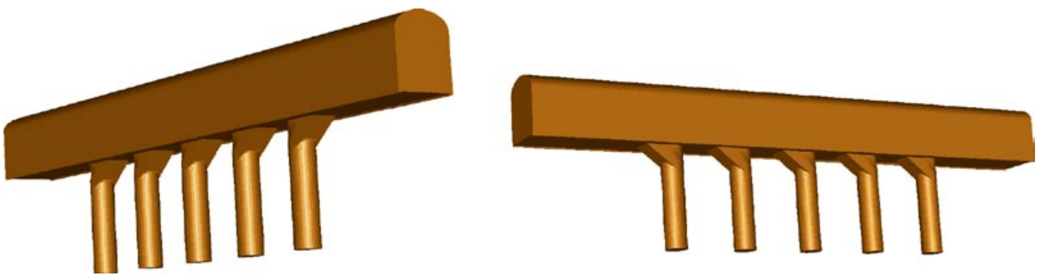


Figure 4-7. Illustration of model used in the calculations. The tunnel is 60 m and includes five deposition holes. The model includes the removed edge of the deposition hole, which is seen in the figures.

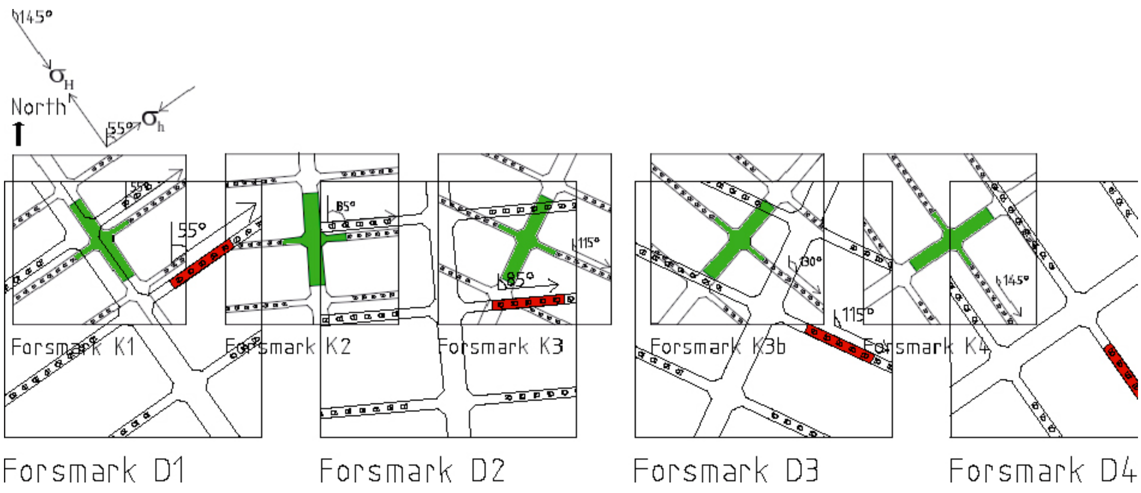


***Figure 4-8.** Illustration of the different cases for calculation of spalling in deposition holes.*

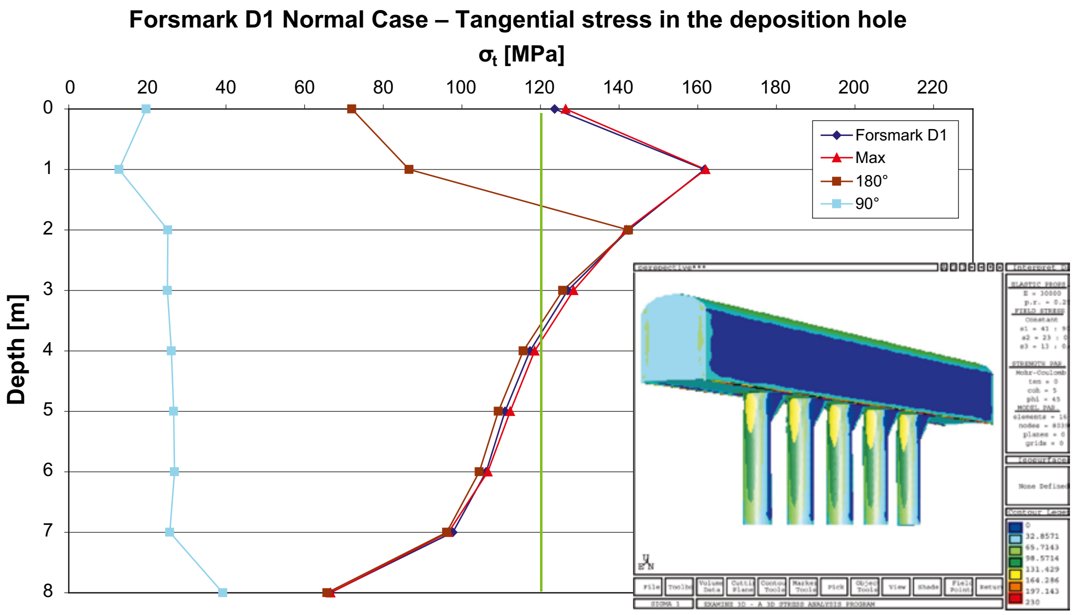


***Figure 4-9.** Example of calculated result for Case D1, i.e. where the deposition tunnel is orthogonal to the maximum principal stress. The figure shows the stress distribution along the deposition hole for a section 0°, 90° and 180° relative the tunnel length axis and also the calculated maximum value. The expected spalling strength (120 MPa) is marked with a green line.*

The results of the elastic three-dimensional study on the deposition holes at the expected stress level are summarized in Table 4-12 where the maximum calculated stress in the modelled deposition holes are presented. The maximum stress occurs at about 1 m from the top of the deposition holes. In Table 4-12 the depth interval where the stresses are noted as being higher than 120 MPa is also presented. Figure 4-10 shows the stress distribution in the deposition hole in calculation of Case D1 and illustrates the difference between an outer and an inner hole.

Table 4-12. Calculated result from the 3D study of spalling in deposition holes at the expected stress level.

Case	Maximum stress [MPa]	Depth interval where the stress is higher than [120 MPa]
D1	162	Deposition hole, 0–3.5 m
D2	159	Deposition hole, 0–3.3 m
D3	127	Deposition hole, 1–2.5 m
D4	87	–

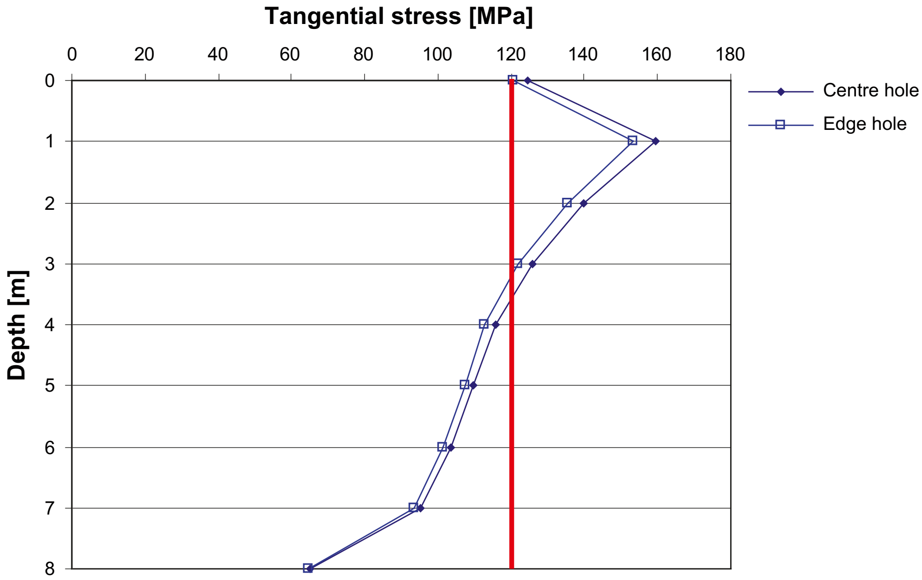


Figure 4-10. Example of calculated result for Case D1, i.e. where the deposition tunnel is orthogonal to the maximum principal stress and at the expected stress level. The two curves are the centre hole and one of the outer holes. The expected spalling strength (120 MPa) is marked with a red line.

The calculated maximum tangential stress on the boundary of the depositions hole as function of orientation of the deposition tunnels is illustrated Figure 4-11. It is noticed that the spalling strength is exceeded at an orientation of around ±25° the trend of the maximum horizontal stress. In the diagram the spalling depth (S_d) evaluated according to Equation 4-5 /Martin 2005/ is also shown.

$$S_d = a\left(0.5\frac{\sigma_{sm}}{\sigma_{\theta\theta}} - 0.52\right) \; for \; \sigma_{\theta\theta} \succ \sigma_{sm} \qquad (4\text{-}5)$$

where

ạ = radius of the deposition hole

$\sigma_{\theta\theta}$ = tangential stress

σ_{sm} = spalling strength

The results of the elastic three-dimensional study on the deposition holes at the elevated stress level are presented in Table 4-13 where the maximum calculated stress in the modelled deposition holes are presented. In Table 4-13 the depth interval where the stresses are noted as being higher than 120 MPa is also presented. It is found that in Case D1E–D3E the spalling strength is exceeded almost over the full depth of the hole.

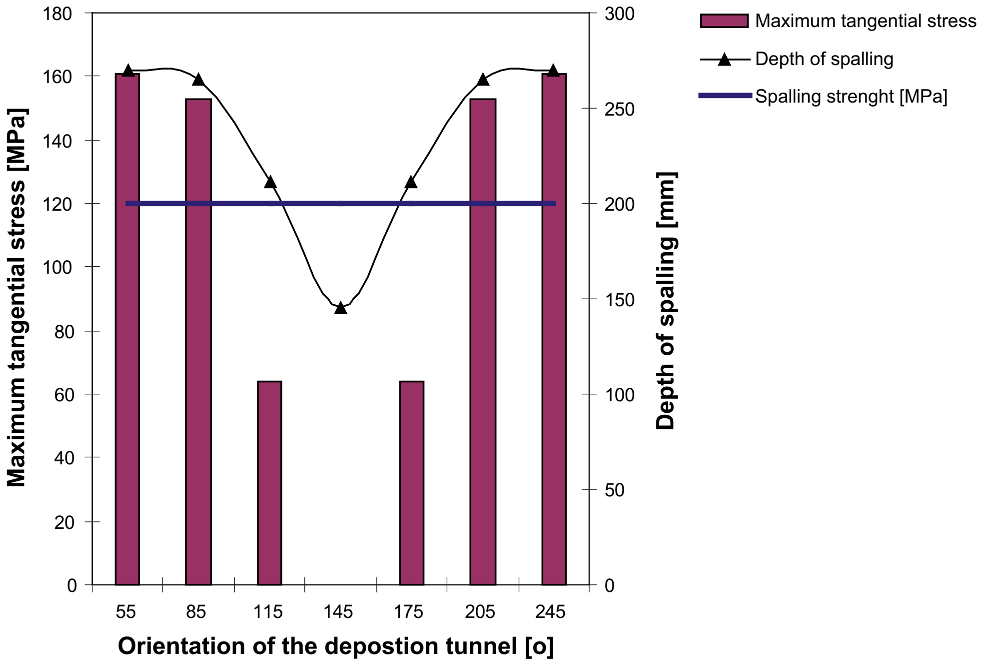


Figure 4-11. *Maximum tangential stress and depth of spalling at the most likely stress level based on the three-dimensional calculations as function of orientation of the deposition tunnel.*

Table 4-13. Calculated result from the 3D study of spalling in deposition holes at the elevated stress level.

Case	Maximum stress [MPa]	Depth interval where the stress is higher than [120 MPa]
D1E	222	Deposition hole, 0–7.0 m
D2E	217	Deposition hole, 0–7.0 m
D3E	175	Deposition hole, 0–7.0 m
D4E	117	–

4.8 Conclusions and discussion

No spalling is anticipated in the parts of the Central area that are located in GT1, provided that the orientation is not orthogonal to the major principal stress orientation. In GT2 and 3, spalling is possible in limited areas if the orientation in orthogonal to the major principal stress. In any other choice of direction spalling is not foreseen at all.

No spalling is found to occur in the tunnels or in the tunnel crossings, independently if the crossings are orthogonal or skewed and independent of the orientation relative to the stress field. It is also shown that the spalling potential in the tunnels can be further reduced using a slightly lower roof profile.

The calculations have been made mainly based on the most likely stress magnitudes. Spalling is not generally found to be a problem in the caverns or the tunnels. There is, however, a possibility that the stresses in general or locally will be higher than the expected. Spalling can therefore not be excluded.

The issue of spalling in deposition holes was studied in a deterministic study. It was found that the factor of safety for domain RFM029 is 1.26, i.e. verges on the limiting value of 1.25.

The spalling potential on the deposition holes was furthermore analysed in a three-dimensional model, based on the most likely and the maximum stress model. If the orientation of the deposition tunnels is in an unfavourable direction relative to the stresses, spalling is expected to occur along the upper part of the deposition hole with the most likely stress magnitudes. This is found to depend on the increased stress concentration that is noticed in the upper part of the deposition holes. With an orientation of the deposition tunnel parallel to the major principal stress, the maximum peak stress is less than the spalling strength. Using to the maximum stress model, spalling in the deposition holes are foreseen in all cases except for an orientation parallel to the major principle stress.

5 Support types

5.1 Introduction

For the current design, SKB proposes five different support types for tunnels and one for caverns in /SKB 2008b/. A summary is given in Table 5-1. The task is to determine the appropriate support measures on the basis of this, considering details such as bolt type, sealing, and length, as well as shotcrete thickness.

Since all parts of the repository except for the deposition tunnels should have a minimum reinforcement of shotcrete (see Reference Design), and that assigned combinations of ground types and ground behaviour only to a limited extent occur as examples in Table 5-1, the support types given by SKB need to be modified. It has also been our ambition to maintain continuity of support types in order to facilitate upgrading based on the observational method in the event that the reinforcement is inadequate. Support types ST1 and ST2 have, therefore, been supplemented with 30 and 50 mm of fibre-reinforced shotcrete, respectively, in the roof and the uppermost metre of the walls, while the deposition tunnels and caverns in the central area were each assigned a support type (see Table 5-2).

The main purpose with the shotcrete is to protect such installations and facilitate maintenance. Since the shotcrete will facilitate the detection of brittle failure, it is also an important part in the application of the observational method. Therefore, it is assessed that a thickness of 30 mm would be fully adequate. This is a minimum thickness, since thinner shotcrete may increase the risk of dehydration and hence loosening.

Since none of the walls according to the Reference Design have fixed installations (except for drainage), there are no arguments for shotcrete on the walls. The motive for the uppermost metre of shotcrete on the walls is entirely due to practical problems to yield the sharp transition between roof and walls, as given in appendix 1 of /SKB 2008a/. However, walls should be thoroughly reinforced with selective bolting.

The most crucial aspects for the quantitative details of the proposed support types have been to facilitate maintenance and protect installations, as well as the application of the observational method. The Q-system has then been used to verify the sufficiency of the suggested reinforcement. Since the Q-system does not consider the abovementioned aspects, the proposed support efforts generally are an over-reinforcement in respect to direct block falls. The safety margins in the proposed reinforcement, strongly suggest that the sufficiency becomes apparent also with alternative analyse methods to the Q-system.

Table 5-1. Summary of support types (ST) proposed by the /SKB 2008b/.

Support type	Description	Example of ground types	Example of ground behaviour
ST1	Spot bolting	GT1	GB1
ST2	Systematic bolting	GT1, GT2	GB1, GB2A
ST3	Systematic bolting + wire mesh	GT1, GT2	GB1
ST4	Systematic bolting + fibre-reinforced shotcrete	GT1, GT2, GT3	GB1, GB2B
ST5	Concrete lining	GT4	GB3
STC	Systematic bolting + fibre-reinforced shotcrete	All	GB1, GB2

Table 5-2. Summary of modified support types (ST).

Support type	Description	Ground types	Ground behaviour
ST1	Fibre-reinforced shotcrete 30 mm in roof + uppermost 1 m of walls. Spotbolting: 1 bolt/50 m² in roof and walls (ø 25 mm, length 3 m).	GT1	GB1, GB2A
ST2	Fibre-reinforced shotcrete 50 mm in roof + uppermost 1 m of walls. Spot bolting: 1 bolt/50 m² in walls (ø 25 mm, length 3 m). Systematic bolting: c/c 2 m in roof (ø 25 mm, length 3 m).	GT2, GT3	GB1, GB2A, GB3B
ST3	Fibre-reinforced shotcrete 75 mm in roof + uppermost 1 m of walls. Spot bolting: 1 bolt/50 m² in walls (ø 25 mm, length 3 m). Systematic bolting: c/c 1 m in roof (ø 25 mm, length 3 m).	GT4	GB1, GB2A, GB3B
ST4	Concrete lining.	GT4	GB2B, GB3A
ST Deposition	Wire mesh in roof + uppermost 1 m of walls for GT2 and GT3. Spot bolting: 1 bolt/50 m² in roof and walls (ø 25 mm, length 3 m).	GT1, GT2, GT3	GB1, GB2A
ST Cavern	Fibre-reinforced shotcrete 50 mm in roof + uppermost 1 m of walls. Spot bolting: 1 bolt/50 m² in walls (ø 25 mm, length 3 m). Systematic bolting: c/c 2 m in roof (ø 25 mm, length 3 m).	GT1, GT2, GT3	GB1, GB2A

5.2 The impact of various ground types

General Q-values for various ground types are given in Table 2-1. GT2 and GT3 have values in the interval 100–40 (‘very good rock’) or 40–10 (‘good rock’). However, since the Q-values are general for each ground type, there may well be local occurrences of weaker, more blocky rocks. Systematic bolting of rock with lower Q-values may hence be needed in order to maintain a load-bearing arch. The differences in reinforcement based on the Q-system are, however, small enough for both GT2 and GT3 to be virtually the same support type.

The expectations of GT1, with a Q-value above 100, is that the rock has very few fractures and that reinforcement of blocks will therefore only be needed in exceptional cases. It should be possible to take care of smaller blocks in the roof and springline with shotcrete, while reinforcement of larger blocks may be supplemented with selective bolting, possibly with 3 m bolts without washers.

In summary, this means that the parts of the installation classified as GT1, under the expected stress conditions, may be treated with ST1. For other combinations of rock classes and ground behaviour, as assumed in the repository under the expected stress conditions (i.e. GT2–GB1, GT2–GB3B, GT3–GB1), it is suggested that ST2 will be a suitable support type. Under unfavourable rock conditions in combination with large spans, it should also be possible to use ST3 for parts of the installation classified as GT2 and GT3. In the case that very poor rock conditions are encountered, for example flowing ground (GB3A), there is also a ST4, which is made up of concrete lining. Table 5-3 gives a summary of the assigned support types for the sub-surface facilities of the repository.

5.3 Spalling

Spalling-induced failure is treated with reinforced shotcrete, wire mesh or bolt reinforcement, using large washers /Stille et al. 2005, Kaiser et al. 1996, Hoek and Brown 1980/. References are mainly practically based, as a theoretical description has not yet been found. Practical experience from mines on great depths shows that a small confinement is sufficient to prevent progressive spalling. In /Andersson 2007/, it is concluded from the Apse tunnel that small confinement from a rubber bladder was enough to stop spalling.

Based on the stress analyses it is assessed that spalling will occur very locally and there is no reason to reinforce for a progressive fracture process. Tough and well-applied shotcrete reinforcement is thought to be suitable from a rock mechanical viewpoint. This is, however, included in all proposed support types in Table 5-2, with the exception of deposition tunnels where shotcrete is not permitted.

Table 5-3. Support types assigned for various facility parts of the repository.

Facility part	ST1 [m]	ST2 [m]	STC [m]	STD [m]
Ramp (model file: 191BR_00_3001)				
Tunnel (5.5 m wide)[1]	3,707	353	–	–
Tunnel (6.0 m wide)	113	6	–	–
Tunnel (7.0 m wide)	651	47	–	–
Passing places (8.0 m wide)	123	12	–	–
Niche (5.5 m wide)	19	1	–	–
Niche (7.0 m wide)	123	6	–	–
Niche (10.0 m wide with 5x5x16 m below)	35	3	–	–
Ventilation (model file: 191BC_00_3002)				
Shaft (ø 1.5 m)	245	13	–	–
Shaft (ø 2.5 m)	365	81	–	–
Shaft (ø 3.5 m)	364	82	–	–
Shaft (ø 4.5 m)	24	1	–	–
Tunnel (4.0 m wide)	893	115	–	–
Central area (model file: 191BC_00_3001)				
Skip shaft (ø 5.0 m)	492	32	–	–
Elevator shaft (ø 6.0 m)	411	83	–	–
Silo (ø 9.5 m)	21	1	–	–
Tunnel (3.0 m wide)	120	14	–	–
Tunnel (4.0 m wide)	28	1	–	–
Service tunnel (4.0 m wide)	475	25	–	–
Tunnel (5.1 m wide)	46	2	–	–
Tunnel (7.0 m wide)[2]	729	183	–	–
Caverns (13.0–16.0 m wide)	–	–	541	–
Sump (12.0 m wide)	–	–	20	–
Electricity hall (7.0 m wide)	–	–	21	–
Crushing hall (10.3 m wide)	–	–	22	–
Deposition area				
Ventilation shafts SA01 and SA02 (ø 3.0 m)	833	102	–	–
Main tunnel (10.0 m wide)	4,497	1,976	–	–
Transport tunnel (7.0 m wide)	3,876	753	–	–
Deposition tunnel (4.2 m wide)	–	–	–	61,189

5.4 Shafts, caverns and deposition tunnels

Deposition tunnels have been given one type of reinforcement, ST Deposition. One reason is that the use of shotcrete is not permitted. The significantly shorter lifetime compared with other parts of the installation is another reason. The orientation of the tunnels also means that any need for reinforcement to prevent spalling may be disregarded. It is, therefore, considered that selective bolting is fully adequate reinforcement for the combinations of ground types and ground behaviour assigned to the deposition tunnels. Occurrences of poorer rock quality as in GT2 and GT3 will be treated with wire mesh.

Although the orientation not directly promotes gravitational block falls, it is recommended to threat all shafts with shotcrete, due to both the height of possible rock falls and maintenance difficulties, especially in the skip and elevator shaft. Also, because space is restricted, a shorter bolt length than in other parts of the repository may be required.

A general support type, corresponding to ST2, should be applied to the so-called caverns in the central area. The definition of caverns includes parts of the installation where constant activity is thought to occur. This applies mainly to nine caverns with spans of 13–16 m, but also to smaller spaces, which cannot be defined as tunnels or shafts. Apart from the Elevator hall, which is intersected by deformation zone ZFMNNW1205, all of the central area caverns are located in very to extremely good rock (95% GT1 and 5% GT2). With regard to working environment and maintenance aspects, as well as, for example special requirements regarding fire safety, bolting and reinforced shotcrete to the same extent as GT2 are proposed.

5.5 Drainage

The need of draining of shotcrete needs to be considered. Fracture domain FFM01 has permeability very similar to that of intact rock. Similarly, the steep deformation zones, which are mainly made up of sealed fractures, have a relatively low transmissivity according to /SKB 2008b/. The need for draining at repository depth is therefore thought to be small, assuming that GT4 is not encountered. Near the surface in fracture domain FFM02, transmissivity is significantly higher, especially near flat-lying deformation zones. A need for drainage is therefore expected in the part of the ramp that encounters GT2 in FFM02.

6 System behaviour

System behaviour refers to the interaction between reinforcement and the rock mass. The intention is to show that the system is stable, i.e. that the proposed reinforcement will work in relation to ground behaviour.

The analysis is carried out for (a) the most probable system behaviour, and (b) the most unfavourable system behaviour.

6.1 Analysis methods

In accordance with /SKB 2008a/, analyses should be applied in rock reinforcement design work to verify the system behaviour, i.e. the interaction between the ground behaviour of the construction measures.

The system behaviour is analysed using different methods for various parts of the repository. In the upper parts of the repository, down to a depth of around 40–50 m (i.e., fracture domain FFM02), the comparison is generally based on reinforcement experience from the SFR, which according to /SKB 2008b/ is a relevant comparison. In the deeper parts, reinforcement solutions are verified by individual analytical calculations.

Three methods are applied for analyses:

- Experience from comparable excavations.
- The Q-system.
- Analytical calculations of load-bearing capacity for rock reinforcement.

6.1.1 Experience from SFR

A description of reinforcement experiences from SFR is given in /Carlsson and Christiansson 2007/. In SFR, four rock classes with description and reinforcement in accordance with Table 6-1 are applied.

Table 6-1. Summary of rock classes and installed reinforcement in the SFR /Carlsson and Christiansson 2007/.

Rock class	Description	Reinforcement
1	Sparsely fractures rock, may contain all dominant fracture sets, but seldom as significant clusters.	Bolting to maximum 1 bolt/4 m². Shotcrete: 0–50 mm.
2	Clusters of sub-horizontal to gently dipping fractures forming minor deformation zones. Local occurrence of vertical fracture sets form lenses of crushed rock. Indications of heterogeneous and discontinuous fracture zones.	Bolting: 1/4 m² – 1/2 m². Shotcrete: 50–80 mm fibre-reinforced.
3	Clusters of steeply dipping fractures, locally forming smaller deformation zones; the majority strike north-eastward and are sealed with calcite + laumontite. Also, less frequent north-westerly striking fractures, locally forming deformation zones. In additon, altered amphibolite dykes, striking north-south.	Bolting: 1/4 m² – 1/2 m². Shotcrete: 50–80 mm fibre-reinforced.
4	Major deformation zones: Singö zone; heterogeneous, with core zone of clay alteration and crushed rock.	Bolting: 1/4m² – pre-bolting. Shotcrete: 80 mm fibre-reinforced to 300 mm shotcrete arch.

6.1.2 The Q-system

The Q-system /Barton et al. 1974/ and its recommendations for rock reinforcement have be used for the analyses. The starting points are the Q-value given in /SKB 2007b/ and the geometries given in the reference design. It should, however, be emphasized that the Q-system has been used primarily to verify the suggested reinforcement, which rather aims at facilitating the maintenance and being part of the observational method than to prevent direct block falls.

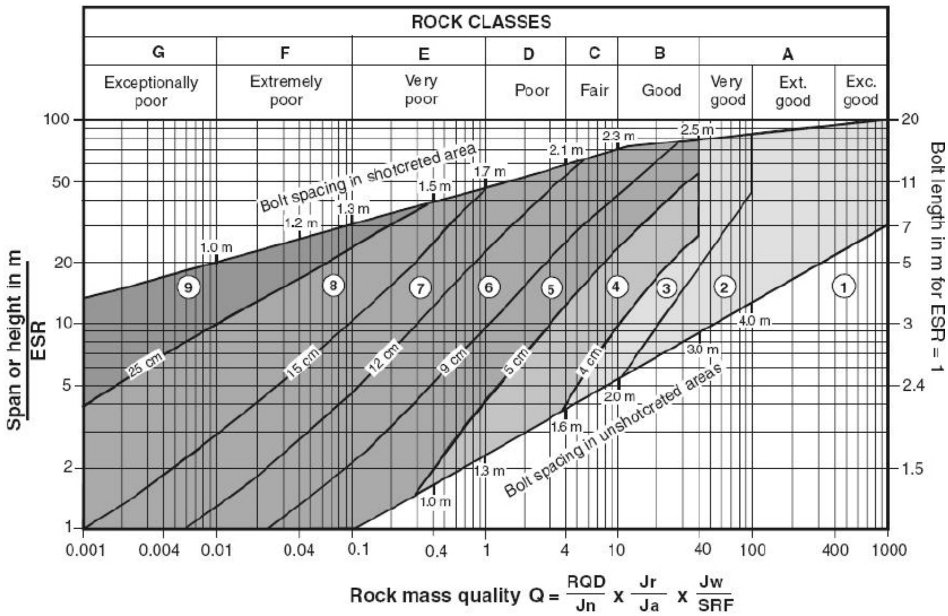
The Q-system is well known and accepted as the classification system for Scandinavian conditions. The objections and criticisms that are often put forward chiefly concern the application of Q-system support types or other deviations in the applicability of the Q-system. The criticism of the support types is that they give an over-reinforced and expensive system.

The Q-system is based on following up a number of tunnel projects where reinforcement was set in relation to various rock parameters. The result was a diagram with Q-value on the x-axis and tunnel dimension on the y-axis which enabled the proposed reinforcement to be read off. A major updating of the Q-system was presented by /Grimstad and Barton 1993/ and several supplements have since been published. The 1993 version is the one used in the present work Figure 6-1.

6.1.3 Analytical calculations on load bearing capacity on reinforcement

Simple analytical calculation is used here to estimate the reinforcement effect that can be required or attained from various approaches. Analyses are performed in accordance with Banverket's Design Instructions /Lundman 2006/ and the reinforcement of individual blocks. The analyses do not account for the stress situation found at this depth. More advanced analyses; using numerical models could be made to include if the blocks are locked by high stresses if the blocks are pushed out by the stresses. The analyses are considered relevant to give an indicative picture of the block sizes supported by the proposed reinforcement.

The analyses are carried out inversely, in other words the total load bearing properties of a bolt or shotcrete are converted to a block size. This approach provides an estimate of the block sizes handled by the bolt reinforcement.



REINFORCEMENT CATEGORIES:

1) Unsupported
2) Spot bolting
3) Systematic bolting
4) Systematic bolting, (and unreinforced shotcrete, 4 - 10 cm)
5) Fibre reinforced shotcrete and bolting, 5 - 9 cm
6) Fibre reinforced shotcrete and bolting, 9 - 12 cm
7) Fibre reinforced shotcrete and bolting, 12 - 15 cm
8) Fibre reinforced shotcrete, > 15 cm, reinforced ribs of shotcrete and bolting
9) Cast concrete lining

Figure 6-1. Support types of the Q-system /Grimstad and Barton 1993/.

Shotcrete

The following expression /Fredriksson 1994/ is used to evaluate the capability of the shotcrete to carry the block with respect to adhesion failure:

$$W \le \frac{\sigma_{adk} \cdot \delta \cdot O_m}{\gamma_n \cdot \eta \cdot \gamma_m} \quad , \tag{6-1}$$

where

W = weight of the block [N]

σ_{adk} = characteristic adhesion strength [Pa]

δ = shotcrete carrying thickness layer [m]

O_m = circumference of load-bearing surface between shotcrete and rock [m]

γ_n, η, γ_m = partial coefficients

The following values are used for the estimate:

- The weight of the block is calculated on the assumption of a block with a shape of a pyramid. The sides have a particular angle (α), see Figure 6-2.
- The characteristic adhesion strength is recommended to be 0.5 MPa corresponding to common practice.
- The shotcrete carrying thickness is assumed to be half of the thickness of the shotcrete layer based on the values given in /Holmgren 1979/.
- Partial coefficients product is set to 1.5 (Safety Class 3), which is considered reasonable due to the type of facility and a life of 100 years.

Figure 6-3 shows the results of the calculation graphically for three different thicknesses of shotcrete: 30, 50 and 70 mm. Assuming a side angle of the block between 45° and 60°, 30 mm of well-applied shotcrete can support a block volume of around 1.2–1.5 m^3.

Rock bolts

The load bearing capacity of the bolt (B_{max}) is calculated using Equation 6-2 where it is assumed that the bolt is fully grouted but not tensioned.

$$B_{max} = \sigma \cdot A/F_s \tag{6-2}$$

where

σ = yield limit [Pa]

A = area [m^2]

F_s = factor of safety

Based on a diameter of 25 mm, a yield limit of 500 MPa and a safety factor of 1.5, the load bearing capacity is estimated to be 160 kN.

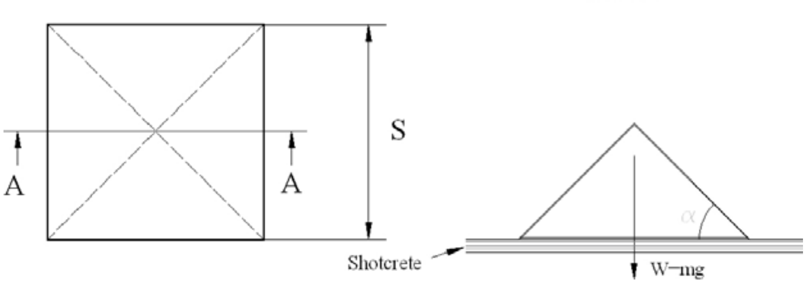


***Figure 6-2.** Illustration of a block in shape of a pyramid. Modified from /Lundman 2007/.*

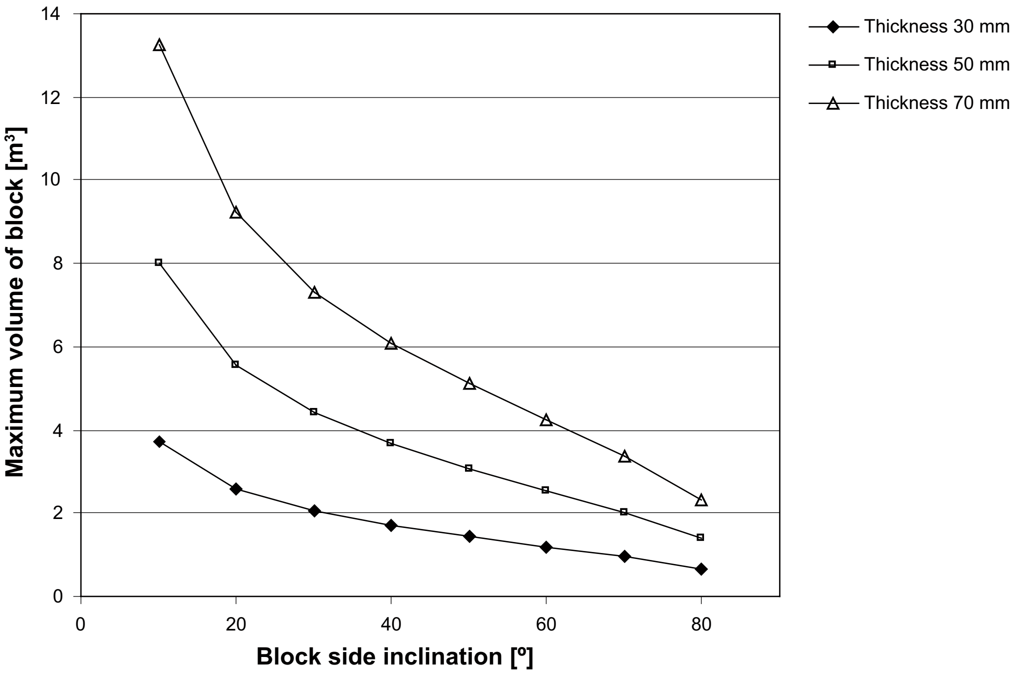


Figure 6-3. *Calculated values of block size carried by the shotcrete.*

The volume of the block that can be carried by the bolt is calculated using Equation 6-3.

$$V = \frac{B_{max}}{\gamma} \qquad (6\text{-}3)$$

where

γ = unit weight of the block [N/m^3]

Using the unit weight 27 kN/m^3 it is found that a block with the volume of 5.9 m^3 is carried by the bolt.

It must also be verified that the bolt length is long enough to penetrate and provide enough reinforcement for the block. If it is assumed that the block is shaped like a pyramid according to Figure 6-4, the block volume is described by Equation 6-4.

$$V = \frac{S^3 \cdot \tan\alpha}{3} \qquad (6\text{-}4)$$

In Figure 6-4 the results of calculations are presented for 2, 3 and 4 m long bolts and assuming at least 1 m support length in the rock. It is noted that the bolt length is the limiting factor for blocks with a steep side. For 3 m long bolts in particular, blocks with a side inclination of around 45° are fully supported but with steeper sides the block volume starts to be limited.

Effect of minimum reinforcement

The intention of the installed reinforcement is partly to provide support against rock failure, and partly to reduce the need for maintenance and periodic inspection. Choice of type of reinforcement also has a purpose. A reason for using shotcrete is to facilitate checks on fracture formation for example.

The reinforcing effect obtained should deal with the most probable case, but not the worst case. Instead, the principles of the observational method should be applied, i.e. if it is noted by measurements or observations that reinforcement is insufficient, it should then be increased.

According to calculations (Figure 6-4), one bolt of three metres length is sufficient to hold a block of about 6 m^3. This value is based on the geometrical assumption that the bolt should pass through and reach a satisfactory anchoring length.

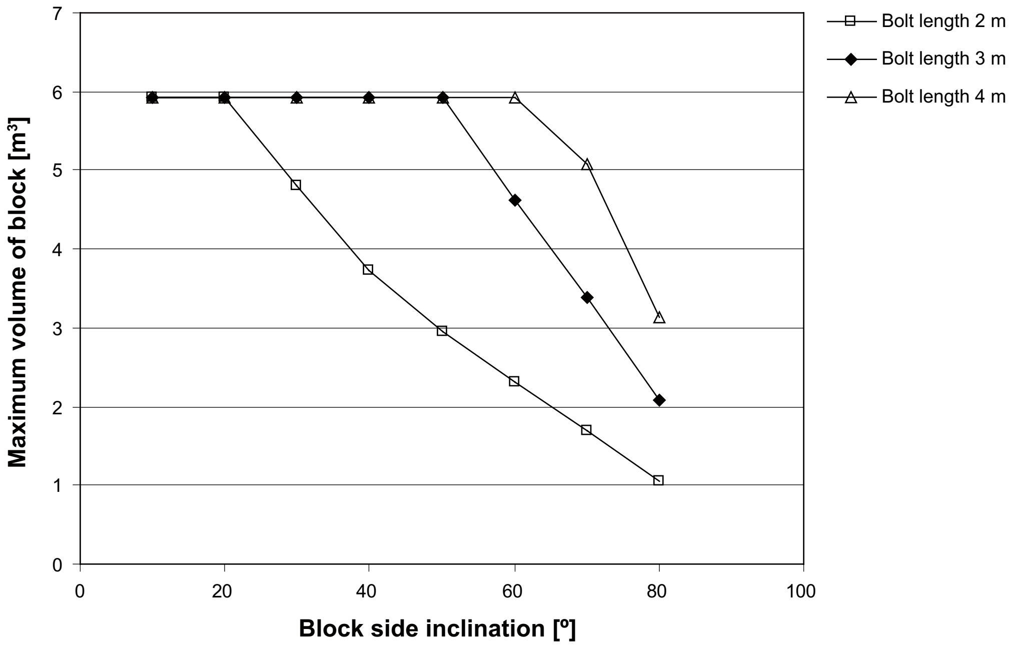


Figure 6-4. *Calculation of maximum block volume for 2, 3 and 4 m bolts and for different dip on block side. It is noted that the bolt length is the limiting factor for blocks with a steep side.*

A 30 mm thick shotcrete can also support a block of approximately 0.5 m^3 if the form of the block is conical with a 45–60° side inclination. The indicative calculations of the block sizes that can be handled by the minimum reinforcement are used to give acceptable limits for block sizes in the application of the observational method.

6.2 Most probable system behaviour

The expected distribution of ground behaviour was established in Chapter 4. The foreseen failures are gravity driven, mostly discontinuity controlled (i.e. GB1). In relation to this, support types were established in Chapter 5.

Since a vast majority of the underground facility is expected to fall within GB1 it is essential that the system behaviour of ST1 and GB1 is established as being within acceptable limits.

6.2.1 Experience from SFR

A comparison with the ground types given in /SKB 2008b/ shows that GT 1 is somewhat better or closely matched to rock class 1 in the SFR. Also, there is a general consensus regarding GT2 and GT3. It has, however, been assumed that what in the SFR consists of rock class 3 is made up of both GT2 and GT3 in the present study. Rock class 4 in the SFR corresponds to GT4, the occurrence of which is limited to larger deformation zones with a respect distance.

From a reinforcement point of view, rock class 1 in the SFR is higher than ST1, with a maximum of 1 bolt/4 m^2, compared with an estimate of 1 bolt/50 m^2. The proposed systematic bolting in ST2, which covers both GT2 and GT3, is on the other hand almost identical (c/c 2 m) with that of the SFR (1 bolt/2–4 m^2). The total amount of reinforcement proposed is therefore slightly less than that for SFR.

The parts of the repository located in FFM02 (i.e., approximately the same depth as SFR), covers about 450 m of the ramp, as well as the upper parts of the elevator, skip, and ventilation shafts. Regarding the shafts, there is a blasted shaft to a pump station. No appreciable stability problems occurred in this shaft.

Also a comparison for the ramp is highly relevant. The entrance to the ramp has an orientation that coincides with the flat-lying, transmissive structures that characterise FFM02, and is assumed to dip steeply towards the south-east or south. Occurrence of sub-horizontal fractures in the tunnel roof normally need systematic bolting. In the SFR, fallout in the roof and towards the springline where the tunnels coincide with swarms of these flat-lying fractures was commonly noted /Carlsson and Christiansson 2007/. The predicted proportion of the ramp where this type of fracture swarms (GT2–GB3B) occur is 10%. Additional reinforcement in the walls around the actual entrance could be necessary, but this can be dealt with as part of the observational method.

Another situation that required extra reinforcement activity in the form of systematic bolting during the construction of the SFR was where the orientation of the tunnels coincided with the occurrence of steep, north-easterly striking fractures. This type of fracture zone dominates the modelled zones in the deposition area. Parts of the main tunnels have been deliberately localized to the north-easterly striking zones in order to avoid loss of deposition holes; a total of 35% of the main tunnels lie within and parallel to these zones. Overbreak and need for bolting could be expected in other parts of the repository with north-easterly stretches in the central area and ramp. Parts of the installation in both these areas do not, however, coincide with any modelled north-easterly zones.

The fractures striking north-eastwards, parallel to the tunnels, often caused fallout up to the springline in SFR /Carlsson and Christiansson 2007/. This occurred up to deviations of 5–20° between tunnel and zone. This fact justifies an increase in the proposed reinforcement of the walls to systematic bolting in the parts of the main tunnels that coincide with the modelled zones striking north-eastwards. For both the tunnels and any further occurrences in the central area and ramp, any decision on systematic bolting will be made after application of the observational method.

Problems with increased rock stresses have been almost non-existent during the construction of the SFR. Spalling in the tunnel roof has occurred, but can be related to significant stress concentrations /Carlsson and Christiansson 2007/.

6.2.2 Analytical calculations

In Section 6.1.3 it was established the shotcrete is capable of carrying the load from a 0.5 m^3 (~0.014 MN) block with a common safety margin and that singular 3 m bolts carry a block of around 6 m^3 (~0.162 MN). Comparing these blocks with the estimation of block foreseen by /Martin 2007/ and shown in Figure 6-5 it is found that the majority of all blocks are carried by the general support ST1.

The trend of the deposition tunnels are around 20° and of the main tunnels 110°. This means that the orientation of the tunnels do not fall within the most critical span based on the result in /Martin 2007/.

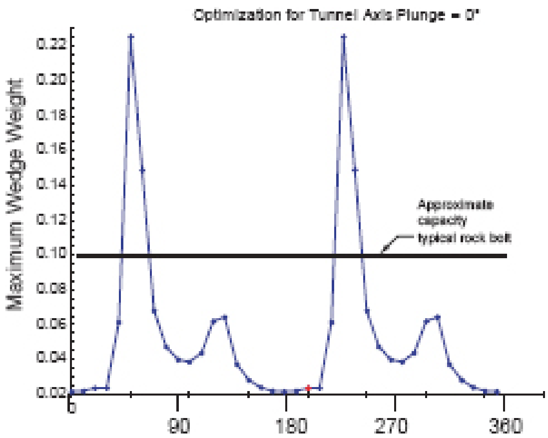


Figure 6-5. *Maximum weight of potential wedges /Martin 2007/. The unit for the y-axis is MN. The trend of the tunnel of the tunnel is shown on the x-axis.*

6.3 Most unfavourable system behaviour

An assessment of the most unfavourable system behaviour, compared with the probable behaviour described above, shows that the condition, which may be changed, is the occurrence of spalling, and that the proportion of ground types GT 2 and GT 3 may increase.

To investigate how rock reinforcement is possibly affected if the most unfavourable system behaviour should occur, a comparison has been made with the Q-system reinforcement proposals for the ground types that do occur.

For the Q-value, spans and ground types that also dominate in the most unfavourable system behaviour, the Q-system shows that no reinforcement apart from selective bolting in larger caverns is needed. For the parts where rock class GT 3 occurs, the Q-system indicates that shotcrete and systematic bolting may be needed in the lower parts of the Q-value interval for this class, and for spans greater than 6 m.

Based on the findings that a small confinement is sufficient to prevent progressive spalling /cf. Andersson 2007/, an increased amount of spalling is handled by the shotcrete reinforcement that is already included in the minimum reinforcement. This may possibly mean that the shotcrete area may need to be increased from covering only the roof to including parts of the walls.

Practical experience from mines on great depths shows that it is concluded from the Apse tunnel that small confinement from a rubber bladder was enough to stop spalling.

The minimum reinforcement (ST 1) that should be applied in the whole sub-surface repository, irrespective of system behaviour, apart from in the deposition tunnels, is shotcrete and selective bolting in the roof. This reinforcement is sufficient in comparison to Q-system to handle the conditions that may occur in the most unfavourable system behaviour. In the proposed support types, bolt lengths greater than 3 m have also been assumed, which is longer than given by the reinforcement recommendations of the Q-system, except in the largest caverns. This means that the proposed support types already cover the most unfavourable system behaviour.

With the resistance offered by shotcrete, spalling should be limited, according to results in /Andersson 2007, Kaiser et al. 1996/. The tunnel behind the face will not suffer full stress since some stresses are transferred in the rock in front of the face. Assuming that an undisturbed stress field is reached approximately maximum 2 blast lengths behind the front (interpreted from /Chang 1994/), no supporting shotcrete should be necessary close to the face to prevent spalling. Since the shotcrete is part of the minimum reinforcement, it is inferred that there will be no changes in support type, whether spalling occurs or not.

Possible problems related to the spalling is, therefore, expected to be restricted to facility parts without the minimum reinforcement, i.e. deposition holes and tunnels. The analyses showed low stresses in the deposition tunnels when these are oriented sub-parallel to the major horizontal stress and spalling is therefore not foreseen at all. If spalling occurs, wire mesh is used as reinforcement. This will not prevent spalling as shotcrete is expected to do, but will allow a safe working environment.

7 Reinforcement quantities

7.1 General

This section covers calculated amounts of reinforcement and the conditions on which they are based. For a description of support types and the activities included in them, reference is made to the previous chapter.

7.2 Compilation of amounts

The various parts of the repository include several different objects such as tunnels, shafts, sumps, caverns, etc. These are summarised and reported in the tables previously given in Chapters 3–5.

When calculating the amount of reinforcement, a number of assumptions have been made regarding the parts of the object that need to be reinforced. Several of these are given and discussed in Chapter 5. In accordance with the Reference Design, a number of parts of the installation have an access road or a floor of concrete 0.43 m above the theoretical bottom contour. It is therefore recommended that the bottom 0.5 m of the walls is not reinforced in tunnels and caverns. This also applies to deposition tunnels, even though in accordance with /SKB 2008a/ they do not have a raised access road.

The use of wire mesh is recommended in the roof and uppermost 1 m of the walls of deposition tunnels that occur in GT2 and GT3. Similar to Layout D1 /Brantberger et al. 2006/, it is assumed that 0.5 rock bolt/m^2 and a bolt length of 0.5 m are suitable for fixing the mesh.

In the tables below, where the amount of reinforcement is summarized, the figures are rounded off except in Table 7-2. In this table the numbers from the calculations are presented for traceability reasons.

The compilations given in Table 7-1 report the total amount of reinforcement per functional area. In Table 7-2 the amount of reinforcement per facility part are presented.

In Table 7-3, Table 7-4 and Table 7-5 the amount of subsidiary material in shotcrete, bolt cement, bolts and wire mesh are presented. This material meets the requirements supplied by SKB for low pH and other functional requirements.

Table 7-1. Compilation of reinforcement amounts for the functional areas of the repository.

Functional area	No of bolts	Quantity of shotcrete [m^3]	Quantity of wire mesh [m^2]
Ramp/access	2,300	1,500	–
Central area, including ventilation	4,400	2,200	–
Deposition area, including SA01 and SA02	26,000	4,700	56,500
Total	32,700	8,400	56,500

Table 7-2. Compilation of reinforcement amounts for different facility parts of the repository.

Facility part	No of bolts	Quantity of shotcrete [m³]	Quantity of wire mesh [m²]
Ramp (model file: 191BR_00_3001)			
Tunnel (5.5 m wide)[1]	1,762	1,144	–
Tunnel (6.0 m wide)	46	34	–
Tunnel (7.0 m wide)	315	224	–
Passing places (8.0 m wide)	71	48	–
Niche (5.5 m wide)	7	6	–
Niche (7.0 m wide)	53	42	–
Niche (10.0 m wide with 5x5x16 m below)	23	16	–
Ventilation (model file: 191BC_00_3002)			
Shaft (ø 1.5 m)	24	38	–
Shaft (ø 2.5 m)	70	118	–
Shaft (ø 3.5 m)	98	165	–
Shaft (ø 4.5 m)	7	11	–
Tunnel (4.0 m wide)	372	231	–
Central area (model file: 191BC_00_3001)			
Skip shaft (ø 5.0 m)	165	257	–
Elevator shaft (ø 6.0 m)	186	311	–
Silo (ø 9.5 m)	13	20	–
Tunnel (3.0 m wide)	37	25	–
Tunnel (4.0 m wide)	8	6	–
Service tunnel (4.0 m wide)	146	109	–
Tunnel (5.1 m wide)	15	13	–
Tunnel (7.0 m wide)[2]	634	317	–
Caverns (13.0–16.0 m wide)	234	487	–
Sump (12.0 m wide)	74	16	–
Electricity hall (7.0 m wide)	48	11	–
Crushing hall (10.3 m wide)	70	15	–
Deposition area			
Ventilation shafts SA01 and SA02 (ø 3.0 m)	176	0	–
Main tunnel (10.0 m wide)	7,660	3,108	–
Transport tunnel (7.0 m wide)	2,936	1,601	–
Deposition tunnel (4.2 m wide)	15,175	0	56,500

Table 7-3. Compilation of the amount of subsidiary material in shotcrete for functional areas of the repository (rounded numbers).

Subsidiary material	kg/m³ or % from SKB	Ramp/access		Central area, including ventilation		Deposition area, including SA01 and SA02	
		[ton]	[m³]	[ton]	[m³]	[ton]	[m³]
Water	158	239	239	340	340	744	744
Ordinary Portland cement CEM I 42.5	210	318	151	452	215	989	471
Silica fume	140	212	101	301	143	659	314
Coarse aggregate (5–11)	552	836	492	1,187	698	2,600	1,529
Natural sand (0–5)	1,025	1,552	913	2,205	1,297	4,227	2,839
Quarts filler (0–0.25) or Limestone filler (0–0.5)	250	379	189	538	269	1,177	589
Superplasticiser “Glennium 51” from Degussa	3	4.5	4.5	6.5	6.5	14	14
Air entraining agent “Sika AER S”	2.5	3.8	3.8	5.4	5.4	12	12
Accelerator “Sigunit” from Sika or AF 2000 from Rescon	7%[1]	0.1	0.1	0.2	0.2	0.3	0.3

[1] Tests performed have given values between 4–10%. An average value of 7% was however chosen for these calculations.

Table 7-4. Compilation of the amount of subsidiary material in bolt holes for functional areas of the repository, (water binder ratio 0.475) (rounded numbers).

Subsidiary material	kg/m³ or % from SKB	Ramp/access		Central area, including ventilation		Deposition area, including SA01 and SA02	
		[ton]	[m³]	[ton]	[m³]	[ton]	[m³]
Cement	340	15	7	28	13	98	47
Silica	226.7	10	5	19	9	65	31
Water	266.6	12	12	22	22	77	77
Glennium 51	4	0.2	0.2	0.3	0.3	1	1
Quarts filler	1,324	57	29	109	54	381	191

Table 7-5. Compilation of the amount of subsidiary material, rockbolts and wire mesh, for functional areas of the repository (rounded numbers).

Subsidiary material	Ramp/access [ton]	Central area, including ventilation [ton]	Deposition area, including SA01 and SA02 [ton]
Rock bolts (l=3 m, d=25 mm, 4 kg/m³)	27	52	182
Wire mesh (1.7 kg/m²)	–	–	96
Fixing bolts (29329 pcs)			28,217

8 Constructability and uncertainties

In this report, 'constructability' means the possibility of producing the structure of the repository while meeting requirements. The term 'uncertainty' refers to the factors that might affect the production structurally or contractually.

An identification and evaluation of risk has been performed and is further treated in the risk report. However, an assessment of the overall uncertainty identified and the effect that deviations from an expected condition may have on constructability etc is carried out below from a rock reinforcement perspective.

8.1 Uncertainties

There are several uncertainties connected with the analysis of reinforcement requirements. The following can be listed from an overall perspective:

- The Ground Types presented in /SKB 2008b/ indicate good to excellent rock from a construction point of view. The uncertainties are according to /SKB 2008b/ found in potential horizontal jointing, which are not considered in particular in the design of the reinforcement.
- The stress analyses are based on the expected stresses and a worst case scenario given in /SKB 200b/. Based on these analyses it has been found that spalling should not be a critical issue in the tunnels and in the central area. The uncertainty span in both magnitude and orientation of the stress field however give a wide range of possible ground behaviour in relation to the layout and it could be found that spalling is more frequent than anticipated in the analyses.
- The analyses on the system behaviour have not included the stresses in the block analyses. There is also a limited understanding on how the reinforcement, in particular the shotcrete and the adhesion strength, can decrease with time.

In summary, an assessment of constructability and observation parameters based on the following uncertainties has been carried out:

- More wedging stability or blocky rock than expected.
- More spalling encountered than expected.
- Function of the reinforcement over time.

8.2 Effect on constructability

As identified at the beginning of this chapter, constructability is a measure of whether or not it is possible to construct the repository. The assessments of constructability given below are made on the basis of the overall uncertainties identified.

Wedge stability and blocky rock is potentially increasing the necessary amount of reinforcement. In the present design of the reinforcement there is a load bearing capacity for wedges and blocks but, in case of frequent horizontal jointing, this needs to be increased. The effect on constructability is however judged small to insignificant since thicker shotcrete and more bolting can be used. However, the effect on production time, costs and material should be recognised.

Spalling is a critical question since this issue is not common in underground construction. The vast majority of spalling occurs in deep mining. If spalling in the tunnels occur it is however expected that the designed minimum reinforcement should be capable of handling this, and therefore it should not affect the constructability. If it is found that the designed reinforcement cannot handle the spalling, the countermeasure is to increase the shotcrete, and to use washers as well as systematic bolting. The consequences of extensive spalling in the deposition hole are significantly more serious from a repository perspective. However, this is not a rock reinforcement issue, because the current reference design does not permit reinforcement in deposition holes. Extensive spalling may therefore adversely affect constructability under current requirements.

During the lifetime of the facility the load bearing capacity of the installed reinforcement can decrease. There is limited experience of the function of shotcrete and bolts, especially after using low-pH grouts, which needs special attention. Common practise is however periodical inspections to verify the function, which reveals any kind of degradation.

From the rock reinforcement perspective, it is assessed that there is no risk that a structurally unstable repository could be produced. The collective experience from rock-working is sufficient to state this definitively. There are, however, practical examples of how poorer-than-expected rock conditions produce contractual difficulties that result in the financial risk of some aspects being too extensive.

8.3 Observation parameters and acceptable limits

For brittle failures, which include wedge stability and spalling, the critical parameters are stress and block size /Stille and Holmberg 2007/. This means that an observation programme should include checks on the stresses and block sizes on which the design is based and that this must be verified in connection with tunnelling. If the conditions deviate, a more suitable reinforcement solution should be chosen.

Geological mapping is proposed when determining the critical parameters for block size. This should work when checking whether conditions encountered lie within acceptable limits. Here, acceptable limits mean that reinforcement should work for the worst case rock classes encountered (i.e., in the worst case of failure behaviour).

Checks on stress conditions are thought to be possible with two indirect methods of checking whether or not spalling occurs. One method is acoustic measurement of the number of microseismic events in connection with tunnelling in order to see whether they diminish in a way similar to that given by /Andersson 2007/. This is not to our knowledge tested in tunnelling activities but could be further looked into. The other method is to drive a pilot tunnel with an 'unfavourable' cross-section, i.e. with a cross-section that favours spalling failures. If spalling failures do not appear, or the reinforcement solutions work, acceptable limits have been verified. If the reinforcement solutions do not work, a stronger reinforcement should be tested and verified.

Spalling in the deposition hole may be a critical factor for the whole repository if it is significant, i.e. occurs in the lower part of the holes. Analyses carried out show that with the chosen layout the stress pattern is on a level where spalling may begin to occur in the upper parts of the deposition holes. A restricted safety margin therefore exists for spalling deeper down in the hole. If spalling occurs at the start of drilling of the deposition holes, there is a possibility of aiming the tunnels parallel to the maximum horizontal stress and not at the 18–22° that now applies.

9 Comments and conclusions

This report has presented a survey of expected rock conditions and suitable reinforcement methods for them in terms of requirements for the final repository. The work has been carried out in accordance with the instructions in /SKB 2008a/ with the support of other literature and input data documents.

In all, the conclusion from the study is that the current site adaptation is suitable from a rock reinforcement perspective. There cannot be foreseen any difficulties concerning rock reinforcement; potential block falls or minor spalling are considered well within the experience of underground construction work. Since the deposition tunnels are oriented sub-parallel to the major horizontal stress, spalling in the roofs that could cause an unsafe working environment, is found to be avoided. Spalling in the deposition holes could occur according to the analyses in the upper part of the hole if the stresses are higher than expected.

Since spalling in deposition holes is a facility-critical issue, the recommendations for continued studies are to focus on spalling stability. This type of failure is moreover not as well known in underground construction as block stability. Even if the analyses in this study have found spalling failures to be handled within the layout and design, there are uncertainties identified.

In relation to the earlier design in Stage D1 certain differences have occurred. The amount of reinforcement thought to be necessary is less than that estimated in Stage D1 /Brantberger et al. 2006/. This is thought to have several causes.

With the conclusion of the complete site survey, a clearer description of the rock has been obtained, which has reduced the uncertainties. Additionally, very favourable rock conditions have been established.

Another reason is application of the observational method, which permits insecure conditions to be retrospectively reinforced on the basis of observations. This means that the amount of reinforcement at this design stage may be based on expected amounts without an actual safety margin.

A general over-reinforcement relative to the estimated stability has however been proposed for the whole roof and the shafts, with the intention of protecting installations, ensuring safe working conditions and minimising the need for periodic maintenance. This minimum reinforcement has been proposed to be 30 mm of shotcrete, which is not intended to have any direct reinforcing effect for block falls. Minimum reinforcement also has the advantage during inspections in that fracture formation can be noted.

10 References

Andersson C J, 2007. Rock Mass Response to Coupled Mechanical Thermal Loading Äspö Pillar Stability Experiment, Sweden. Doctoral Thesis. Div of Soil and Rock Mechanics, Royal Institute of Technology, Stockholm, Sweden. ISSN 1650-9501.

Barton N, Lien R, Lunde J, 1974. Engineering Classification of Rock Masses for the Design of Tunnel Support. Rock Mechanics 6, 186–236.

Carlsson A, 1979. Characteristic features of a superficial rock mass in Southern central Sweden – Horizontal and sub-horizontal fractures and filling material. Striae 11.

Carlsson A, Christiansson R, 2007. Construction experiences from underground works at Forsmark. SKB R-07-10, Svensk Kärnbränslehantering AB.

Chang Y, 1994. Tunnel Support with shotcrete in weak rock – a rock mechanics study. Doctoral thesis, Div of Soil and Rock Mechanics, Royal Institute of Technology, Stockholm.

Fredriksson A, 1994. Bergmekanik. Dimensionering av sprutbetong. Södra Länken, PM 1994-02-15.

Grimstad E, Barton N, 1993. Updating the Q-system for NMT. Proc to the Int Symp on Sprayed Concrete, Oslo.

Hansson B, Magnusson J, Söderlund P, 2008. Underground Design, Forsmark, Layout D2. Layout Studies. SKB R-08-113, Svensk Kärnbränslehantering AB

Hoek E, Brown E T, 1980. Underground excavations in rock. Chapman & Hall, ISBN 0419160302.

Holmgren J, 1979. Punch loaded shotcrete linings on hard rock. Doctoral thesis, Rapp nr 121:6n, FortF/F dnr 2492F, Stockholm.

Johansson J, Olsson L, 2008. Underground Design, Forsmark, Layout D2. Risk Analysis. SKB R-08-116, Svensk Kärnbränslehantering AB.

Kaiser P K, McCreath D R, Tannant D D, 1996. Rockburst research handbook. Volume 2. Rockburst reinforcement. Prepared for sponsors of the Canadian Rockburst Research program 1990–1995, GRC, Laurentian University, Sudbury, Canada.

Lundman P, 2006. Projekteringsanvisningar – Dimensionering av det bärande huvudsystemet för bergtunnlar. Report BB 06-04, Banverket.

Martin CD, 1997. Seventeenth Canadian Geotechnical Colloquium: The effect of cohesion loss and stress path on brittle rock strength. Canadian Geotechnical Journal 34, 698–725.

Martin D, 2005. Preliminary assessment of potential underground stability (wedge and spalling) at Forsmark, Simpevarp and Laxemar sites. SKB R-05-71, Svensk Kärnbränslehantering AB.

Palmstrom A, Stille H, 2007. Ground behaviour and rock engineering tools for underground excavations. Tunnelling and Underground Space Technology 22, 363–376.

RocScience, 1998. Examine^{3D} version 4.0. Manual. RocScience Inc.

RocScience, 2007. Examine^{2D} version 7.0. Manual. RocScience Inc.

SKB, 2006. PSAR Allmän del Kapitel 3 Krav och Konstruktionsförutsättningar. Svensk Kärnbränslehantering AB

SKB, 2007a. Miljöprogram för kärnbränsleprojektet. Svensk Kärnbränslehantering AB.

SKB, 2007b. Mall för motivering av layout och teknisk lösning. Svensk Kärnbränslehantering AB.

SKB, 2008a. Final Repository Facility. Underground Design Permises/D2. SKB R-07-33, Svensk Kärnbränslehantering AB.

SKB, 2008b. Forsmark Site Engineering Report. Guidelines for Underground Design Step D2. SKB R-08-83, Svensk Kärnbränslehantering AB.

Stille H, Eriksson M, Nord G, 2005. Kompendium i Bergmekanik. Dept of Soil and Rock Mechanics, KTH, Stockholm.

Stille H, Holmberg M, 2007. Observationsmetodens grunder och dess tillämpning på design av konstruktioner i berg. SveBeFo 80, Stockholm 2007. ISSN 1104-1773.

Appendix A

Tangential stress in depositions holes

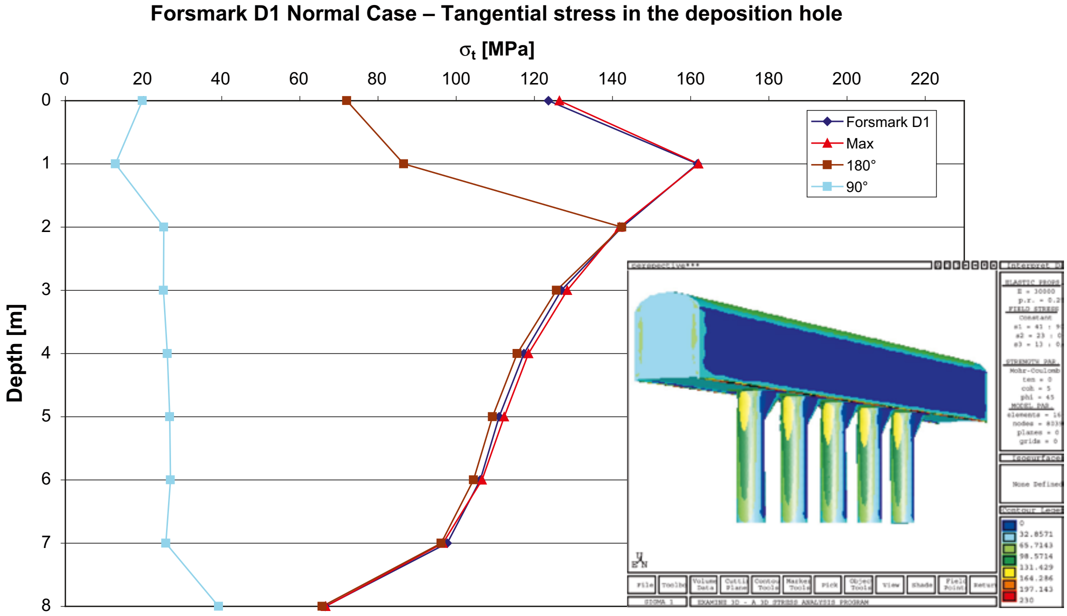


Figure A-1. Normal case D1.

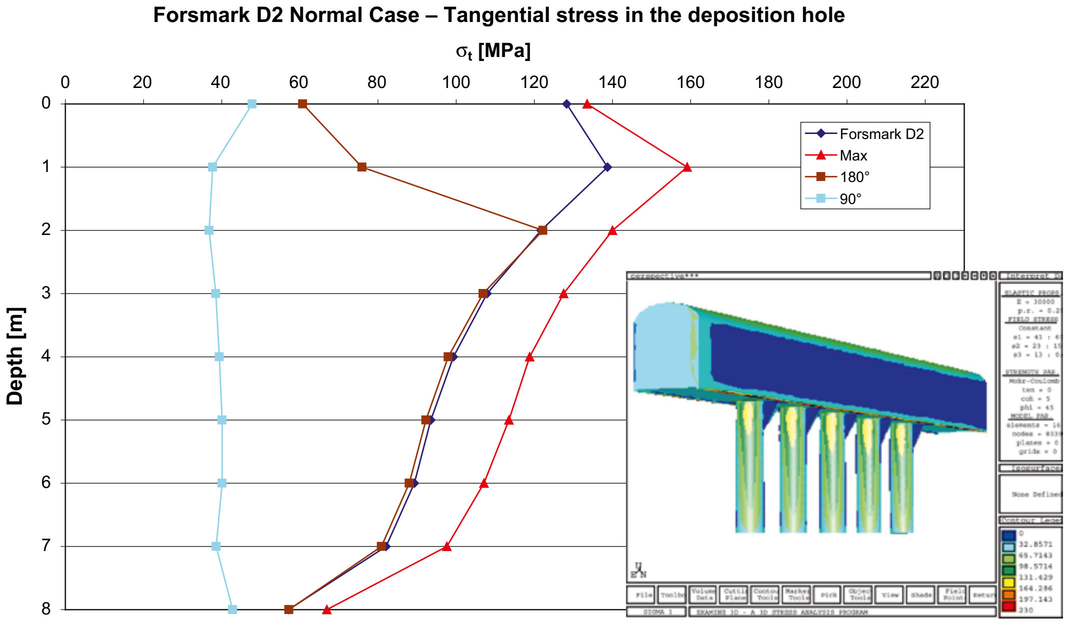


Figure A-2. Normal case D2.

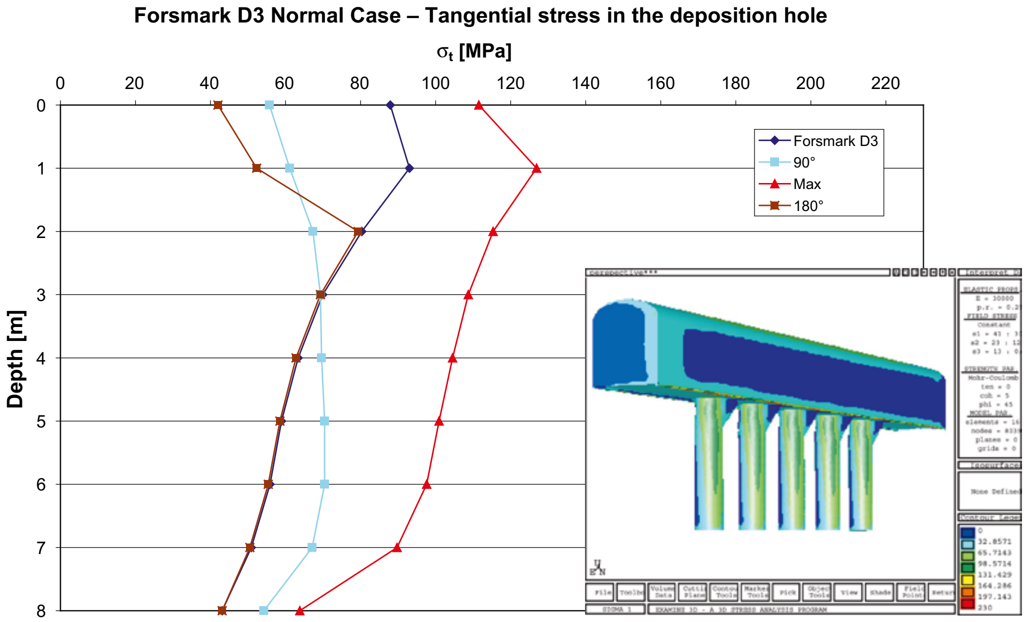


Figure A-3. Normal case D3.

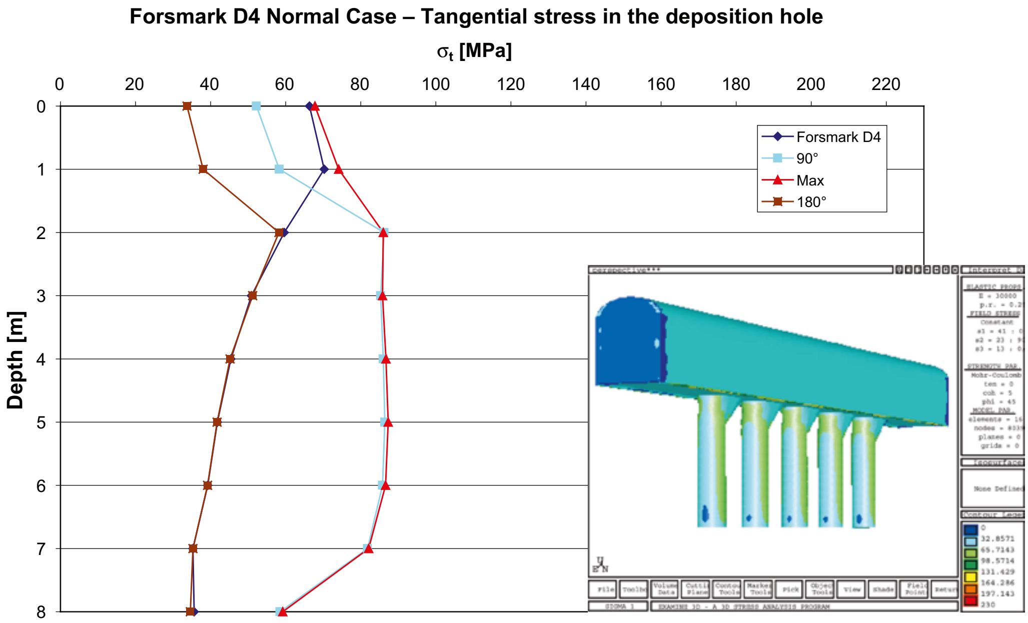


Figure A-4. Normal case D4.

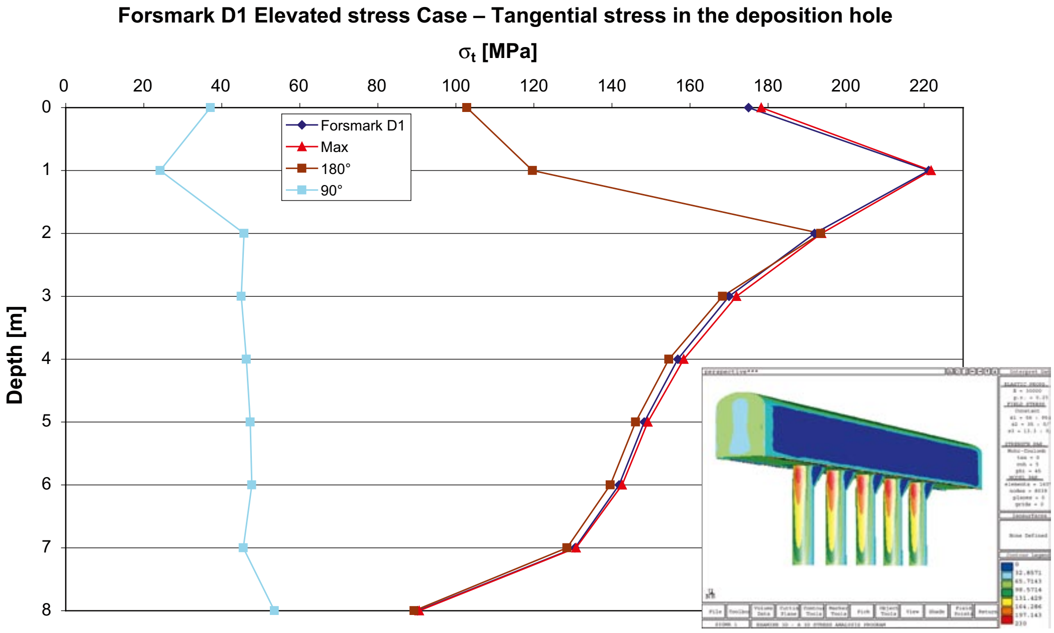


Figure A-5. Elevated stress D1.

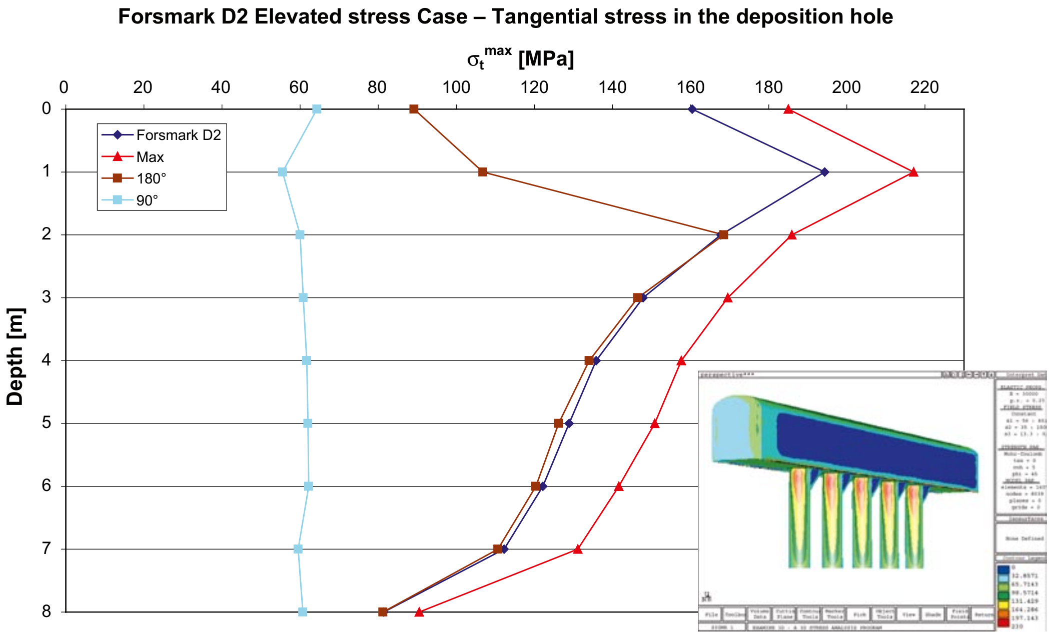


Figure A-6. Elevated stress D2.

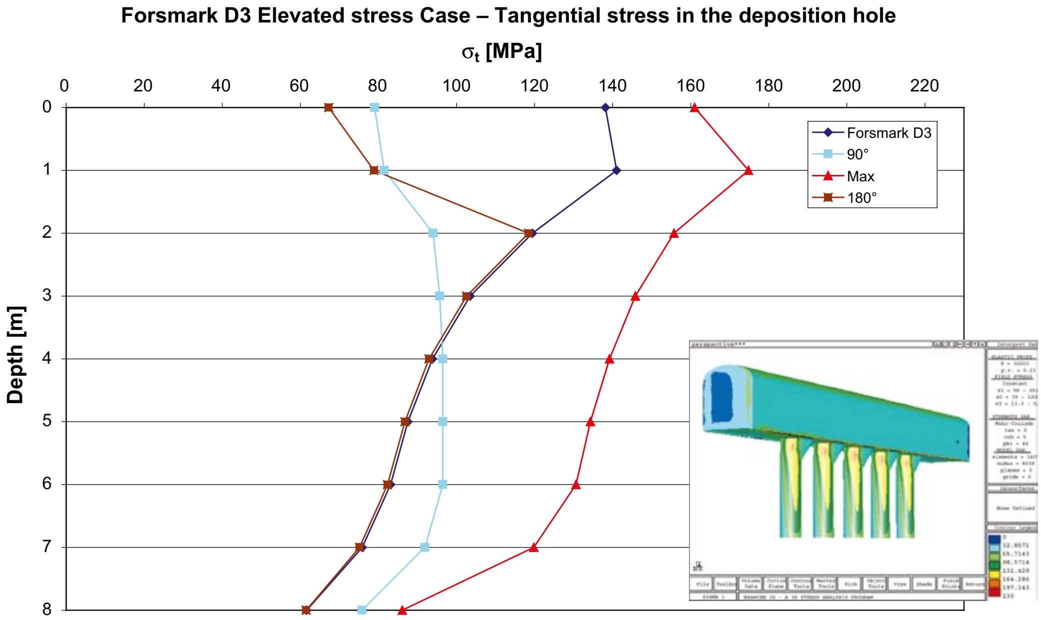


__Figure A-7.__ Elevated stress D3.

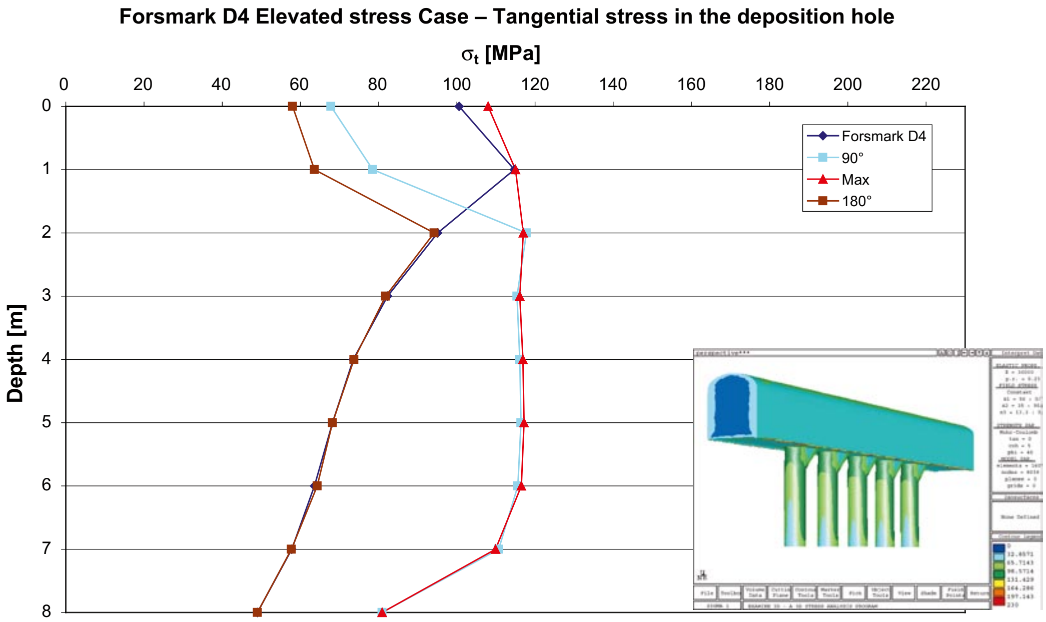


__Figure A-8.__ Elevated stress D4.