International
Progress Report

IPR-03-01

Äspö Hard Rock Laboratory

Äspö Pillar Stability Experiment

Feasibility Study

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February 2003

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Report no.	No.
IPR-03-01	F86K
Author	Date
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Checked by	Date
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Approved	Date
Christer Svemar	2003-05-21

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Keywords: Numerical modelling, stress, spalling

Sammanfattning

Denna rapport är en översiktlig rimlighetsstudie inför genomförandet av Äspö Pillar Stability Experiment.

En av målsättningarna med experiment är att demonstrera möjligheten att prediktera brott i en uppsprucken bergmassa. En annan viktig målsättning är att kunna demonstrera den inverkan backfillen har på bildandet av mikrosprickor i bergmassan närmast deponeringshålen.

Dessa syften uppnås genom att det för experimentet tillreds en ny tunnel. I sulan på den nya tunneln borras två stora hål nära varandra. Ett av hålen skall vid försöket ha ett invändigt vattentryck på 1 MPa som skall simulera återfyllnaden i en tunnel. Berget i pelaren mellan hålen skall värmas upp och de tillskottsspänningar som tillkommer genom bergets termiska expansion skall driva berget till brott i hålväggarna. Inga större bergutfall förväntas då berget går i brott varför akustisk emissionsteknik skall användas för moneteringen av experimentet och kommer att vara den moneteringsutrustning som i huvudsak skall användas för att utvärdera experimentet.

Förstudien har kommit till följande slutsatser:

- Den nya tunneln bör byggas på 450 meters avvägning på Äspölaboratoriet.
- Den nya tunneln bör ha en höjd på 7,5 m och en bredd på 5 m. Tunnelgolvet skall vara halvcirkelformat.
- Pelarbredden bör vara 1 m.
- Håldiametern bör vara 1,8.
- Det kan vara nödvändigt att borra slitsar i borrhålsväggarna mitt emot pelaren. Dessa slitsar medför att större laster kommer att ledas in i pelaren och säkerställa att den går i brott vid uppvärmningen.

Rimlighetsstudien identifierar vidare översiktligt risker med projektet samt beskriver en strategi för karakterisering och modellering av experimentet. En kort beskrivning av experimentets utförande, nödvändig moneteringsutrustning och dess placering ingår även.

Summary

This report is a study of the feasibility for performing a pillar stability experiment at Äspö HRL.

One objective with the Äspö Pillar Stability Experiment is to demonstrate the capability to predict spalling in a fractured rock mass. Another important objective is to demonstrate the effect the backfill has on the propagation of micro cracks in the rock mass closest to the deposition holes.

To achieve these objectives a new tunnel will have to be excavated. Two holes in the new tunnel's floor will be bored and a pillar will be created between them. One of the holes will be confined with 1 MPa water pressure in a liner to simulate backfill. The rock mass in the pillar will be heated and the additional thermal stresses shall force the rock in the hole walls to spall. The spalling will be monitored with an acoustic emission system, which will be the main instrumentation for evaluation of the experiment.

The pre-study has come to the following conclusions:

- The new tunnel for the experiment should be located at the 450 m level at Äspö Hard Rock Laboratory
- The new tunnel should have a height of 7.5m and a width of 5m. The tunnel's floor should be curved.
- The width of the pillar should be 1m.
- The diameter of the holes should be 1.8m
- Slots may have to be drilled in the hole walls, the keyhole principle, to create high stresses enough in the pillar before heating.

The study identifies risks in general and describes the strategy for the characterisation and modelling of the experiment as well as the way the experiment should be practically realised. A draft description of the monitoring equipment and their placement is also included.

Contents

1 Introduction

This report is a feasibility study for the project, Äspö Pillar Stability Experiment, APSE.

1.1 Objectives

The APSE is a rock mechanics experiment which can be summarised in the following three main objectives:

1. Demonstrate the capability to predict spalling in a fractured rock mass
2. Demonstrate the effect of backfill (confining pressure) on the rock mass response
3. Comparison of 2D and 3D mechanical and thermal predicting capabilities

1.2 Experiment layout

To perform the experiment a new tunnel will have to be excavated. That is necessary since the stress situation at the experiment location not should be influenced by the existing tunnel geometrys. In the tunnel two large holes will be drilled so that a slim pillar is created between them. The volume around the pillar and the holes will be instrumented with monitoring equipment and heated by electrical heaters located in boreholes.

1.2.1 Geometry and execution

Figure 1 presents one possible geometry for the experiment. During the execution one of the holes will be confined by water pressure in a liner that will simulate backfill. The pillar will then be heated and hence subjected to additional stresses that shall induce spalling in the unconfined borehole's wall. When a steady state is reached the confinement pressure will be lowered to predetermined levels where the numerical models predict spalling.

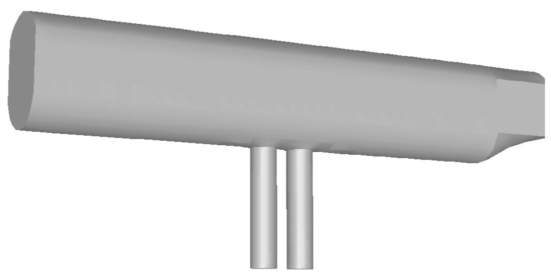

Figure 1. *Possible geometry for the experiment with a slender pillar between two 1.8m diameter holes.*

1.2.2 Verification of the experiment results

The spalling will mainly be monitored with acoustic emission, which together with thermocouples and convergence/displacement measurements are the monitoring instruments needed to evaluate the experiment. If breakouts occur in the large holes it be documented.

1.3 Purpose of the report

This feasibility study is an assessment of the possibility to successfully perform the experiment briefly described above. The main issue in this report is the design of the new tunnel and the pillar. The design is important since it has to be shown that it is possible to achieve a stress level in the pillar before heating that is high enough to ensure that spalling will occur when the rock mass is heated.

The report also presents three alternative experiment locations and briefly discusses the characterisation needed to be able to select a final site.

2 AECL's Heater Failure Test

The first large scale work on thermal response was made by AECL in 1993-1996 in their Heated Failure Test, Read 1997.

The objective with the experiment was to investigate the effects of thermal loading on progressive failure and excavation damage development.

The strategy adopted for the Heated Failure Tests was to monitor the development of excavation damage, characterise the extent of damage in situ and then use numerical modelling to assess the relation between damage development and near-field in situ stresses. Monitoring was conducted using an acoustic emission (AE) system, extensometers, convergence arrays, piezometers and thermocouples/thermocouples. Characterisation activities included geological mapping and photography/videotaping of lithology, induced fractures and breakouts in each of five observation holes.

600mm diameter boreholes were chosen for the experiment. The electrical heaters used had an effect of approximately 3kW and it took approximately 20 days the heat up the hole wall to 85 degrees with almost full effect. The temperature in the heater holes was approximately 500 degrees C.

The experiment confirmed that the spalling extended further in the borehole when the rock mass was heated (in relation to the damaged zone due to redistribution of stresses during excavation). Small pressure changes in a confined hole (water pressure in a vinyl liner) lead to large differences in acoustic emission event rates.

The additional thermal stress from an increase of temperature in the rock from 15 to 85 degrees was 15-20 MPa.

2.1 Justification of the Äspö Pillar Stability Experiment

The Heated Failure Test (HFT) was successfully accomplished and had results in quite good agreement with the model. It might therefore seem unnecessary to perform a similar experiment at Äspö HRL. There are though three major differences in the conditions for the two experiments that justifies the APSE.

- HFT was performed in an almost totally unfractured rock mass
- The in situ stresses in relation the rock strength in URL are so high that the rock mass response is brittle and not elastic
- The APSE will demonstrate the effect of backfill at a maximum confining pressure of 1000 kPa compared to the 100 kPa used at URL. At Äspö the effect of the backfill on the micro cracking will be reliable determined since both a confined and un-confined hole will be tested at the same time in as similar geological environment as possible.

A theoretical relationship between the stress and strain on a core when determining Young's modulus is presented in *Figure 2*. The figure also includes the principal AE event rate when an accelerating number of micro cracks are created as the rock mass response turns brittle.

The in situ stresses at Äspö HRL at 450m depth subjects the rock to stresses that are on the limit of its elastic response. If the stresses are increased slightly more, the response will be brittle. The in situ stresses at URL are so high that failure will occur for almost any change in a existing geometry. The zone between the elastic conditions at Äspö HRL and the failure conditions at URL (eg. the zone where the rock mass response to additional loading becomes more and more brittle) is called the transitional zone.

According to Martin et al. (2001) it is likely that the pillar between the deposition holes will encounter transitional conditions during the life span of the repository. The stress situation directly after placement of a canister will be in the same range as at Äspö (elastic). The swelling pressure, the thermal load and glaciation load (note: the thermal load and the glaciation load will not occur at the same time) will induce such stresses in the pillar that the stress-strain relationship might enter the transitional zone.

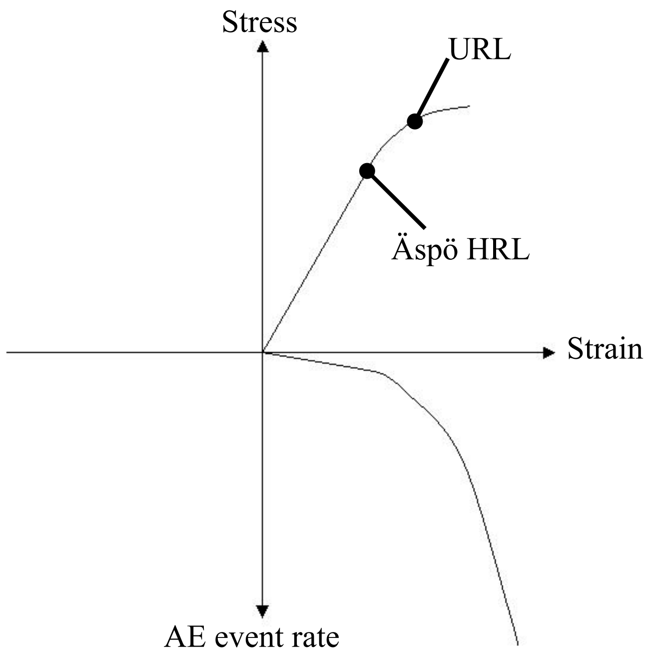

Figure 2. *Relationship between stress and strain on a compressed core. Included is the relative rock stress situation at Äspö HRL and at the URL. The principal increasing AE event rate when micro cracks are formed is also included.*

Very limited rock mechanics research on the rock mass response in the transitional zone (accelerating frequency of micro-cracking) has been carried out. It is therefore important to gain knowledge in this field since the spacing of the canister holes gives a great impact on the total costs for the deep repository. If verified numerical models can be used to optimise the spacing, the design will probably be much more cost effective than the alternative, which is empirical formulas developed for mining conditions. These formulas will be rougher and an unnecessary high safety factor might be the result.

The rock where the HFT was performed was almost unfractured. Since any given rock mass at a Swedish repository site will contain a number of fractures the URL results can't be applied directly. The Äspö experiment will indicate if the rock mass response will be roughly the same as at URL even though it will include some fractures.

The acoustic emission system can rather exactly pinpoint the location of an acoustic event. Valuable information will therefore also be achieved regarding the behaviour of any induced or pre-existing fracture in the pillar.

3 Risks for the outcome of the APSE

A number of risks that could jeopardise the outcome of the experiment have been discussed. The identified major risks and some countermeasures are listed below.

1. Excavation damage in the floor will make the rock mass Young's modulus lower and reduce the increase of stresses when the rock is heated.
   This effect can dramatically be reduced if a curved floor is blasted out carefully in the tunnel as a separate bench.
2. The stress in the pillar before heating may be too low and spalling will not occur when heated.
   The design of the tunnel and pillar will ensure sufficient high stress levels in the pillar to exceed crack initiation stress. The stress level on the hole walls shall be approximately 120 MPa before heating.
3. The selected area is not homogenous from a thermal point of view.
   Characterisation of the rock mass in different stages of the project will indicate anisotropy as early as possibly.
4. Geological conditions.
   Single larger discontinuities will obviously effect the experiment if located in the pillar but the effect of different rock types, especially on the thermal properties, is somewhat uncertain.

Especially the excavation of the new tunnel might effect a number of other experiments. The risks for these experiments are further discussed in Section 4.4.

4 Alternative experiment locations

Since the experiment is dependent of the in-situ stresses the experiment should be located where they are as high as possible. The location of the experiment should therefore be somewhere at the 450 m level. At this level there are there possible sites for the experiment which are presented in Figure 3. Included in the picture is also the direction of the major principal stress, S1.

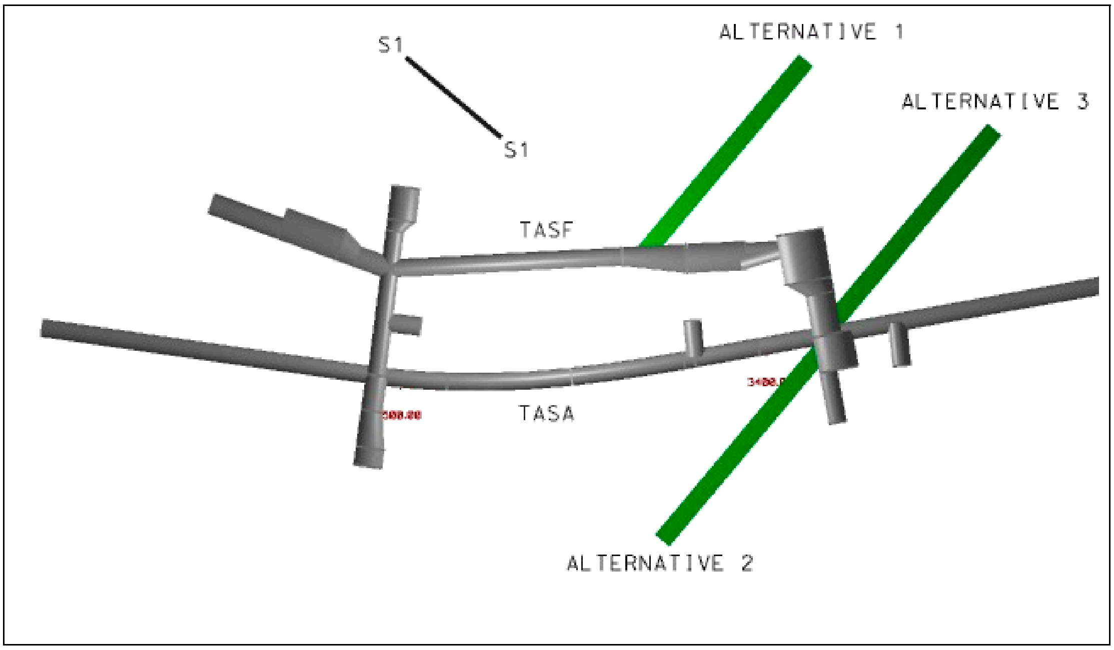

Figure 3. *Location of the three possible experiment locations and the bearing of the major principal stress at the 450m level.*

4.1 Geological knowledge in the areas of the alternatives

This section will summarise the structural geological and rock stress knowledge in the area as it was in the beginning of 2002.

4.1.1 Structural geology

A summary of the main structures, and their occurrence in the area is presented in *Figure 4* and *Figure 5* (Maersk-Hansen, Hermanson, 2002).

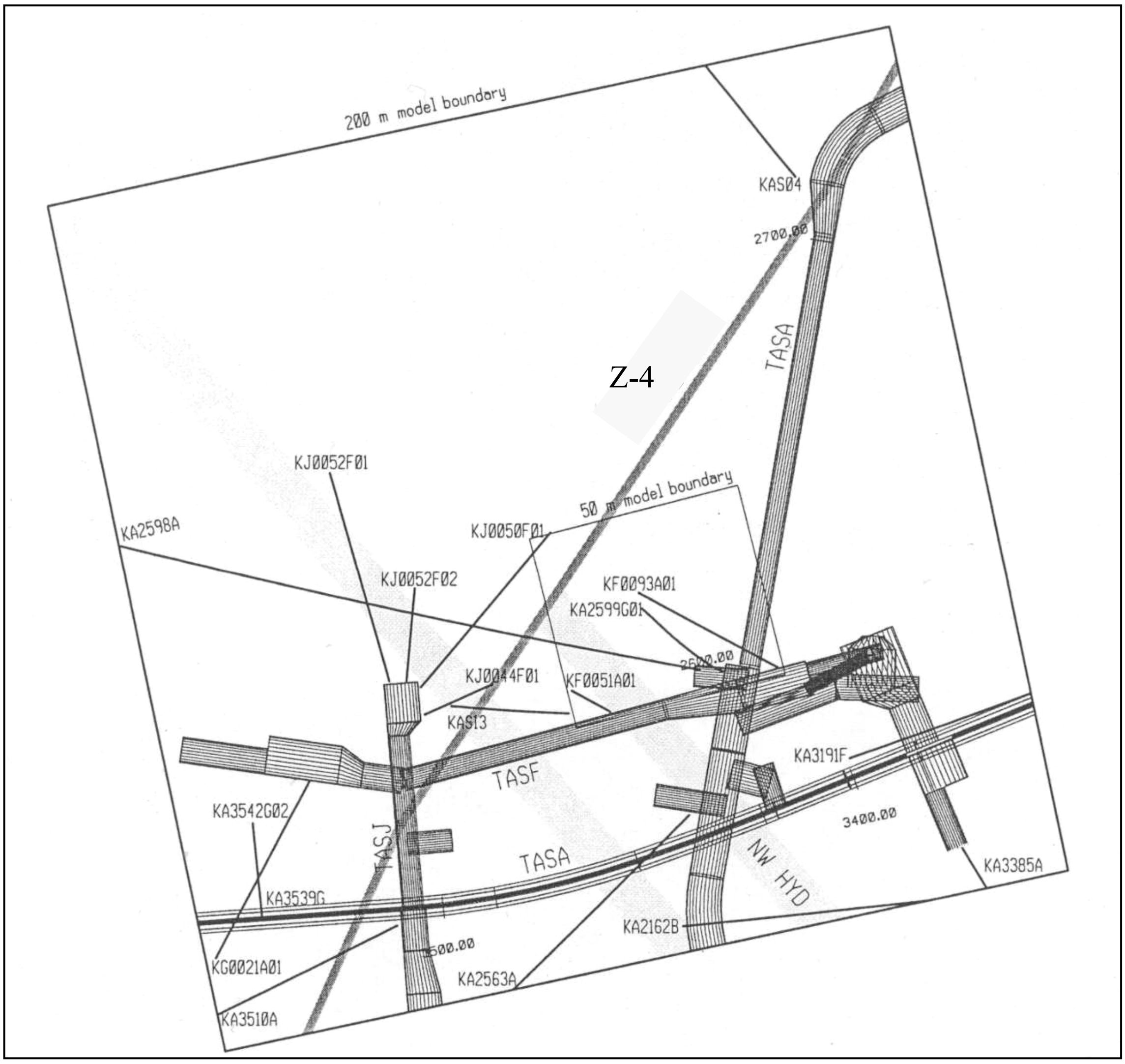

Figure 4. *Tunnels, boreholes and major structures in the proposed areas for the experiment.*

Z-4 is the most dominant structure in the area. It is verified in the tunnels and in borehole KA3510A. Z-4 is also supported by indications in the boreholes KAS04, KJ0044F01, KF0093A01, KJ0050F01 and TASA-tunnel chainage 2740m. Z-4 dips almost vertically and is indicated by an approximately 5m wide fractured zone. The definition of a fracture zone is according to Rhén et al. p.43 (1997) “a zone with the characteristic that the intensity of natural fractures is at least twice that of the surrounding rock”.

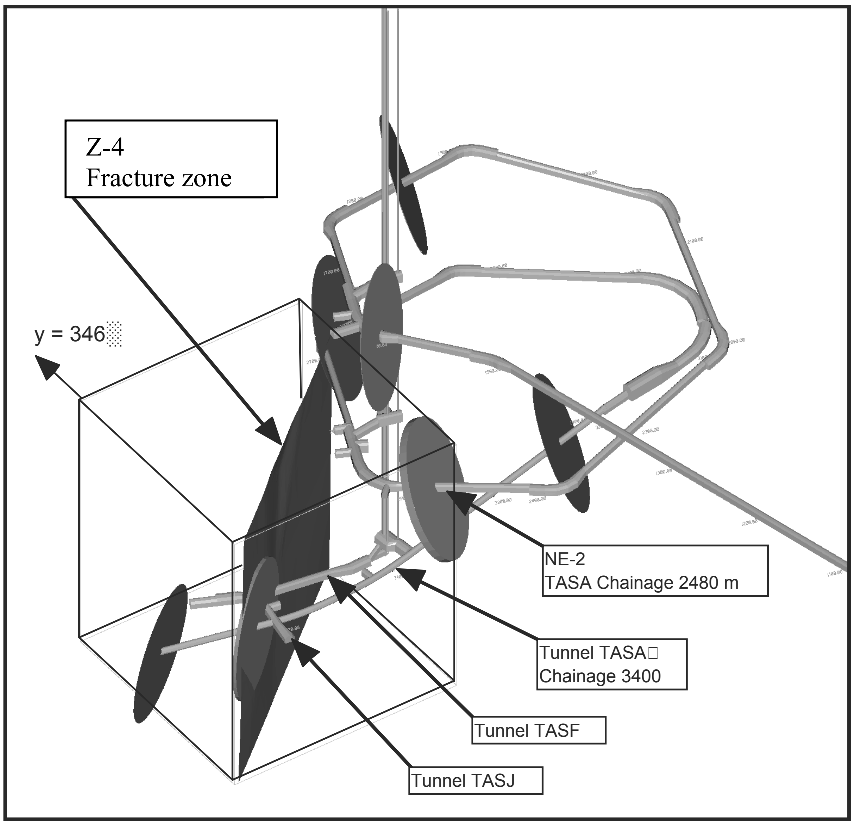

Figure 5. *Structural model showing Z-4, NE-2 and its indications (discs) in tunnels and boreholes.*

Four water-bearing parallel fractures trending NW cuts through both the TASA and TASF-tunnels and is terminated at Z-4. There also seems to be another conductive fracture group parallel with the first one located approximately 30m West of it that does not terminate at Z-4.

A stereonet contour plot of joints in TASA- chainage 2625 - 2700, TASF-, TASG-, TASI-, TASJ-tunnel and the hoist shaft from elevation –350m to the bottom of the shaft is presented in *Figure 6*. The major parts of the fractures are sub-horizontal. There are also two almost sub-vertical fracture sets trending N-S and NW-SE respectively.

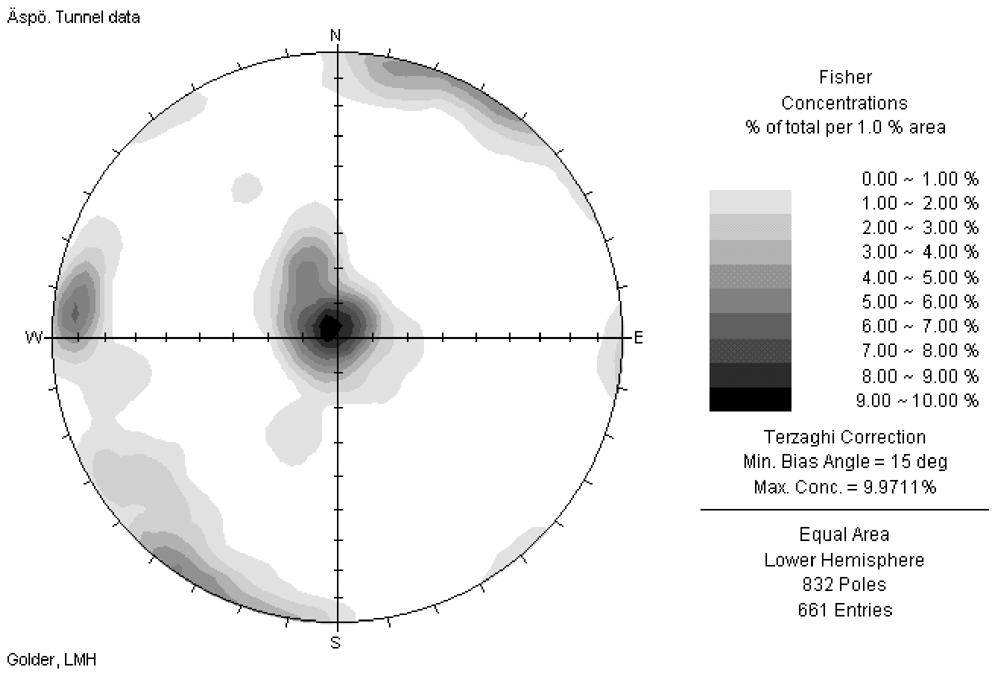

Figure 6. *Stereonet contour plot of joints in Tunnels TASA chainage 2625-2700, TASF, TASI, TASJ and the hoist shaft from elevation –350m to the floor of the shaft at –450m.*

4.1.2 Rock Stress

Christiansson & Janson (2002) have compiled the stress measurements in the area (boreholes KA2599G01 and KF0093A01). The result is presented in ***Figure** 7* and *Figure 8*.

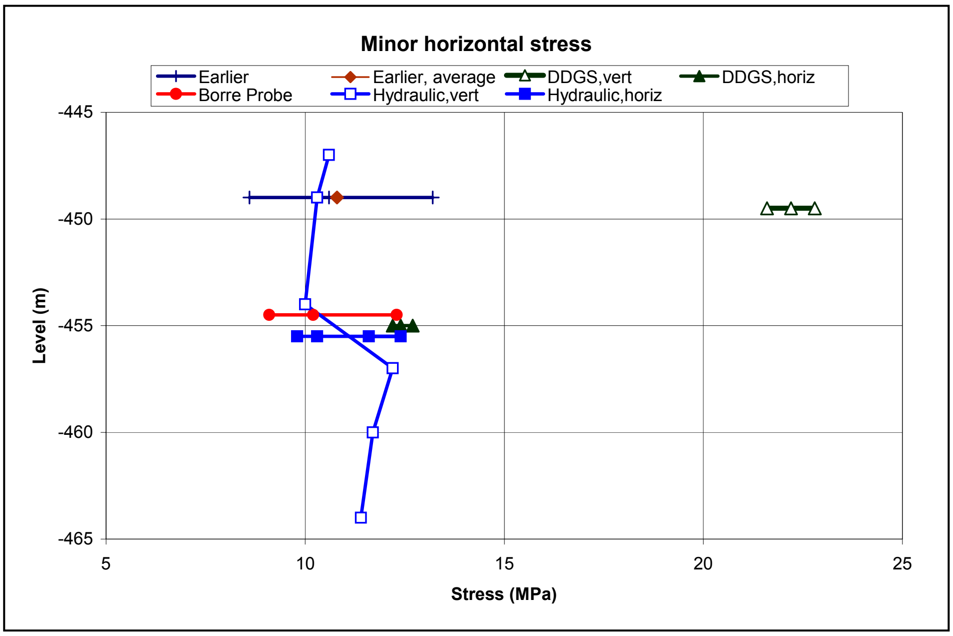

***Figure 7.** The minor horizontal stress measurement results in the vicinity of the alternative tunnel locations.*

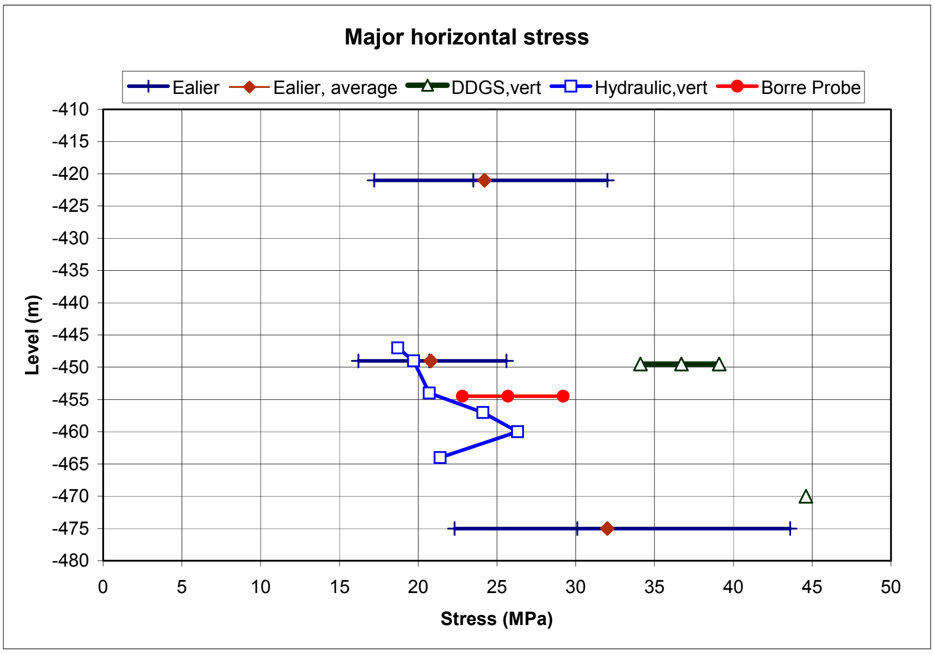

***Figure 8.** The major horizontal stress measurement results in the vicinity of the alternative tunnel locations.*

The trend and plunge from the three dimensional rock stress measurements has been used to orient the stress field. Vertical rock stress measurements indicate that the minor horizontal stress is lower than the vertical stress. Measurements also indicates that the maximum principal stress is 27 +/- 2 MPa. This has been taken into account when the following resulting stress tensor has been derived and used for most of the numerical calculations in this report (Table 1).

Table 1. The stress tensor used for the calculations of the new tunnel.

	Magnitude, MPa	Trend, degrees N over E	Plunge, degrees below horizontal
Sigma 1	25	133	19
Sigma 2	12	49	42
Sigma 3	10	234	33

4.2 Pros and cons with the three alternatives

Advantages and disadvantages for the three experiment locations presented in *Figure 3* are listed in Table 2.

Table 2. Advantages and disadvantages for the use of alternative 1, 2 or 3 as an experiment location.

	Advantages	Disadvantages
Alternative 1	Extensive rock stress measurements made in the immediate vicinity	Close to the lowest sump. Action plan for eventual pump stop needed.
	Boreholes in the area indicates that there only are few water conducting features	Several other experiments in the area, risk for conflicts
	Gases from the blasting can easily be removed by use of the existing ventilation system	Difficult to transport the boring machine for the deposition holes to the site
	The geology and the rock mechanics in the area is well known from boreholes and tunnel mapping. Nothing implies that the geology in the area will cause any problems.	
Alternative 2 and 3	Gases from the blasting can be removed by use of the existing ventilation system with only small adjustments of it	No rock stress measurements in the immediate vicinity
	The experiment area is easily accessed from the elevator stop	The area is quite wet with a lot of water bearing features
	Transports to the area will not disturb work at the 450m level	No boreholes in the immediate vicinity except for the TBM-tunnel

The best site for the experiment can not be determined without site specific data from the different alternatives. It is mainly the disturbance on the hydraulic boundary conditions for other experiments that can’t be assessed without hydraulic tests in core boreholes drilled along the tunnel axis of the different alternatives.

4.3 Calculations of stress effects of existing tunnels

De layouts of the three alternatives are chosen so that they are perpendicular to the major principal stress, which is trending approximately N130E. Alternatives 1 and 3 are therefore trending N040E and alternative 2 N220E.

It is important that the stress field in the vicinity of the existing tunnels does not effect the stress field around the planned new tunnel. If there is an interference between the stress fields the modelling will be very difficult and it is not likely that good results will be obtained. The stress fields around alternatives 1 and 3 and the existing tunnels have therefore been investigated. Alternative 2 does not need to be modelled since it is located further away from the tunnels and shafts than alternative 3. Alternative three can therefore be regarded as a worst case scenario among the two alternatives.

Figure 9 presents the result from the stress analysis of the rock mass between alternative 1 in the F-tunnel (TASF) and the hoist and ventilation shafts.

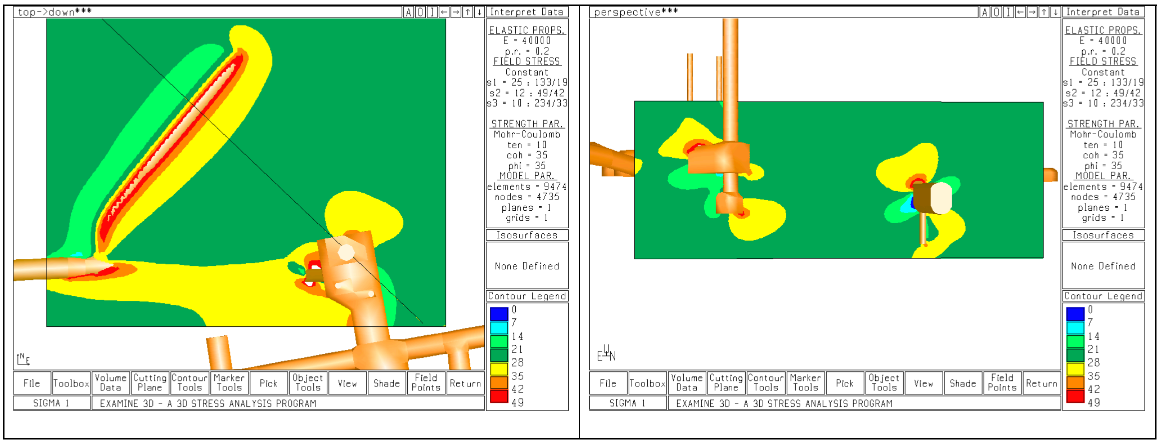

Figure 9. *Cutting planes presenting the stress field for alternative 1 in the rock mass between TASF and the hoist shaft. Note that the in situ stress of 25 MPa is within the dark green span. The right picture is a view perpendicular towards the diagonal line in the left picture.*

As can be seen in Figure 9 the stress situation in the rock mass between the tunnels is unaffected. It can also be concluded that the start of the new tunnel in the F-tunnel can be changed rather much without risking to cause interference effects between it and the existing tunnels.

The redistribution of stresses (from an excavation of alternative 1) around the existing TASF-tunnel will not create volumes with high stress concentrations or low confinement. It is therefore likely that very little extra support in TASF will be necessary because of a tunnel in alternative 1.

Figure 10 presents the result from the stress analysis of the rock mass between alternative 3 in the A-tunnel (TASA) and the hoist and ventilation shafts.

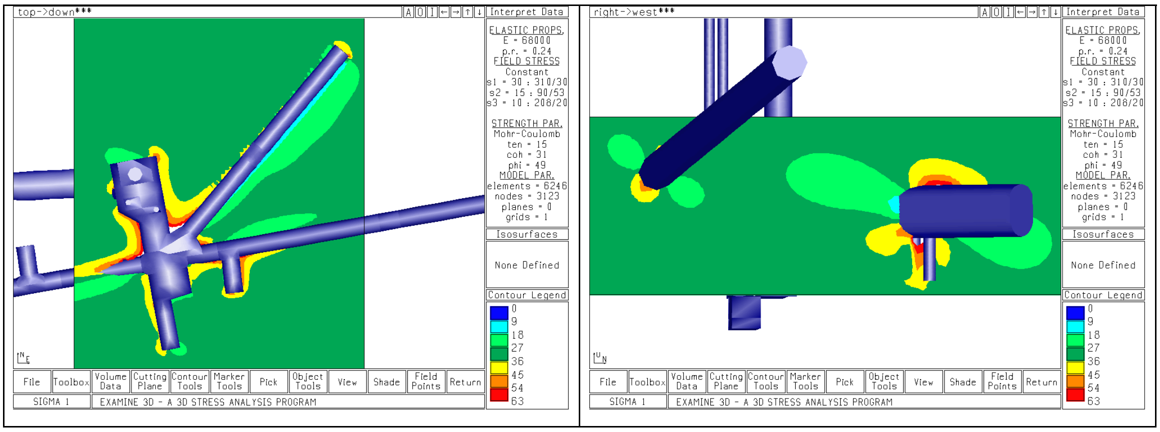

__Figure 10.__ Cutting planes presenting the stress field for alternative 3 in the rock mass between TASA and the hoist shaft. Note that the in situ stress of 30 MPa is within the dark green span.

Also for this alternative the experiment area will not be located in a stress field that is disturbed by the existing tunnels. The disturbance from the existing geometry's will stretch approximately 20 m into the alternative 3 access tunnel.

The tunnel crossing where the alternative 3 tunnel starts will have a rather large span and some unloading of the roof will take place. The unloaded volume will though be rather small and no stability problems are expected.

Besides the effect from the stress field around the existing tunnels the new experiment tunnel has to be long enough to ensure that its end effects does not disturb the stress field in the area of the pillar. *Figure 11* presents the stress situation along the new tunnel. The stress pattern in the pillar looks different compared to the plots presented in Section 5 since a less dense grid was used when these calculations were made.

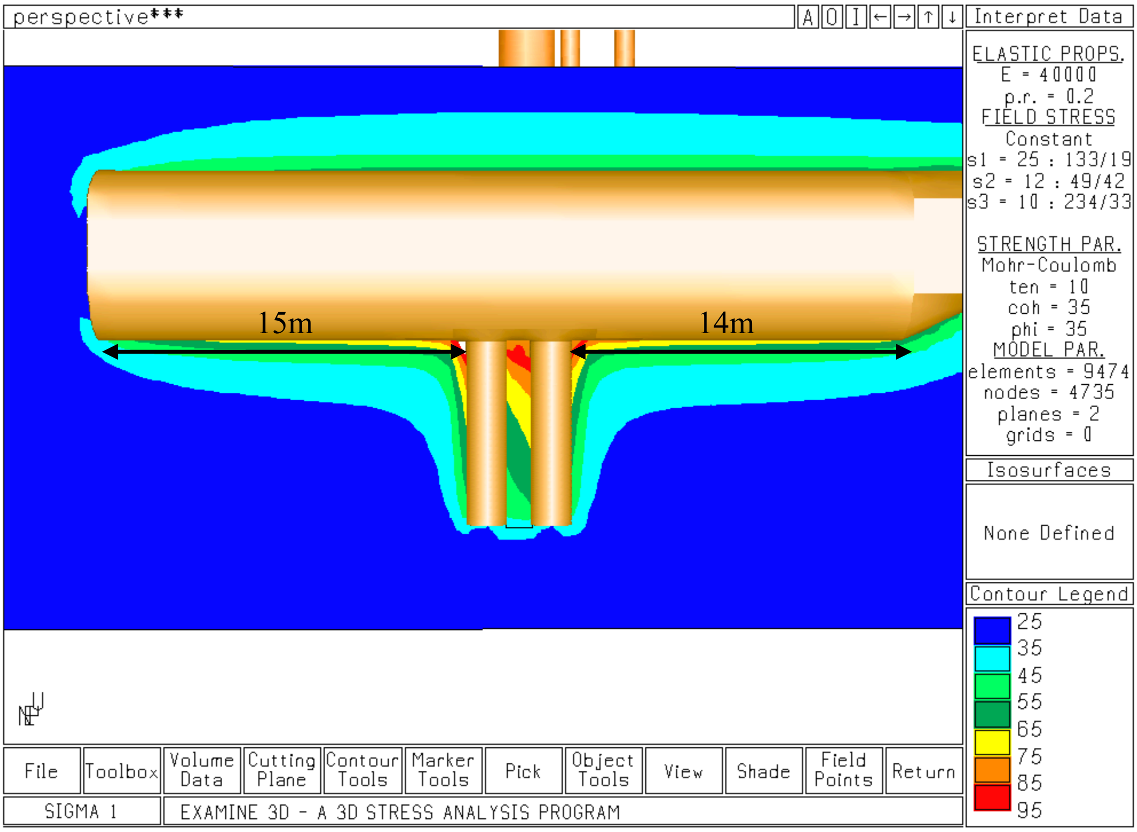

Figure 11. *The stress situation in a plane along the new tunnels alignment.*

The distance between the two holes and the ends of the part of the tunnel with the curved floor is approximately 15m. This distance can be reduced to approximately 10m and still avoid end effects from the tunnel. It is however recommended that the tunnel not should be optimised in length since a single discontinuity may make it necessary to move the pillar a few meters back or forth in the tunnel to avoid the discontinuity.

4.4 Other experiments at the 450 m level

There are several ongoing experiments in the area where the new tunnel is proposed to be located. The experiments in question are the:

1. Prototype Repository,
2. Long term test of buffer material,
3. Microbe,
4. Chemlab and the
5. Matrix fluid experiment.

It has to be investigated if the vibrations and water pressure peaks in the fractures originating from the blasting of the new tunnel might effect or damage these experiments. If this is the case the whole new tunnel might have to be excavated very carefully or the location of the new tunnel changed to an area further away from these experiments.

5 Preliminary design

This section will present a preliminary design of the experiment and describe the iterative process used to achieve an as favourable experiment geometry as possible.

5.1 Geometry and its effect on pillar stress

Since the far field stresses are fixed it is only possible to achieve higher stresses by diverting and concentrating the existing far field stresses. The air in a tunnel opening can't take loads from the far field stresses, hence the load from them must be taken by the rock close to the opening. To illustrate this *Figure 12* presents the stress concentrations around a circular tunnel opening.

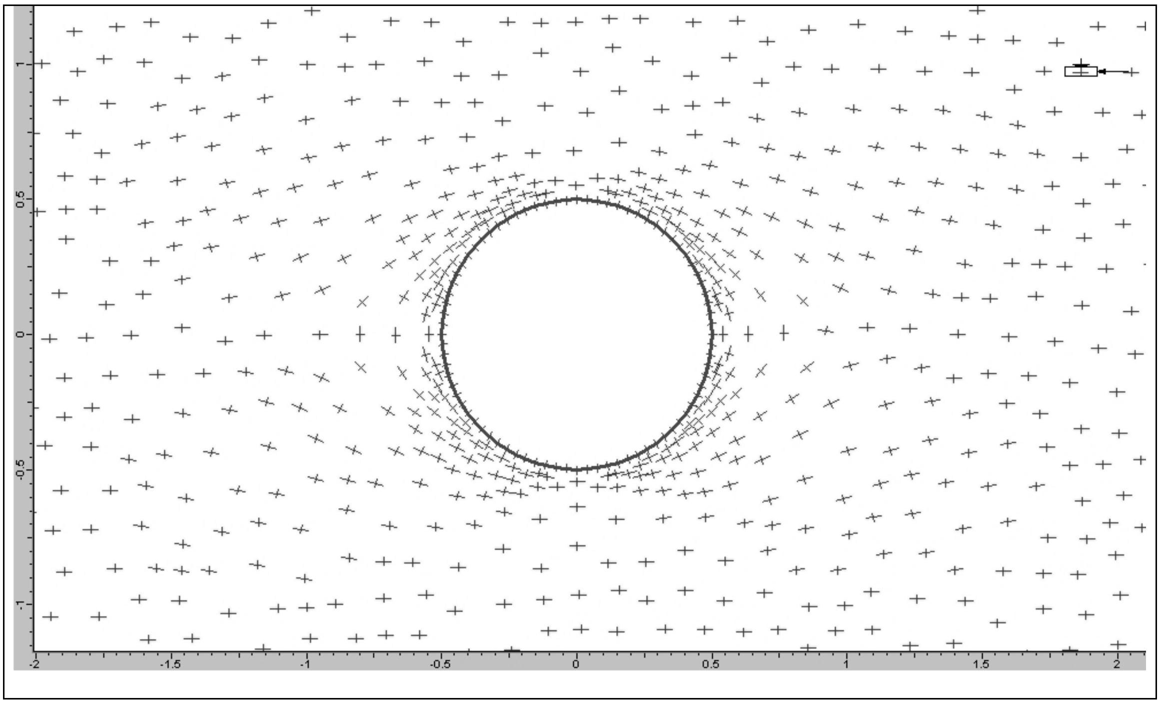

Figure 12. *The stress field around a circular tunnel opening. The direction of the major principal stress is horizontal.*

As can be seen on the stress trajectories they are diverted from the centre and concentrated around the top and floor of the tunnel. The same principle applies for all tunnel shapes in all stress fields.

To design the stress level in the pillar the four different variables listed below were used. Their respective influence on the stress situation in the pillar is described in the following sections

1. Tunnel size
2. Tunnel shape
3. Pillar width
4. Hole geometry

5.1.1 Tunnel size

Especially the height of the tunnel determines the stress at the top and floor of a tunnel. *Figure 13* presents the stress situation around two tunnel openings in the same stress field but with different height.

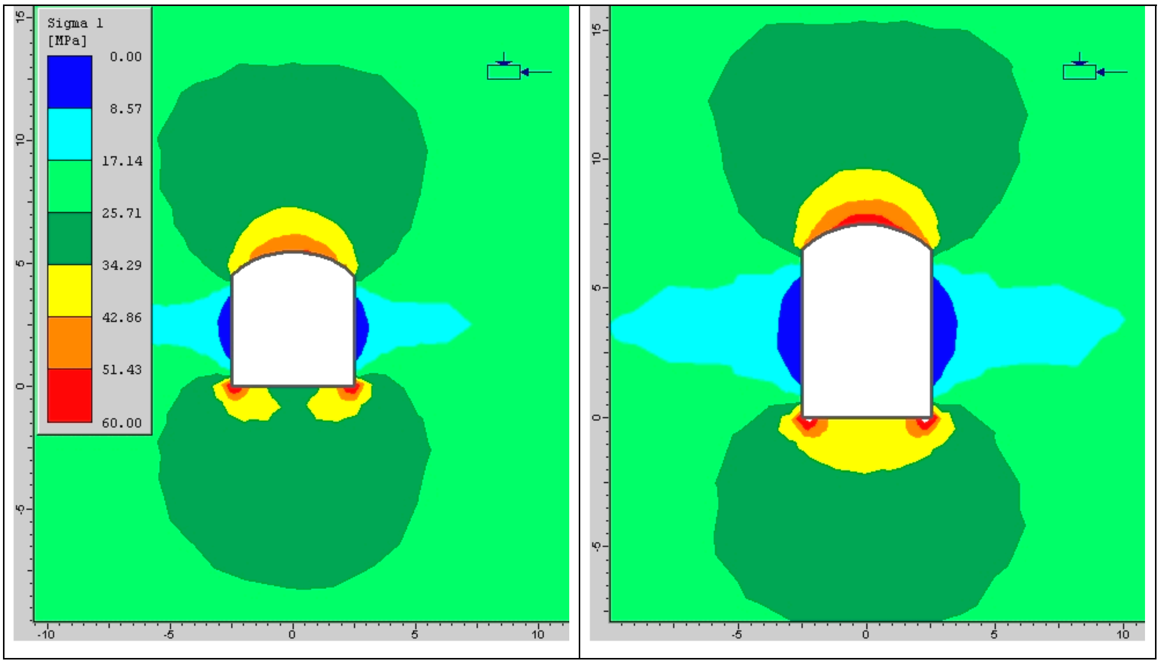

Figure 13. *Stress situation around a 5.5m high tunnel (left) and a 7.5m high tunnel (right). The magnitude of the far field stress is the same in the two pictures. The major principal stress is horizontal.*

As can be seen in the figure the two extra meters of height increases the stress levels in the floor and top of the tunnel significantly.

5.1.2 Tunnel shape

A conclusion that can be made from *Figure 13* is that the tunnel shape effects the stress distribution around an opening. The rounded top of the tunnel has concentrated the stresses to the highest point. In the tunnel's floor, the stresses are concentrated around the corners. It is favourable to have a similar concentration of the stresses in the tunnel floor as in the tunnel top of two reasons:

1) the stresses in the top of the pillar is increased

2) the stresses in the centre of the pillar will be more evenly distributed.

A comparison of the stress situation in a flat and curved tunnel floor is presented in *Figure 14*. The tunnel heights as well as the far field stresses are the same in both cases. Only the shape of the tunnel floor is different.

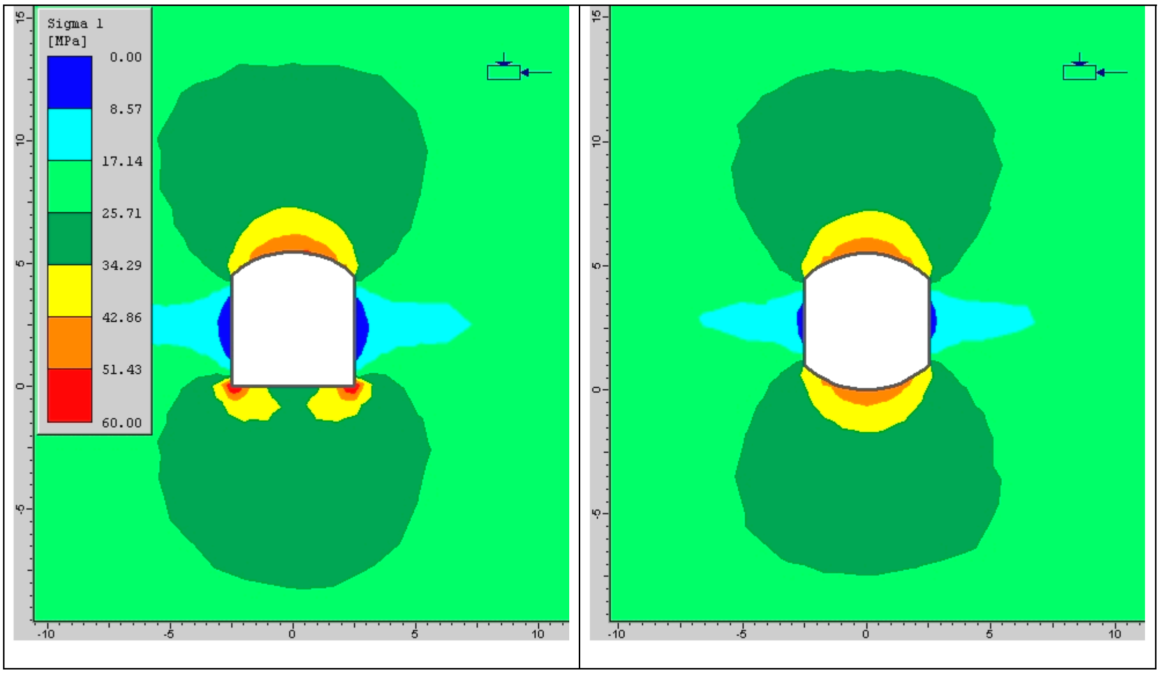

Figure 14. *Comparison of the stress situation in the floor of equally high tunnels but with different shape.*

The curved tunnel floor creates a favourable uniform stress situation and higher stress concentrations in the pillar. The curved floor ought to be excavated as a separate bench, after completion of a pilot tunnel, in order to minimise the excavation damage.

5.1.3 Pillar width

A study of the pillar width's influence on the stress situation in it is presented in *Figure 15*. It is concluded that the stresses increase dramatically when the width is decreased. From a practical point of view, the pillar width should not be much less than 1 m. If designed narrower, single discontinuities in the pillar might have large undesirable effects on the outcome of the experiment. It is therefore recommended that the pillar width is set to 1m.

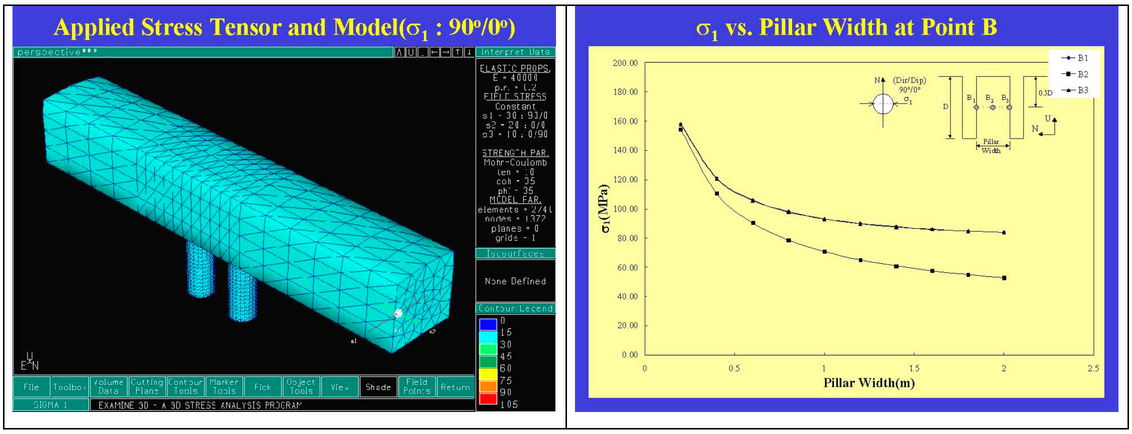

Figure 15. *Stress in pillar versus pillar width approximately 1m down the hole.*

5.1.4 Hole geometry

The holes that will be bored to create the pillar will of course effect the stress situation in it. Larger holes will, in analogy with the reasoning regarding the tunnel height, create higher stresses in the pillar for the same pillar width. A hole diameter of 1.8m is chosen as a reference diameter for the calculations since that is the size of the deposition holes in the future deep repository.

Figure 16 demonstrates that it is possible to control the stress situation in the pillar by drilling or sawing of slots in the hole walls, the keyhole principle. The length of the slots will determine the magnitude of the extra stresses that will be induced. In this case 0.9m deep slots have been modelled.

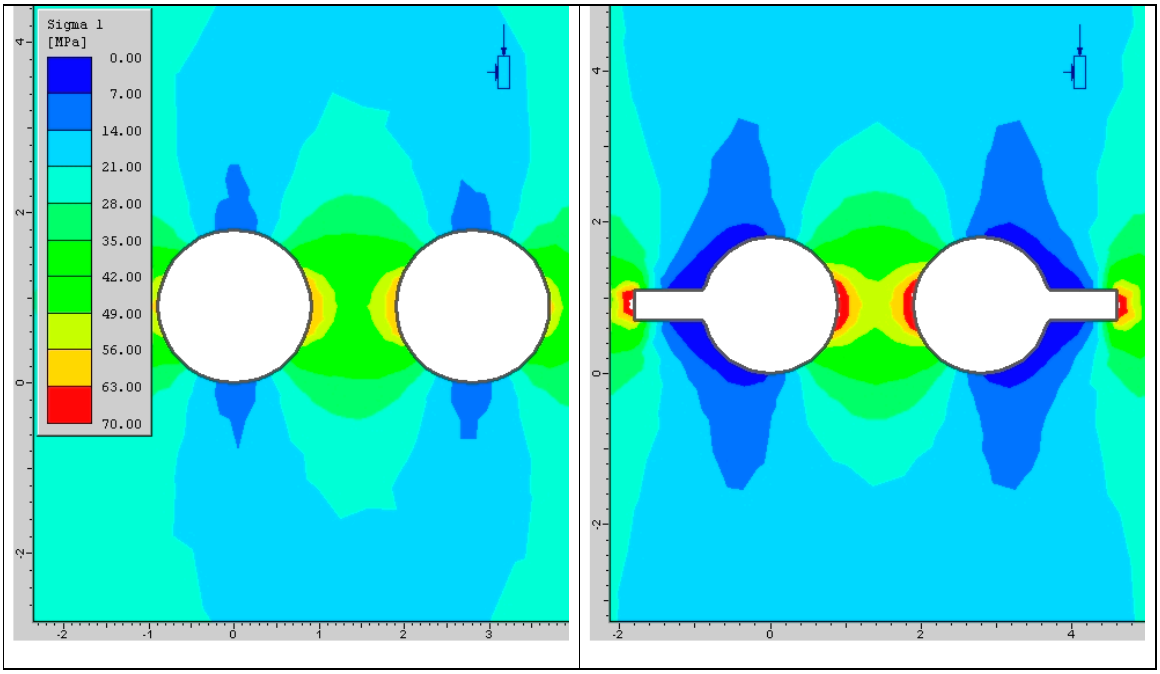

Figure 16. *Comparison of the stress field in the pillar with and without the keyhole principle in the same stress field. The major principal stress direction is from top to bottom. The hole diameter is the same in the two figures.*

5.2 Pillar and tunnel design

When evaluating the results from the different models the stress situation approximately 1m down the pillar is compared. The reason for this is that one can expect some excavation damage in the first 0.5m of the pillar. The objective with the modelling is that the stresses in the pillar ought to be approximately 120 MPa at 1m depth along the hole walls. It is assessed that the stresses then are high enough to ensure spalling in the pillar when it later is heated.

All the results presented in this section except those in *Figure 17* are calculated using the general geometry presented in *Figure 18*. Only the dimensions of the tunnel and holes are changed between the different realisations.

As a first reference case an approximately 5 by 5m tunnel with two 8m deep 1.8m diameter holes was modelled. The pillar between the holes was 1m. The stress on the hole walls will be approximately 100 MPa, Figure 17.

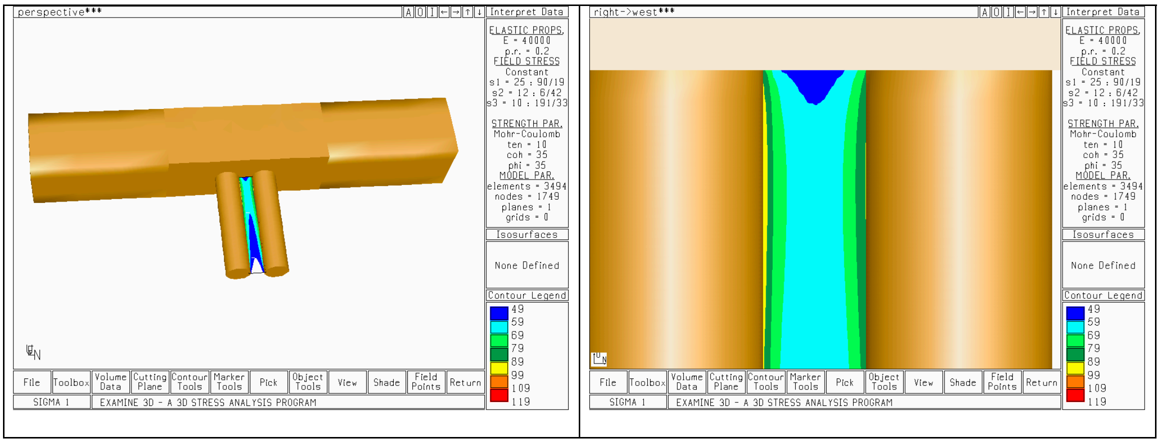

Figure 17. *Stress level in a 1m pillar between two 1.8m diameter boreholes.*

As mentioned above the stress level in the pillar prior to heating should be in the order of 120 MPa. To increase the stress in the pillar a new model was built. The tunnel height was increased to 7.5m and the floor curved according to the conclusions in Section 5.1.2. The result is presented in Figure 18.

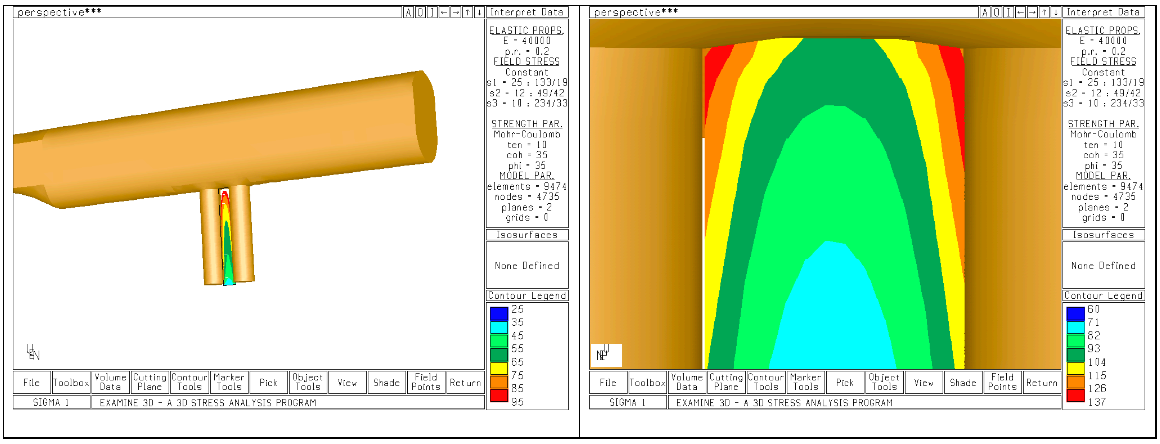

Figure 18. *Stress level in the pillar when the tunnel floor is curved and the height increased from 5 to 7.5m. The first 1.2 meters of the pillar shown in the right figure.*

As can be seen in Figure 18 the stresses in the first meter down the hole are significantly increased compared to the 5m tunnel (*Figure 17*). At 1m depth the stress is approximately 115 MPa on the hole wall. These stresses are however a little bit to low. A realisation was therefore made where the tunnel height was increased by 1m to 8.5m. A comparison of the stress levels in the pillar with 7.5 and 8.5m tunnel height is presented in Figure 19.

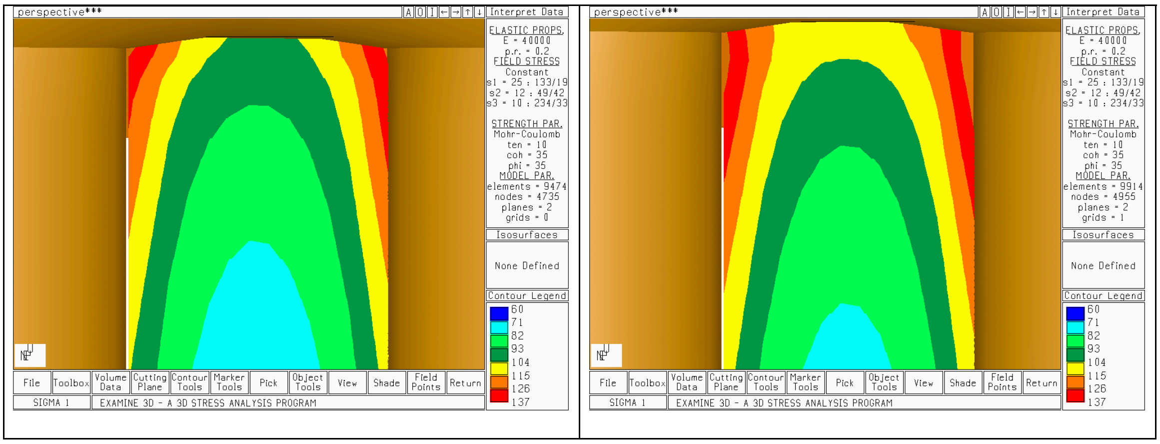

Figure 19. *The stress levels in the first 1.2m of the pillar when the tunnel height is 7.5m (left) and 8.5m (right).*

The one-meter higher tunnel increases the stresses in the pillar to a very moderate extent. The cost for excavating the extra rock volume to make such a small increase in stresses is too high to make this an alternative design. The stress levels achieved are very close too the ones needed but not quite high enough and alternative solutions to increase the stresses in the pillar will anyhow be needed.

A model according to the keyhole principle was therefore built and realised, Figure 20 and *Figure 21*.

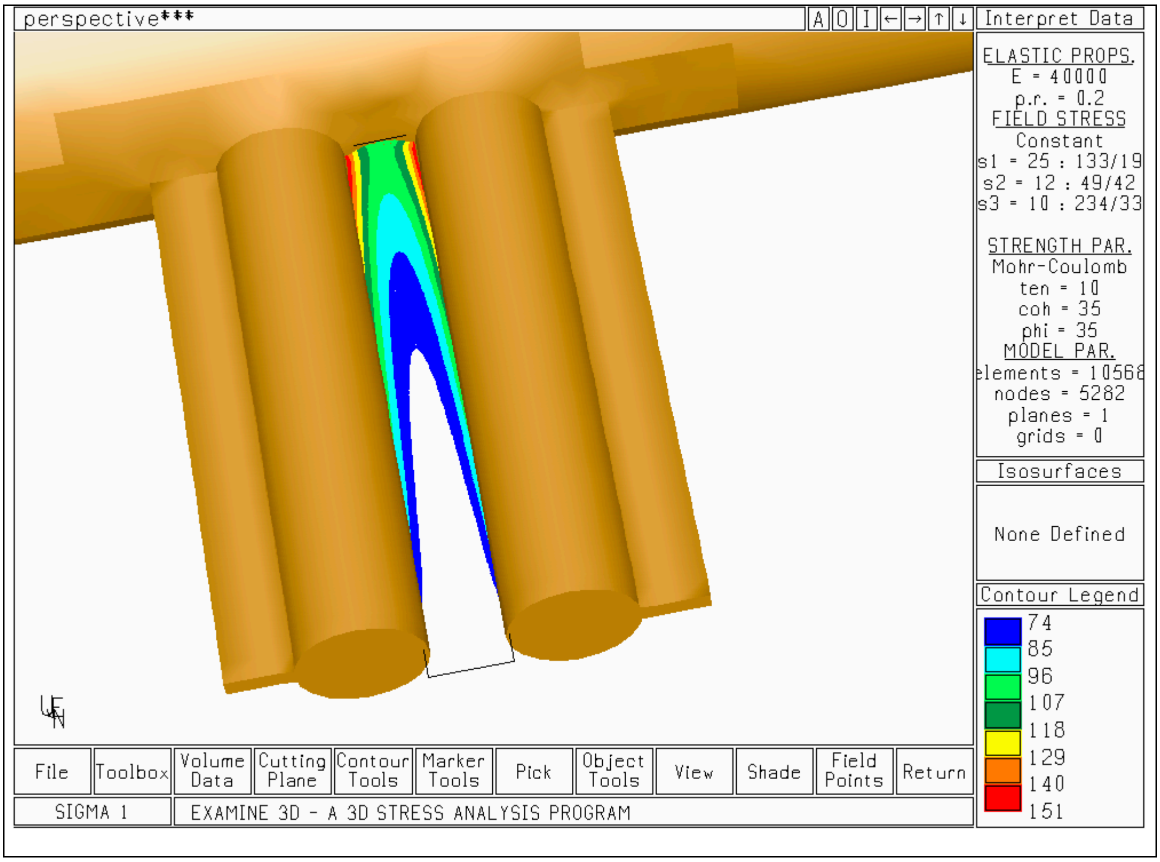

Figure 20. *A 0.9m deep slot in the 1.8m diameter boreholes and the stresses obtained.*

A comparison of the stresses in the pillar without and with a 0.9m deep slot is presented in Figure 21.

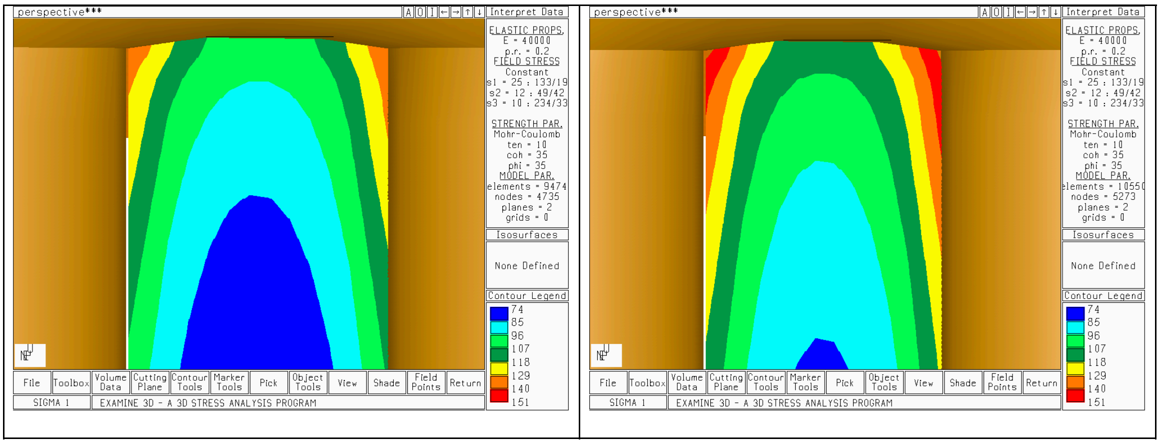

Figure 21. *The difference in stress levels without slot (left) and with a 0.9m deep slot (right). The first 1.2m of the pillar shown.*

As can be seen in Figure 21 there is a large increase in the stress levels when the slot is included. The stress levels are also high enough to create spalling, even without heating. This will however not be a problem since the stress levels in the pillar easily can be optimised by choosing the optimal depth of the slot.

During certain circumstances it might be troublesome to bore 1.8m diameter holes. Holes with a diameter of 1.0m are easier to bore and are here therefore the backup alternative. Scooping calculations have been made to see the stresses in the pillar when the hole diameter is 1.0m, Figure 22. The figure also includes the results from calculations where a 0.8m deep slot have been incorporated in the model with the 1.0m diameter holes.

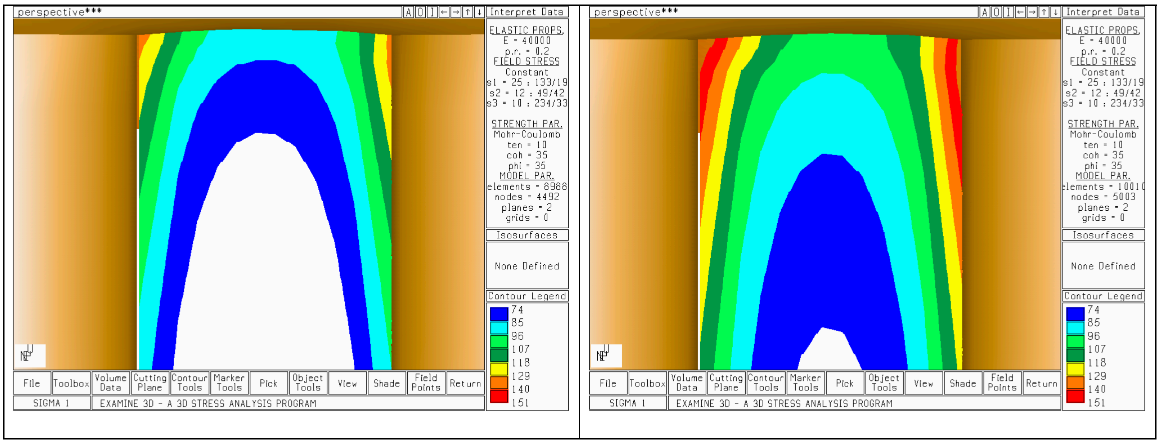

Figure 22. *Stress levels in the first 1.2m of the pillar when the hole diameter is 1m. Left, result without and right, result with a 0.8m deep keyhole slot.*

With the keyhole slot the stress level will be high enough for the experiment. The zone where the spalling will occur will though be narrower in a 1.0m hole than in a 1.8m hole. A 1.0m hole experiment will therefore be more difficult to monitor since less spalling will occur. A 1.8m diameter hole should therefore be the reference alternative.

5.3 Confinement

To apply an artificial backfill pressure of 1 MPa in one of the holes a watertight membrane will be used. The membrane will see too that no water seeps into the rock mass and as a consequence lowers the confinement pressure. The required water pressure in the membrane can be achieved by using Nitrogen gas to pressurise the water in the same way as in the packers at Äspö HRL.

5.4 Design recommendations

Based on the results from the numerical modelling presented these design recommendations are issued:

- The experiment tunnel should be oval in shape, 35m long and 5m wide by 7.5m high. Another approximately 20m length of tunnel is needed to get the start of the experiment tunnel away from the influence the existing tunnels on the stress field. This tunnel should be of standard dimension, 5m wide by 5.5m in height.

- The holes shall have depth ranging between 6 to 8m and a diameter of 1.8m, the pillar between the holes shall be 1m in width

- Slots, approximately 0.9m deep shall be drilled or sawn in the hole walls opposite the pillar

A section of the recommended tunnel shape is presented in *Figure 23*.

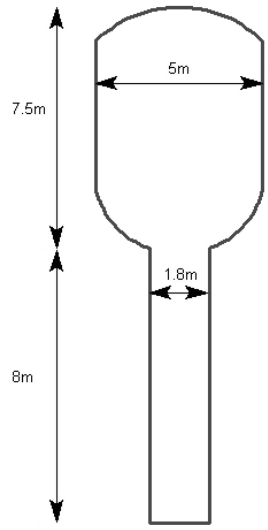

Figure 23. *Section of the recommended tunnel shape.*

6 Characterisation

The 450 m level is quite well defined from a geological and rock mechanical point of view. Further characterisation will though be needed. The current available and the needed additional information are summarised in this section.

6.1 Available information

In the TASJ tunnel there are two boreholes (KJ0050F01 and KJ0045F01) located close

to the experiment area. The borehole KF0093A01 approaches the experimental area from the TASF tunnel and the borehole KA2599G01 approaches the experiment area almost vertically from the 340m level. KAS02 reaches the experiment volume from the surface. The location of the above mentioned boreholes is presented in *Figure 24* and the characterisation made in them is presented in Table 3.

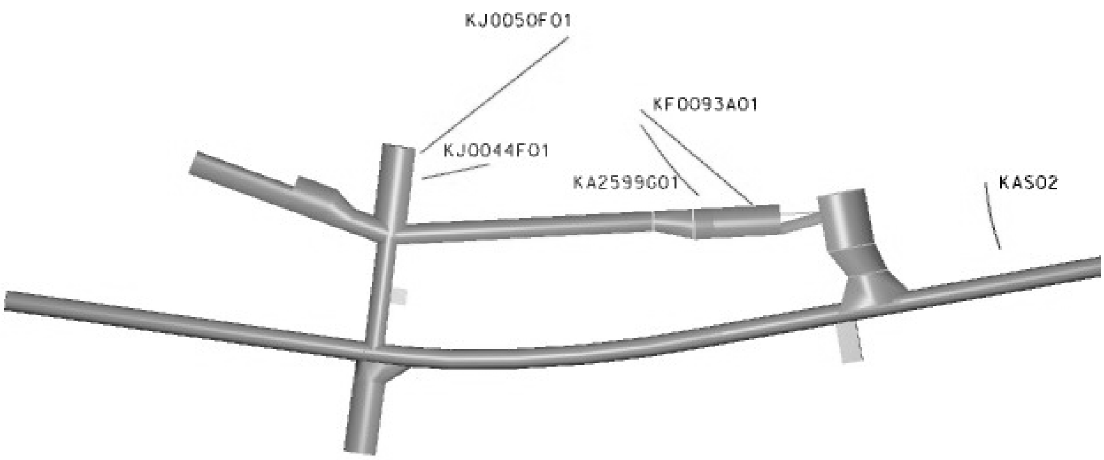

Figure 24. *The location of the five boreholes in the area of the experiment, including the location of the new tunnel.*

Table 3. The five boreholes in the area of the experiment and their characterisation.

Borehole	Type of characterisation made
KJ0050F01	BIPS- and core-logging
KJ0044F01	BIPS- and core-logging
KF0093A01	Biaxial strain, BIPS- and core-logging, Hydraulic fracturing, Overcoring in 2D and 3D, Seismic core log, Thin section,
KA2599G01	Biaxial strain, BIPS- and core-logging, Density, Hydraulic fracturing, Overcoring in 2D, Porosity, Pressure build up test, Seismic core log, Thermal properties, Thermal response, Thermal expansion, Thin section
KAS02	BIPS- and core-logging

There are also data from laboratory tests and other measurements and assessments of the rock's properties available. This data can be used for scooping numerical modelling but for a final prediction, data derived from samples taken in the immediate vicinity of the pillar is necessary.

6.2 Needed information

More site specific information is needed to be able to produce good predictions for the experiment. The area will therefore be further characterised in different steps depending on how far the project has progressed. A summary of the different disciplines and the additional and complementary parameters they will need for the predictions is summarised in Table 4.

Table 4. Compilation of disciplines and the parameters needed for the predictions.

Discipline	Parameter
Rock mechanics	In situ stress
	Uniaxial strength
	Young's modulus
	Poisson's ratio
	Core density
	Density log in borehole
	Thermal expansion
	Thermal conductivity
Geology	Core mapping
	BIPS (Televiewer)
	Tunnel mapping
Hydrogeology	Inflow measurements in borehole
	Pressure build up tests in borehole

6.3 General characterisation program

To characterise the rock mass in the proposed areas for the experiment core boreholes should be drilled in an early stage. The boreholes should be aligned with the centre of the planned tunnel alternatives a few meters above the floor. The location of the alternatives is presented in a top view in Figure 25.

Most of the parameters listed in Table 4 will be determined in these boreholes. Parameters that needs be determined in a laboratory will be measured on core samples taken from the core at the chosen alternative.

The density log is necessary since different rock types have significant differences in their coefficient of thermal conductivity. If the density log indicate a large density variability close the proposed experimental area it may be inappropriate to site the experiment there. This is especially true if the coefficient of the thermal expansion follows the same pattern as the thermal conductivity.

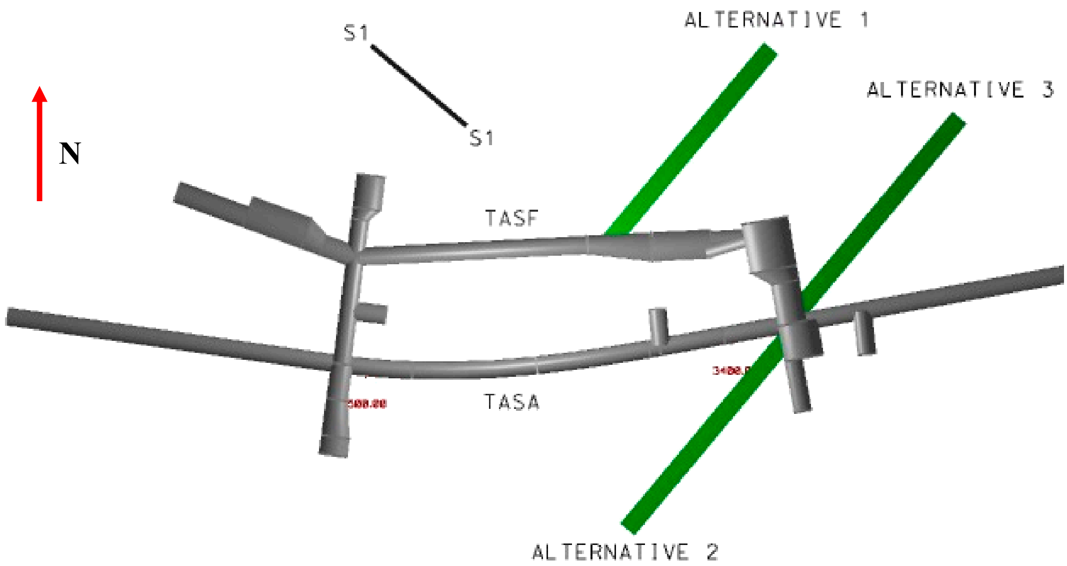

Figure 25. *Green lines indicating the proposed alternatives of the tunnel. The core boreholes should be located within the green lines.*

After a complete characterisation of the boreholes no further characterisation will be made until the tunnelling is completed. Then, a detailed geological characterisation with a shortest fracture trace length of 0.5m will performed. The tunnel floor in the proposed experiment area will be mapped in even more detail. The exact location of the pillar will be determined with the fracture mapping used as a base. When the pillar location is determined additional core drilling will be performed in the periphery of the future holes close to the pillar wall. Core drilling will also be made for the instrumentation and the heaters. Samples from these cores will be used to determine the rock mechanical (except in situ stress and density log) and the geology parameters.

7 Instrumentation and heaters

To assess the outcome of the experiment three types of instrumentation is needed. The most important of these is the acoustic emission (AE) system. AE is the most important system since the stresses in the pillar will most likely not be large enough to create fallouts in the borehole. The occurrence of spalling will instead be detected by an increased event rate in the AE log. AE will also be used to localise the events in the rock mass and to measure the seismic velocity. The seismic velocity measured close to the boreholes walls will indicate the depth of the damaged zone.

The second type of instrumentation needed is a number of thermocouple arrays. There will be two arrays in each hole attached to the walls and a few arrays in boreholes close to the pillar.

The third type of instrumentation is displacement/convergence measurements of the tunnel during excavating and in the open hole during heating.

In addition to the instruments, heaters will be needed. Based on experiences from AECL's heated failure experiment it is preliminary assessed that four heaters with an effect of 3000kW will be adequate. The temperature in the heater holes might reach as high as 500 degrees but the temperature at the hole walls will be limited to 90 degrees. A draft set up of the instruments geometry is presented in Figure 26 and a more detailed, but still a draft set up, of the acoustic emission system is presented in *Figure 27*, Young (2002). In *Figure 27* a reference hole depth of 8m is used. The set up of the instruments would change very little if the hole depth is reduced to 6m.

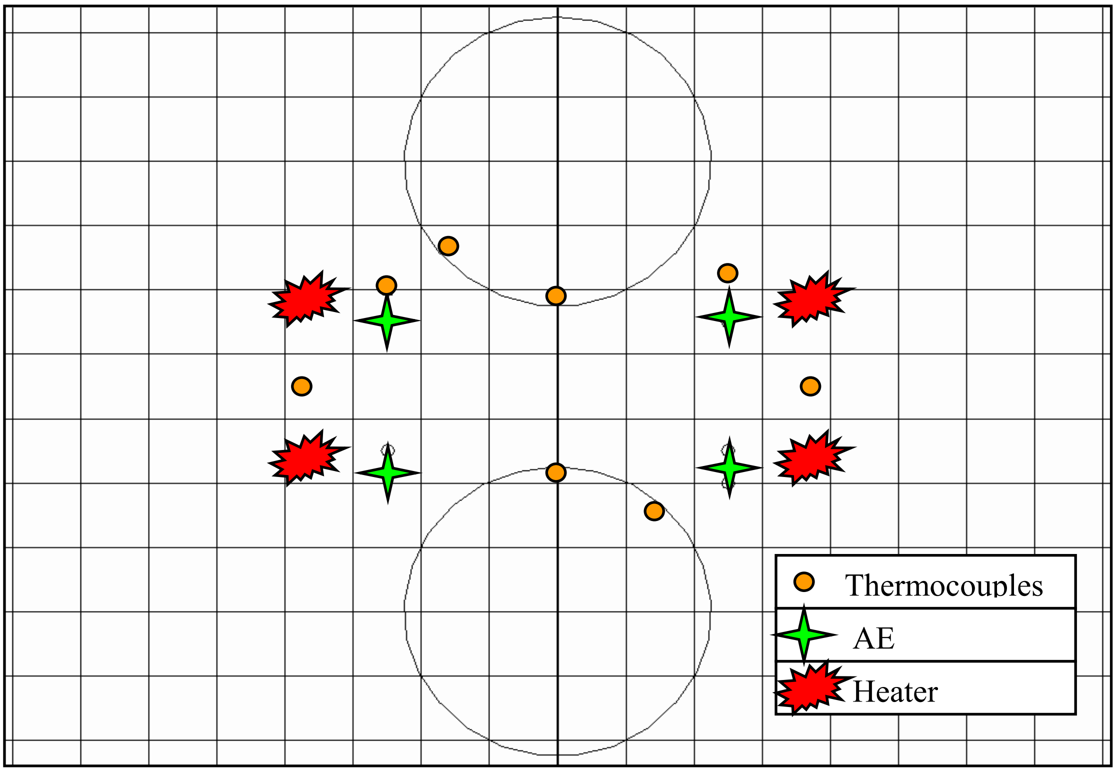

Figure 26. *Draft set up of the instrument geometry. The grid spacing is 0.4m.*

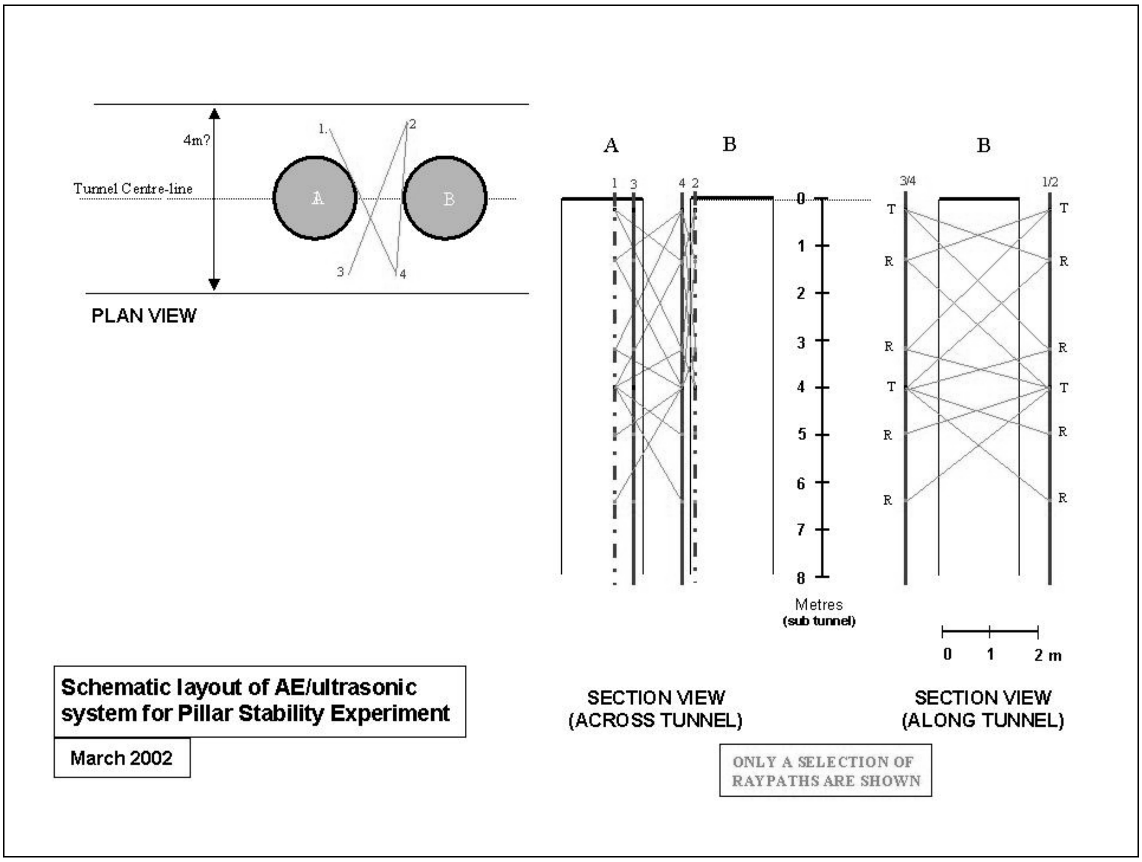

Figure 27. *Draft diagram of the set up of the acoustic emission system.*

The array consists of transmitters and receivers installed in 4 boreholes around the pillar. The position of boreholes 1, 2 and 4 is chosen to given raypaths that pass tangentially to the deposition hole walls. Borehole 3 is positioned so that raypaths passing through the central part of the pillar can also be investigated. The sensors in borehole 3 also provide good coverage for the detection and location of AE events occurring within the pillar.

Each borehole would contain 2 transmitters and 4 receivers. These are used in the same way as the system installed for similar measurements in the Retrieval and Prototype Tunnels. In this case the arrangement of transmitters and receivers within the borehole is a little different to the previous cases. The transmitters are positioned so that there is one close to the borehole collar and one about half-way down the borehole. This provides improved ray-coverage over the upper part of the pillar/deposition hole walls, which will allow us to image and investigate the spalling process in more detail. The lower 2 metres of the deposition hole is less well imaged.

8 Modelling and visualisation

Different tools will be used to predict the outcome of the experiment and to evaluate the results. This section will discuss the planned modelling and visualisation stages of the project.

8.1 Modelling

Several different types of modelling tools will be used. The modelling stages will in short be the following.

1. The geometry of the tunnels and holes will be designed with respect to the stress situation in the pillar and the disturbance from the stress field in the existing tunnels.
2. The information from the first core hole will be used for a thermal model that will be used for the design of the placement of the heaters and thermocouple arrays in relation to the large bore holes.
3. When the geometry is preliminary set a Microstation model will be created. It will cover the new tunnel and the location of the different holes for instrumentation.
4. After completion of the tunnelling the geological mapping will be used for creating a Discrete Fracture Network-, DFN-model. This work will then be used to determine the exact location of the experiment.
5. The final numerical modelling of the rock mass response to heating will be made after the results from the lab tests on the instrumentation and heater hole cores are finished.

The results from these modelling stages will be reported before the experiment starts. The report shall predict at what temperature/stress level the spalling will occur and how far into the rock the spalling will go. The vertical extent of the spalling into the hole shall also be predicted.

8.2 Visualisation

A 3D visualisation tool will be needed to present the results from the predictive modelling and the information from the monitoring. It is necessary to visualise the results in 3D to get a good understanding of the AE and displacement distribution both in space and time.

The information that will be used to assess the outcome of the experiment is listed below.

1. Geology
2. Fracture mapping/DFN-model
3. Geometry
4. Acoustic Emission
5. Seismic velocity
6. Stress contours
7. Convergence/displacement

9 Realisation

This section will pinpoint most of the important practical steps in the realisation of the APSE. The tunnelling is further described in Section 9.1.

1. Drilling of core hole in the location of the bench in the new tunnels proposed alignment including characterisation
2. Tunnelling including:
   - Pre-grouting
   - Excavation of pilot tunnel
   - Excavation of bench
3. Geological mapping of the new tunnel
4. Casting of concrete in the area of the holes and filling up the rest of the tunnel with gravel
5. Drilling of instrumentation and heating holes, including characterisation
6. Installation of instrumentation
7. Drilling of slot by the large holes to be bored
8. Boring of the first hole
9. Installation of confinement equipment in the first hole and test of it. Setting of confinement pressure to 1 MPa
10. Drilling of the second hole. Rock mass response monitored by AE
11. If the AE event rate is too low when the second hole is bored and it is assessed that the stresses in the pillar are to low the slots can be extended to increase the stress. Numerical calculations together with AE and extension of the slots will make it possible to get precisely the stress level desired.
12. Controlled heating of the pillar until failure in the wall of the unconfined hole. The temperature on the hole wall will be heated to approx. 90 degrees. A steady thermal state will then be held until the rate of the AE events drops significantly.
13. Lowering of the confinement pressure in the first hole to a level determined by the predictive modelling. If the AE event rate does not increase the pressure will be lowered until spalling occur.
14. Decommissioning of the experiment

9.1 Tunnelling

The tunnelling work will be performed through the three activities: pre-grouting, excavation of pilot tunnel and excavation of the bench. The three activities are briefly described in the sections below.

9.1.1 Pre-grouting

According to the present characterisation data for the proposed area of the new tunnel, amounts of water seepage unlikely. For precaution, there will though be exploratory percussion drilling in front of the tunnel. Excluding the core borehole drilled for characterisation another six percussion-drilled holes is needed. The first three holes will be drilled from the start of the new tunnel and they will be approximately 35m long respectively. The last three holes will be drilled when the tunnel has advanced 25m. Also these three last holes will be 35m long respectively. The approximate location and extension of the six holes is presented in *Figure 28*.

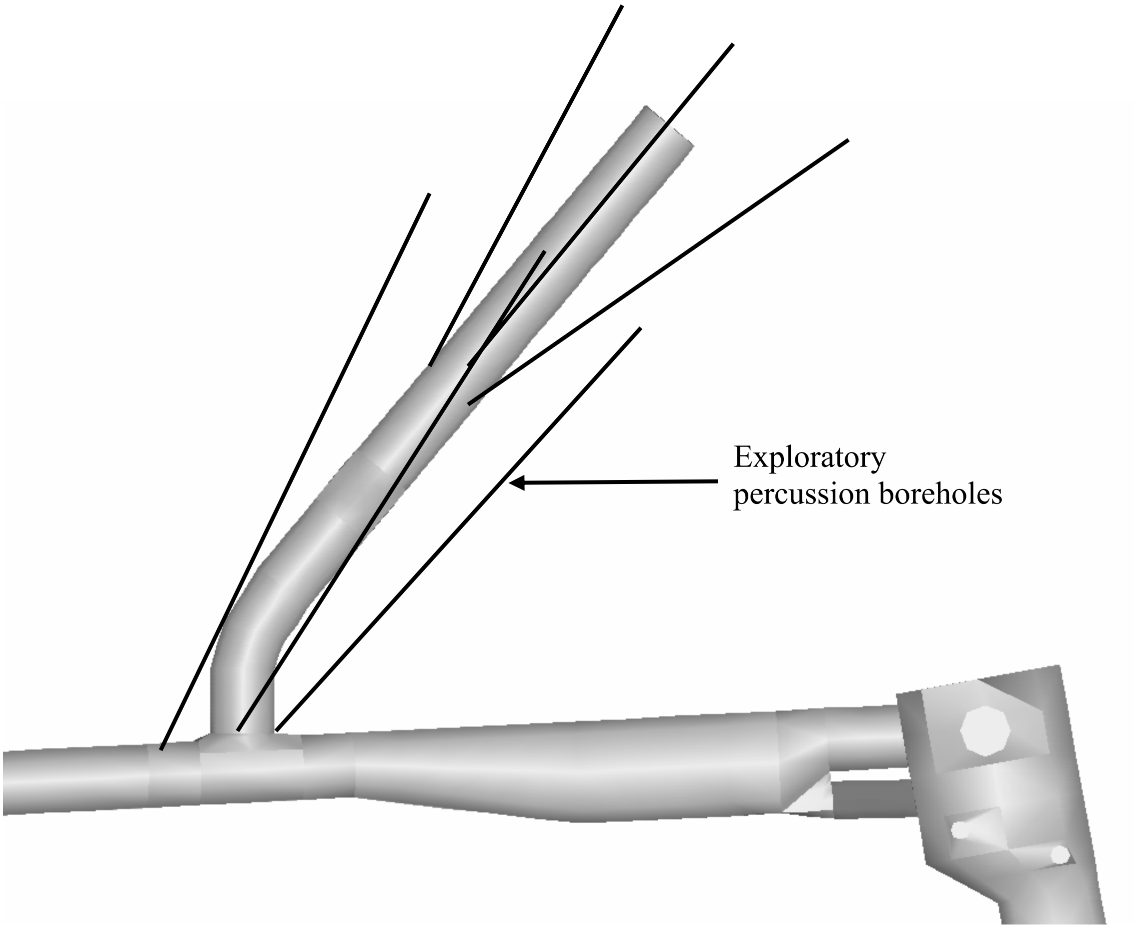

Figure 28. *Approximate location and extension of the exploratory percussion boreholes. Alternative 1 taken as an example.*

If the seepage in one or several of the boreholes is assessed to create problems in the new tunnel, pre-grouting will be performed. If it is decided to pre-grout a few more holes is needed to achieve a good grouting result. The location of these holes will be determined on site. When the pre-grouting has been performed one or two boreholes will be drilled along the tunnel to see if the desired effect was achieved. The location of this/these borehole/s will also be decided on site.

9.2 Excavation of pilot tunnel and bench

Approximately 60m of new tunnel will have to be built. Of these 60m the last 35m shall have a height of 7.5m and a carefully blasted curved floor. To make the tunnelling as cheap as possible it is proposed that the tunnelling is performed in two stages. First a standard 5 by 5.5m-pilot tunnel is excavated through standard drilling and blasting techniques. Then, a 2m deep curved floor is excavated very carefully to minimise the damage in the new floor. *Figure 29* presents this staged excavation.

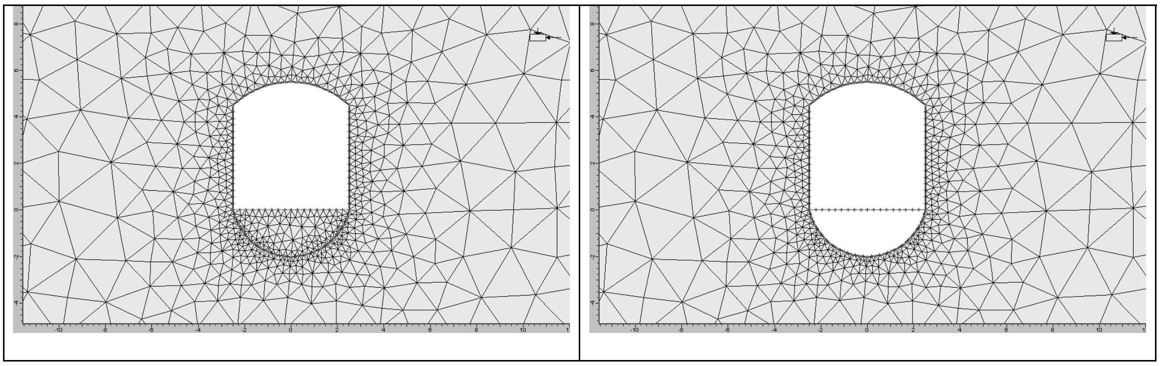

Figure 29. *Finite element model of the staged excavation showing the pilot/standard tunnel to the left and the full contour of the experiment part of the tunnel to the right.*

10 Conclusions

The results from scooping calculations made with the boundary element code Examine3D and the finite element code Phase2 leads to the following conclusions:

1. The new tunnel for the experiment can be placed in the existing facility at the 450 m level. The stress field around the current tunnel system will not effect the experiment area of the new tunnel in any of the three alternatives.

2. It is possible to create high stresses enough in the pillar before heating to ensure that spalling will take place. To achieve this, no extreme geometry is needed and standard drilling, cutting and rock excavation equipment can be used.

3. The following design of the experiment tunnel and holes is proposed:

   - The experiment tunnel in the vicinity of the pillar should be oval in shape, 35m long and 5m wide by 7.5m high. The experiment volume needs to be located in a stress field that isn’t disturbed by the existing tunnels. The access tunnel to the experiment part of the tunnel should be of standard dimension, 5m wide by 5.5m in height. The total tunnelling length for the experiment is assessed to be approximately 60 – 70 meters.

   - The holes shall have depth 6 to 8m and a diameter of 1.8m, the pillar between the holes shall be 1m in width

   - Slots, approximately 0.9m deep will probably be drilled or sawn in the hole walls opposite the pillar

4. Acoustic emission and convergence measurements are important tools to confirm the spalling tendency

There is nothing in the numerical calculations that indicate that the experiment would fail because spalling isn’t achieved when heating is taking place.

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Summary

This report is a study of the feasibility for performing a pillar stability experiment at Äspö HRL.

One objective with the Äspö Pillar Stability Experiment is to demonstrate the capability to predict spalling in a fractured rock mass. Another important objective is to demonstrate the effect the backfill has on the propagation of micro cracks in the rock mass closest to the deposition holes.

To achieve these objectives a new tunnel will have to be excavated. Two holes in the new tunnel's floor will be bored and a pillar will be created between them. One of the holes will be confined with 1 MPa water pressure in a liner to simulate backfill. The rock mass in the pillar will be heated and the additional thermal stresses shall force the rock in the hole walls to spall. The spalling will be monitored with an acoustic emission system, which will be the main instrumentation for evaluation of the experiment.

The pre-study has come to the following conclusions:

- The new tunnel for the experiment should be located at the 450 m level at Äspö Hard Rock Laboratory
- The new tunnel should have a height of 7.5m and a width of 5m. The tunnel's floor should be curved.
- The width of the pillar should be 1m.
- The diameter of the holes should be 1.8m
- Slots may have to be drilled in the hole walls, the keyhole principle, to create high stresses enough in the pillar before heating.

The study identifies risks in general and describes the strategy for the characterisation and modelling of the experiment as well as the way the experiment should be practically realised. A draft description of the monitoring equipment and their placement is also included.

Contents

1 Introduction

This report is a feasibility study for the project, Äspö Pillar Stability Experiment, APSE.

1.1 Objectives

The APSE is a rock mechanics experiment which can be summarised in the following three main objectives:

1. Demonstrate the capability to predict spalling in a fractured rock mass
2. Demonstrate the effect of backfill (confining pressure) on the rock mass response
3. Comparison of 2D and 3D mechanical and thermal predicting capabilities

1.2 Experiment layout

To perform the experiment a new tunnel will have to be excavated. That is necessary since the stress situation at the experiment location not should be influenced by the existing tunnel geometrys. In the tunnel two large holes will be drilled so that a slim pillar is created between them. The volume around the pillar and the holes will be instrumented with monitoring equipment and heated by electrical heaters located in boreholes.

1.2.1 Geometry and execution

Figure 1 presents one possible geometry for the experiment. During the execution one of the holes will be confined by water pressure in a liner that will simulate backfill. The pillar will then be heated and hence subjected to additional stresses that shall induce spalling in the unconfined borehole's wall. When a steady state is reached the confinement pressure will be lowered to predetermined levels where the numerical models predict spalling.

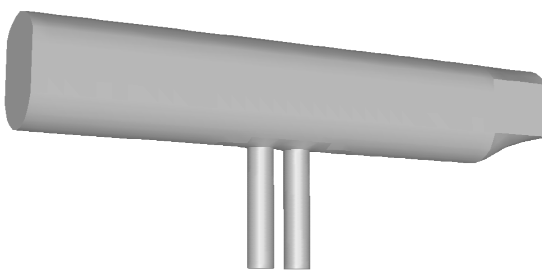

Figure 1. *Possible geometry for the experiment with a slender pillar between two 1.8m diameter holes.*

1.2.2 Verification of the experiment results

The spalling will mainly be monitored with acoustic emission, which together with thermocouples and convergence/displacement measurements are the monitoring instruments needed to evaluate the experiment. If breakouts occur in the large holes it be documented.

1.3 Purpose of the report

This feasibility study is an assessment of the possibility to successfully perform the experiment briefly described above. The main issue in this report is the design of the new tunnel and the pillar. The design is important since it has to be shown that it is possible to achieve a stress level in the pillar before heating that is high enough to ensure that spalling will occur when the rock mass is heated.

The report also presents three alternative experiment locations and briefly discusses the characterisation needed to be able to select a final site.

2 AECL's Heater Failure Test

The first large scale work on thermal response was made by AECL in 1993-1996 in their Heated Failure Test, Read 1997.

The objective with the experiment was to investigate the effects of thermal loading on progressive failure and excavation damage development.

The strategy adopted for the Heated Failure Tests was to monitor the development of excavation damage, characterise the extent of damage in situ and then use numerical modelling to assess the relation between damage development and near-field in situ stresses. Monitoring was conducted using an acoustic emission (AE) system, extensometers, convergence arrays, piezometers and thermocouples/thermocouples. Characterisation activities included geological mapping and photography/videotaping of lithology, induced fractures and breakouts in each of five observation holes.

600mm diameter boreholes were chosen for the experiment. The electrical heaters used had an effect of approximately 3kW and it took approximately 20 days the heat up the hole wall to 85 degrees with almost full effect. The temperature in the heater holes was approximately 500 degrees C.

The experiment confirmed that the spalling extended further in the borehole when the rock mass was heated (in relation to the damaged zone due to redistribution of stresses during excavation). Small pressure changes in a confined hole (water pressure in a vinyl liner) lead to large differences in acoustic emission event rates.

The additional thermal stress from an increase of temperature in the rock from 15 to 85 degrees was 15-20 MPa.

2.1 Justification of the Äspö Pillar Stability Experiment

The Heated Failure Test (HFT) was successfully accomplished and had results in quite good agreement with the model. It might therefore seem unnecessary to perform a similar experiment at Äspö HRL. There are though three major differences in the conditions for the two experiments that justifies the APSE.

- HFT was performed in an almost totally unfractured rock mass
- The in situ stresses in relation the rock strength in URL are so high that the rock mass response is brittle and not elastic
- The APSE will demonstrate the effect of backfill at a maximum confining pressure of 1000 kPa compared to the 100 kPa used at URL. At Äspö the effect of the backfill on the micro cracking will be reliable determined since both a confined and un-confined hole will be tested at the same time in as similar geological environment as possible.

A theoretical relationship between the stress and strain on a core when determining Young's modulus is presented in *Figure 2*. The figure also includes the principal AE event rate when an accelerating number of micro cracks are created as the rock mass response turns brittle.

The in situ stresses at Äspö HRL at 450m depth subjects the rock to stresses that are on the limit of its elastic response. If the stresses are increased slightly more, the response will be brittle. The in situ stresses at URL are so high that failure will occur for almost any change in a existing geometry. The zone between the elastic conditions at Äspö HRL and the failure conditions at URL (eg. the zone where the rock mass response to additional loading becomes more and more brittle) is called the transitional zone.

According to Martin et al. (2001) it is likely that the pillar between the deposition holes will encounter transitional conditions during the life span of the repository. The stress situation directly after placement of a canister will be in the same range as at Äspö (elastic). The swelling pressure, the thermal load and glaciation load (note: the thermal load and the glaciation load will not occur at the same time) will induce such stresses in the pillar that the stress-strain relationship might enter the transitional zone.

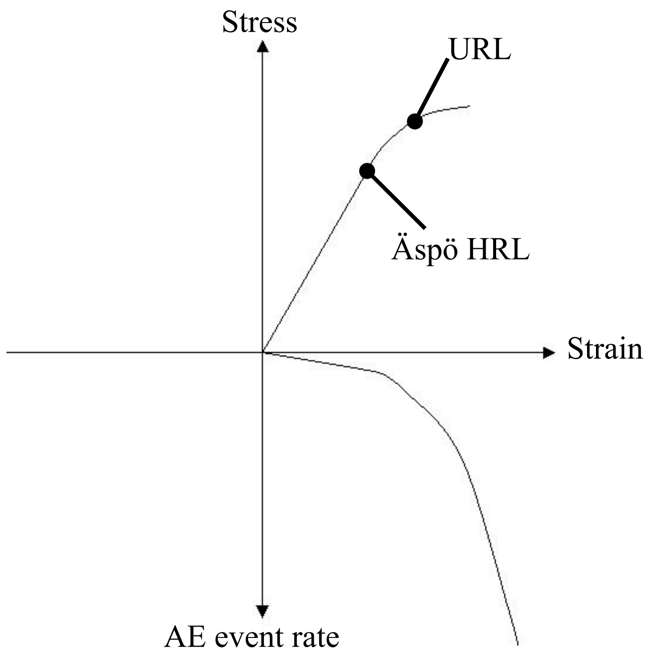

Figure 2. *Relationship between stress and strain on a compressed core. Included is the relative rock stress situation at Äspö HRL and at the URL. The principal increasing AE event rate when micro cracks are formed is also included.*

Very limited rock mechanics research on the rock mass response in the transitional zone (accelerating frequency of micro-cracking) has been carried out. It is therefore important to gain knowledge in this field since the spacing of the canister holes gives a great impact on the total costs for the deep repository. If verified numerical models can be used to optimise the spacing, the design will probably be much more cost effective than the alternative, which is empirical formulas developed for mining conditions. These formulas will be rougher and an unnecessary high safety factor might be the result.

The rock where the HFT was performed was almost unfractured. Since any given rock mass at a Swedish repository site will contain a number of fractures the URL results can't be applied directly. The Äspö experiment will indicate if the rock mass response will be roughly the same as at URL even though it will include some fractures.

The acoustic emission system can rather exactly pinpoint the location of an acoustic event. Valuable information will therefore also be achieved regarding the behaviour of any induced or pre-existing fracture in the pillar.

3 Risks for the outcome of the APSE

A number of risks that could jeopardise the outcome of the experiment have been discussed. The identified major risks and some countermeasures are listed below.

1. Excavation damage in the floor will make the rock mass Young's modulus lower and reduce the increase of stresses when the rock is heated.
   This effect can dramatically be reduced if a curved floor is blasted out carefully in the tunnel as a separate bench.
2. The stress in the pillar before heating may be too low and spalling will not occur when heated.
   The design of the tunnel and pillar will ensure sufficient high stress levels in the pillar to exceed crack initiation stress. The stress level on the hole walls shall be approximately 120 MPa before heating.
3. The selected area is not homogenous from a thermal point of view.
   Characterisation of the rock mass in different stages of the project will indicate anisotropy as early as possibly.
4. Geological conditions.
   Single larger discontinuities will obviously effect the experiment if located in the pillar but the effect of different rock types, especially on the thermal properties, is somewhat uncertain.

Especially the excavation of the new tunnel might effect a number of other experiments. The risks for these experiments are further discussed in Section 4.4.

4 Alternative experiment locations

Since the experiment is dependent of the in-situ stresses the experiment should be located where they are as high as possible. The location of the experiment should therefore be somewhere at the 450 m level. At this level there are there possible sites for the experiment which are presented in Figure 3. Included in the picture is also the direction of the major principal stress, S1.

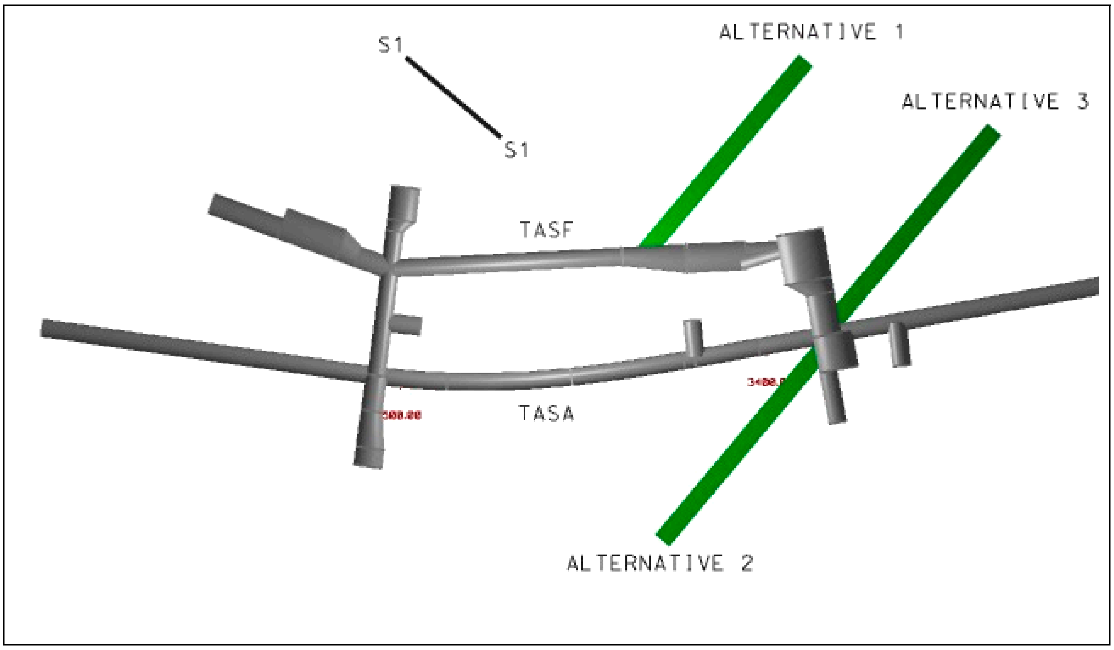

Figure 3. *Location of the three possible experiment locations and the bearing of the major principal stress at the 450m level.*

4.1 Geological knowledge in the areas of the alternatives

This section will summarise the structural geological and rock stress knowledge in the area as it was in the beginning of 2002.

4.1.1 Structural geology

A summary of the main structures, and their occurrence in the area is presented in *Figure 4* and *Figure 5* (Maersk-Hansen, Hermanson, 2002).

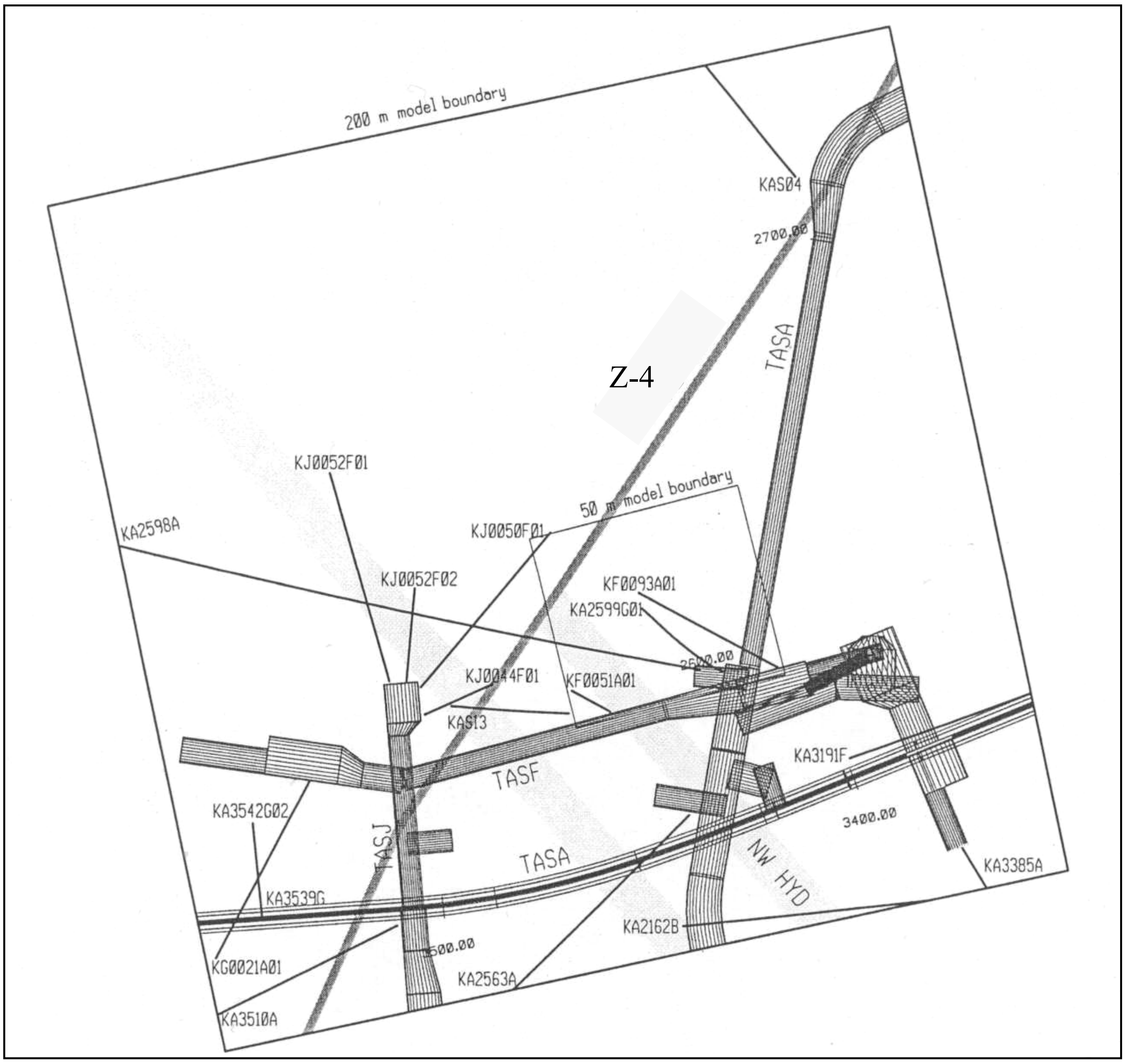

Figure 4. *Tunnels, boreholes and major structures in the proposed areas for the experiment.*

Z-4 is the most dominant structure in the area. It is verified in the tunnels and in borehole KA3510A. Z-4 is also supported by indications in the boreholes KAS04, KJ0044F01, KF0093A01, KJ0050F01 and TASA-tunnel chainage 2740m. Z-4 dips almost vertically and is indicated by an approximately 5m wide fractured zone. The definition of a fracture zone is according to Rhén et al. p.43 (1997) "a zone with the characteristic that the intensity of natural fractures is at least twice that of the surrounding rock".

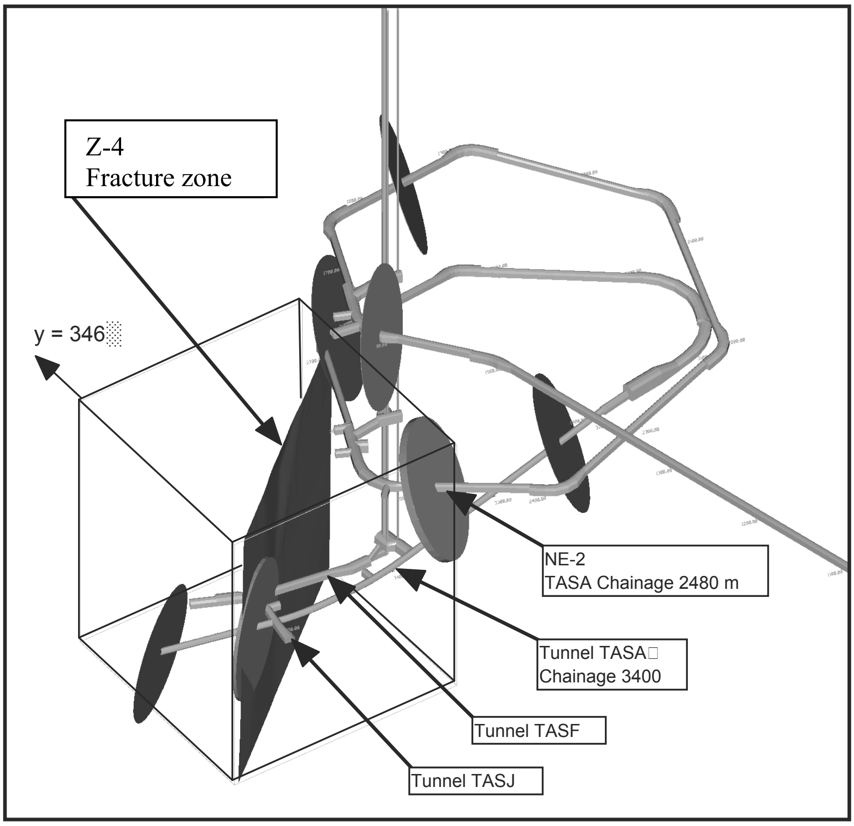

Figure 5. *Structural model showing Z-4, NE-2 and its indications (discs) in tunnels and boreholes.*

Four water-bearing parallel fractures trending NW cuts through both the TASA and TASF-tunnels and is terminated at Z-4. There also seems to be another conductive fracture group parallel with the first one located approximately 30m West of it that does not terminate at Z-4.

A stereonet contour plot of joints in TASA- chainage 2625 - 2700, TASF-, TASG-, TASI-, TASJ-tunnel and the hoist shaft from elevation –350m to the bottom of the shaft is presented in *Figure 6*. The major parts of the fractures are sub-horizontal. There are also two almost sub-vertical fracture sets trending N-S and NW-SE respectively.

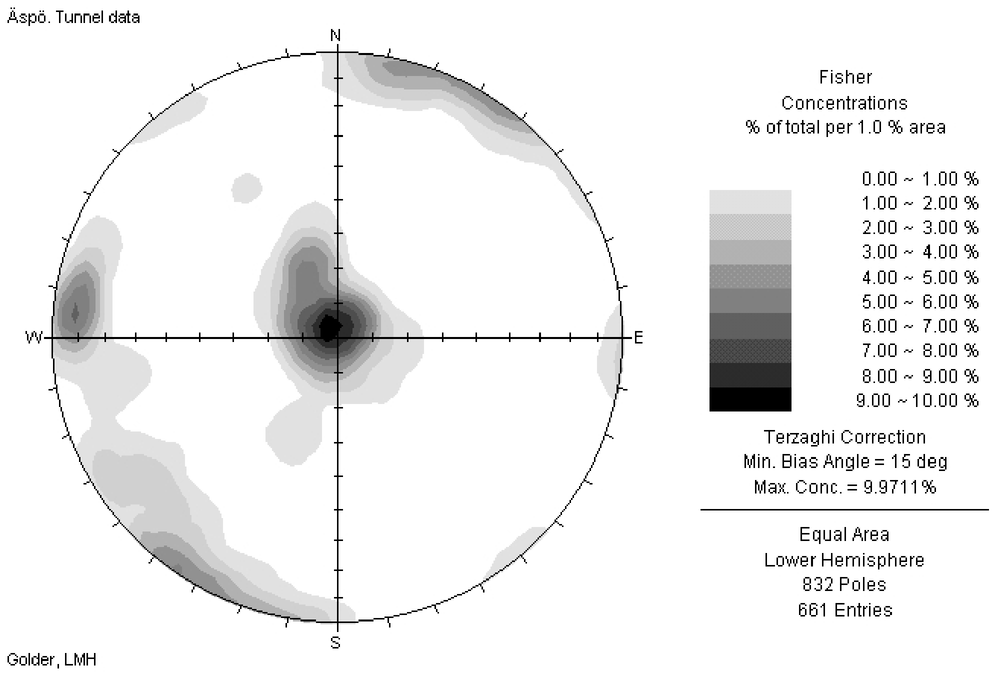

Figure 6. *Stereonet contour plot of joints in Tunnels TASA chainage 2625-2700, TASF, TASI, TASJ and the hoist shaft from elevation –350m to the floor of the shaft at –450m.*

4.1.2 Rock Stress

Christiansson & Janson (2002) have compiled the stress measurements in the area (boreholes KA2599G01 and KF0093A01). The result is presented in ***Figure*** *7* and *Figure 8*.

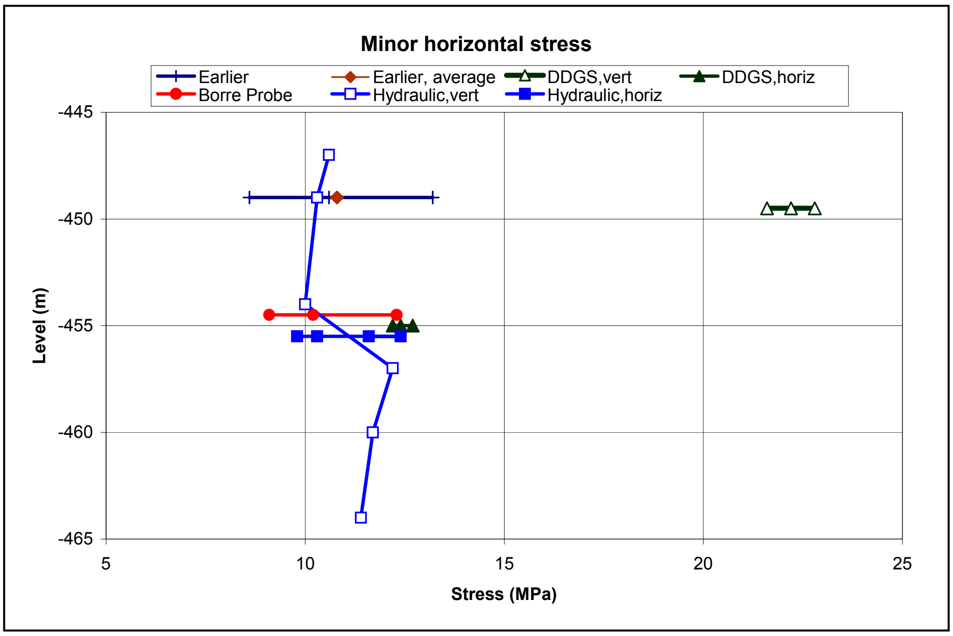

Figure 7. *The minor horizontal stress measurement results in the vicinity of the alternative tunnel locations.*

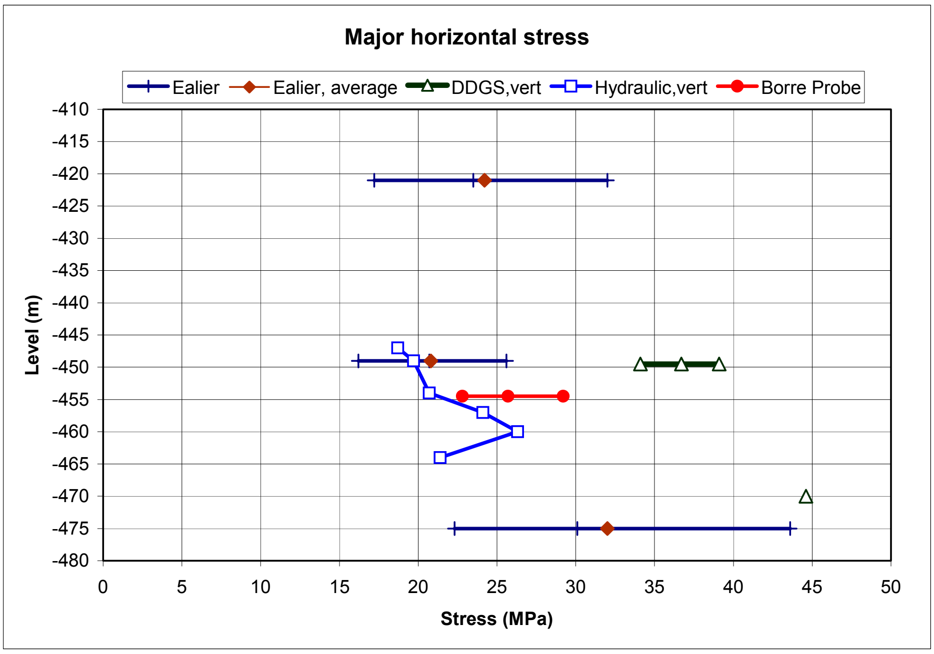

Figure 8. *The major horizontal stress measurement results in the vicinity of the alternative tunnel locations.*

The trend and plunge from the three dimensional rock stress measurements has been used to orient the stress field. Vertical rock stress measurements indicate that the minor horizontal stress is lower than the vertical stress. Measurements also indicates that the maximum principal stress is 27 +/- 2 MPa. This has been taken into account when the following resulting stress tensor has been derived and used for most of the numerical calculations in this report (Table 1).

Table 1. The stress tensor used for the calculations of the new tunnel.

	Magnitude, MPa	Trend, degrees N over E	Plunge, degrees below horizontal
Sigma 1	25	133	19
Sigma 2	12	49	42
Sigma 3	10	234	33

4.2 Pros and cons with the three alternatives

Advantages and disadvantages for the three experiment locations presented in *Figure 3* are listed in Table 2.

Table 2. Advantages and disadvantages for the use of alternative 1, 2 or 3 as an experiment location.

	Advantages	**Disadvantages**
Alternative 1	Extensive rock stress measurements made in the immediate vicinity	Close to the lowest sump. Action plan for eventual pump stop needed.
	Boreholes in the area indicates that there only are few water conducting features	Several other experiments in the area, risk for conflicts
	Gases from the blasting can easily be removed by use of the existing ventilation system	Difficult to transport the boring machine for the deposition holes to the site
	The geology and the rock mechanics in the area is well known from boreholes and tunnel mapping. Nothing implies that the geology in the area will cause any problems.	
Alternative 2 and 3	Gases from the blasting can be removed by use of the existing ventilation system with only small adjustments of it	No rock stress measurements in the immediate vicinity
	The experiment area is easily accessed from the elevator stop	The area is quite wet with a lot of water bearing features
	Transports to the area will not disturb work at the 450m level	No boreholes in the immediate vicinity except for the TBM-tunnel

The best site for the experiment can not be determined without site specific data from the different alternatives. It is mainly the disturbance on the hydraulic boundary conditions for other experiments that can’t be assessed without hydraulic tests in core boreholes drilled along the tunnel axis of the different alternatives.

4.3 Calculations of stress effects of existing tunnels

De layouts of the three alternatives are chosen so that they are perpendicular to the major principal stress, which is trending approximately N130E. Alternatives 1 and 3 are therefore trending N040E and alternative 2 N220E.

It is important that the stress field in the vicinity of the existing tunnels does not effect the stress field around the planned new tunnel. If there is an interference between the stress fields the modelling will be very difficult and it is not likely that good results will be obtained. The stress fields around alternatives 1 and 3 and the existing tunnels have therefore been investigated. Alternative 2 does not need to be modelled since it is located further away from the tunnels and shafts than alternative 3. Alternative three can therefore be regarded as a worst case scenario among the two alternatives.

Figure 9 presents the result from the stress analysis of the rock mass between alternative 1 in the F-tunnel (TASF) and the hoist and ventilation shafts.

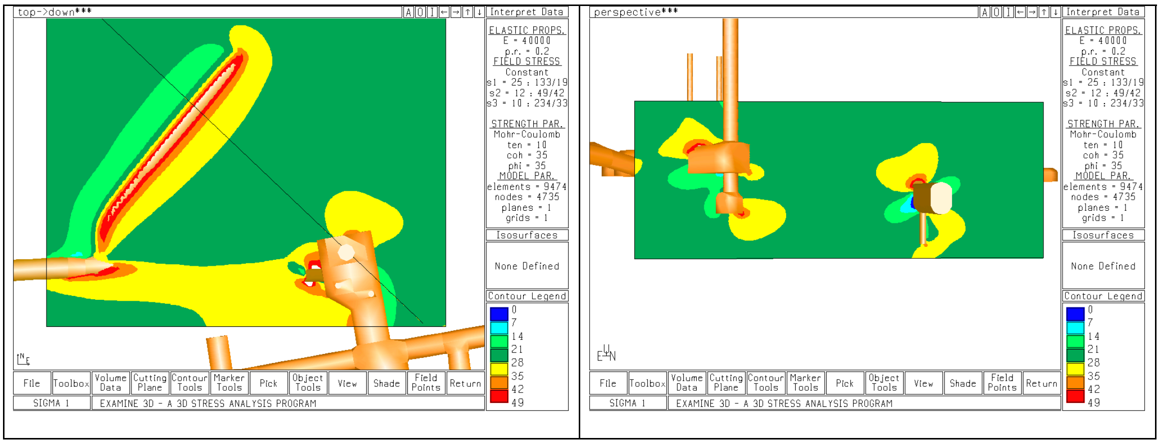

Figure 9. *Cutting planes presenting the stress field for alternative 1 in the rock mass between TASF and the hoist shaft. Note that the in situ stress of 25 MPa is within the dark green span. The right picture is a view perpendicular towards the diagonal line in the left picture.*

As can be seen in Figure 9 the stress situation in the rock mass between the tunnels is unaffected. It can also be concluded that the start of the new tunnel in the F-tunnel can be changed rather much without risking to cause interference effects between it and the existing tunnels.

The redistribution of stresses (from an excavation of alternative 1) around the existing TASF-tunnel will not create volumes with high stress concentrations or low confinement. It is therefore likely that very little extra support in TASF will be necessary because of a tunnel in alternative 1.

Figure 10 presents the result from the stress analysis of the rock mass between alternative 3 in the A-tunnel (TASA) and the hoist and ventilation shafts.

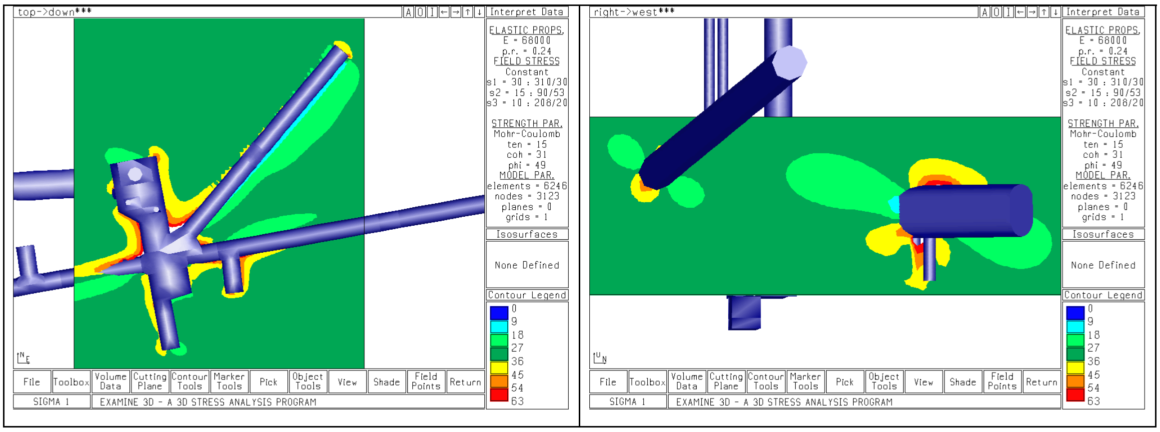

Figure 10. *Cutting planes presenting the stress field for alternative 3 in the rock mass between TASA and the hoist shaft. Note that the in situ stress of 30 MPa is within the dark green span.*

Also for this alternative the experiment area will not be located in a stress field that is disturbed by the existing tunnels. The disturbance from the existing geometry's will stretch approximately 20 m into the alternative 3 access tunnel.

The tunnel crossing where the alternative 3 tunnel starts will have a rather large span and some unloading of the roof will take place. The unloaded volume will though be rather small and no stability problems are expected.

Besides the effect from the stress field around the existing tunnels the new experiment tunnel has to be long enough to ensure that its end effects does not disturb the stress field in the area of the pillar. *Figure 11* presents the stress situation along the new tunnel. The stress pattern in the pillar looks different compared to the plots presented in Section 5 since a less dense grid was used when these calculations were made.

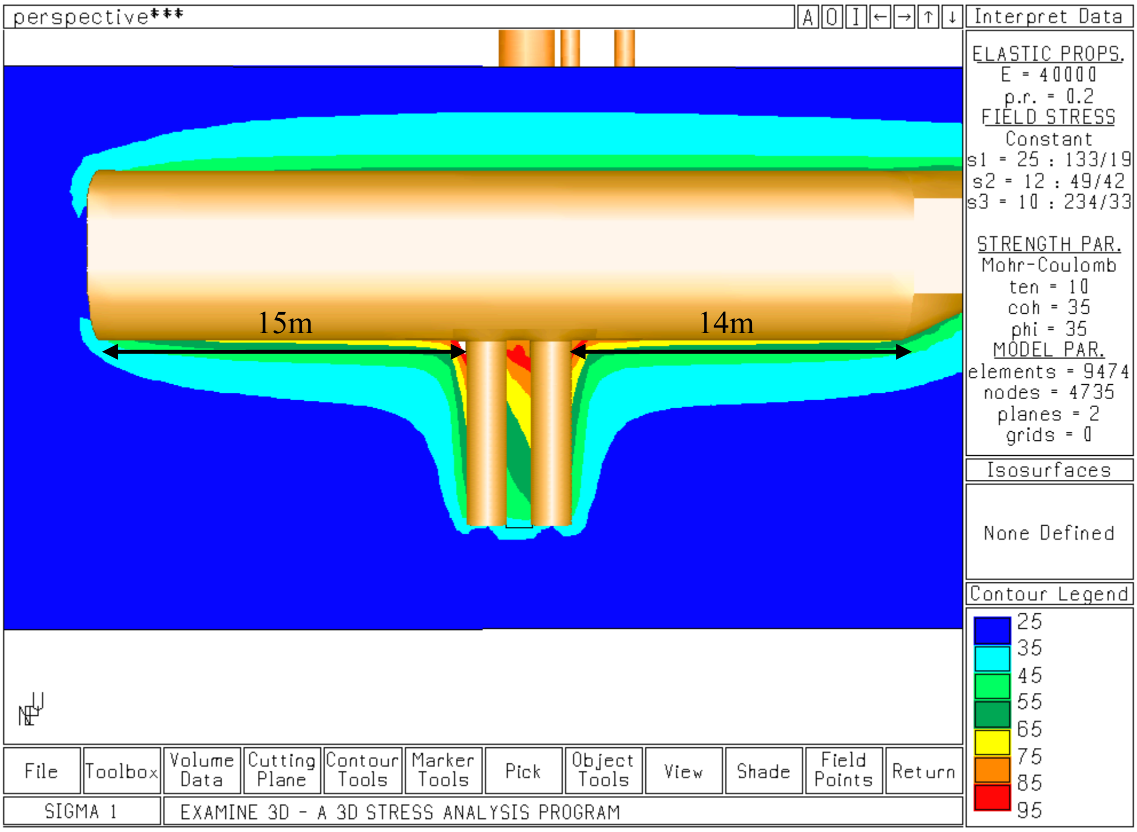

Figure 11. *The stress situation in a plane along the new tunnels alignment.*

The distance between the two holes and the ends of the part of the tunnel with the curved floor is approximately 15m. This distance can be reduced to approximately 10m and still avoid end effects from the tunnel. It is however recommended that the tunnel not should be optimised in length since a single discontinuity may make it necessary to move the pillar a few meters back or forth in the tunnel to avoid the discontinuity.

4.4 Other experiments at the 450 m level

There are several ongoing experiments in the area where the new tunnel is proposed to be located. The experiments in question are the:

1. Prototype Repository,
2. Long term test of buffer material,
3. Microbe,
4. Chemlab and the
5. Matrix fluid experiment.

It has to be investigated if the vibrations and water pressure peaks in the fractures originating from the blasting of the new tunnel might effect or damage these experiments. If this is the case the whole new tunnel might have to be excavated very carefully or the location of the new tunnel changed to an area further away from these experiments.

5 Preliminary design

This section will present a preliminary design of the experiment and describe the iterative process used to achieve an as favourable experiment geometry as possible.

5.1 Geometry and its effect on pillar stress

Since the far field stresses are fixed it is only possible to achieve higher stresses by diverting and concentrating the existing far field stresses. The air in a tunnel opening can’t take loads from the far field stresses, hence the load from them must be taken by the rock close to the opening. To illustrate this *Figure 12* presents the stress concentrations around a circular tunnel opening.

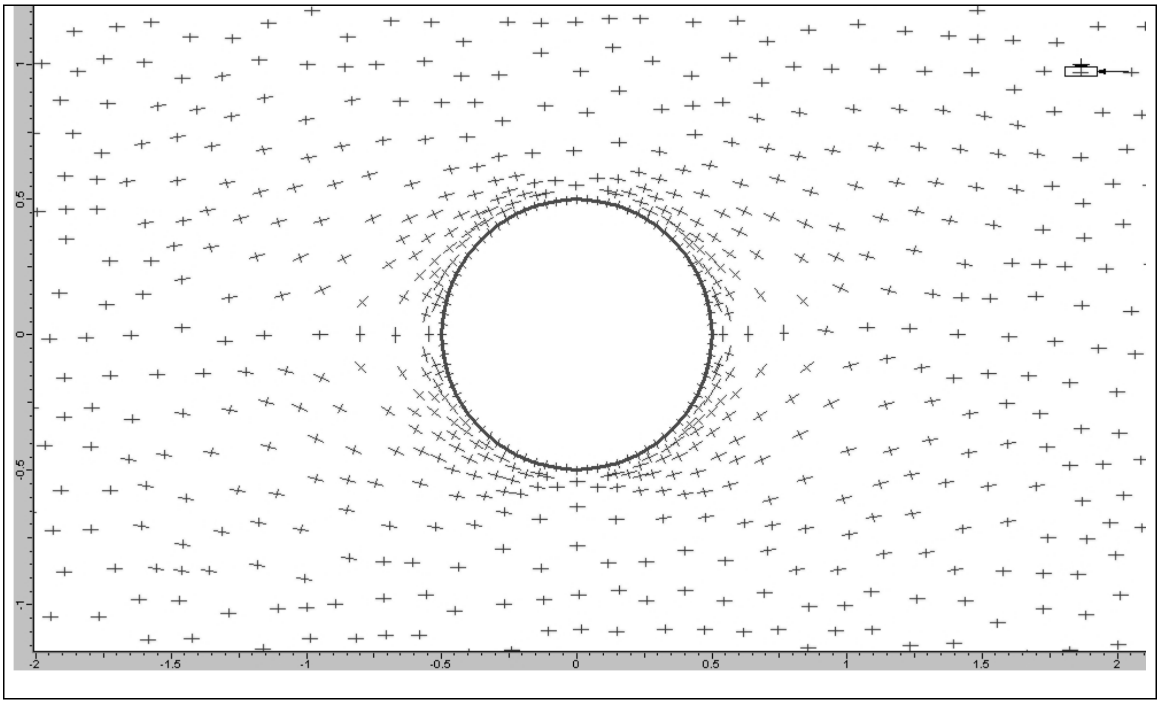

Figure 12. *The stress field around a circular tunnel opening. The direction of the major principal stress is horizontal.*

As can be seen on the stress trajectories they are diverted from the centre and concentrated around the top and floor of the tunnel. The same principle applies for all tunnel shapes in all stress fields.

To design the stress level in the pillar the four different variables listed below were used. Their respective influence on the stress situation in the pillar is described in the following sections

1. Tunnel size
2. Tunnel shape
3. Pillar width
4. Hole geometry

5.1.1 Tunnel size

Especially the height of the tunnel determines the stress at the top and floor of a tunnel. *Figure 13* presents the stress situation around two tunnel openings in the same stress field but with different height.

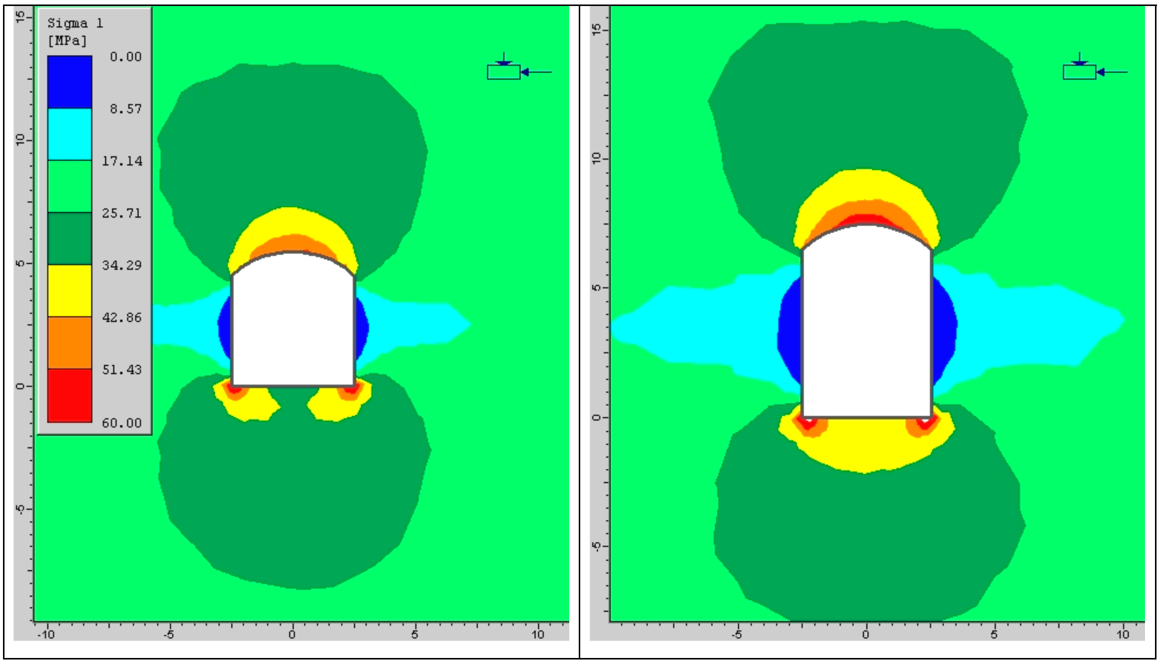

Figure 13. *Stress situation around a 5.5m high tunnel (left) and a 7.5m high tunnel (right). The magnitude of the far field stress is the same in the two pictures. The major principal stress is horizontal.*

As can be seen in the figure the two extra meters of height increases the stress levels in the floor and top of the tunnel significantly.

5.1.2 Tunnel shape

A conclusion that can be made from *Figure 13* is that the tunnel shape effects the stress distribution around an opening. The rounded top of the tunnel has concentrated the stresses to the highest point. In the tunnel's floor, the stresses are concentrated around the corners. It is favourable to have a similar concentration of the stresses in the tunnel floor as in the tunnel top of two reasons:

1) the stresses in the top of the pillar is increased

2) the stresses in the centre of the pillar will be more evenly distributed.

A comparison of the stress situation in a flat and curved tunnel floor is presented in *Figure 14*. The tunnel heights as well as the far field stresses are the same in both cases. Only the shape of the tunnel floor is different.

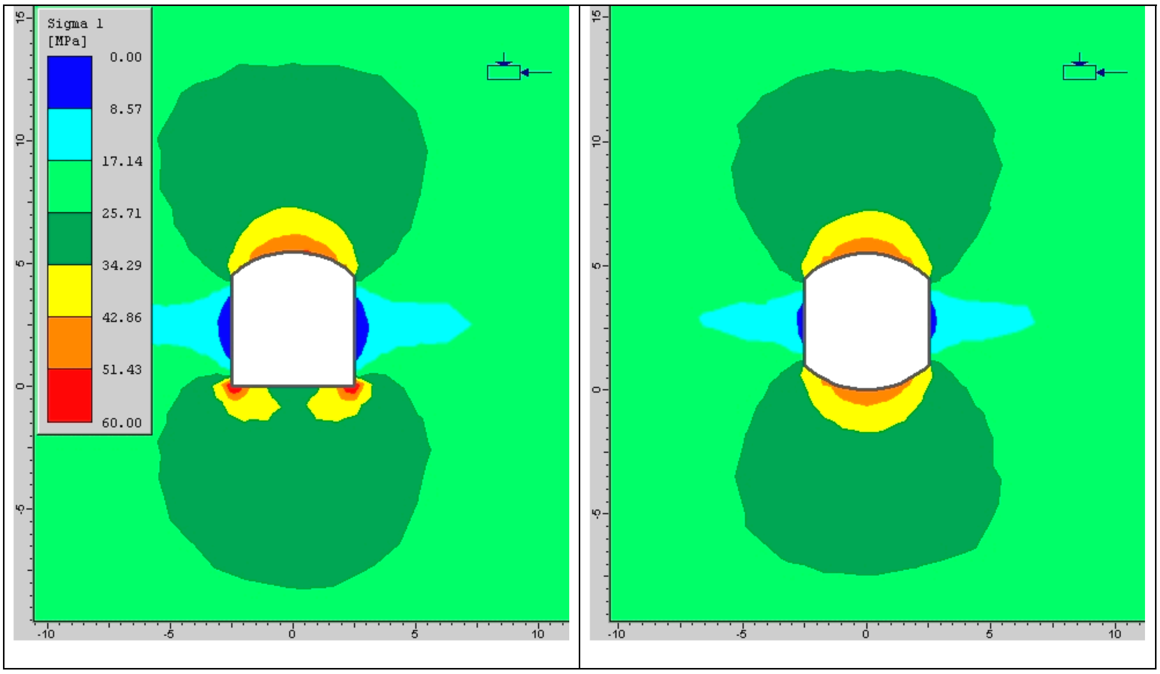

Figure 14. *Comparison of the stress situation in the floor of equally high tunnels but with different shape.*

The curved tunnel floor creates a favourable uniform stress situation and higher stress concentrations in the pillar. The curved floor ought to be excavated as a separate bench, after completion of a pilot tunnel, in order to minimise the excavation damage.

5.1.3 Pillar width

A study of the pillar width's influence on the stress situation in it is presented in *Figure 15*. It is concluded that the stresses increase dramatically when the width is decreased. From a practical point of view, the pillar width should not be much less than 1 m. If designed narrower, single discontinuities in the pillar might have large undesirable effects on the outcome of the experiment. It is therefore recommended that the pillar width is set to 1m.

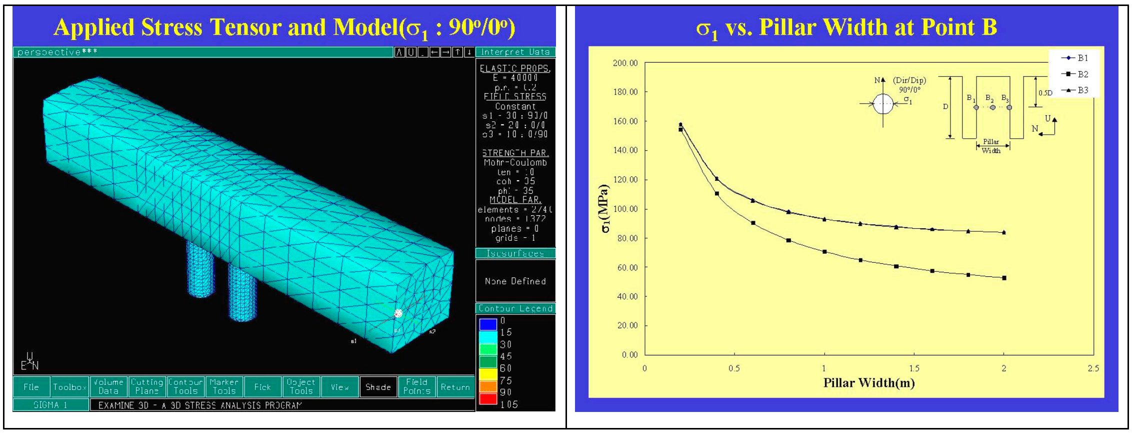

Figure 15. Stress in pillar versus pillar width approximately 1m down the hole.

5.1.4 Hole geometry

The holes that will be bored to create the pillar will of course effect the stress situation in it. Larger holes will, in analogy with the reasoning regarding the tunnel height, create higher stresses in the pillar for the same pillar width. A hole diameter of 1.8m is chosen as a reference diameter for the calculations since that is the size of the deposition holes in the future deep repository.

Figure 16 demonstrates that it is possible to control the stress situation in the pillar by drilling or sawing of slots in the hole walls, the keyhole principle. The length of the slots will determine the magnitude of the extra stresses that will be induced. In this case 0.9m deep slots have been modelled.

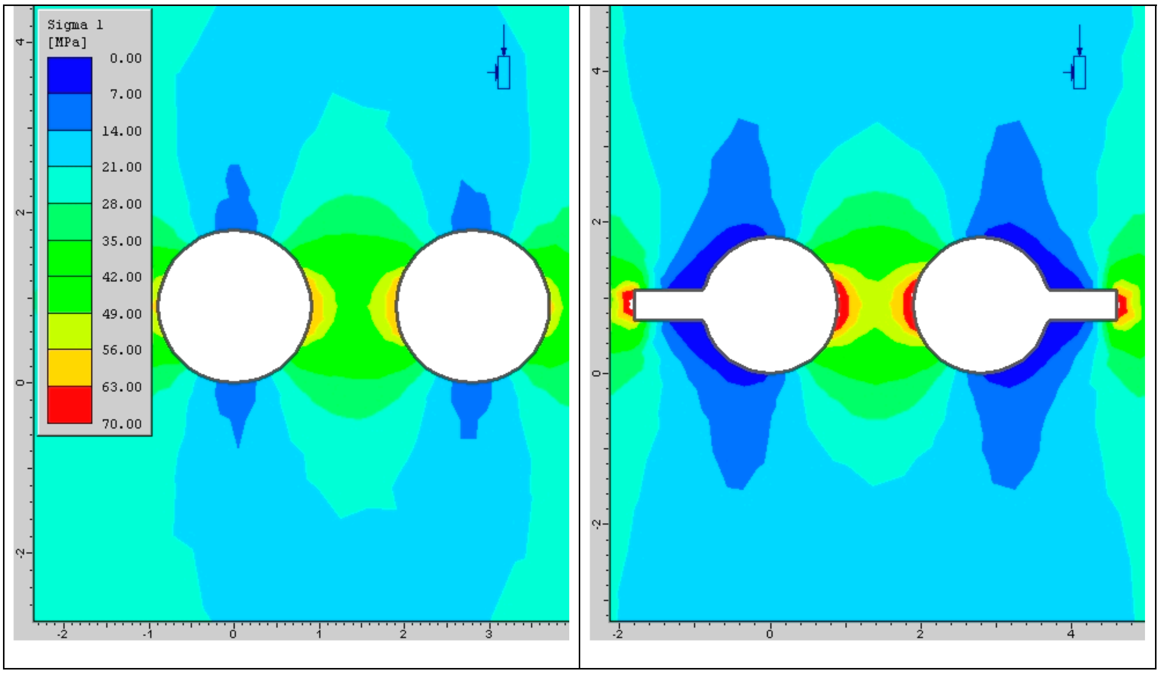

Figure 16. Comparison of the stress field in the pillar with and without the keyhole principle in the same stress field. The major principal stress direction is from top to bottom. The hole diameter is the same in the two figures.

5.2 Pillar and tunnel design

When evaluating the results from the different models the stress situation approximately 1m down the pillar is compared. The reason for this is that one can expect some excavation damage in the first 0.5m of the pillar. The objective with the modelling is that the stresses in the pillar ought to be approximately 120 MPa at 1m depth along the hole walls. It is assessed that the stresses then are high enough to ensure spalling in the pillar when it later is heated.

All the results presented in this section except those in *Figure 17* are calculated using the general geometry presented in *Figure 18*. Only the dimensions of the tunnel and holes are changed between the different realisations.

As a first reference case an approximately 5 by 5m tunnel with two 8m deep 1.8m diameter holes was modelled. The pillar between the holes was 1m. The stress on the hole walls will be approximately 100 MPa, Figure 17.

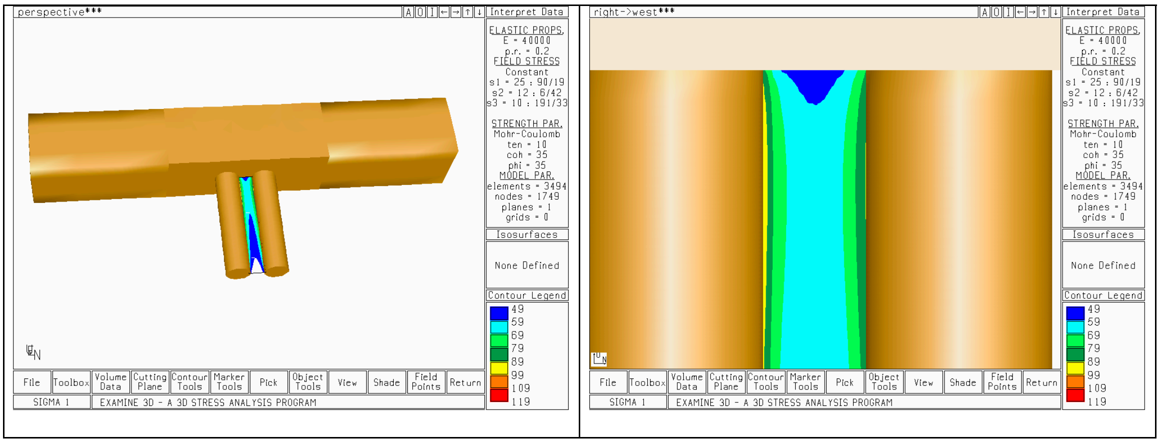

Figure 17. *Stress level in a 1m pillar between two 1.8m diameter boreholes.*

As mentioned above the stress level in the pillar prior to heating should be in the order of 120 MPa. To increase the stress in the pillar a new model was built. The tunnel height was increased to 7.5m and the floor curved according to the conclusions in Section 5.1.2. The result is presented in Figure 18.

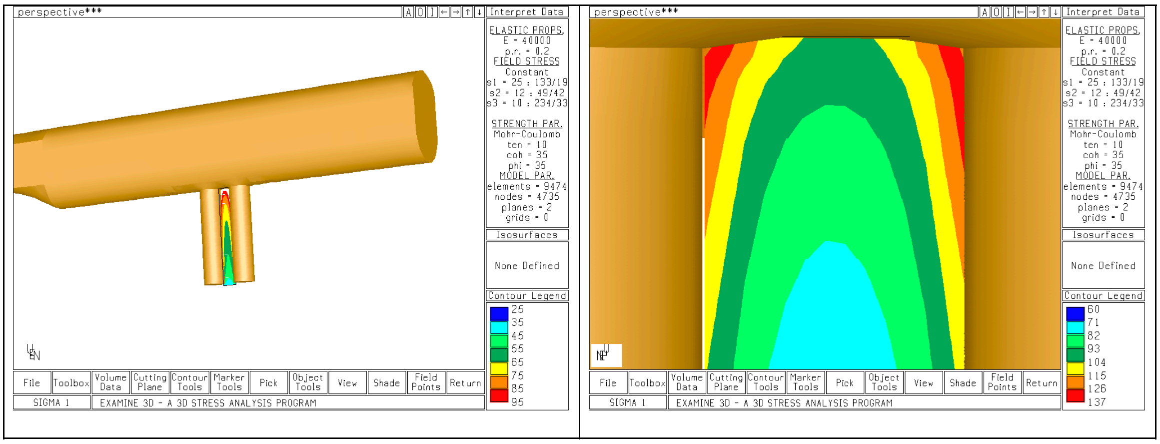

Figure 18. *Stress level in the pillar when the tunnel floor is curved and the height increased from 5 to 7.5m. The first 1.2 meters of the pillar shown in the right figure.*

As can be seen in Figure 18 the stresses in the first meter down the hole are significantly increased compared to the 5m tunnel (*Figure 17*). At 1m depth the stress is approximately 115 MPa on the hole wall. These stresses are however a little bit to low. A realisation was therefore made where the tunnel height was increased by 1m to 8.5m. A comparison of the stress levels in the pillar with 7.5 and 8.5m tunnel height is presented in Figure 19.

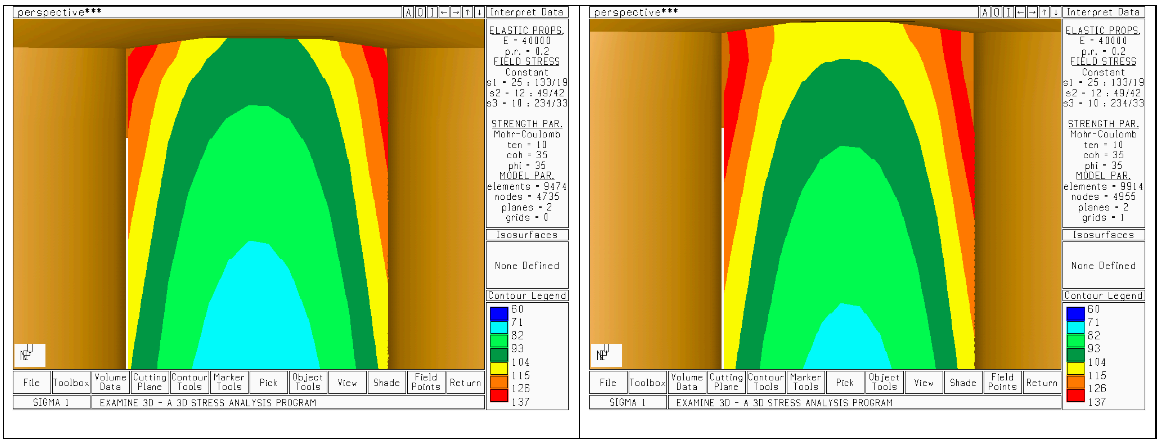

Figure 19. *The stress levels in the first 1.2m of the pillar when the tunnel height is 7.5m (left) and 8.5m (right).*

The one-meter higher tunnel increases the stresses in the pillar to a very moderate extent. The cost for excavating the extra rock volume to make such a small increase in stresses is too high to make this an alternative design. The stress levels achieved are very close too the ones needed but not quite high enough and alternative solutions to increase the stresses in the pillar will anyhow be needed.

A model according to the keyhole principle was therefore built and realised, Figure 20 and *Figure 21*.

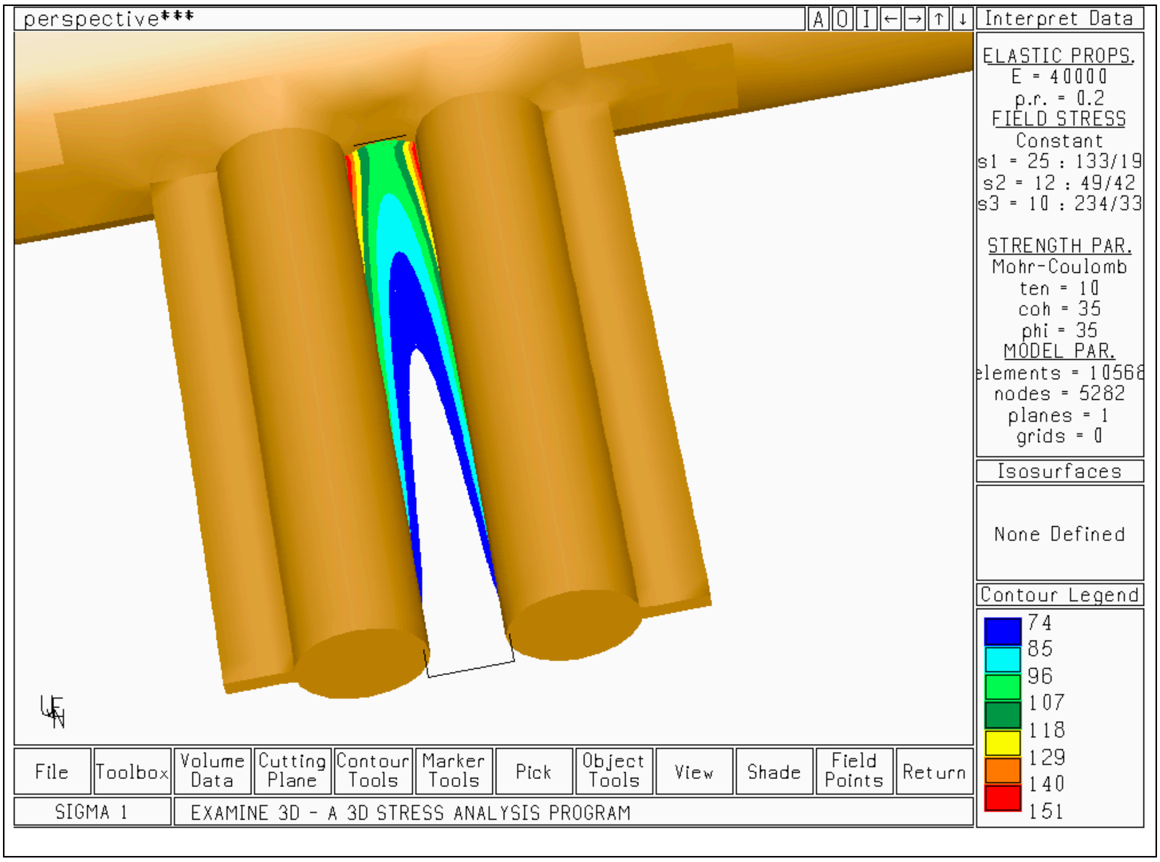

Figure 20. *A 0.9m deep slot in the 1.8m diameter boreholes and the stresses obtained.*

A comparison of the stresses in the pillar without and with a 0.9m deep slot is presented in Figure 21.

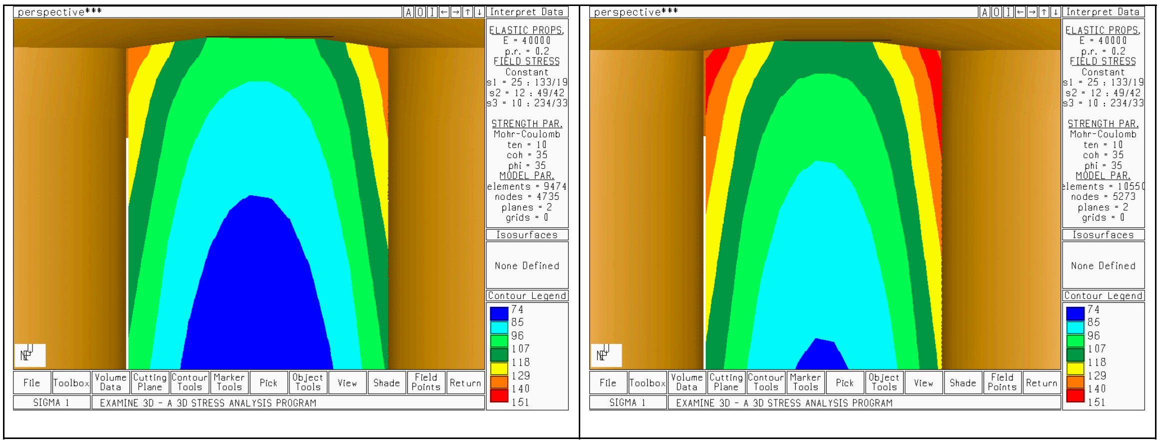

Figure 21. *The difference in stress levels without slot (left) and with a 0.9m deep slot (right). The first 1.2m of the pillar shown.*

As can be seen in Figure 21 there is a large increase in the stress levels when the slot is included. The stress levels are also high enough to create spalling, even without heating. This will however not be a problem since the stress levels in the pillar easily can be optimised by choosing the optimal depth of the slot.

During certain circumstances it might be troublesome to bore 1.8m diameter holes. Holes with a diameter of 1.0m are easier to bore and are here therefore the backup alternative. Scooping calculations have been made to see the stresses in the pillar when the hole diameter is 1.0m, Figure 22. The figure also includes the results from calculations where a 0.8m deep slot have been incorporated in the model with the 1.0m diameter holes.

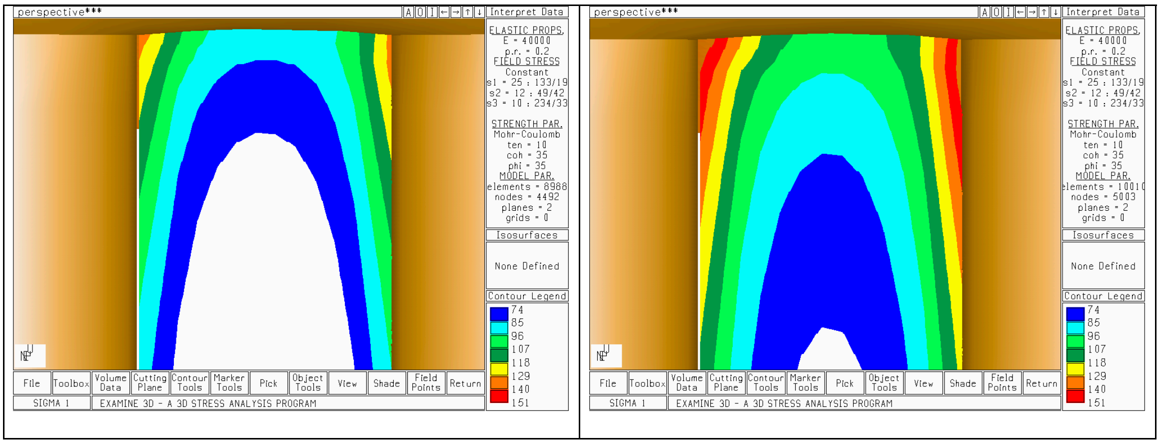

Figure 22. *Stress levels in the first 1.2m of the pillar when the hole diameter is 1m. Left, result without and right, result with a 0.8m deep keyhole slot.*

With the keyhole slot the stress level will be high enough for the experiment. The zone where the spalling will occur will though be narrower in a 1.0m hole than in a 1.8m hole. A 1.0m hole experiment will therefore be more difficult to monitor since less spalling will occur. A 1.8m diameter hole should therefore be the reference alternative.

5.3 Confinement

To apply an artificial backfill pressure of 1 MPa in one of the holes a watertight membrane will be used. The membrane will see too that no water seeps into the rock mass and as a consequence lowers the confinement pressure. The required water pressure in the membrane can be achieved by using Nitrogen gas to pressurise the water in the same way as in the packers at Äspö HRL.

5.4 Design recommendations

Based on the results from the numerical modelling presented these design recommendations are issued:

- The experiment tunnel should be oval in shape, 35m long and 5m wide by 7.5m high. Another approximately 20m length of tunnel is needed to get the start of the experiment tunnel away from the influence the existing tunnels on the stress field. This tunnel should be of standard dimension, 5m wide by 5.5m in height.

- The holes shall have depth ranging between 6 to 8m and a diameter of 1.8m, the pillar between the holes shall be 1m in width

- Slots, approximately 0.9m deep shall be drilled or sawn in the hole walls opposite the pillar

A section of the recommended tunnel shape is presented in *Figure 23*.

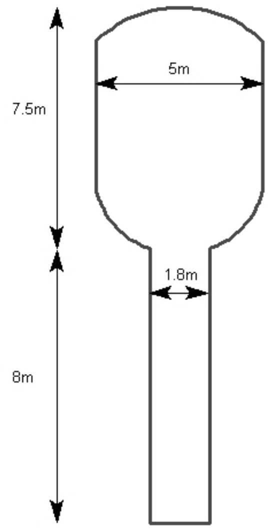

Figure 23. *Section of the recommended tunnel shape.*

6 Characterisation

The 450 m level is quite well defined from a geological and rock mechanical point of view. Further characterisation will though be needed. The current available and the needed additional information are summarised in this section.

6.1 Available information

In the TASJ tunnel there are two boreholes (KJ0050F01 and KJ0045F01) located close

to the experiment area. The borehole KF0093A01 approaches the experimental area from the TASF tunnel and the borehole KA2599G01 approaches the experiment area almost vertically from the 340m level. KAS02 reaches the experiment volume from the surface. The location of the above mentioned boreholes is presented in *Figure 24* and the characterisation made in them is presented in Table 3.

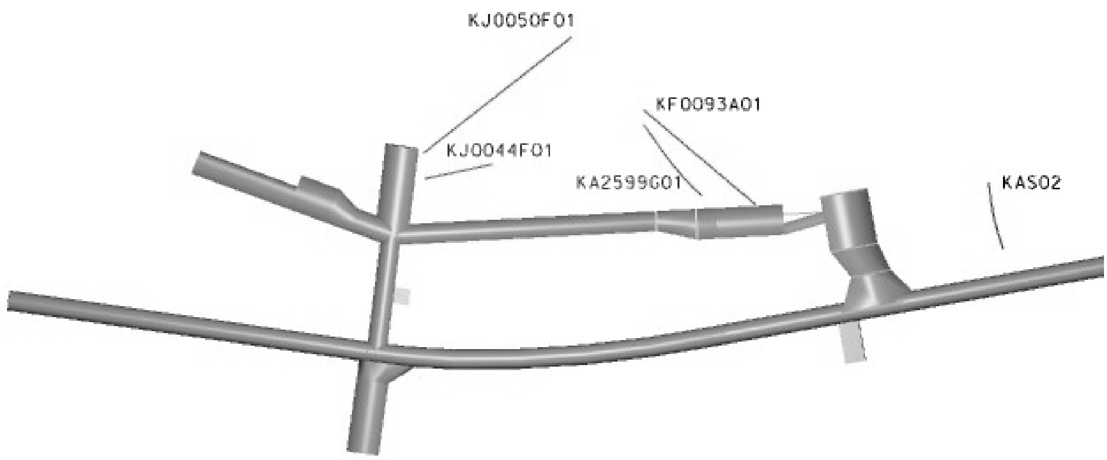

Figure 24. *The location of the five boreholes in the area of the experiment, including the location of the new tunnel.*

Table 3. The five boreholes in the area of the experiment and their characterisation.

Borehole	Type of characterisation made
KJ0050F01	BIPS- and core-logging
KJ0044F01	BIPS- and core-logging
KF0093A01	Biaxial strain, BIPS- and core-logging, Hydraulic fracturing, Overcoring in 2D and 3D, Seismic core log, Thin section,
KA2599G01	Biaxial strain, BIPS- and core-logging, Density, Hydraulic fracturing, Overcoring in 2D, Porosity, Pressure build up test, Seismic core log, Thermal properties, Thermal response, Thermal expansion, Thin section
KAS02	BIPS- and core-logging

There are also data from laboratory tests and other measurements and assessments of the rock's properties available. This data can be used for scooping numerical modelling but for a final prediction, data derived from samples taken in the immediate vicinity of the pillar is necessary.

6.2 Needed information

More site specific information is needed to be able to produce good predictions for the experiment. The area will therefore be further characterised in different steps depending on how far the project has progressed. A summary of the different disciplines and the additional and complementary parameters they will need for the predictions is summarised in Table 4.

Table 4. Compilation of disciplines and the parameters needed for the predictions.

Discipline	Parameter
Rock mechanics	In situ stress
	Uniaxial strength
	Young's modulus
	Poisson's ratio
	Core density
	Density log in borehole
	Thermal expansion
	Thermal conductivity
Geology	Core mapping
	BIPS (Televiewer)
	Tunnel mapping
Hydrogeology	Inflow measurements in borehole
	Pressure build up tests in borehole

6.3 General characterisation program

To characterise the rock mass in the proposed areas for the experiment core boreholes should be drilled in an early stage. The boreholes should be aligned with the centre of the planned tunnel alternatives a few meters above the floor. The location of the alternatives is presented in a top view in Figure 25.

Most of the parameters listed in Table 4 will be determined in these boreholes. Parameters that needs be determined in a laboratory will be measured on core samples taken from the core at the chosen alternative.

The density log is necessary since different rock types have significant differences in their coefficient of thermal conductivity. If the density log indicate a large density variability close the proposed experimental area it may be inappropriate to site the experiment there. This is especially true if the coefficient of the thermal expansion follows the same pattern as the thermal conductivity.

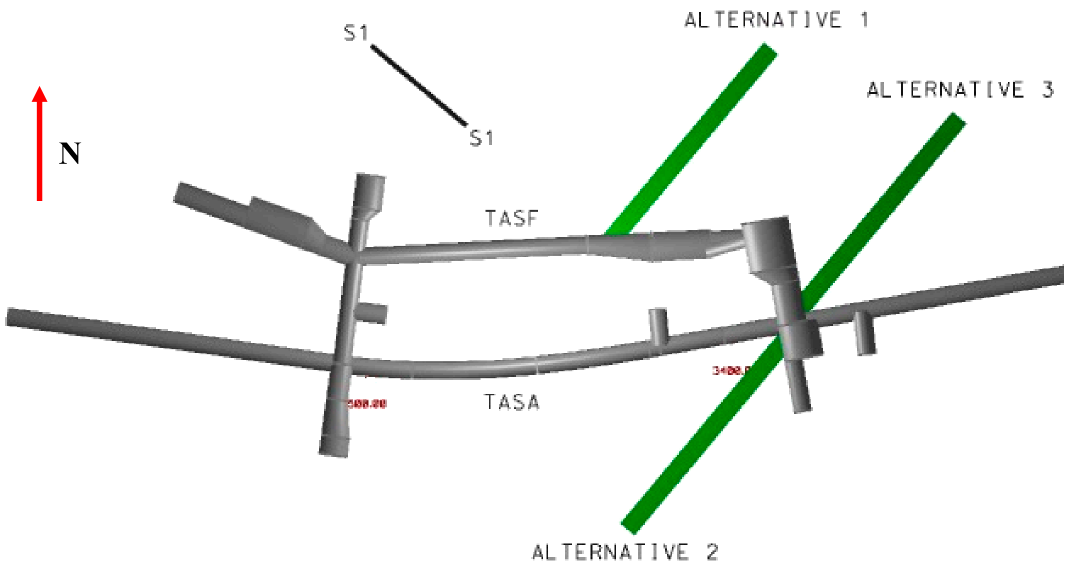

Figure 25. *Green lines indicating the proposed alternatives of the tunnel. The core boreholes should be located within the green lines.*

After a complete characterisation of the boreholes no further characterisation will be made until the tunnelling is completed. Then, a detailed geological characterisation with a shortest fracture trace length of 0.5m will performed. The tunnel floor in the proposed experiment area will be mapped in even more detail. The exact location of the pillar will be determined with the fracture mapping used as a base. When the pillar location is determined additional core drilling will be performed in the periphery of the future holes close to the pillar wall. Core drilling will also be made for the instrumentation and the heaters. Samples from these cores will be used to determine the rock mechanical (except in situ stress and density log) and the geology parameters.

7 Instrumentation and heaters

To assess the outcome of the experiment three types of instrumentation is needed. The most important of these is the acoustic emission (AE) system. AE is the most important system since the stresses in the pillar will most likely not be large enough to create fallouts in the borehole. The occurrence of spalling will instead be detected by an increased event rate in the AE log. AE will also be used to localise the events in the rock mass and to measure the seismic velocity. The seismic velocity measured close to the boreholes walls will indicate the depth of the damaged zone.

The second type of instrumentation needed is a number of thermocouple arrays. There will be two arrays in each hole attached to the walls and a few arrays in boreholes close to the pillar.

The third type of instrumentation is displacement/convergence measurements of the tunnel during excavating and in the open hole during heating.

In addition to the instruments, heaters will be needed. Based on experiences from AECL's heated failure experiment it is preliminary assessed that four heaters with an effect of 3000kW will be adequate. The temperature in the heater holes might reach as high as 500 degrees but the temperature at the hole walls will be limited to 90 degrees. A draft set up of the instruments geometry is presented in Figure 26 and a more detailed, but still a draft set up, of the acoustic emission system is presented in *Figure 27*, Young (2002). In *Figure 27* a reference hole depth of 8m is used. The set up of the instruments would change very little if the hole depth is reduced to 6m.

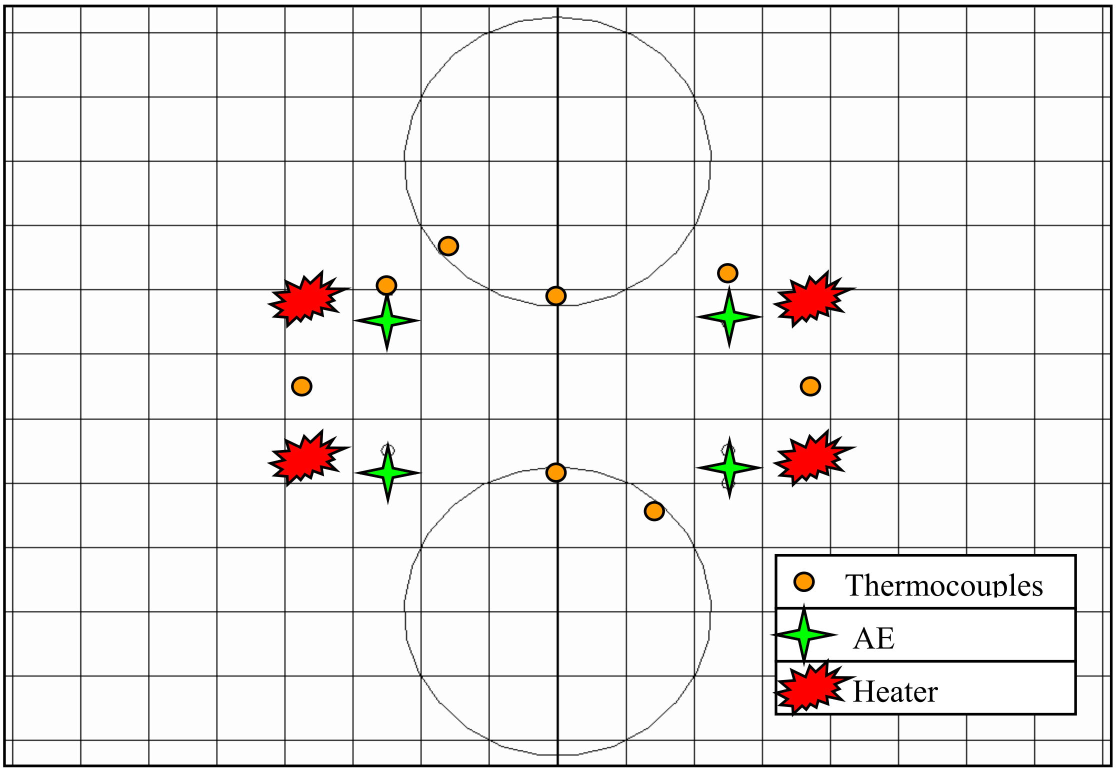

Figure 26. *Draft set up of the instrument geometry. The grid spacing is 0.4m.*

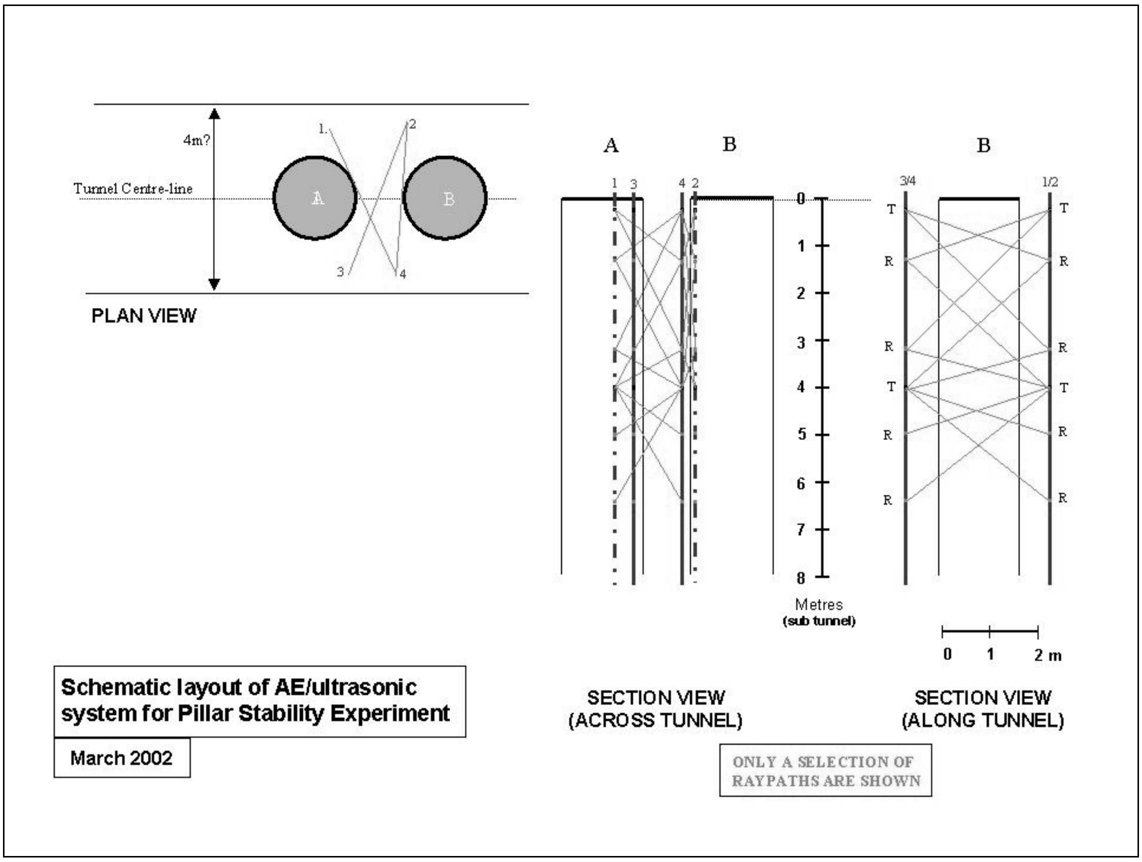

Figure 27. *Draft diagram of the set up of the acoustic emission system.*

The array consists of transmitters and receivers installed in 4 boreholes around the pillar. The position of boreholes 1, 2 and 4 is chosen to given raypaths that pass tangentially to the deposition hole walls. Borehole 3 is positioned so that raypaths passing through the central part of the pillar can also be investigated. The sensors in borehole 3 also provide good coverage for the detection and location of AE events occurring within the pillar.

Each borehole would contain 2 transmitters and 4 receivers. These are used in the same way as the system installed for similar measurements in the Retrieval and Prototype Tunnels. In this case the arrangement of transmitters and receivers within the borehole is a little different to the previous cases. The transmitters are positioned so that there is one close to the borehole collar and one about half-way down the borehole. This provides improved ray-coverage over the upper part of the pillar/deposition hole walls, which will allow us to image and investigate the spalling process in more detail. The lower 2 metres of the deposition hole is less well imaged.

8 Modelling and visualisation

Different tools will be used to predict the outcome of the experiment and to evaluate the results. This section will discuss the planned modelling and visualisation stages of the project.

8.1 Modelling

Several different types of modelling tools will be used. The modelling stages will in short be the following.

1. The geometry of the tunnels and holes will be designed with respect to the stress situation in the pillar and the disturbance from the stress field in the existing tunnels.
2. The information from the first core hole will be used for a thermal model that will be used for the design of the placement of the heaters and thermocouple arrays in relation to the large bore holes.
3. When the geometry is preliminary set a Microstation model will be created. It will cover the new tunnel and the location of the different holes for instrumentation.
4. After completion of the tunnelling the geological mapping will be used for creating a Discrete Fracture Network-, DFN-model. This work will then be used to determine the exact location of the experiment.
5. The final numerical modelling of the rock mass response to heating will be made after the results from the lab tests on the instrumentation and heater hole cores are finished.

The results from these modelling stages will be reported before the experiment starts. The report shall predict at what temperature/stress level the spalling will occur and how far into the rock the spalling will go. The vertical extent of the spalling into the hole shall also be predicted.

8.2 Visualisation

A 3D visualisation tool will be needed to present the results from the predictive modelling and the information from the monitoring. It is necessary to visualise the results in 3D to get a good understanding of the AE and displacement distribution both in space and time.

The information that will be used to assess the outcome of the experiment is listed below.

1. Geology
2. Fracture mapping/DFN-model
3. Geometry
4. Acoustic Emission
5. Seismic velocity
6. Stress contours
7. Convergence/displacement

9 Realisation

This section will pinpoint most of the important practical steps in the realisation of the APSE. The tunnelling is further described in Section 9.1.

1. Drilling of core hole in the location of the bench in the new tunnels proposed alignment including characterisation
2. Tunnelling including:
   - Pre-grouting
   - Excavation of pilot tunnel
   - Excavation of bench
3. Geological mapping of the new tunnel
4. Casting of concrete in the area of the holes and filling up the rest of the tunnel with gravel
5. Drilling of instrumentation and heating holes, including characterisation
6. Installation of instrumentation
7. Drilling of slot by the large holes to be bored
8. Boring of the first hole
9. Installation of confinement equipment in the first hole and test of it. Setting of confinement pressure to 1 MPa
10. Drilling of the second hole. Rock mass response monitored by AE
11. If the AE event rate is too low when the second hole is bored and it is assessed that the stresses in the pillar are to low the slots can be extended to increase the stress. Numerical calculations together with AE and extension of the slots will make it possible to get precisely the stress level desired.
12. Controlled heating of the pillar until failure in the wall of the unconfined hole. The temperature on the hole wall will be heated to approx. 90 degrees. A steady thermal state will then be held until the rate of the AE events drops significantly.
13. Lowering of the confinement pressure in the first hole to a level determined by the predictive modelling. If the AE event rate does not increase the pressure will be lowered until spalling occur.
14. Decommissioning of the experiment

9.1 Tunnelling

The tunnelling work will be performed through the three activities: pre-grouting, excavation of pilot tunnel and excavation of the bench. The three activities are briefly described in the sections below.

9.1.1 Pre-grouting

According to the present characterisation data for the proposed area of the new tunnel, amounts of water seepage unlikely. For precaution, there will though be exploratory percussion drilling in front of the tunnel. Excluding the core borehole drilled for characterisation another six percussion-drilled holes is needed. The first three holes will be drilled from the start of the new tunnel and they will be approximately 35m long respectively. The last three holes will be drilled when the tunnel has advanced 25m. Also these three last holes will be 35m long respectively. The approximate location and extension of the six holes is presented in *Figure 28*.

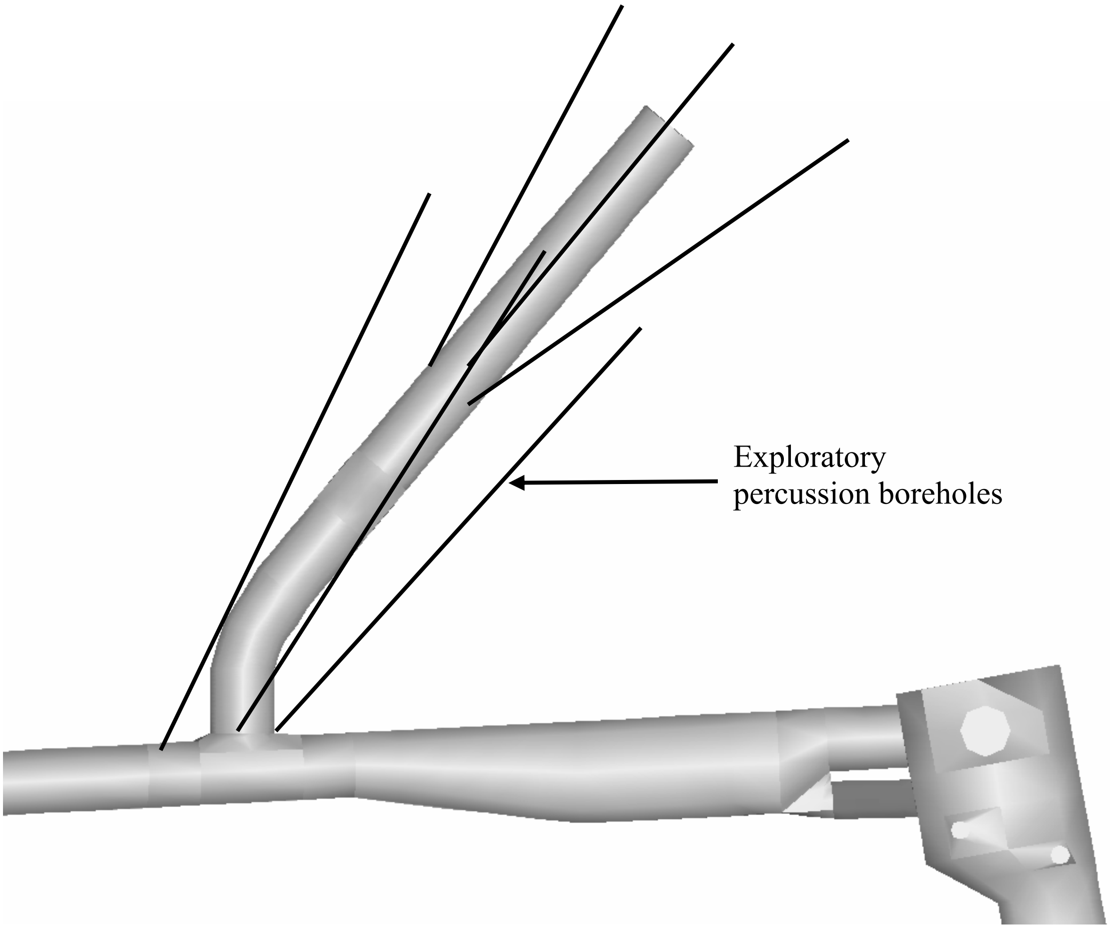

Figure 28. *Approximate location and extension of the exploratory percussion boreholes. Alternative 1 taken as an example.*

If the seepage in one or several of the boreholes is assessed to create problems in the new tunnel, pre-grouting will be performed. If it is decided to pre-grout a few more holes is needed to achieve a good grouting result. The location of these holes will be determined on site. When the pre-grouting has been performed one or two boreholes will be drilled along the tunnel to see if the desired effect was achieved. The location of this/these borehole/s will also be decided on site.

9.2 Excavation of pilot tunnel and bench

Approximately 60m of new tunnel will have to be built. Of these 60m the last 35m shall have a height of 7.5m and a carefully blasted curved floor. To make the tunnelling as cheap as possible it is proposed that the tunnelling is performed in two stages. First a standard 5 by 5.5m-pilot tunnel is excavated through standard drilling and blasting techniques. Then, a 2m deep curved floor is excavated very carefully to minimise the damage in the new floor. *Figure 29* presents this staged excavation.

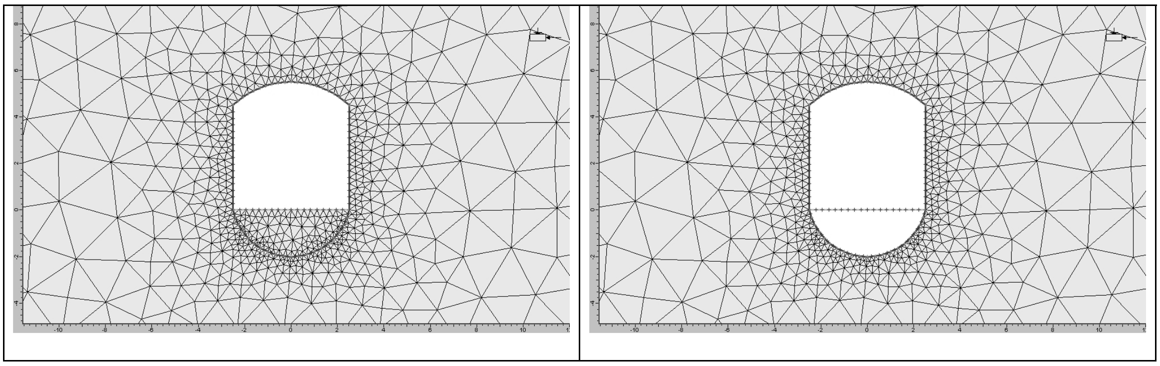

Figure 29. *Finite element model of the staged excavation showing the pilot/standard tunnel to the left and the full contour of the experiment part of the tunnel to the right.*

10 Conclusions

The results from scooping calculations made with the boundary element code Examine3D and the finite element code Phase2 leads to the following conclusions:

1. The new tunnel for the experiment can be placed in the existing facility at the 450 m level. The stress field around the current tunnel system will not effect the experiment area of the new tunnel in any of the three alternatives.

2. It is possible to create high stresses enough in the pillar before heating to ensure that spalling will take place. To achieve this, no extreme geometry is needed and standard drilling, cutting and rock excavation equipment can be used.

3. The following design of the experiment tunnel and holes is proposed:

   - The experiment tunnel in the vicinity of the pillar should be oval in shape, 35m long and 5m wide by 7.5m high. The experiment volume needs to be located in a stress field that isn't disturbed by the existing tunnels. The access tunnel to the experiment part of the tunnel should be of standard dimension, 5m wide by 5.5m in height. The total tunnelling length for the experiment is assessed to be approximately 60 – 70 meters.

   - The holes shall have depth 6 to 8m and a diameter of 1.8m, the pillar between the holes shall be 1m in width

   - Slots, approximately 0.9m deep will probably be drilled or sawn in the hole walls opposite the pillar

4. Acoustic emission and convergence measurements are important tools to confirm the spalling tendency

There is nothing in the numerical calculations that indicate that the experiment would fail because spalling isn't achieved when heating is taking place.

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