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Test with different stress measurement methods in two orthogonal bore holes in Äspö HRL

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Summary

Within the scope of work, to provide the necessary rock mechanics support for the site investigations, the Swedish Nuclear Fuel and Waste Management CO (SKB) has studied some available pieces of equipment for in situ stress measurements in deep boreholes. A project with the objective to compare three different pieces of equipment for in situ stress measurements under similar conditions has been carried out.

The main objective for the project is to compare the three different pieces of equipment for in situ stress measurements and find a strategy for SKB's Site Investigations to determine the state of stress in the rock mass. Two units of equipment use the overcoring method while the third uses the hydraulic fracturing method.

The overcoring was performed by AECL, using Deep Doorstopper Gauge System (DDGS), and SwedPower, using their triaxial strain measuring instrument (Borre Probe). MeSy Geo Systeme GmbH performed the hydraulic fracturing. The DDGS system is a new method to SKB while the experience of the SwedPower overcoring and the hydraulic fracturing methods are long.

The tests were performed in the same orthogonal boreholes at Äspö Hard Rock Laboratory (HRL), Oskarshamn, Sweden. The measured results have been verified against known conditions at the Äspö HRL.

The stress measurement results are summarised in Table I, for the minimum horizontal, the maximum horizontal and the vertical stresses.

Table I. Summary of the results from the measuring of the stresses in the vertical and horizontal borehole, at level –450 m.

Method	Minimum horizontal stress, MPa	Maximum horizontal stress, MPa	Vertical stress, MPa
DDGS, vertical hole	22.2 ± 0.6	36.7 ± 2.6	–
HF, vertical hole	11.0 ± 1.2	21.8 ± 4.5	–
DDGS, horizontal hole	12.4 ± 0.3	–	32.6 ± 5.6
Borre Probe	10.2 ± 2.1	25.8 ± 3.5	18.0 ± 8.8
HF, horizontal hole	11.0 ± 1.4	–	19.8 ± 1.6
Theoretical	–	–	12.2
Former measurements	10.5 ± 3.0	21.0 ± 5.0	15.0 ± 4.5

The results from the three in situ stress measurement methods rose more questions than answers. Which illustrate the complexity to determine the in situ stresses in a rock mass. To understand the difference in results and answer the questions, it was necessary to do deeper investigations such as laboratory tests and theoretical calculations such as

- geological structure model,
- analysis of the influence of a nearby fracture,
- P-wave measurements,
- uniaxial tests on small cores from the HQ-3 core,
- theoretical and numerical analyses of the hole bottom (theoretical strains, stress concentrations and microcracking),
- auditing of DDGS measurements results and assumptions in the DDGS analyse and
- microscopy investigations on the cores.

The following conclusions have been drawn, based on the stress measurements and deeper investigations:

- The following stress state is obtained at the target volume at about –455 m. The minimum horizontal stress is between 10 and 13 MPa, which is lower than the theoretical vertical stress. The maximum horizontal stress is 24 ±5 MPa, most likely within the upper range. The vertical stress is between 15 and 20 MPa, most probably is this value only local due to the presence of a nearby fracture.
- The local disturbance of the stress field in the rock mass, due to discontinuities has been demonstrated. This also indicates one of the problems with stress measurements in boreholes.
- In the area with significant anisotropic stress conditions all the tested methods were able to determine the orientation of the principal major horizontal stress within ± 10°.
- The microscopy investigations confirm two sets of microcracks in the overcored core. One set was parallel and near the bore hole bottom and one set was perpendicular to the bottom and located a bit away from the hole bottom.
- The results from the overcoring may be influenced by microcracks, causing additional non-elastic strains, see /Martin and Christiansson, 1991/. Only the results from the DDGS seem to have been influenced, indicating that the hollow cylinder of a 3D stress cell may be less sensitive for stress induced sample disturbance than core samples from the 2D Doorstopper cell.
- The determination of Young's modulus in a medium grained crystalline rock with heterogeneity may not be trivial using core samples. The results from the determination influence the calculated stresses.

- Hydraulic fracturing most likely measures the most correct value of the minimum horizontal stress, provided that the induced fracture is aligned with the borehole.

- If the rock behaves reasonable elastic the overcoring methods provide stress magnitudes with an uncertainty of 15–20%. It seems likely that the overcoring methods may overestimate the stress magnitudes at large depth, due to the influence of microcracks.

- A good understanding of the geology in the scale from mineralogical heterogeneity to possible discontinuities is important for the judgement of the reliability of the results of the stress measurement. The difficulty in understanding all geological variabilities in the vicinity of a borehole must however add a general uncertainty to the stress measurement results.

The conclusions have direct impact on the stress measuring program that will be carried out in the planned SKB Site Investigation program for the deep repository for spent fuel.

Sammanfattning

En av de bergmekaniska huvudparametrarna vid Svensk Kärnbränslehantering AB:s (SKB) platsundersökningarna (PLU) är att bestämma det ursprungliga (*in-situ*) spänningstillståndet i bergmassan. Spänningstillståndet, tillsammans med bergets hållfasthetsegenskaper avgör om det förekommer risk för smällberg på lagringsnivån, ett förhållande som inte kan accepteras om dessa förhållanden är allmänt utbredda.

SKB har i ett projekt studerat lämplig mätningsutrustning och metodik för att bestämma bergmassans *in-situ* spänningar i djupa borrhål. De två vanligaste principerna för direkt bergspänningsmätning är överborrning och hydraulisk spräckning. Två av de testade metoderna tillhörde överborrningsprincipen och en av dem hydraulisk spräckning. Den ena överborrningen var en trådtöjningsgivare limmad på borrhålsbotten, en 2D metod kallad Deep Doorstopper Gauge System (DDGS) och utförd av AECL (Atomic Energy of Cananada Limit) från Kanada. Den andra överborrning var med en triaxial töjningsgivare limmad i ett pilothål, en 3D metod kallad Borre Probe och utförd av SwedPower AB från Sverige. Den hydrauliska spräckningen genomfördes av MeSy Geo Systeme GmbH från Tyskland. SKB:s erfarenhet av spänningsmätning med Borre Probe och hydraulisk spräckning är lång medan spänningsmätning med DDGS är ny för SKB.

Huvudsyftet med projektet är att jämföra de tre mätmetoderna med varandra under lika förhållande och föreslå en strategi inför SKB:s bergspänningsmätningar vid platsundersökningarna.

Mätningarna har utförts i två ortogonala borrhål, ett vertikalt och ett horisontellt, i Äspö Hard Rock Laboratory (HRL), Oskarshamn. Resultaten från mätningarna har vidare jämförts med tidigare gjorda spänningsmätningar i Äspö HRL.

Spänningsresultaten från projektet är sammanställt i tabellen nedan, de redovisade spänningarna är minsta horisontalspänningen, största horisontalspänningen och vertikalspänningen.

Sammanställning av uppmätta spänningar, minsta och största horisontalspänningen samt vertikal spänningen, vid nivå –450 m.

Metod och hål	Minsta horisontal spänningen, MPa	Största horisontal spänningen, MPa	Vertikal spänningen, MPa
DDGS, vertikalt hål	22.2 ± 0.6	36.7 ± 2.6	–
HF, vertikalt hål	11.0 ± 1.2	21.8 ± 4.5	–
DDGS, horisontellt hål	12.4 ± 0.3	–	32.6 ± 5.6
Borre Probe	10.2 ± 2.1	25.8 ± 3.5	18.0 ± 8.8
HF, horisontellt hål	11.0 ± 1.4	–	19.8 ± 1.6
Teoretisk	–	–	12.2
Tidigare mätningar	10.5 ± 3.0	21.0 ± 5.0	15.0 ± 4.5

Analysen av resultaten från de tre mätmetoderna gav upphov till fler frågor än svar, vilket illustrerar komplexabiliteten att bestämma *in-situ* spänningen i en bergmassa. För att bättre förstå skillnaden mellan resultaten från mätmetoderna och få svar på frågeställningarna har det varit nödvändigt att göra fördjupade undersökningar. De fördjupade undersökningarna har varit följande:

- upprättande av en struktur geologisk modell,
- inverkan av närliggande sprickplan,
- P-vågsmätningar genom borrkärnor,
- enaxliga tryckförsök på mindre kärnor, utborrade från borrhålskärnan,
- teoretisk och numerisk spänningsanalys av borrhålsbotten (teoriska töjningar, spänningskoncentrationer och mikrosprickor),
- tunnslipsanalys av överborrade kärnor,
- utomstående granskning av resultaten från spänningsmätningarna, speciellt DDGS testen,

Baserat på spänningsmätningarna och de fördjupade undersökningarna har följande slutsatser gjorts:

- Spänningstillståndet i den undersökta bergmassan, på nivå –455 m, har uppskattats till mellan 10 till 13 MPa för minsta horisontal spänningen, vilket är lägre än den teoretiska vertikal spänningen. Största horisontalspänningen är 24 ±5 MPa, med störst sannolikhet i det högre området. Den vertikal spänningen är mellan 15 till 20 MPa. Troligtvis beror de höga värdena, jämfört med tidigare mätningar, på ett närliggande sprickplana.
- Den fördjupade undersökningen visar att lokala störning kan påverka spänningsfältet. Detta är ett generellt problem vid spänningsmätningar i borrhål.
- I en volym med viss anistropi är det möjligt att bestämma orienteringen av största horisontalspänningen med osäkerheten ± 10°.
- Tunnslipsanalysen åskådliggör två grupper av mikrosprickor i de överborrade kärnorna. En grupp är lokaliserad parallellt och strax under borrhålsbotten där DDGS:s töjningsmätarna är placerade. Den andra gruppen är vinkelrät borrhålsbotten och sprickorienteringen överensstämmer med den observerade anistropin som erhölls vid P-vågsmätningen.
- Mikrosprickorna påverkar troligtvis resultaten från DDGS testen, sprickorna ger icke-elastiska töjningar. Däremot påverkas inte resultaten från den triaxiala töjningsgivaren (Borre Probe), limmad i ett pilothål, av mikrosprickorna.
- Bestämningen av elasticitetsmodulen i ett medelkornigt kristallint berg med en viss heterogenitet är inte helt trivial på borrkärnor. Bestämningen påverkar direkt den beräknade spänningen.

- För bestämning av minsta horisontella spänningen är hydraulisk spräckning mest lämplig, om den inducerade sprickan är orienterad längs borrhålet.

- Om bergmassans beteende i huvudsak är elastisk kan överborrningsmetoden ge ett spänningsvärde med en osäkerhet på 15 till 20%. Vid förekomst av mikrosprickor vid stort djup fås en överskattning av spänningarnas storlek.

- En bra förståelse av den gällande geologin från heterogenitet på mineralkornsskala till möjliga zoner är viktigt för att kunna utvärdera resultaten från spänningsmätningarna på ett tillfredsställande sätt. Svårigheten att få med alla inverkande faktorer från geologin gör att resultaten från spänningsmätningarna alltid innehåller en viss osäkerhet.

Projektet och dess slutsatser har direkt inverkan på det spänningsmätningsprogram som skall genomföras vid SKB:s planerade platsundersökningar.

Contents

1 Introduction

1.1 Background

As a part of the Swedish nuclear waste disposal program a series of site investigations are planned. For each of the studied site a design work will be carried out as a base for studies of Constructability, Environment Impact Assessment and Safety Assessment.

The state of stress together with the strength of the rockmass is the two key parameters to determine if the risk of spalling exists. It is not acceptable if the risk for spalling is common at the depth of a contemplated repository, see /Andersson et al, 2000/.

In situ stress measurements is needed to determine the origin state of the stress in the rock mass in the vicinity of the final repository. Some of the available equipment for in situ stress measurements has been studied, by the Swedish Nuclear Fuel and Waste Management Co (SKB), within the scope of works to provide the necessary rock mechanics support for the site investigations. A project with the objective to compare three different pieces of equipment for in situ stress measurements in deep boreholes under similar conditions has been carried out.

The project was carried out during year 2001 within the framework of development of methods for the site investigations.

1.2 Objectives

The two most common methods to measure in situ stress in the rockmass are overcoring and hydraulic fracturing. To be able to handle the known uncertainties in each of the methods, both of them are intended to be used during the site investigations /SKB, 2001/.

The main objective is to compare three different pieces of equipment for in situ stress measurements and find a strategy for SKB's Site Investigations to determine the state of stress in the rock mass. Two units of equipment use the overcoring method while the third uses the hydraulic fracturing method.

The overcoring was performed by AECL, using Deep Doorstopper Gauge System (DDGS) /Thompson and Martino, 2000/, and SwedPower, using their triaxial strain measuring instrument (Borre Probe) /Hallbjörn et al, 1990/. MeSy Geo Systeme GmbH performed the hydraulic fracturing. The DDGS system is a new method while the experience of the SwedPower overcoring and hydraulic fracturing methods are long.

Another purpose is to investigate how well the three different methods works under the same conditions. The tests were performed in the same orthogonal boreholes at Äspö Hard Rock Laboratory (HRL), Oskarshamn and Sweden. The measured results have been verified against known conditions at the Äspö HRL.

2 Conditions

2.1 Selected location

2.1.1 Criteria for the location

Three major criteria had to be fulfilled to locate the stress measurements: the geometry, the geology and the occurrence of water.

There was a requirement that the two boreholes for the test should be perpendicular and that the distance between the measurements should be kept as small as possible. It was preferable if the horizontal borehole could be drilled parallel or perpendicular to one of the principal stress directions.

The drilling equipment required that there was at least 6 m free space behind the start of the borehole in line of the hole. The borehole and the tests had to be located in a volume where they would not affect other ongoing or planned experiments and it was preferable if the holes could be used to some other experiment.

Since the new stress measurements were supposed to be compared to former measured values it was important that the tests were performed under similar circumstances, e.g. at approximately same depths.

The test should be performed in a rockmass that is in conjunction with the most common at Äspö, i.e. Äspö diorite. It was also preferable if the rockmass had a low degree of fracturing. The test should be performed within a rock unit with known or interpreted boundaries such as fracture zones.

A rockmass with low water conductance and little water bearing fractures were preferable.

The vertical borehole was supposed to be used for a thermal testing after completion of the stress measurements, see /Sundberg, 2002/. This test required that the length of the vertical hole was at least 60 m long.

2.1.2 Test location

The location that best fulfils the criteria above is a rock mass in Äspö HRL between –420 m and –470 m level north of the F-tunnel, see Figure 2-1. The rock mass is known to be homogeneous and relatively dry and it was therefore chosen for the tests.

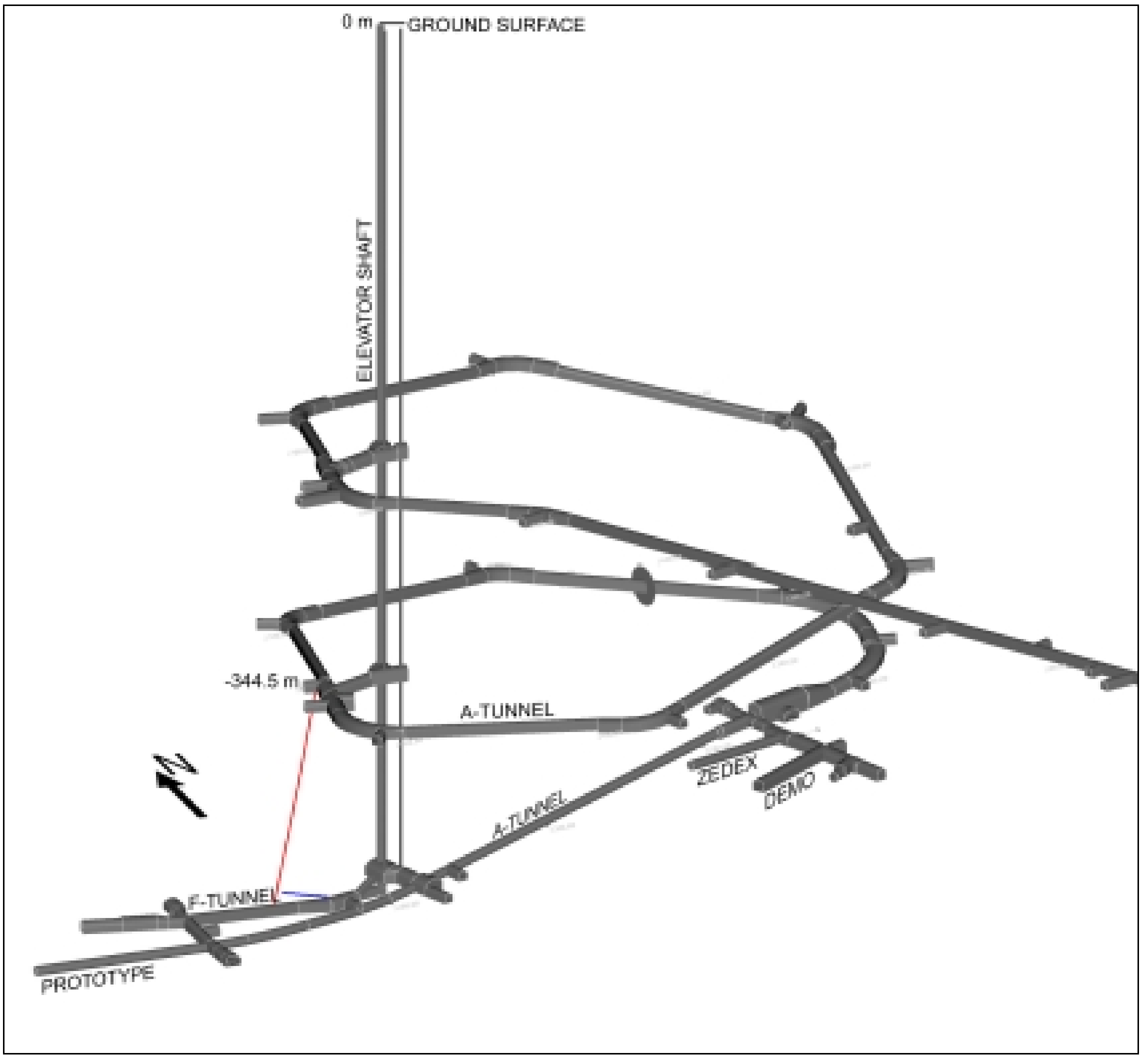

Figure 2-1. *View of Äspö HRL from SSW, the test location is placed north of the F-tunnel.*

The vertical borehole is about 130 m long and starts from the access ramp at the –344.5 m level. The horizontal hole is about 35 m long and starts in the F-tunnel just beside the located area at –456 m, see Figure 2-1.

Table 2-1 contains the start co-ordinates (in Äspö96 co-ordinate system) bearings, inclinations and lengths of the two boreholes.

Table 2-1. The start co-ordinates (Äspö96 co-ordinate system), bearings, inclinations and lengths of the boreholes.

Borehole	Name	X (m)	Y (m)	Z (m)	Bearing	Inclination	Length
Vertical	KA2599G01	7302.46	2039.95	–344.51	310.4°	80.1°	129.4 m
Horizontal	KF0093A01	7300.82	2052.64	–455.95	310.0°	–1.9°	36.5 m

The geometry of the two holes in Äspö HRL is shown in Figure 2-2.

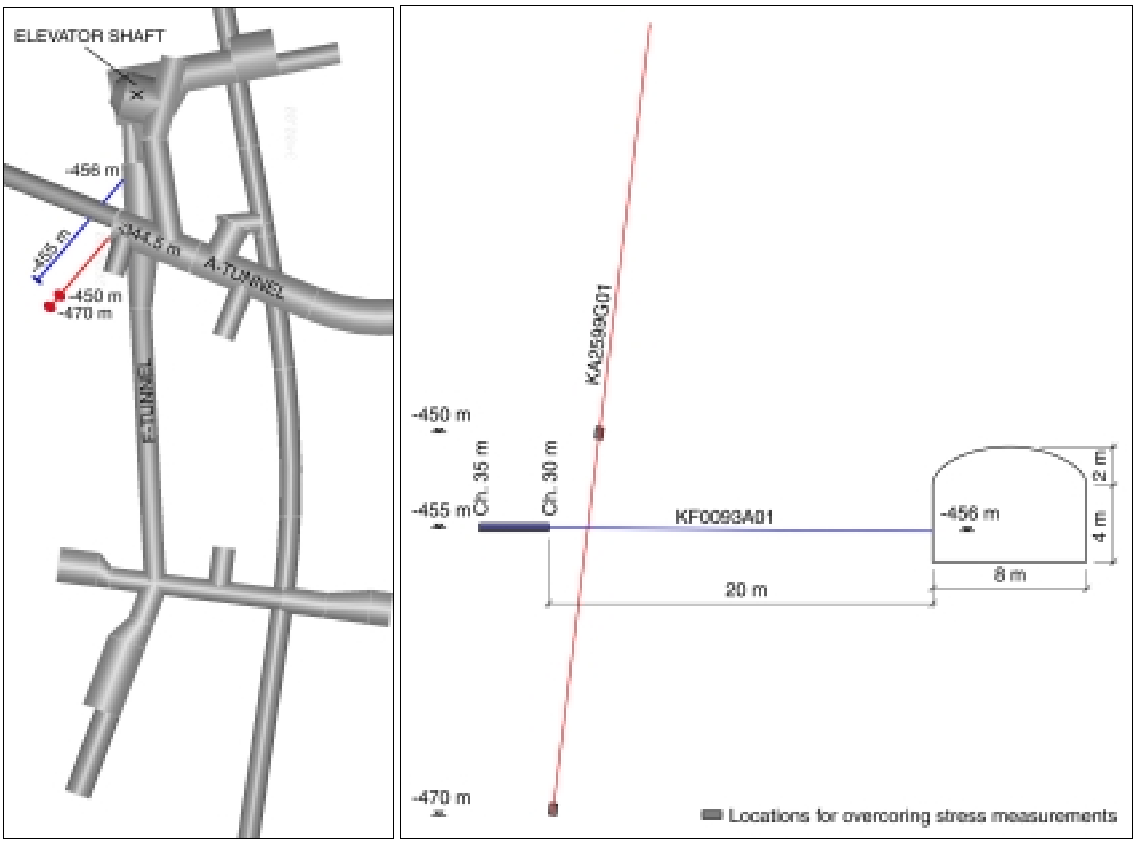

Figure 2-2. *The location of boreholes; CAD view from above (left) and sketch from west (right).*

3 Site conditions – earlier stress measurements and geological studies

3.1 Äspö stress database

The results from previous stress measurements include the two surface boreholes KAS02 and KAS05 together with boreholes in the vicinity of the DEMO, ZEDEX and Prototype tunnels. The Äspö HRL and locations for stress measurements are plotted in Figure 3-1. The dots for KAS02 and KAS05 denote where the boreholes cross the –450 m level.

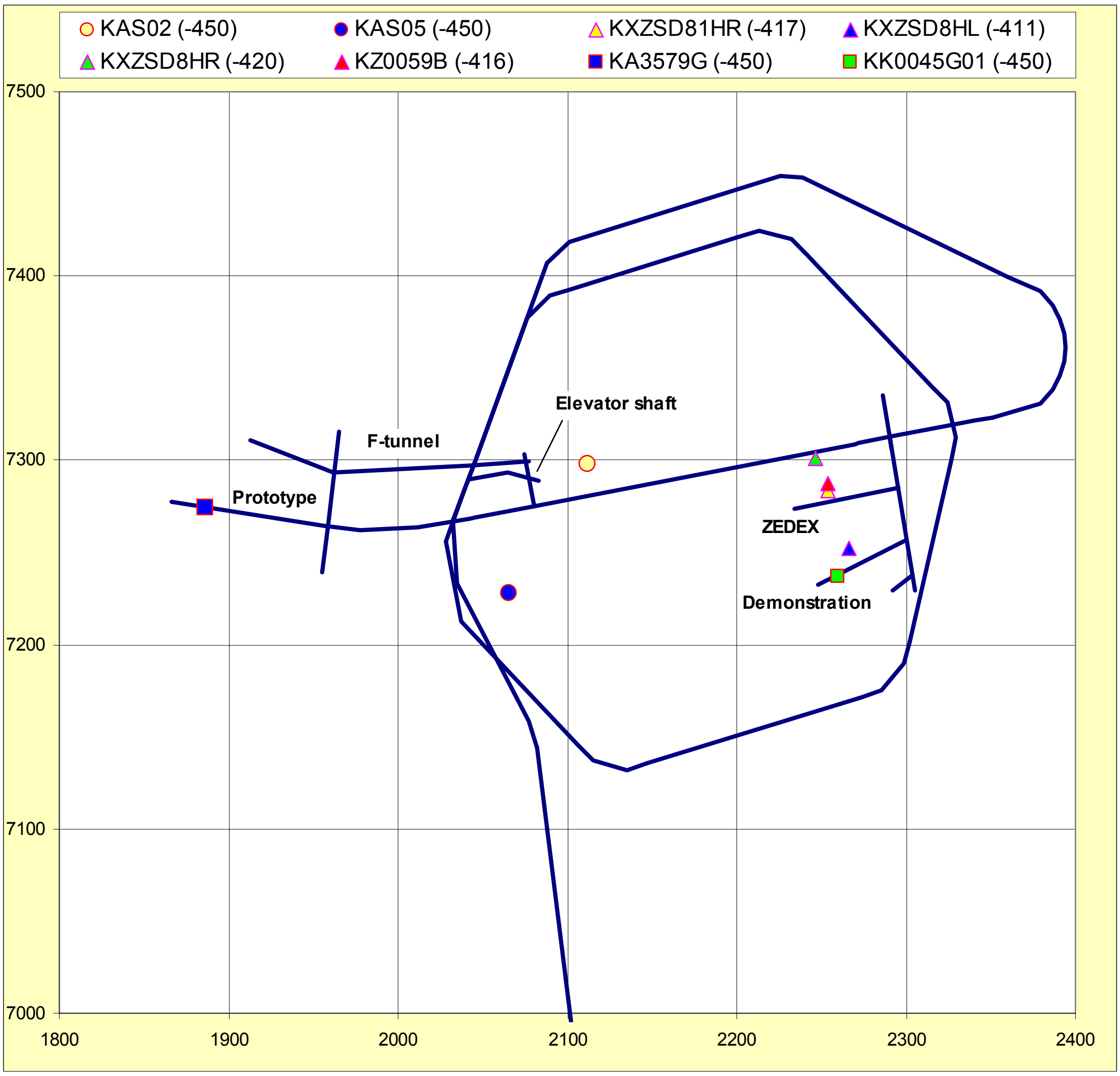

Figure 3-1. *Overview of Äspö HRL. The dots denote where stress measurements have been carried out. The dots for KAS02 and KAS05 are though where the boreholes cross the level –450 m.*

The stress measurements that are evaluated in this study are from the boreholes shown in Table 3-1. In the borehole KAS02 the measurements were performed using hydraulic fracturing while overcoring was used in all other studied boreholes. Measurements in the vicinity of the tunnel walls are not included due to disturbed conditions near large cavities. The stress measurements are carried out from level –300 m down to –600 m with a concentration around –420 m to –480 m. The approximated levels are listed in Table 3-1.

The measurements in the boreholes KAS02, KAS05 and Prototype (KA3579G) are performed in the same rock unit, according to a rock block model from 1998 by Raymond Munier, SKB, personal communication. The measurements in the ZEDEX and Demo tunnel are performed in a different rock unit.

Table 3-1. Summary of boreholes where stress measurements have been performed.

Bore hole name	Project	Type of method	Approximated level for measurements, m
KAS02	Surface borehole	Hydraulic fracturing	–300 to –600
KAS05	Surface borehole	Overcoring (SwedPower)	ca –345
KXZSD81HR	ZEDEX	Overcoring (CSIRO)	ca –417
KXZSD8HL	ZEDEX	Overcoring (CSIRO)	ca –411
KXZSD8HR	ZEDEX	Overcoring (CSIRO)	ca –420
KZ0059B	ZEDEX	Overcoring (SwedPower)	ca –416
KA3579G	Prototype	Overcoring (SwedPower)	–469 to –471
KK0045G01	Demo	Overcoring (SwedPower)	–448 to –452 and –479 to –481

3.2 Former stress measurement results

The former stress measurement results are presented in vertical and horizontal plane, so the 2D and 3D stress measurement methods can be compared.

3.2.1 Vertical stress

The measurements in KAS02 together with the ones in the boreholes from the Prototype tunnel is assumed to be in the same rock unit, while the measurements in the bore holes from the Zedex and Demo tunnels are performed in another unit. Figure 3-2 shows how the vertical stresses vary towards depth in the different boreholes and locations.

In KAS02 the stress increases linearly with depth. This is because the stress is assumed to correspond to the weight of the overlaying rockmass. The stress is 11.3, 12.1 and 12.7 MPa at the levels –420, –450 and –470 respectively. The vertical stresses in the holes from the Prototype tunnel vary between 14.1 and 20.5 MPa at the –470 m level.

The scatter was large for the measured stresses in the boreholes from the Zedex-tunnel and therefore a single interpreted value is shown. The magnitude is 16.2 MPa.

In the borehole KK0045G01, in the Demo tunnel, the vertical stresses vary between 10.5 and 18.1 MPa at the –450 m level and vary between 10.8 and 20.1 MPa at the –480 m level.

The vertical stress increases towards depth and has a scatter that is about 15 to 20 MPa. The averages of the measured stresses are slightly higher than the calculated.

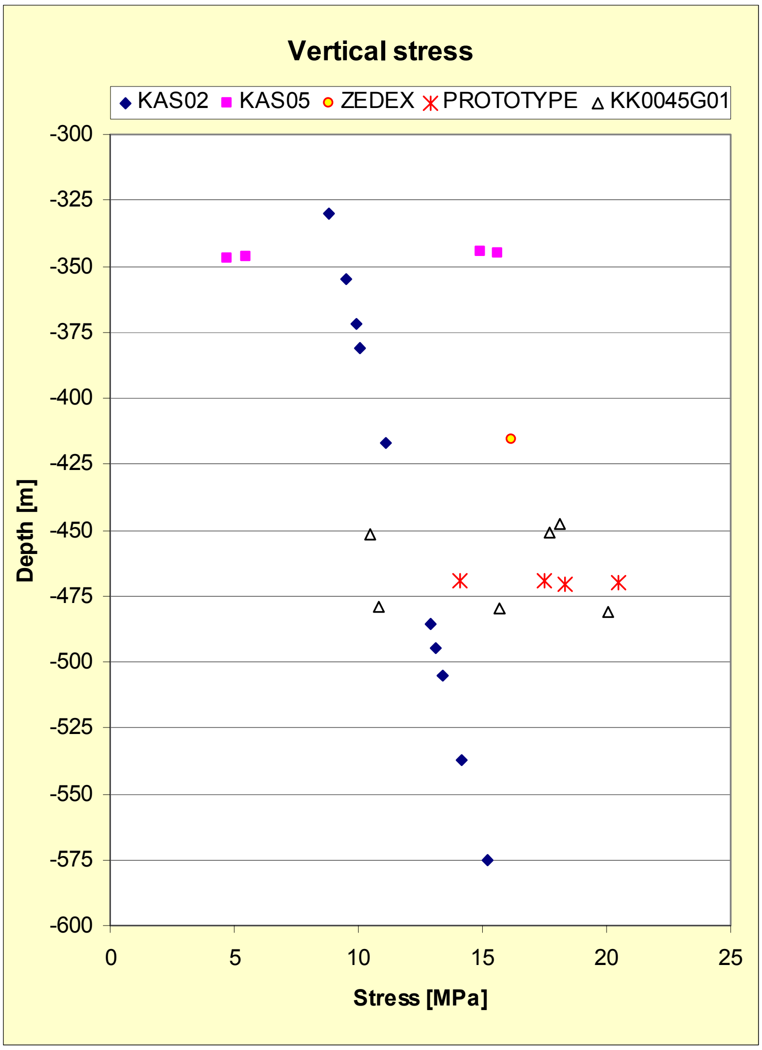

Figure 3-2. *Vertical stress as a function of depth.*

3.2.2 Maximal horizontal stress

Figure 3-3 shows how the maximal horizontal stresses vary towards depth in the different boreholes and locations.

The results from the stress measurements in KAS02 are well assembled along a line that increase a bit more than linearly towards depth. The stress is ca 17 MPa at the –420 m level. The stress measurements around the Prototype tunnel, which is in the same rock unit as KAS02, vary between 29 and 44 MPa at the –470 m level. The average stress around Prototype is about 20%, 5 MPa, larger than the interpolated stress in KAS02 at the same level.

The results from the stress measurements in the ZEDEX tunnel has been interpreted to 32 MPa at the level –416 m. This is twice as much as measured in KAS02 at the same level.

The measurements indicate good match between the stresses in the KK0045G01 and KAS02. At the level –450 m the measurements in KK0045G01 vary between 16 and 26 MPa and at the level –480 m they vary between 22 and 28 MPa.

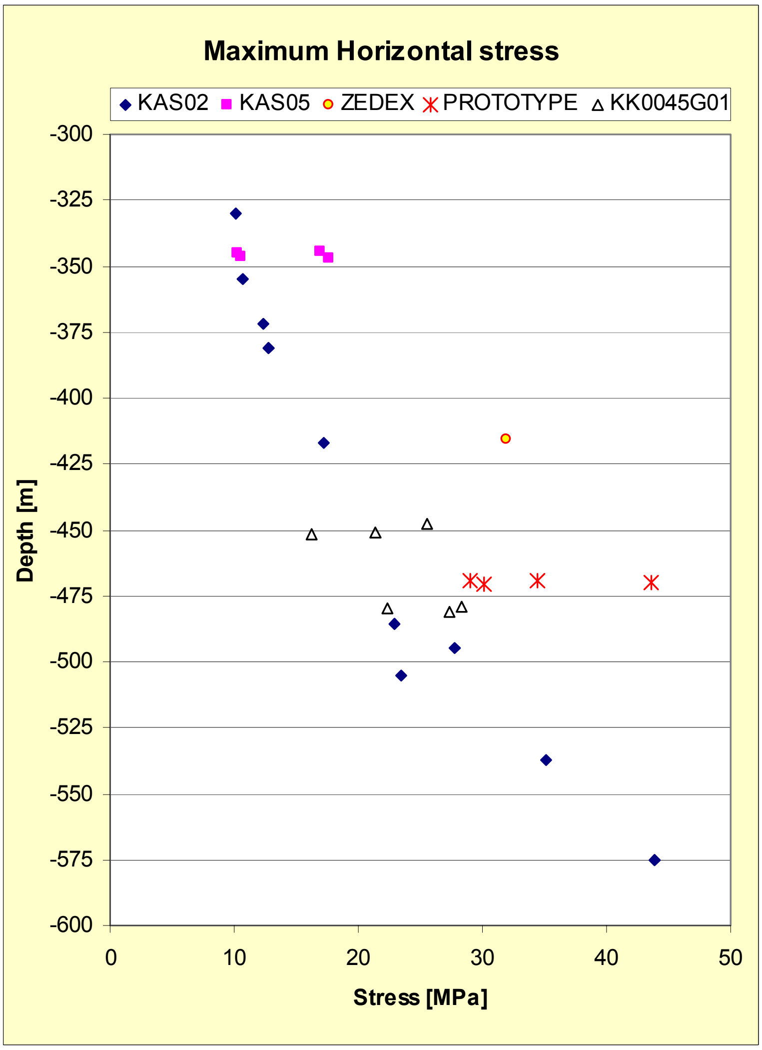

Figure 3-3. Maximum horizontal stress versus depth.

There are large differences between the results from the stress measurements in the different boreholes. The differences are larger within the same rock unit than between the two different units.

3.2.3 Minimum horizontal stress

Figure 3-4 shows how the minimum horizontal stresses vary towards depth in the different boreholes and locations.

The maximum horizontal stress shows minor scatter both in absolute and relative numbers. The scatter is less than 40% from the average value at each depth. The stress increase more than linear towards depth.

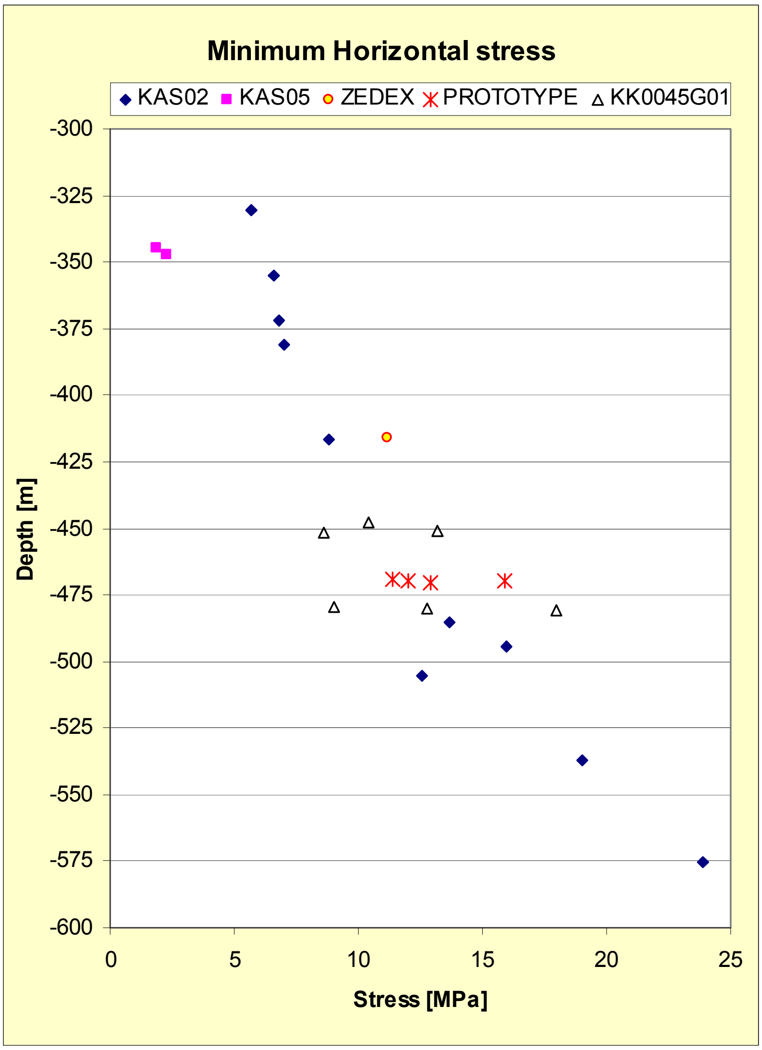

Figure 3-4. Minimum horizontal stress versus depth.

3.2.4 Orientation of maximum horizontal stress

The orientation of the maximum horizontal stress is shown in Figure 3-5 (in Äspö96 local co-ordinate system). The orientation varies between 120 and 160° for the measurements in KAS02 and between 135 and 150° in the surroundings of the Prototype tunnel. Likewise the scatter of the orientation is small for the measurements in KK0045G01, except for one value.

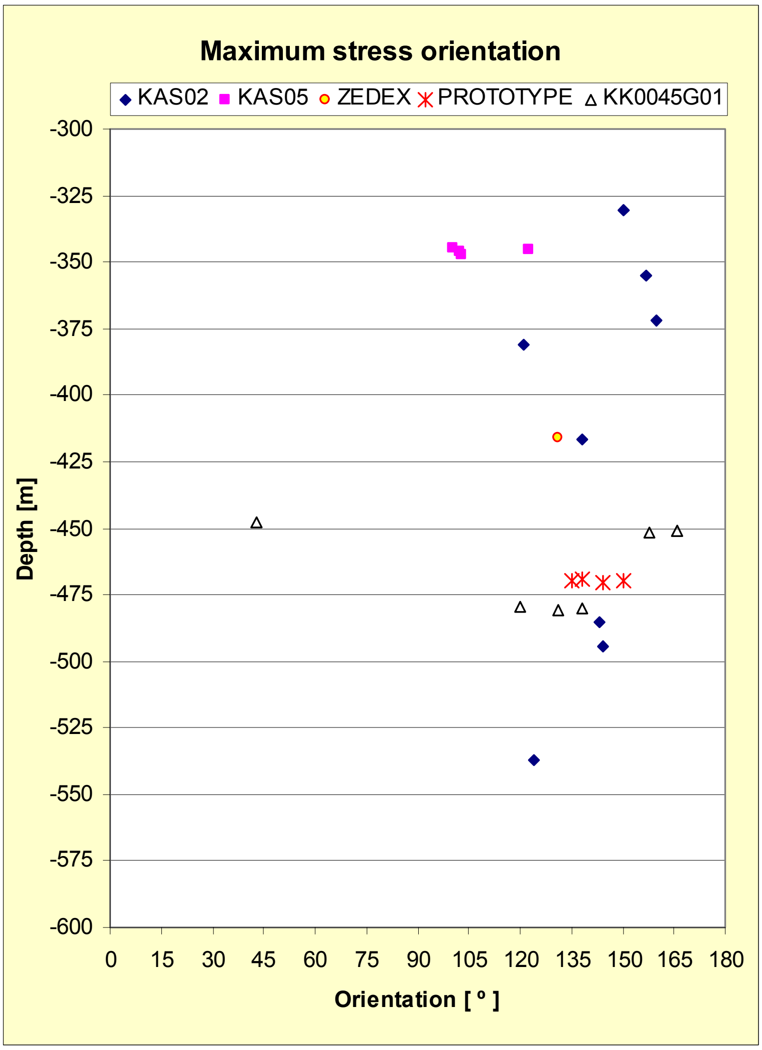

Figure 3-5. Orientation of maximum horizontal stress (Äspö96 co-ordinate system).

3.2.5 Conclusions and comment from earlier stress studies

The results from the stress measurements give an well-assembled picture for the vertical and the minimum horizontal stress, with a tendency of higher stresses and larger dispersion towards depth, see Figure 3-6. On the contradictory the maximum stress has a large dispersion and no clear depth dependence.

The expected vertical stress at the level –450 m is 14–21 MPa, maximum horizontal stress between 16 and 26 MPa together with a minimum stress between 9 and 14 MPa. The orientation of the maximum stress in expected to be around 120 to 150°.

Studies from /Ask, 2001/, concerning the results from hydraulic data, indicate that the magnitude of minimum horizontal stress is of the same order of magnitude as the vertical stress down to 400–500 m depth. Below 500 m depth the vertical stress is the minimum stress, at least down to approximately 900 m depth. /Ask, 2001/ also includes an investigation of geological structures that possibly influence the regional stress field. Based on the hydraulic stress data, it is suggested that one or more sub-horizontal zones, associated with fine-grained granite, disturb the stress field below the island of Äspö.

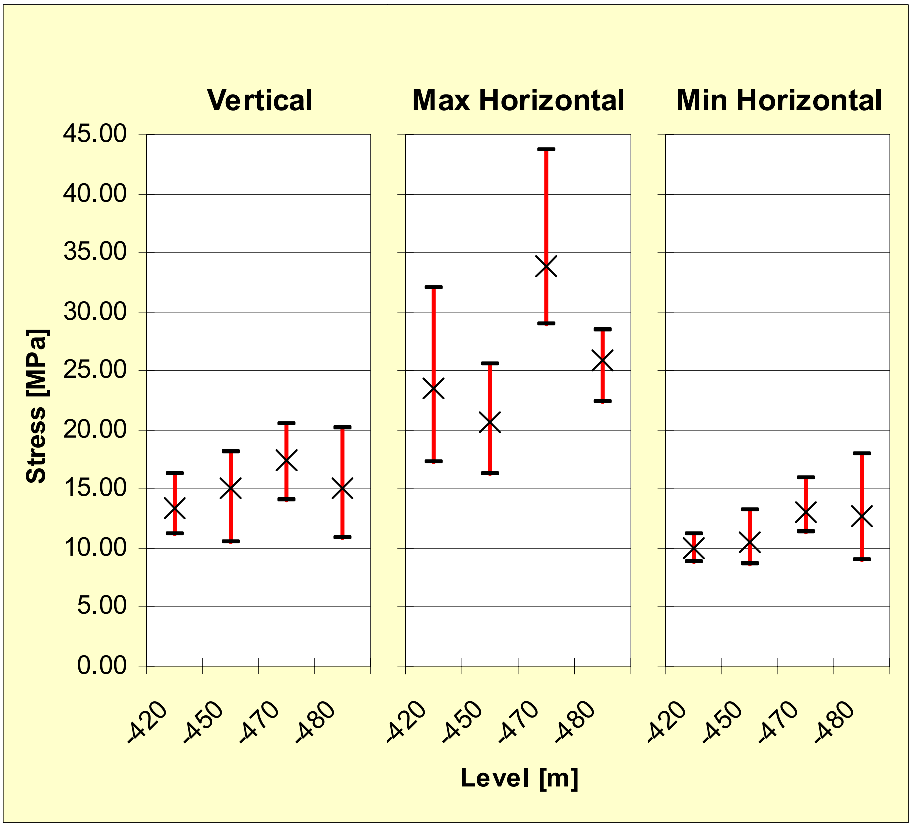

__Figure 3-6.__ The geometric mean and dispersion of the stress measurements at different depths.

/Lundholm, 2000/ has also studied the rock stresses from earlier stress measurements at Äspö HRL. The difference between hydraulic fracturing and overcoring results should be a result of stress-induced microcracks, which might be initiated during the overcoring.

In the Simevarp domain which include the Äspö HRL, fracture zones are expected to be the major cause for stress variations /Hakami et al, 2002/. The expected spatial variation, for the stress measurements, around the mean minimum horizontal stress is ± 15% /SKB, 2002/. But it is also shown that the result from overcoring data are more scattered than the hydraulic fracturing data, this is ascribed to the smaller scale for the overcoring method tests. Hakami has also done an estimation of the stresses in the Simevarp domain, see Table 3-2.

Table 3-2. Model for in situ stress magnitudes in the Simpevarp domain /Hakami et al, 2002/, z = depth.

Parameter	σ1	σ2	σ3
Mean stress magnitude, MPa	$0.066 \cdot z + 3$	$0.027 \cdot z$	$0.022 \cdot z + 1$
Uncertainty, 0–500 m	±25%	±25%	±25%
Uncertainty, 500–2000 m	±40%	±25%	±40%
Spatial variation, rock mass	±15%	±15%	±15%
Spatial variation, fracture zones	±50%	±50%	±50%

/Ljunggren et al, 1998/ carried out a statistical analysis on the stresses from earlier measurements at Äspö. The analysis of the minor principal stress showed that the average is almost the same for the two methods. The variance is, though, large for the overcoring measurements and small for the hydraulic fracturing. The large variance is due to the microstructure of the rockmass and it is suggested that at least 3 tests shall be performed to get good results.

3.3 Geological structure model and water in the boreholes

/Hansen and Hermanson, 2002/ have performed a geological structure study around the test area. The rock around the stress measurement area, within a 50 m scale block, consists of massive granodiorite (Äspö diorite) of good quality, except for small vertical slabs of finegrained granite. One of them is seen in the horizontal hole between hole length, Ch, 29.3 m and 30.6 m. The Äspö diorite is foliated after Ch 30.6 m.

The nearest dominant structure is a zone that is sub-vertical and striking NE. The nearest zone is about 15 m away from the end of the horizontal hole.

The fracture frequency, in the 50-m scale block, is low. It is measured to 0.5 fractures/m in the vertical hole and 0.2 fractures/m in the horizontal.

Figure 3-7 shows contours of poles of more than 900 fractures observed in tunnels around the tested area.

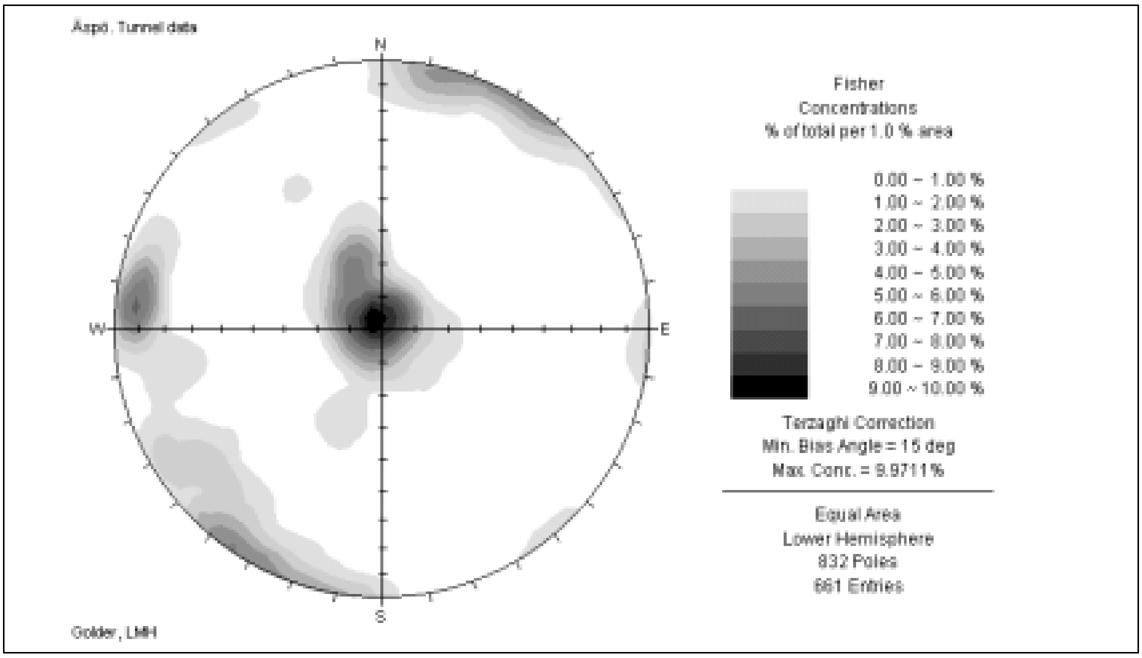

Figure 3-7. *Stereonet contour plot of fracturs in Tunnels TASA ch 2625-2700, TASF, TASG, TASI, TASJ, and hoist shaft (TASH) from elevation –350 metres to bottom at elevation –450 metres /Hansen and Hermanson, 2002/.*

The tunnel data showing a strong concentration of sub-horizontal fractures while borehole data show concentrations of North-South striking fractures with moderate dip mainly towards the West.

According to /Hansen and Hermanson, 2002/ the deviation is an artefact of the boreholes and the tunnels being in different rock mass and also because of the bias in the orientation of the boreholes. Most of the boreholes are oriented West-north-west, and may cause an over-representation of NS striking fractures. There is also a lack of sub-vertical boreholes causing a corresponding lack of intersections with horizontal fractures.

Data from boreholes KF0093A01, the horizontal hole, and KA2599G01, the vertical hole, do not indicate any significant deviation in the orientation distribution compared to the 200 m model.

Two fractures, J1 and J2, are interpreted by Hansen and Hermanson to intersect both boreholes, see Figure 3-8. One of them intersects the vertical hole at level –433.1 m and the horizontal at Ch 16.3 m having orientation N/mod. E. The other fracture is oriented NNW/steep E, and intersects the vertical hole at level –454.5 m and the horizontal at Ch 29.3 m.

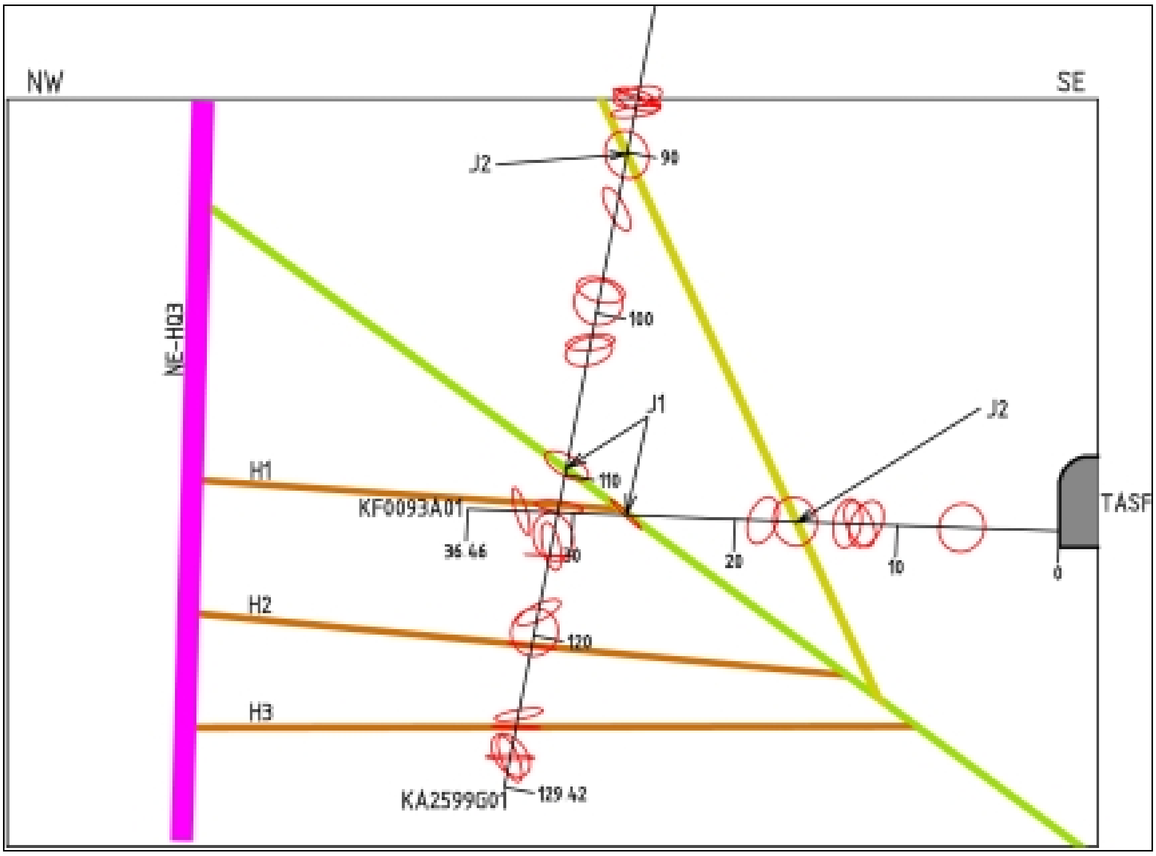

Figure 3-8. *Section in vertical plane along the boreholes. View approximately from west.*

Sudden losses in pressure for the drilling water were observed during the drilling of the vertical hole. Normally the drilling was stopped, and a measuring of the inflow to the borehole was done. The total measured inflow to the vertical hole was 36 l/min of which 32 l/min comes from the depth between –371 m and –374 m. From level –409 m and deeper there were no further observations of inflow to the hole.

A pressure build up test was performed in the vertical borehole. During the test the total inflow of water to the hole was monitored during at least one hour. Then the hole was closed and the pressure build up was monitored. The total inflow to the hole was measured to 40.8 l/min. The pressure build up is shown in Figure 3-9.

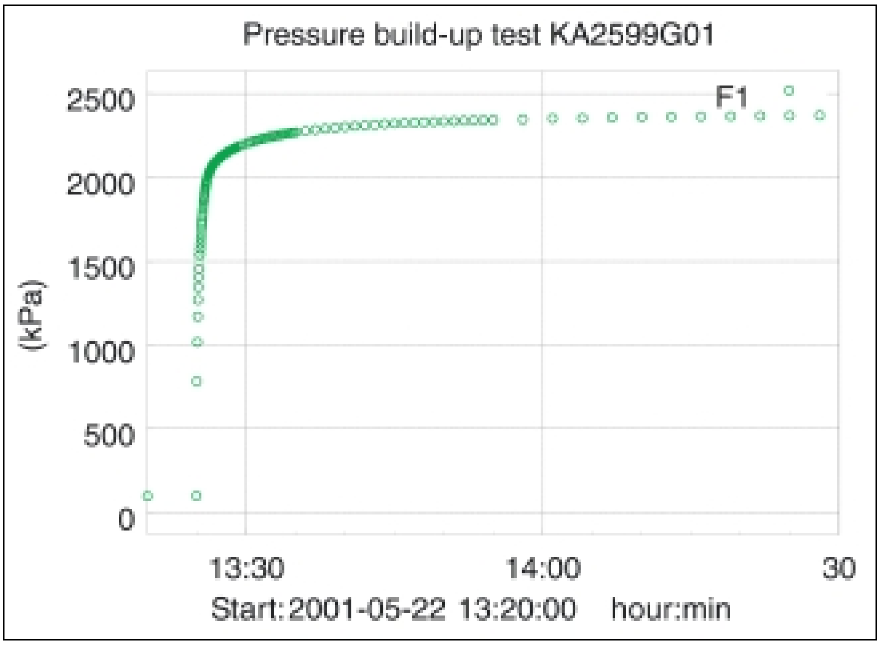

Figure 3-9. *Pressure build-up test in the vertical borehole.*

The pressure build up test shows that the water pressure in the vertical borehole is about 24 Bar, measured at level –344 m. An interpretation of the pressure build up curve, gives a transmissivity of about $2 \cdot 10^{-7}$ m^2/s.

The horizontal hole was dry after drilling.

Prior to the hydrofrac/hydraulic injection tests, short pressure pulse tests were carried out to test the suitability of the test interval for the subsequent fracturing test and to determine the in situ rock mass permeability in the vicinity of the borehole wall. The results of the pressure pulse tests gave an average hydraulic conductivity of $2.2 \cdot 10^{-10}$ m/s between level –447 m and –465 m ($T = 4 \cdot 10^{-9}$ m^2/s).

4 Description of the test methods

4.1 Borehole size

An international standard, called HQ, was chosen as the size of the borehole. The size was chosen to fit the equipment that AECL has developed for in situ stress measurements. Boreholes drilled with HQ-3 equipment gives a hole with an outer diameter of 96 mm and a core diameter of 61.1 mm.

4.2 DDGS, Deep Doorstopper Gauge System

At URL, in Canada, AECL has developed a 2D overcoring method that measures the stresses perpendicular to the borehole, i.e. it measures the horizontal stresses in a vertical borehole /Thompson and Martino, 2000/. The method is based on a combination of the overcoring method Doorstopper Gauge /Amedei and Stephansson, 1997/ and a system for registration in deep boreholes. The device utilises an Intelligent Acquisition Module (IAM), a remote battery-powered data logger that collects and stores strain data during stress measurement test. The method has been tested in, for example, URL in Canada and is able to perform measurements down to a depth of 1000 m depth. The equipment is developed to be a robust method in relatively high stressfields for measurements in ϕ 96 mm (HQ-3) boreholes. The method has never been tested by SKB before.

The principle of the method, see Figure 4-1, is that the strain gauge from the Doorstopper is glued onto the flat bottom of the borehole. During the overdrilling the changes in the strains are recorded with the data logger, IAM, which is mounted directly on the Doorstopper in the borehole. After the overdrilling, the core together with the measuring equipment is retrieved using a wire line system and the data from the overcoring is transferred to a computer. Normally a bi-axial pressure test will be done directly after up take. In the bi-axial test the Doorstopper gauges is used.

Calculating the in situ stress using overcoring relies on the difference between the strain in loaded and unloaded state, the gauge measures the magnitude and the orientation of the stresses in 2D.

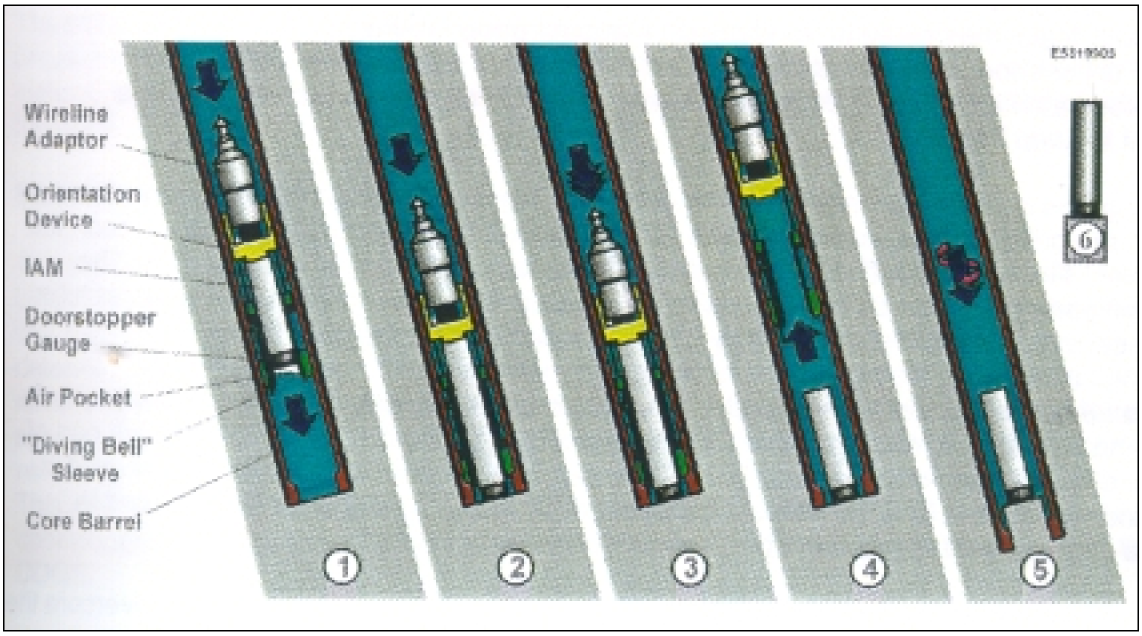

Figure 4-1. *Principal installation of DDGS /Thompson and Martino, 2000/. 1) After flattens and cleaning of the bottom, the instruments are lowered down the hole with the wire line cables. 2) When the DDGS is in the bottom the orientation of the measurement is noted in the orientation device and the strain sensor is glued. 3) The IAM and Doorstopper gauge are removed from the installation equipment. 4) The installation assembly is retrieved with the wire line system. 5) The monitoring and overdrilling start, the strain change in the bottom is measured by the time. 6) When the overdrill is finished the core and measurements is take up and a bi-axial pressure test will be done to estimated the Young's modulus.*

4.3 Borre Probe, Triaxial strain cell

A common overcoring method in Scandinavia is the Tri-axial Strain Cell (Borre Probe), developed by SwedPower /Hallbjörn et al, 1990/. The method can be used in long and water filled holes.

The main advantage of the method is that it only needs one borehole to determine the complete stress field. The major disadvantage, compared to DDGS, is that the system of drill rods needs to be disassembled and assembled every time that the tri-axial strain cell is going to be installed.

The principle of the method, see Figure 4-2, is that a large-diameter hole is drilled to the measuring depth in the volume of rock in which stresses will be determined. Then a small pilot hole is drilled in the bottom of the previous hole. A cell is attached to the wall of the pilot hole. Then an instrumented device, that can measure displacements, is inserted into the pilot hole. To the last, the drilling of the large-diameter hole is resumed and the changes of displacement within the instrumented device are recorded.

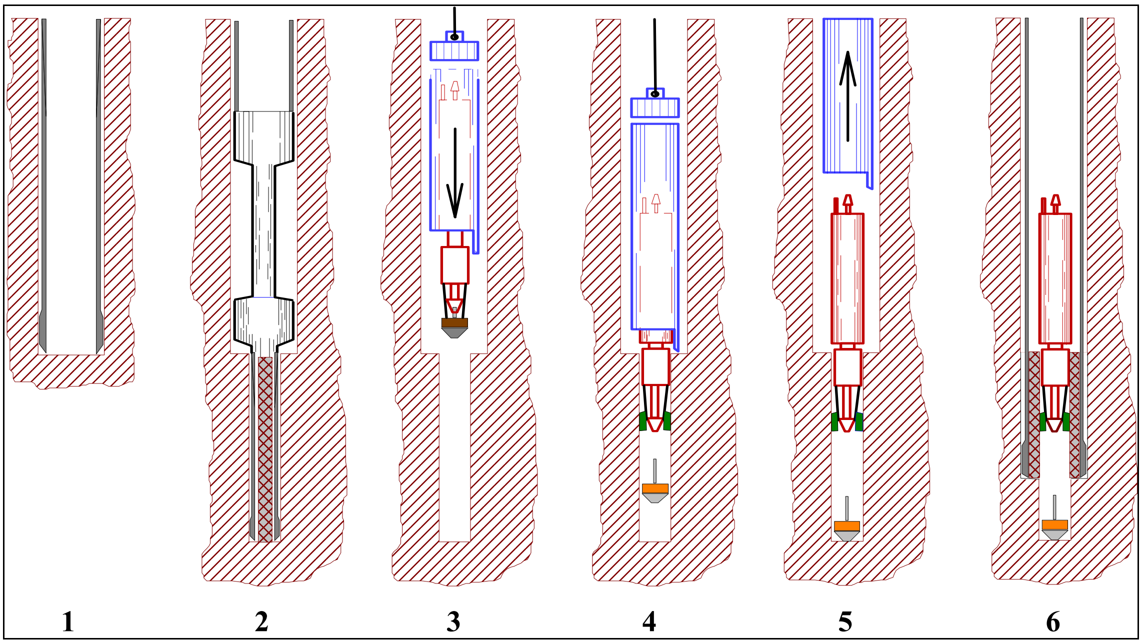

Figure 4-2. *Base steps for the SwedPowers overcoring method (from SwedPowers instructions): 1) Advance of ϕ 76 mm main borehole to measurement depth. 2) Drill ϕ 36 mm pilot hole and recover core for appraisal. 3) Lower Bore Probe in installation tool down the hole using a wire or glass-fibre rods (in sub-horizontal boreholes). 4) Probe releases from installation tool. Gages bonded to pilot-hole wall under pressure from the nose cone. 5) Raise installation tool. Probe bonded in place. 6) Overcore the Bore Probe and recover to surface in core barrel.*

The principle of all overcoring methods is that the differences in strain are measured in loaded and unloaded state. The knowledge of the changes in strain together with the deformation properties, i.e. Young's modulus and Poisons ratio, makes it possible to calculate the size and direction for the major stresses.

4.4 Hydraulic fracturing

Hydraulic fracturing is the most common in situ method to measure the state of stress in boreholes. The method is relatively simple and cost effective and it can be used in short holes as well as in long. It has been used for over 50 years at many different sites and conditions /Amadei and Stephansson, 1997/. A major advantage is that it can be used in already drilled boreholes.

Hydraulic fracturing involves the isolation of a part of a borehole using inflatable straddle packers and subsequent pressurisation of the hole until the rock fractures, see Figure 4-3. If an axial fracture appears, the pressure record obtained during the test can be used to determine the magnitude of the minor principle stresses in the plane normal to the borehole axis. During a hydraulic fracture test, pressure versus time is recorded.

The magnitude of the minor principal stress component can be determined directly from recorded shut-in pressure. The magnitude of the major principle stress can be calculated from relationships involving the fracture initiation pressure, the fracture reopening and the tensile strength of the rock. An impression packer together with a compass or a borehole scanner can be used to determine the orientation of the major principal stress in the plane normal to the borehole axis. The vertical stress is assumed to be due to the weight of the overburden rock.

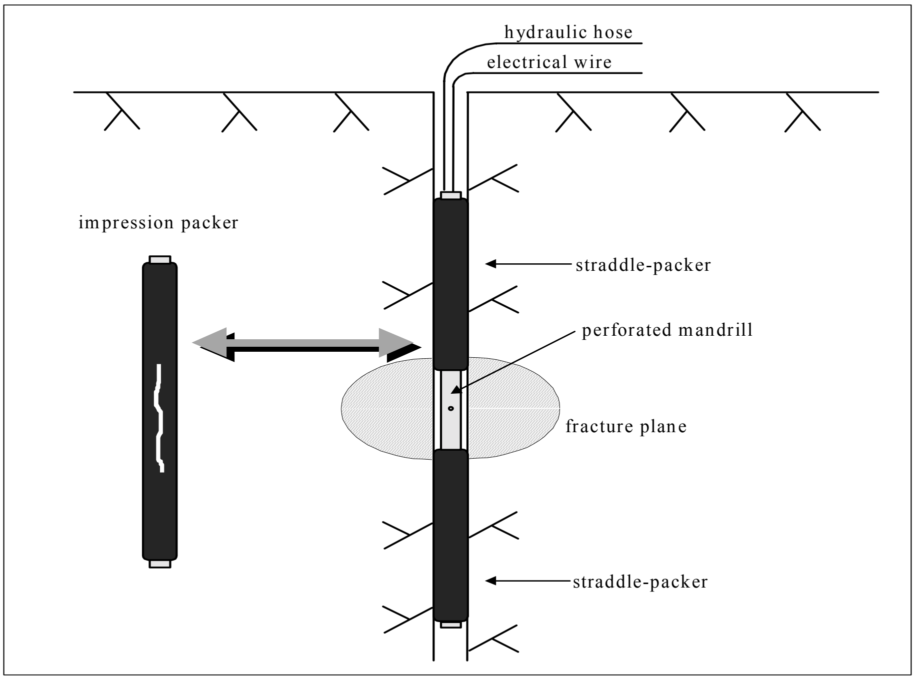

Figure 4-3. *Straddle packer for hydraulic fracturing and impression packer for fracture orientation.*

5 Test Results

5.1 General description of the drilling and tests

The HQ-3 drilling was performed by Drillcone core AB with equipment from Hagby. An electric hydraulic core bore machine, Onram 2000 CCD, was used for the drilling.

The drilling began with the vertical hole that was drilled to the first test level at –450 m i.e. 107 meter core drilling. The first stress measurements, DDGS, were performed using overcoring of the instrumented bottom of the borehole. After the tests at level –450 m the hole was drilled another 20 m where the second DDGS stress measurements took place at level –470 m.

Thereafter the drilling equipment was moved to the F-tunnel where the horizontal hole was drilled. There was one location for DDGS stress measurements in this hole, between hole length, Ch, 30 and 31 m. After the DDGS tests the Borre Probe tests started in the horizontal hole, between Ch 32 and 35 m.

Later the hydraulic fracturing were performed, first in the horizontal hole, between Ch 22 and 32 m, and secondly in the vertical hole, between level –448 and –466 m.

During all drilling the core was handled by the site geologist/drilling co-ordinator and placed in boxes for later mapping.

A more detail time schedule of the drilling and tests is summarised in Appendix 1 and 2.

All orientations are given in Äspö96 local co-ordinate system.

5.2 DDGS

This chapter briefly presents the results from the stress measurements performed with the DDGS by AECL. The results are both from the vertical borehole, KA2599G01, and the horizontal borehole, KF0093A01. This text is an extract from AECL's PM, see Appendix 3.

One DDGS test, from cleaning the bottom to retrieval of the core, takes about four hours.

The DDGS tests were performed at three different locations, 2 locations in the vertical hole, see section 5.2.1, and one in the horizontal hole, see section 5.2.2.

5.2.1 Results from the vertical hole

Six DDGS tests were attempted at five locations between level –449.91 m and –450.56 m, and three successful tests were obtained. Of ten started tests at eight locations only one successful test was obtained between level –470.37 m and –471.52 m. A summary of the attempted DDGS tests is shown in Table 5-1.

Table 5-1. Summary of DDGS overcores test at the different depths for the vertical borehole.

Test Level (m)	Date	Remarks
–449.91	9 May	Successful
–450.06	10 May	Data logger improperly programmed
–450.25	10 May	Glue failure
–450.40	11 May	Trigger failure
–450.40	11 May	Successful
–450.56	12 May	Successful
–470.37	14 May	Glue failure
–470.53	15 May	Successful
–470.70	15 May	Trigger failure
–470.70	16 May	Trigger failure
–470.70	16 May	Rock fragments at borehole bottom
–470.86	16 May	Broken battery clip in data logger
–471.03	17 May	Glue failure
–471.21	17 May	Glue failure
–471.37	17 May	Glue failure
–471.52	18 May	Glue failure

Bi-axial pressure test was conducted on each test specimen, using a hydraulically actuated Hoek cell. The signal conditioner for the pressure transducer failed after the first test. Subsequent tests were conducted without the pressure transducer, producing time versus pressure data, which were converted manually to strain versus pressure and ultimately modulus versus pressure.

A summary of the measured Young's moduli is shown in Table 5-2.

Table 5-2. Summary of measured elastic moduli for the vertical borehol.

Test Level (m)	E, gauge 3 (V) (GPa)	E, gauge 1 (H) (GPa)	E, gauge 2 (45) (GPa)	E, gauge 4 (135) (GPa)	E average (GPa)
–449.91	80	72.5	77	75	76
–450.40	94	68	76	92	83
–450.56	66	59	57	69	63
–470.53	57	120	87	–	88

It is a significant variation in the average moduli and degree of anisotropy between the four tests. The modulus ranges from an average of 63 GPa at –450.56 m to 88 GPa at –470.53 m. The test samples from –449.91 m and –450.56 m exhibit fairly isotropic behaviour. The test at –450.40 m is moderately anisotropic and test at –470.53 m is highly anisotropic. There was no difference in the overcore samples visible to the naked eye.

The calculation of in situ stresses in the plane perpendicular to the borehole axis was done using the methodology outlined in /Thompson and Martino, 2000/. The strain values were obtained from the overcore data. The modulus used for each test was the average, as determined, from bi-axial pressure test on corresponding overcore sample. The Poisson's ratio was assumed to be 0.25 and the vertical stress was calculated from the weight of the overburden. The calculated stresses from the tests are shown in Table 5-3.

Table 5-3. Summary of calculated stresses for the vertical borehole (σ_H is the maximum horizontal stress, σ_h is the minimum horizontal stress and σ_V is the vertical stress).

Test Level (m)	σ_H (MPa)	σ_h (MPa)	σ_V (MPa)	σ_H / σ_V	σ_h / σ_V	Azimuth of σ_H
–449.91	37.0	22.3	12.1	3.1	1.8	111°
–450.40	39.1	22.8	12.1	3.2	1.9	133°
–450.56	34.1	21.6	12.1	2.8	1.8	135°
Average at level –450 m	**36.7**	**22.2**	**12.1**			**126°**
–470.53	44.6	20.4	12.3	3.6	1.7	122°

5.2.2 Results from the horizontal hole

Eight DDGS tests were attempted between Ch 28.87 m and 31.05 m, and three successful tests were obtained. A summary of the DDGS tests attempted is shown in Table 5-4.

Table 5-4. Summary of DDGS overcores test depths for the horizontal borehole.

Hole length (m)	Date	Remarks
28.87	28 May	Glue failure
29.25	28 May	Glue failure
29.42	29 May	Glue failure
29.92	29 May	IAM failed to download data
30.07	30 May	Dimple on bottom caused gauge to break
30.23	30 May	Successful
30.89	30 May	Successful
31.05	31 May	Successful

Biaxial pressure test was conducted on each test specimen, using a hydraulically actuated Hoek cell. The signal conditioner for the pressure transducer had failed earlier, and hence the tests were conducted without the pressure transducer. The Produced time versus pressure data were manually converted to strain versus pressure and ultimately modulus versus pressure. A summary of the measured Young's moduli is shown in Table 5-5.

Table 5-5. Summary of measured elastic moduli for the horizontal borehole.

Hole length (m)	E, gauge 3 (V) (GPa)	E, gauge 1 (H) (GPa)	E, gauge 2 (45) (GPa)	E, gauge 4 (135) (GPa)	E average (GPa)
30.23	52	36	36	45	42
30.89	60	37	32	83	53
31.05	61	44	45	50	50

The average modulus for the three tests was 42, 53 and 50 GPa from shallowest to deepest. All the three test samples had a high anisotropic behaviour, especially at Ch 30.89 m.

The calculation of in situ stresses in the plane perpendicular to the borehole axis was done using the methodology outlined in /Thompson and Martino, 2000/. The strain values were obtained from the overcore data. The modulus used for each test was the average, as determined, from bi-axial pressure test on corresponding overcore sample. The Poisson's ratio was assumed to be 0.25. The stress perpendicular to the plane of the gauge was the average of the σ_H values determined from the measurements in the vertical borehole (38.7 MPa). The azimuth of the horizontal borehole was chosen to be roughly parallel to the direction of σ_H as determined in the vertical borehole, see Table 5-3. The calculated stresses from the tests are contained in Table 5-6.

Table 5-6. Summary of calculated stresses for the horizontal borehole (σ_H is the maximum horizontal stress, σ_h is the minimum horizontal stress and σ_V is the vertical stress).

Hole length (m)	σ_H (MPa)	σ_h (MPa)	σ_V (MPa)	σ_H / σ_V	σ_h / σ_V	Direction of σ_V*
30.23	38.7	12.2	27.0	1.4	0.5	0°
30.89	38.7	12.7	32.6	1.2	0.4	–6°
31.05	38.7	12.2	38.1	1.0	0.3	–8°
Average at level –455 m	**38.7**	**12.4**	**32.6**			

* Clockwise from vertical

5.2.3 Conclusions of the results from DDGS tests

- 7 of totally 24 test attempts were successful in the two boreholes.
- The average stress magnitudes from the tests in the vertical hole are; σ_H = 37 MPa, σ_h = 22 MPa, and σ_v = 12 MPa at level –450 m together with σ_H = 45 MPa, σ_h = 20 MPa, and σ_v = 12 MPa at level –470 m, see Table 5-3.
- The relationship σ_H/σ_V in the vertical hole is roughly 3.0 and σ_h/σ_V is around 1.8.
- For the horizontal hole the average stress magnitudes are; σ_H = 39 MPa, σ_h = 12 MPa, and σ_v = 33 MPa (range from 20 to 38 MPa) at level –455 m, see Table 5-6.

- The relationship σ_H/σ_V in the horizontal hole is roughly 1.2 and σ_h/σ_V is around 0.4.
- It is noted that the magnitude of $\sigma_{v,}$ in the horizontal hole, is 170% higher than the stress corresponding to the overburden pressure.
- No consistency between the results from the vertical and horizontal hole.
- The direction of σ_H is 126º, in the vertical hole.
- The bi-axial test results are indistinct. The values of Young's modulus between the vertical and horizontal hole differ and the modulus was variable in different direction. The bi-axial tests indicate rock anisotropy.

5.3 Borre Probe

This chapter briefly presents the results from the SwedPower overcoring rock stress measurements conducted in the horizontal borehole, KF0093A01. This text is an extract from SwedPowers PM, see Appendix 4.

One test, from drilling the pilot hole in bottom to retrieval of the core, takes about fifteen hours.

5.3.1 Results from the horizontal hole

Three successful results were conducted from four test attempts and six pilot holes between Ch 32.14 and 35.38 m. Table 5-7summarizes the general information from the test.

Table 5-7. Summary of Borre Probe tests for the horizontal borehole.

Hole length (m)	Date	Remarks
31.51	31 May	Failure in pilot core, instability in drilling equipment
32.14	1 June	Successful
32.70	2 June	Successful
33.23	3 June	Core failure
33.62	3 June	Failure in pilot core, natural fractures
35.38	6 June	Successful

The overcore rock samples from test 32.14 m, 32.70 m and 35.38 m were suitable for bi-axial testing. Table 5-8 shows the values of Young's modulus, E, and Poisson's ratio, ν, as interpreted from the bi-axial tests. The average values of E and ν are 54 GPa and 0.2 respectively.

Table 5-8. Results from biaxial tests on overcore rock samples from the horizontal borehole.

Hole length (m)	E (GPa)	Poisson's ratio, ν
32.14	51	0.19
32.70	60	0.23
35.38	51	0.19
Average:	**54**	**0.20**

The results from the bi-axial tests are not distinct neither for E, nor ν. Depending on individual gauge rosette, located at 120º angles on the overcore cylinder, secant E-values taken from the loading cycle and processed in the stress analysis vary between 41 GPa and 74 GPa. The variance is large and is an indicator of heterogeneity or anisotropy.

The results from the horizontal borehole are supposed to represent the virgin stress field at depths around level –455 m.

Table 5-9 to Table 5-11 show the results from the tests. All orientations are given in the Äspö 96 local co-ordinate system.

The average magnitudes for the primary stress field have been obtained by transformation of all applicable results to one common coordinate system, and then solving the average stress tensor for its eigen values.

Table 5-9. Primary stress field, principal stress magnitudes as determined by overcoring.

Hole length (m)	σ_1 (MPa)	σ_2 (MPa)	σ_3 (MPa)
32.14	32.5	13.8	8.7
32.70	36.0	17.7	8.9
35.38	23.2	14.2	6.9
Average level –455 m	**29.8**	**14.8**	**9.4**

Table 5-10. Primary stress field, principal stress orientations as determined by overcoring. Orientations are given as trend/plunge of the stress vectors σ_1, σ_2 and σ_3, respectively.

Hole length (m)	σ_1 Trend/plunge	σ_2 Trend/plunge	σ_3 Trend/plunge
32.14	307/38	096/48	204/16
32.70	310/38	114/51	214/08
35.38	308/10	044/30	204/58
Average at level –455 m	**310/31**	**088/52**	**206/21**

Table 5-11. Summary of calculated stresses for the horizontal hole (σ_H is the maximum horizontal stress, σ_h is the minimum horizontal stress and σ_V is the vertical stress).

Hole length (m)	σ_H (MPa)	σ_h (MPa)	σ_V (MPa)	σ_H / σ_V	σ_h / σ_V	Azimuth of σ_H
32.14	25.3	9.2	20.4	1.2	0.5	123
32.70	29.2	9.1	24.3	1.2	0.4	127
35.38	22.8	12.3	9.2	2.5	1.3	125
Average at level –455 m	**25.7**	**10.2**	**18.0**			**125**

5.3.2 Conclusions of the results from Borre Probe tests

- In the horizontal borehole, three of six drilled pilot holes were successful.
- σ_1 is as an average around 30 MPa, with a maximum value of 36 MPa. The relationship σ_1/σ_2 is roughly 2.0 whereas σ_1/σ_3 is around 3.0.
- The principal stresses are neither horizontal, nor vertical. σ_1 trends 309º with a dip around 30º, seeTable 5-9.
- Average magnitudes in the vertical and horizontal planes are; σ_H = 26 MPa, σ_h = 10 MPa, and σ_v = 18 MPa, see Table 5-10.
- The results at borehole length 35.38 m differ from the results at 32.14 m and 32.70 m, especially in σ_h and σ_v.
- It is noted that the magnitude of σ_v is 50% higher than the stress corresponding to the overburden pressure.
- The direction of σ_H is uniform at 125º, see Table 5-10. Transformed with respect to magnetic north the results yield a NW-SE direction for the maximum horizontal stress.
- The bi-axial test results are indistinct. Values for Young's modulus are generally in the lower region of the interval of E-values found for core samples from the Äspö HRL. The variance between E-values from gauges located 120º apart on the same overcore sample is hard to explain.

5.4 Hydraulic fracturing (HF)

This chapter presents briefly the results from the MeSy's hydraulic fracturing tests in the vertical borehole, KA2599G01, and the horizontal borehole, KF0093A01. This text is an extract from MeSy's IPR-report /Klee and Rummel, 2002/.

One test, including hydrofrac and impression packer test, takes about three hours.

5.4.1 Results from the vertical hole

A total of six hydrofrac/hydraulic injection- and impression packer tests were carried out in the vertical borehole between level –446.9 m and –464.5 m. The graphical test record analysis is given in the MeSy's report /Klee and Rummel, 2002/. The derived characteristic pressure data P_c, P_r, P_{si} and the resulting in situ tensile strength $P_{co} = P_c – P_r$ are summarized in Table 5-12. The data are listed as downhole pressure values. The results from the impression packer tests, conducted to derive the spatial orientation of induced or stimulated fractures, are shown graphically as a pole-plot in Figure 5-1.

Table 5-12. Breakdown pressure P_c, refrac pressure P_r, in situ hydraulic tensile strength $P_{co} = P_c – P_r$, and shut-in pressure P_{si} derived from hydrofrac tests in the vertical borehole (depth is related to the center of the 0.6 m long test interval.

Test level (m)	Date	P_c (MPa)	P_r (MPa)	P_{co} (MPa)	P_{si} (MPa)
–446.9	11 Oct	17.3	13.1	4.2	10.6
–449.4	11 Oct	15.0	11.2	3.8	10.3
–454.0	11 Oct	13.1	9.3	3.8	10.0
–457.4	11 Oct	18.0	12.5	5.5	12.2
–460.5	11 Oct	–*	8.8	–*	11.7
–464.5	11 Oct	18.8	12.8	6.0	11.4

* stimulation of a pre-existing fracture

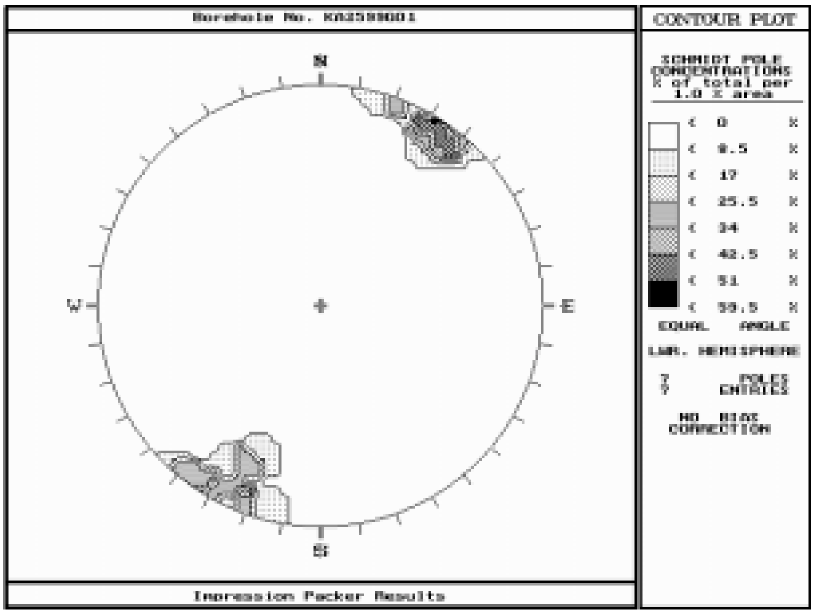

Figure 5-1. *Orientation of induced or stimulated fractures derived from impression packer testing in the vertical borehole (orientation in magnetically north).*

Since the impression packer tests showed that mainly unambiguous sub-vertical (axial) fractures with a consistent orientation of WNW-ESE were induced the stress estimation was conducted on the basis of the "classical" approach, see /Klee and Rummel, 2002/.

The stimulated fractures in Figure 5-1 follow the vertical NW-SE fracture set, that are mapped in the tunnels around the tested area, see Figure 3-7. The fracture set could not be found in the boreholes around the tested area, see discussion in section 3.3.

The resulting stresses σ_h and σ_H is listed in Table 5-13. The vertical stress σ_V* was calculated for a mean overburden rock mass density of 2.7 g/cm^3. The acting maximum horizontal principal stress σ_H is oriented N 131° ± 8° (WNW-ESE), in Äspö96 co-ordinate system.

Table 5-13. Results of the stress evaluation for the vertical borehole (σ_v *: vertical stress calculated for a mean overburden rock mass density of 2.7 g/cm^3, σ_h: minimum horizontal stress, σ_H: maximum horizontal stress, θ_{SH}: strike direction of σ_H).

Test level (m)	σ_H (MPa)	σ_h (MPa)	σ_V* (MPa)	σ_H / σ_V	σ_h / σ_V	$\theta_{SH,}$ N over E (deg)
–446.9	18.7	10.6	11.8	1,6	0,9	141
–449.4	19.7	10.3	11.9	1,7	0,9	137
–454.0	20.7	10.0	12.0	1,7	0,8	114
–457.4	24.1	12.2	12.1	2,0	1,0	127
–460.5	26.3	11.7	12.2	2,2	1,0	130
–464.5	21.4	11.4	12.3	1,7	0,9	130

5.4.2 Results from the horizontal hole

Six hydrofrac- and impression packer tests were conducted in the horizontal borehole between hole length 21.5 m and 32.0 m. The graphical test record analysis is given in MeSy's report /Klee and Rummel, 2002/. The derived characteristic pressure data P_c, P_r, P_{si}, and the resulting in situ tensile strength $P_{co} = P_c - P_r$ are summarized in Table 5-14. The results of the impression packer tests, conducted to derive the spatial orientation of induced or stimulated fractures, are shown graphically in Figure 5-2.

Table 5-14. Breakdown pressure P_c, refrac pressure P_r, in situ hydraulic tensile strength $P_{co} = P_c - P_r$, and shut-in pressure P_{si} derived from hydrofrac tests in the horizontal borehole (depth is related to the center of the 0.6 m long test interval.

Hole length (m)	P_c (MPa)	P_r (MPa)	P_{co} (MPa)	P_{si} (MPa)
21.5	22.4 – 23.5 [a)]	15.3	7.1 – 8.2	18.6
24.0	21.4	16.0	5.4	13.4
26.0	21.8	14.3	7.5	11.6
28.0	20.3	16.6	3.7	12.4
30.0	15.1	11.2	3.9	10.3 [b)]
32.0	16.5	10.9 – 11.5	5.0 – 5.6	9.8

a) no clear breakdown
b) increase of shut-in pressure during the injection cycles

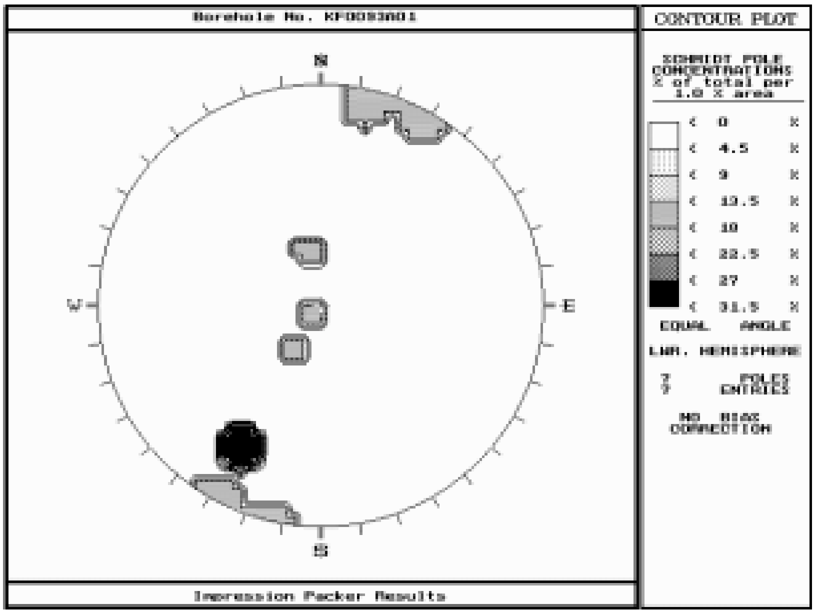

Figure 5-2. Orientation of induced or stimulated fractures derived from impression packer testing in the horizontal borehole (orientation in magnetically north).

The evaluation of the in situ stresses is based under consideration of the direction of the maximum horizontal stress derived in the vertical borehole (θ_{SH} = N 131° ± 8°), the horizontal borehole is orientated parallel to the direction of the maximum horizontal stress σ_H. For the particular case of $\sigma_h < \sigma_v$, axial vertical fractures will be initiated.

Sub-horizontal fractures was initiated at Ch 21.5 and 24.0 m, which most likely depend on that $\sigma_v < \sigma_h$. In this case the shut-in pressure corresponds to the vertical stress, i.e. $P_{si} = \sigma_v$.

However, although breakdown (fracture initiation) events were observed during all tests, the impression packer tests showed axial, steeply inclined fractures only for the test sections between Ch 26.0 m and 32.0 m.

The acting maximum horizontal stress σ_H is oriented N 127° ± 8° (WNW-ESE), in Äspö96 co-ordinate system. The results of the stress estimation for the horizontal borehole are presented in Table 5-15

Table 5-15. Results of the stress evaluation for the horizontal borehole (σ_V*: vertical stress calculated for a mean overburden rock mass density of 2.7 g/cm^3, σ_h: minimum horizontal stress, σ_V: measured vertical stress, θ_{SH}: strike direction of the maximum horizontal stress σ_H).

Hole length (m)	σ_h (MPa)	σ_V (MPa)	σ_V* (MPa)	σ_h / σ_V	θ_{SH} ,N over E (deg)
26.0	11.6	20.5	11.9	0,6	115
28.0	12.4	20.6	11.9	0,6	131
30.0	10.3	19.7	11.9	0,5	130
32.0	9.8	17.9 – 18.5 <18.2>	11.9	0,5	131

<> mean value

5.4.3 Conclusions of the results from the Hydraulic fracturing tests

- In the two boreholes, the vertical and the horizontal, 10 of totally 12 tests gave acceptable results. In the horizontal hole two tests resulted in sub-horizontal fractures, i.e. $\sigma_v < \sigma_h$.
- For the vertical hole the average magnitudes in the vertical- and horizontal planes are: σ_H = 22 MPa, σ_h = 11 MPa, and σ_v = 12 MPa (theoretical) between level –447 m and –465 m, see Table 5-13.
- The relationship σ_H/σ_V in the vertical hole is roughly 1.8 whereas σ_h/σ_V is around 0.9.
- For the horizontal hole the average magnitudes in the vertical- and horizontal planes are: σ_h = 11 MPa, and σ_v = 20 MPa (measured) at level –455 m, see Table 5-15.
- The relationship σ_h/σ_V in the horizontal hole is roughly 0.6.
- It is noted that the measured magnitude of $\sigma_{v,}$ in the horizontal hole, is 80% higher than the stress corresponding to the overburden pressure.
- The direction of σ_H is about 129°.
- It's a good agreement on σ_h between the vertical and horizontal borehole. All results indicate σ_h = 11 ± 1.5 MPa.

6 Comparison of results

All results are presented as horizontal and vertical stress components to enable comparisons with the 2D methods. Only averages of results from a method are compared, because some dispersion of the measured values appears for each method at the same depth. The dispersion is normally between 15–40%.

The comparisons start with the minor horizontal stress. The reason is that the normal stress to an induced fracture is based directly on the measured values from the hydraulic fracturing method. Then follow the major horizontal stress and to the last the vertical stress.

6.1 Minimum horizontal stress

The measured and interpreted minimum horizontal stresses from the three methods in the two boreholes are shown in Figure 6-1. The results are summarised in Table 6-1.

Table 6-1. Summary of the results for measuring minimum horizontal stress in the vertical and horizontal borehole.

Method	DDGS	DDGS	HF	HF	Borre Probe
Test hole	Vertical	Horizontal	Vertical	Horizontal	Horizontal
Stress (MPa)	22.3	12.4	11.0	11.0	10.2

The measured minor horizontal stress between level –450 m and –465 m is about 11 MPa except for the DDGS method in the vertical hole that is about 22 MPa.

The difference in minimum horizontal stress between the single measurements is about 5% for the DDGS method, 35% for the Borre Probe method and 25% for the hydraulic fracturing method see Figure 6-1.

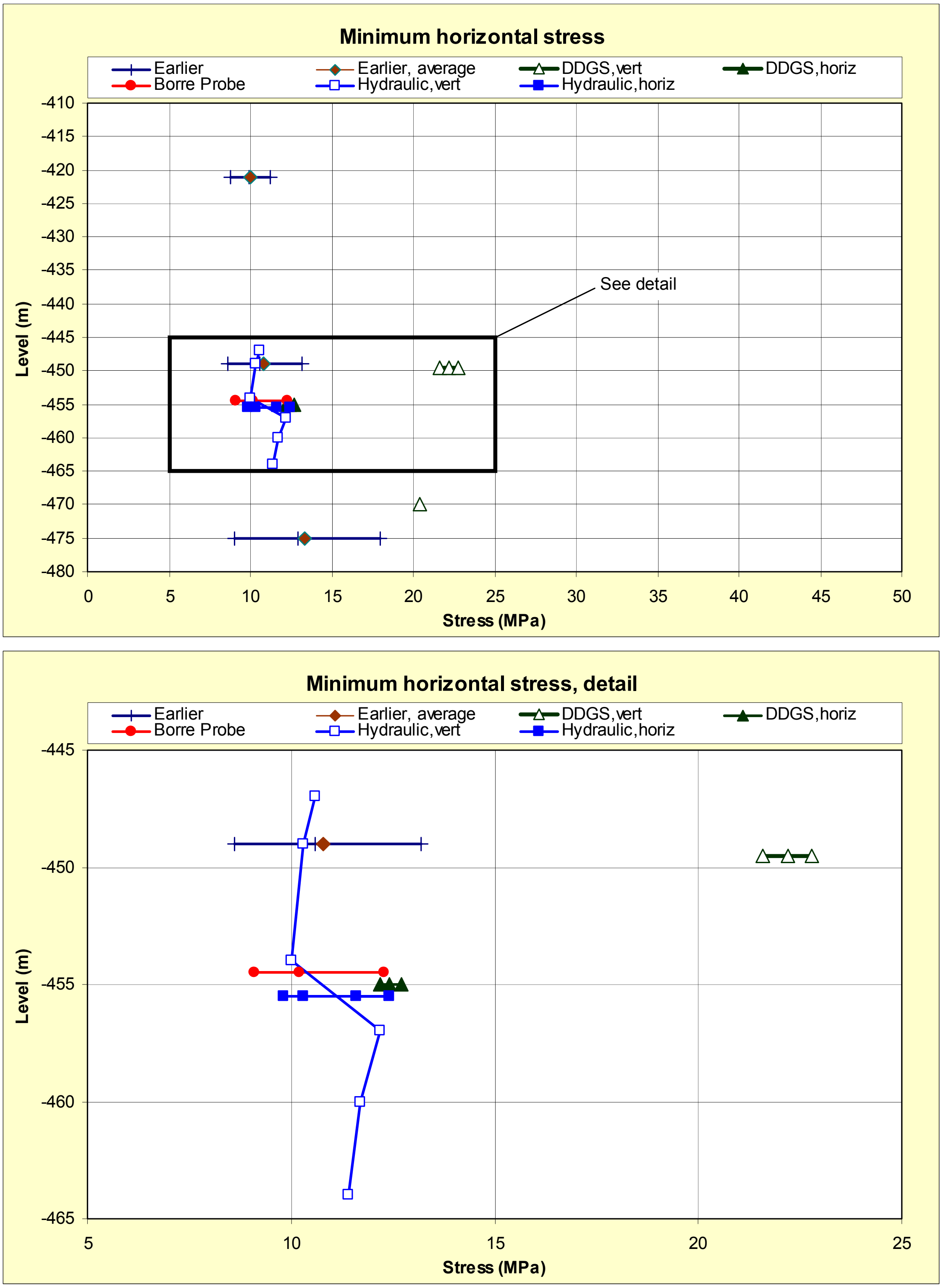

***Figure 6-1.** Measured minor horizontal stress versus depth in the vertical and horizontal borehole.*

6.2 Maximal horizontal stresses

The measured and interpreted maximum horizontal stresses from the three methods in the two boreholes are shown in Figure 6-2. The results are summarised in Table 6-2.

The measured stress is much larger for the DDGS method. The average is 37 MPa, which shall be compared to 26 MPa for the Borre Probe and 22 MPa for hydraulic fracturing. The orientation of the measured maximum horizontal stress varies between 125 and 131° for all three methods.

All measurements of the maximum horizontal stress show large varieties within each method in the single values at the same test location. Between level –447 and –465 m, the maximum difference is 15% for the DDGS in the vertical hole, 30% for the Borre Probe in the horizontal hole and 40% for the hydraulic fracturing in the vertical hole.

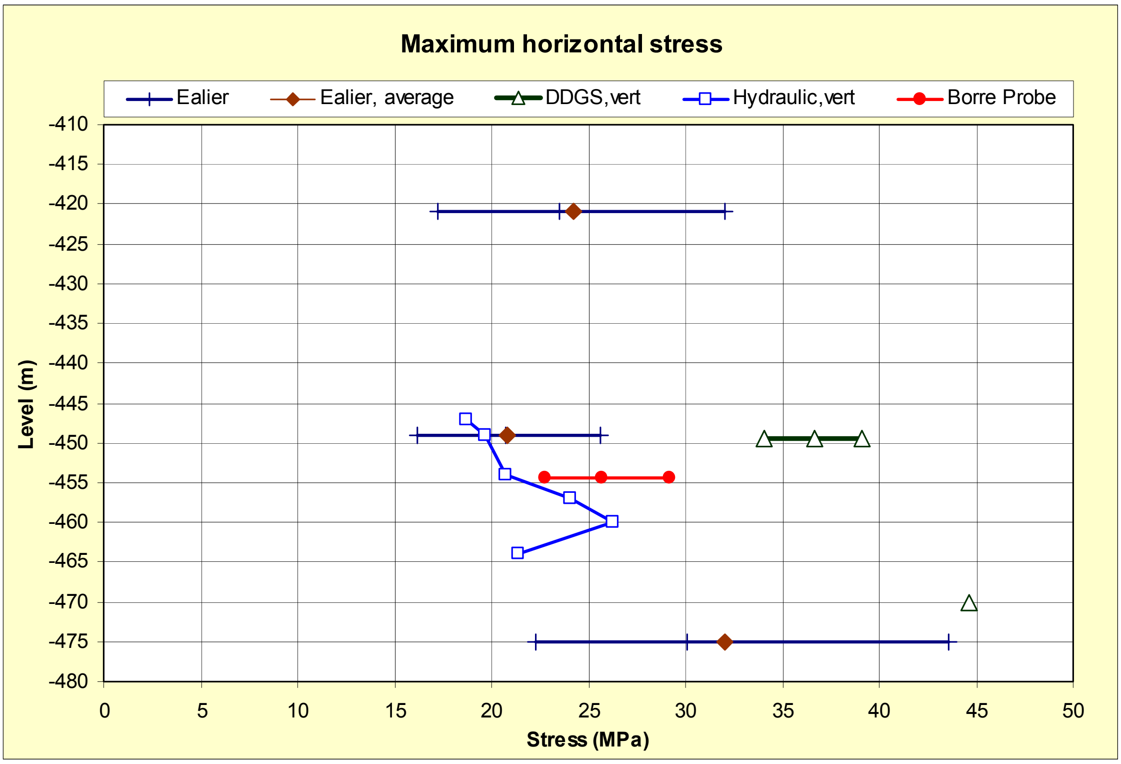

Figure 6-2. Measured and interpreted maximum horizontal stress versus depth in the vertical and horizontal borehole.

Table 6-2. Summary of the results for measuring maximum horizontal stress in the vertical and horizontal borehole.

Method	DDGS	HF	Borre Probe
Test hole	vertical	Vertical	horizontal
Stress (MPa)	36.7	21.8	25.8

6.3 Vertical stresses

The measured and interpreted vertical stresses from the three methods in the two boreholes are shown in Figure 6-3. The results are summarised in Table 6-3.

Most of the results are much higher than the theoretical vertical stress. The interpreted values are much higher from the DDGS method than for the other two methods.

The stress measurements from DDGS and Borre Probe methods show large varieties in the single values within each method, 40% and 160% respectively. However the result from hydraulic fracturing is well assembled around 20 MPa at level –455 m.

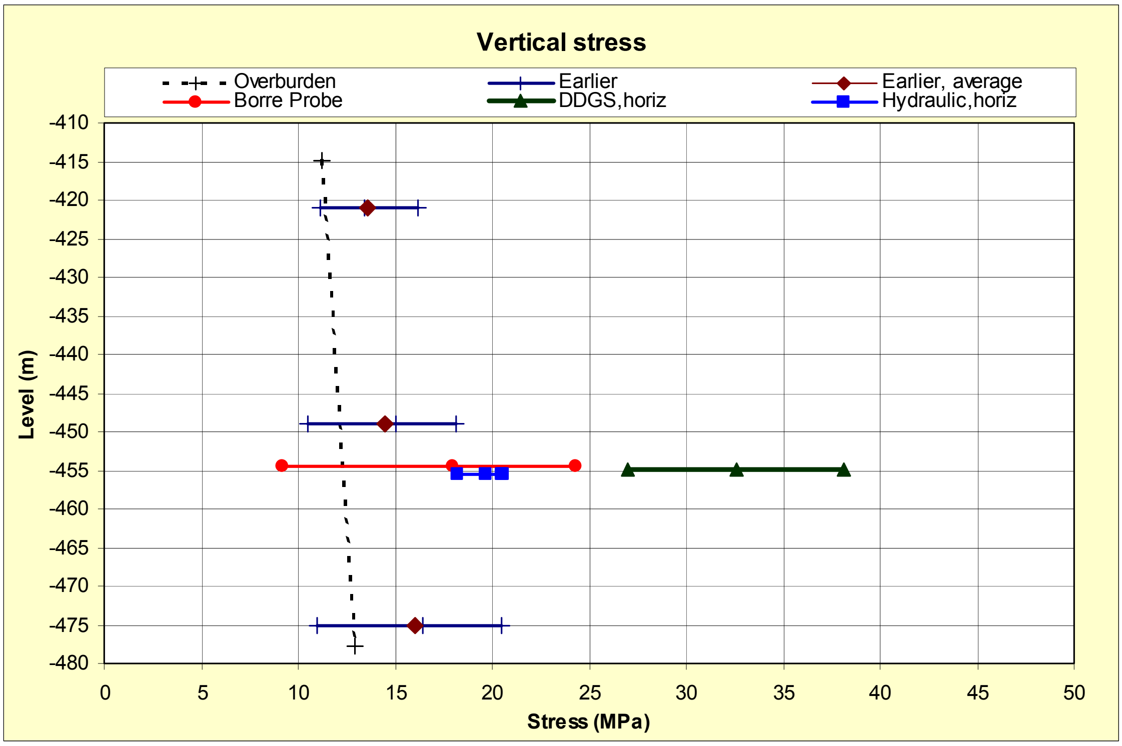

Figure 6-3. *Measured and interpreted vertical stress versus depth in the horizontal borehole.*

Table 6-3. Summary of the results for measuring vertical stress in the vertical and horizontal borehole.

Method	Theoretical	DDGS	HF	Borre Probe
Test hole		Horizontal	Horizontal	Horizontal
Stress (MPa)	12.2	32.6	19.8	18.0

6.4 Young’s modulus

A summary of the measured Young’s modulus from the DDGS and Borre Probe tests is show in Table 6-4.

Table 6-4. Summary of Young´s modulus, from the DDDS and Borre Probe tests.

Test	**E,** vertical hole (MPa)	**E,** horizontal hole (MPa)
DDGS, –450 m	76	42
DDGS, –450 m	83	53
DDGS, –450 m	63	50
DDGS, –470 m	88	–
Borre, –450 m	–	51
Borre, –450 m	–	60
Borre, –450 m	–	51

6.5 Summary of the result comparison

Some reflections from the comparisons above:

- Some dispersion of the measured and interpreted values appear for each method at the same depth, normally between 15–40%, but in one case as much as 160%.
- The hydraulic fracturing gives almost the same value for the minimum horizontal stress in the vertical and the horizontal borehole.
- The Borre Probe and the hydraulic fracturing method results in values in the same range for all three stress components at level –450 m to –460 m.
- Except for the maximum horizontal stress measured in the horizontal borehole the DDGS method constantly result in larger stresses than the other two methods. The factor is 1.4 to 2.0.
- According to the results from the horizontal borehole, all methods measure the vertical stress significantly higher than the weight of the overburden.
- From the comparisons, the following estimations of the stress components has been made: 9 to 13 MPa for the minimum horizontal stress, 20 to 29 MPa for the maximum horizontal stress and 10 to 22 MPa for the vertical stress. In this estimation the result from the DDGS measurement are neglected.

7 Investigations about factors that may influenced the results

7.1 Questions raised from the results

From the result of the stress measurements and the comparison between the methods, the following questions has been brought out:

- The higher stress magnitudes from the DDGS tests, compared to the results from the Borre Probe and hydraulic fracturing tests. This give two hypotheses; one that the majority of the results are most correct (the latest and former results from Borre and hydraulic tests) and one that the DDGS tests is most correct and the other have systematic errors.
- The higher measured vertical stresses compared to the theoretical stress.
- The scattered results from the overcoring.
- The indication of high anisotropy in the horizontal hole.
- The difference in minimum horizontal stress between the vertical and horizontal hole for the DDGS test.
- The difference in interpreted Young's modulus between the vertical and horizontal bore hole.

The possible answer for the questions above is most likely a combination of different factors and, hence, does not have to rely on just one factor. One main group of factors is focused of rock mass conditions, like;

- anisotropy in the rock mass, the stress analyses is based on isotropic conditions,
- high stress concentrations around the borehole, causing a problem with micro fractures,
- the natural statistical scatter for the measured parameters and
- the assumptions and models for the analyses of the results in the actual rock mass.

Another group of factors concerns the technical conditions like;

- the technical success and the measured strains in the DDGS tests and
- determination of Young's modulus.

To try to understand the results and answer the questions deeper studies have been done on the topics:

1. Anisotropy and Young's modulus with two independent methods, P-wave measuring and uniaxial tests.
2. Geological influence, e.g. rock type and fractures.
3. Theoretical strain values and stress concentrations on the borehole bottom using numerical analyse.
4. Microcracking, using numerical analyse and microscopy.
5. An extensive auditing on the DDGS strain recording.

7.2 Anisotropy and Young's modulus, E

The overcoring results rely on the theory of elasticity /Amadei and Stephansson, 1997/, i.e.;

1. the rock must behave reasonable homogenous and elastic to enable the standard analyses for interpreting the results and
2. the method must be able to determine Young's modulus in a controlled way and the results must be reproducible.

The results from the bi-axial tests are indistinct for both the DDGS and Borre Probe tests. The measured values of Young's modulus differ between the vertical and horizontal hole, i.e. the modulus is dependent on the direction. Hence, the bi-axial tests indicate rock anisotropy.

A large variation of Young's modulus was obtained in the bi-axial cell for the tested cores, both in magnitude and orientation. Results from individual gauges in a test indicated anisotropic behaviour. Due to this and the large differences in the measured stresses a deeper investigation was made on the tested cores and cores from nearby. Both P-wave measurements, perpendicular to the core with 30° intervals /Eitzenberger, 2003/, and uniaxial compression tests, on small samples in different directions, were performed using retrieved HQ-3 cores, see Appendix 5.

The test method used for the diametrical measurements of P-wave velocity is developed by LTU /Eitzenberger, 2003/. The measured principle is that transducers and wave-guides are placed opposite to each other with a sample in between, see Figure 7-1.

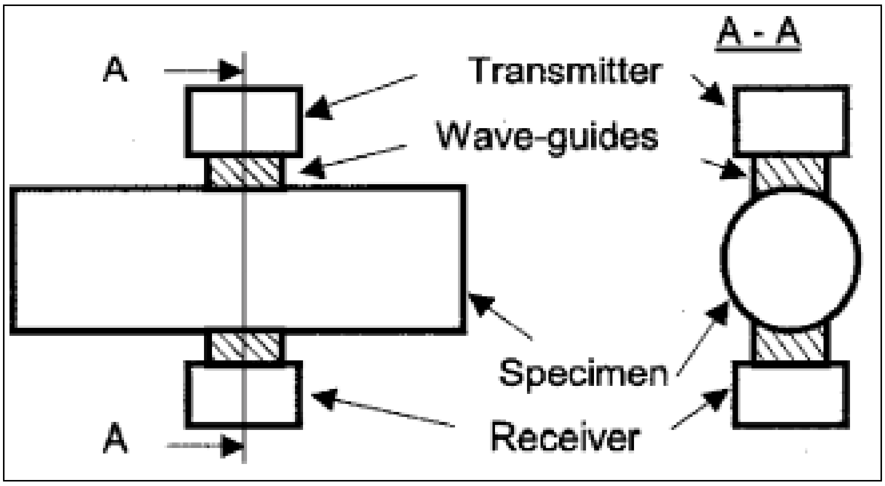

Figure 7-1. *Position of the transducers and wave-guides in the diametrical measurement, developed by LTU /Eitzenberger, 2003/.*

A pulse is generated and propagates from the transmitting transducer, through the sample to the receiver. An oscilloscope displays the measurements and a velocity can be calculated. P-wave velocity measurements are performed every 30º along the circumference of the cores between 0º and 180º to see if the rock sample is anisotropic or not. Based on the measured P-wave, the density and Poisson's ratio Young's modulus can be determined.

The results from the determination of Young's modulus from the vertical hole are presented in Figure 7-2 and from the horizontal hole in Figure 7-3. Young's modulus is 72.5 ±21 MPa and 56.2 ±26 MPa respectively in the vertical and horizontal bore hole.

The main conclusion from the laboratory program is that the local heterogeneity in the rock mass causes the majority of the scattering in Young's modulus. It was reported that the randomly distributed individual large feldspar crystals caused the differences in Young's modulus especially for the uniaxial testing of the 20 mm diameter cores. Considering the small dimensions of the strain gauges used during overcoring (10 mm) it is likely that the results from the overcoring partly suffer of the heterogeneity of the rock. This problem is supposed to be reduced by averaging a number of tests at one level.

Since Young's modulus is in the same range for the DDGS method as for the other methods, the measured strains must be significantly too high for the DDGS tests. There is a small difference in geology between the two nearby orthogonal boreholes and that may have an influence on the results of the DDGS measurements. The effect of the detailed mineralogical compositions can not be neglected.

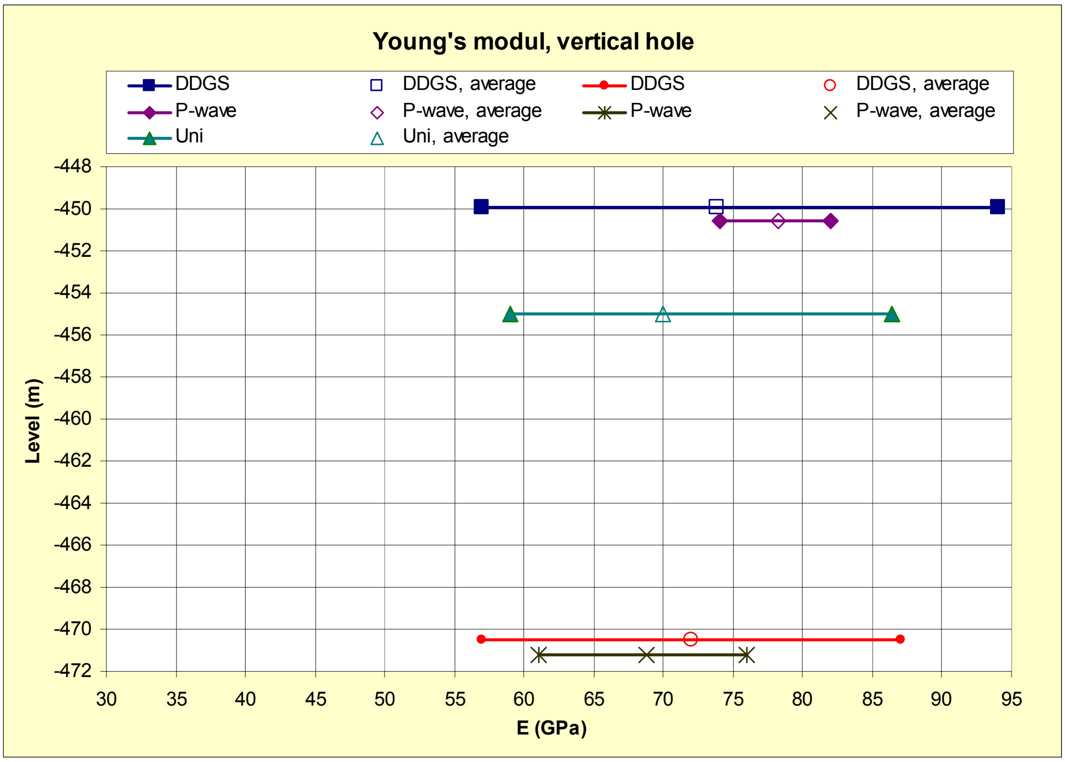

***Figure 7-2.** The results from the determination of Young's modulus in the vertical hole.*

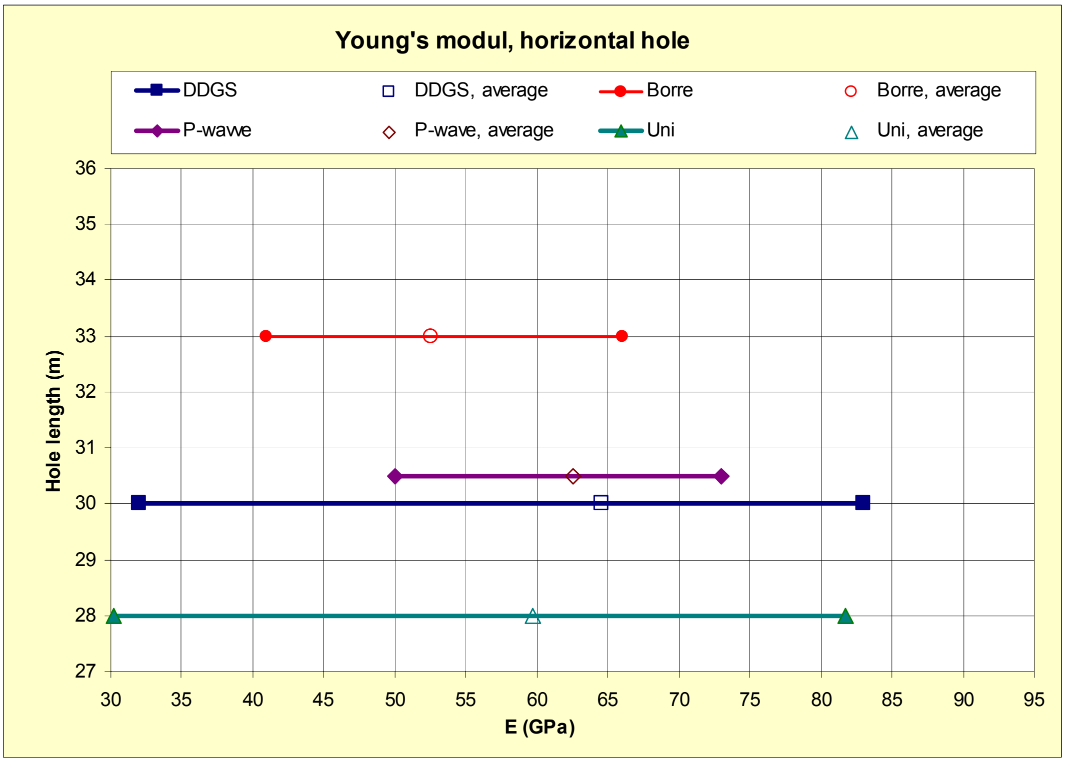

***Figure 7-3.** The results from the determination of Young's modulus in the horizontal hole.*

The degree of anisotropy, measured as E_{max}/E_{min}, from the different investigations, DDGS, Borre Probe, P-wave, and Uniaxial test, are summarized in Table 7-1.

Table 7-1. The average ratio E_{max}/E_{min} from the different investigations.

Hole	DDGS	Borre Probe	P-wave	Uniaxially	Rock type
Vertical	1.31		1.09	1.06	Diorite
Horizontal			1.03	1.21	Diorite
Horizontal	1.44		1.15		Fine grained granite
Horizontal	1.99	1.44	1.22		Foliated diorite

The effect of anisotropy may be important if the ratio of Young's modulus in different directions exceed 1.3–1.5 /Amadei and Stephansson, 1997/. The conclusion from the laboratory tests is that the rock is rather locally inhomogeneous than anisotropic. The results shown in Table 7-1 indicate that the measurements from the foliated diorite, in the horizontal borehole, may have been influenced by heterogeneity in the rock, that especially for the DDGS measurements. The calculation of the in situ stresses based on the field results could however not take account for the possible anisotropy caused by the local heterogeneity in the rock.

The results shown in Figure 7-2, Figure 7-3 and Table 7-1 also indicates the general problem of determination of Young's modulus in the overcore samples. The more heterogeneous the rock mass is and the higher the stress is the more difficult it is to determine Young's modulus, and consequently to calculate the state of stress.

The results from the field methods, DDGS and Borre Probe tests, indicate that the rock cores are anisotropic. The difference between the laboratory and field tests could be a scale problem. The length of strain gauges for the DDGS and Borre tests is about 10 mm and in the laboratory tests the scale is between 21 and 61 mm.

The conclusion is that the field measurements give the local variation in the core, mineral scale, and that the two laboratory tests give an average value for the core, core scale. The measured strain together with the corresponding measured Young's modulus must be measured in the same scale to give relevant stress results, and the calculated stress will probably be less scattered. Logically the calculations of the stresses should be based on the field measurements, though, it can give scattered results depending on the local variations in the core, while the determination of possible rock mass heterogeneity should be based on a laboratory test.

7.3 Geological structures

The geological influence on results such as changing in rock type, vicinity to fracture zones or changed orientation of fractures has been studied. The conclusions are that there is almost the same rock type, granodiorite or fine-grained granite, in all measuring sections, and that no major fracture zones are present in the vicinity, see chapter 3.3.

However there is a sub horizontal fracture (H1) in the vertical hole at –454.6 m, see Figure 3-8. The fracture may influence the results from the measurement in the horizontal hole and also some of the results from the hydraulic fracturing in the vertical hole, Figure 6-1 and Figure 6-2.

A study has been done to estimate the stress disturbance due to the presence of a nearby horizontal fracture, see Appendix 7. In the study it was assumed that a distinct and undulating fracture would cause the stress to locally be concentrated and relaxed. The result of the study propose that the influence is dependent on the distance between the fracture and measurement together with the waviness of the fracture (λ). The conclusion of the study is that one can expect that the effect of the presence of a fracture to become negligible if measurements are done at a distance that are larger than the waviness of the fracture, see Figure 7-4.

The distance between the horizontal hole and horizontal fracture is about 0.5 m and the estimate waviness is between 0.5 and 1.0 m. This implies that the fracture most likely influence the stress measurements in the horizontal hole.

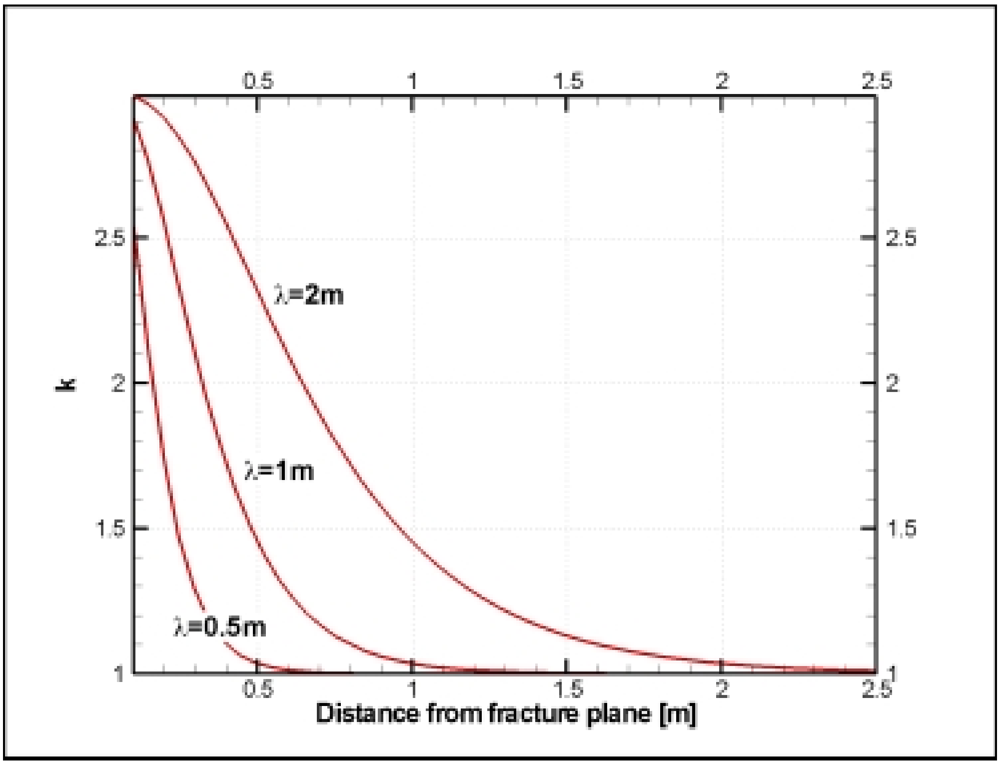

Figure 7-4. *Vertical stress intensification factor (k) vs. distance from the fracture plane under a contact area (λ is the waviness of the fracture).*

7.4 Measured strains and stress concentrations on the bottom of the borehole

The DDGS method resulted in higher stress magnitudes than the other two methods did, and also larger than earlier measurements in the area. The DDGS method calculates the stress using the measured strains. Therefore a study has been carried out to investigate the relationship between the strains and the stresses for the DDGS method, see Figure 7-5. High measured strains can be an effect of microcracking, see section 7.5. The microcracking give an additional non-elasticity for the strains on the hole bottom.

Based on the hypothesis that the measured strains are too high has the calculated stress in the vertical hole to be decreased by roughly 40% to be in the same range as for other measurements. This implies that the strains need to be 50% lower than measured, i.e. the measured strains are a factor 2 too large.

The measured strains by the DDGS method have also been compared with numerical calculations, see Appendix 8. The numerical simulations are done using the computer program "Examine 3D".

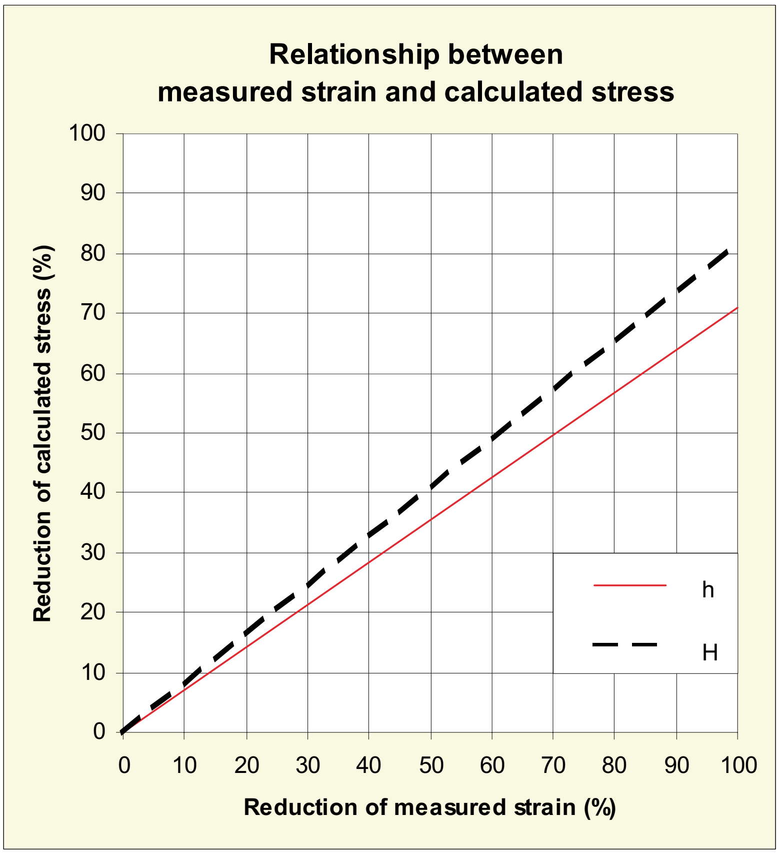

Figure 7-5. Reduction of the calculated stress as a function of the measured strain.

The simulations are made in two steps. The plain hole with the flat bottom is first simulated and then the state after the overcoring is simulated. The strain situation is recorded for the two different states. The results from the measured and simulated strains are shown in Table 7-2 for the vertical hole and in Table 7-3 for the horizontal.

Further, a comparison of the empirical stress concentration factors (a, b and c) is performed using the results from the DDGS and the simulations. The stress concentration factors for the simulations are determined using the calculated stresses in the bottom of the borehole and known relationships /Amadei and Stephansson, 1997/. The stress concentration factors for the DDGS test is estimated using the results presented in Appendix 3. In Table 7-4 the determined and estimated factors are shown.

According to Table 7-2 and Table 7-4 the correlation is fair between the measured and the simulated values for the vertical hole. On the contrary the correlation for the horizontal hole is poor, see Table 7-3 and Table 7-4. The difference may be explained by the measurements in the horizontal hole being affected by some factors, e.g. nearby fractures, anisotropy in the rock mass, micro cracks or some other source of error.

Table 7-2. Measured and simulated strains from the DDGS test in the vertical hole. The strains are also normalised against the stresses in the same direction.

	ε_h (με)	ε_H (με)	ε_h/σ_h (με/MPa)	ε_H/σ_H (με/MPa)
DDGS, average	132	468	5.9	12.7
Numerical calculation	138	395	6.9	13.2

Table 7-3. Measured and simulated strains from the DDGS test in the horizontal hole. The strains are also normalised against the stresses in the same direction.

	ε_V (με)	ε_h (με)	ε_V/σ_V (με/MPa)	ε_h/σ_h (με/MPa)
DDGS	407	–307	12.5	–24.8
Numerical calculation	70	–148	4.7	–7.4

Table 7-4. The determined and estimated empirical stress concentration factors (a, b and c) from the measured and simulated borehole.

	a	b	c
DDGS	1.35	–0.05	–0.65
Examine3D, vertical hole	1.44	0	–0.74
Examine3D, horizontal hole	2.44	0	–1.54

7.5 Microcracking

The deeper investigation on microcracking is carried out in two ways. The first study is theoretical and based on the results from the numerical analysis. The other is based on the observations from the microscopy of the cores.

7.5.1 Theoretical study

Former analyses have been used as references to the theoretical study. For example Posiva Oy made a desktop study on the stress and strain behaviour in crystalline rock /Hakala, 1996/. The study shows that microcracks will be initiated in rock samples when the stress exceeds 15 to 60% of the uniaxial tensile strength. Results from experiments on "Olkiluoto" gneiss from Finland showed that microcracks were initiated when the stresses exceeded 47 to 56% of the tensile strength /Hakala and Heikkilä, 1997/. Further experiments on "Äspö" diorite showed that microcracks were initiated when the stress exceeded 60–70% of the tensile strength /Nordlund et al, 1999/.

The analysis of eventual microcracking before the overcoring is based on the results from the Examine 3D numerical anayses, see chapter 7.4 and Appendix 8. The analysis is done in two ways;

1. using the, so called, "strength factors" (the relation between the present stress and the strength) that is a direct result from the simulation, see Table 7-5 and
2. using the major and minor principal stresses from the simulations as input to a Mohr-Coulomb diagram together with the failure criterium, based on the used strength parameters, see Table 7-6.

Table 7-5. Relationship between maximal stress and the tensile strength, based on the "strength factor".

The horizontal bore hole		The vertical bore hole	
Edge of the hole bottom	Centre of the hole bottom	Edge of the hole bottom	Centre of the hole bottom
50%	25%	65%	55%

Table 7-6. Relationship between the rupture line and the Mohr-Coulomb circles of stress.

The horizontal bore hole		The vertical bore hole	
Edge of the hole bottom	Centre of the hole bottom	Edge of the hole bottom	Centre of the hole bottom
42%	19%	59%	50%

The following conclusions can be drawn from Table 7-5 and Table 7-6:

- The probability for microcracks can be considered as small for the horizontal hole, mostly depending on the favourable directions of the principal stresses. The principal stresses in the plane of the hole bottom is almost equal, 15 and 20 MPa respectively.
- The occurrence of microcracks is most likely in the vertical hole. There are larger difference in the principal stresses in the plane of the hole bottom, 20 an 30 MPa respectively.

7.5.2 Laboratory study

The laboratory study was performed by microscopy investigations, see Appendix 9. The investigations were done on four overcoring cores, two from the vertical hole and one from the horizontal hole. Two microscopy plane were arrange for each core, one plane across the hole bottom and one perpendicular to the hole bottom.

Two dominant microcrack sets were observed from the investigations. One set was parallel and near the bore hole bottom and one set was perpendicular to the bottom and located a bit away from the hole bottom. The perpendicular set was also in agreement with observations from the measurement of the P-wave velocity, i.e. the orientations of the microcracks was parallel to the highest magnitude of the P-wave velocity.

The observed microcracks could have been created in different situations, such as:

1. under the work and preparation of the hole bottom in connection with DDGS test, the work and preparation give high stress concentrations at the hole bottom,
2. created by the rock mass anisotropy,
3. when the core is moved from a high stress situation in the rockmass to an unloaded situation.
4. original from earlier geological processes.

Which alternative or alternatives that are the right is difficult to determine, but the first alternative seem to be the most possible for the set of parallel microcracks near the bore hole bottom. How large impact the microcracks make on the strain measurements is difficult to estimate.

7.6 Other factors that may influence the results

An extensive auditing has been carried out about the results from the stress measuring. The procedures for the actual stress measurements have also been scrutinised, especially for the DDGS that result in the highest stresses, see Appendix 6. The internal strain check, in the DDGS method, did not show that any technical problems had occurred during the tests. Assumptions for the DDGS analysis have also been checked, i.e. error in the assumption of the vertical stress in the vertical hole, Young's modulus and Poisson's ratio. Only minor changes will occur when the assumptions are varied.

/Hakami et al, 2002/ studied the difference in the stress level between the rock units in the Äspö area, using the distinct element code 3DEC. The blocks are defined by the surrounding fracture and shear zones. Due to the geometry, the zones form a wedge, higher stresses are simulated in the block where the measurements are done than in the surrounding blocks at the same level, –420 to –480 m.

In a study by /Rutqvist et al, 2000/ it is showed that the general theory for calculating the major horizontal stress from the hydraulic fracturing suffer from uncertainties in the assumptions; like a linear elastic, homogenous, and isotropic medium together with the fracture reopening. It is probable that the major horizontal stress, determined from the hydraulic fracturing, may be somewhat underestimated when σ_1/σ_3 is close to, or higher than a factor 3.0.

The length of the strain gauges in relationship to the grain size is critical. It is recommended that the length of the gauge is at least 10 times larger than the grain size. The visual inspection of the cores shows that there are large grains of feldspar, up to 20 mm in diameter, see Appendix 6. This shall be compared to the length of the strain gauge, 10 mm.

8 Experiences from the drilling and stress measurement methods

8.1 Drilling Experiences

The chosen drilling standard, HQ-3 and the wire-line system is not common in Scandinavia. There was only one drilling contractor that was able to offer the equipment for the planned test period and hence do the work. The costing was about 50% higher for the HQ-3 method than for the more conventional TM 101, a method with roughly the same bore diameter.

The establishment of the vertical hole started the 2nd of April and the drilling started the 25th of April, see Appendix 1. The establishment of the equipment lasted for about 11 working days, whereas one working day is 10 hours for two drillers. The long time of the establishment was caused by the following reasons:

- Ground stability causing problem to bring the drilling equipment to the right direction, that resulted in a new adjustment of the equipment and casting of a concrete slab.
- Soft rock at the collaring.
- Drilling and gluing of casing.
- Delayed delivery of parts to the drilling equipment.

During the 25th and 26th of April 31 m was drilled and additionally 71.5 m were drilled between the 28th of April and 1st of May. The speed of the drilling was about 2.1 m per hour including the handling of the core. The core was retrieved using a wire line system.

After the rock stress measurements with the DDGS method in the vertical hole the drilling equipment was moved to the area for the horizontal hole. The establishment of the drilling equipment was accomplished without any troubles during two 2 days, totally 2 times 26 hours. During the 21st and 22nd of May 29 m was drilled with an average speed of 2.6 m per hour including handling of the core.

8.2 Stress measurements with DDGS

The testing success rate was lower than expected. Only seven measurements were successful out of 24 attempts. The cause for the large amount of failures originated in many different factors, see Appendix 3

The main reason was that the gluing between the bottom of the hole and the gauges failed during the overcoring. The first thought was that the salt/brackish water in the borehole affected the hardening of the glue. Both SKB and AECL performed tests after the stress measurements and neither could find any evidence that the salt should have affected the hardening of the glue. The glue has been tested in Canada earlier with satisfaction, and the reason why there was a problem at Äspö is not clarified yet.

Further reasons to the large amount of failures can be due to running-in problems between the different conditions in Canadian and Scandinavian such as the fitting of the measuring equipment in the drilling equipment, together with battery and power problems for the data logger. Finally two of the failures were due to rock conditions.

Despite of the problems and the failures there are several positive experiences such as; the wire line technique (ductile and fast handling), the short time for each test (two to three tests per working day) and a well-developed checklist.

8.3 Stress measurements with Borre Probe

The tests were only performed in the relatively short horizontal borehole. This made the measuring simpler than it would have been in long water filled vertical hole. Three measurements were successful out of four gluing trials and six pilot holes. In all the measurements with the Borre Probe lasted for seven days.

The minor problems that occurred during the tests were related to the geological circumstances (fracture and differences in rock type), and stand-by time for delivery and adjustments of the equipment. Further, the biaxial cell did not work with satisfaction and new tests had to be done by SwedPower before the analysis of the measuring could be completed.

8.4 Stress measurements with Hydraulic fracturing

The tests in the two boreholes were performed without any troubles. The 12 measuring sections, including hydrofracting test and impression packer test, was performed during four days. Before the measuring work started some core samples were sent to the contractor. Laboratory tests, such as ultrasonic, density, fracture roughness and hydrofrac, were performed on the cores to predict the fissuring process. Further the cores were examined to decide which sections that were suitable for tests, i.e. a section of 1 m without any fractures.

A well prepared, with the investigated samples, and experienced contractor was one of the major reasons for the problem-free measuring work.

8.5 Laboratory tests

The results, stresses and Young's modules, from the three in situ stress measurement methods rose more questions than answers. Which illustrate the complexity to determine the in situ stresses in a rock mass. To understand the difference in results and answer the questions, it was necessary to do deeper investigations as laboratory tests and theoretical calculations such as

- geological structure model,
- analysis of the near fracture influence,
- P-wave measurements,

- uniaxial tests on small cores from the HQ-3 core,
- theoretical analyses of the hole bottom (theoretical strains, stress concentrations and microcracking),
- auditing of DDGS measurements results and assumptions in the DDGS analyse and
- microscopy investigations on the cores.

These investigations gave a better understanding of the results and answered some of the questions. Some investigations did not give any new information, that had not been confirmed earlier. The most important investigations in this case was the P-wave measurement, the geological analysis (including the microscopy and visually inspections of the cores) and the external auditing. That those investigations were the most important may have been an artefact on the chronological order of the studies.

A disadvantage, with investigations after the measurement tests, is that it takes time and that the cores have to be sent away.

9 Conclusions and Discussion

The following conclusions are based on the actual site conditions; medium grained crystalline rock and high stresses. These conditions are expected for the planned Site Investigations for a deep repository for spent fuel in Sweden. Other conditions that may cause other conclusions have not been considered.

- The following stress state is obtained at the target volume at about –455 m. The minimum horizontal stress is between 10 and 13 MPa, which is lower than the theoretical vertical stress. The maximum horizontal stress is 24 ±5 MPa, most likely within the upper range, e.g. compare the findings of /Andersson et al, 2002/ where it is expected that the major principle stress rather should be in the range 25–30 MPa. The vertical stress is between 15 and 20 MPa, which most probably only is a local value due to the presence of a nearby fracture. These results exclude the measured stresses from the DDGS tests.

- The local disturbance of the stress field in the rock mass, due to discontinuities has been demonstrated. This also indicates one of the problems with stress measurements in boreholes.

- In the area with significant anisotropic stress conditions all the tested methods were able to determine the orientation of the principal major horizontal stress within ± 10°.

- The microscopy investigations confirm two sets of microcracks in the overcored core. One set was parallel and near the bore hole bottom and one set was perpendicular to the bottom and located a bit away from the hole bottom. The later set agree with results from P-wave velocity measurements perpendicular to the core axis.

- The results from the overcoring may be influenced by microcracks, causing additional non-elastic strains, see /Martin and Christiansson, 1991/. Only the results from the DDGS seem to have been influenced, indicating that the hollow cylinder of a 3D stress cell may be less sensitive for stress induced sample disturbance than the 2D Doorstopper cell. The explanation could be that the 3D stress cell is measured over a larger volume and a more simple geometry than the 2D Doorstopper cell.

- The determination of Young's modulus in a medium grained crystalline rock with heterogeneity may not be trivial using core samples. The results from the determination influence the calculated stresses.

- Hydraulic fracturing most likely measures the most correct value of the minimum horizontal stress, provided that the induced fracture is aligned with the borehole.

- If the rock behaves reasonable elastic the overcoring methods provide stress magnitudes with an uncertainty of 15–20%. It seems likely that the overcoring methods overestimate the stress magnitudes at large depth, due to the influence of microcracks.

- A good understanding of the geology in the scale from mineralogical heterogeneity to possible discontinuities is important for the judgement of the reliability of the results of the stress measurement. The difficulty in understanding all geological variabilities in the vicinity of a borehole must however add a general uncertainty to the stress measurement results.

The following conclusions have direct impact on the stress measuring program that will be carried out in the planned SKB Site Investigation program for the deep repository for spent fuel:

- Stress measurements at large depth with high demands on the results ought to be carried out with both hydraulic fracturing and overcoring at the same levels in the borehole. First, the measured minor horizontal stress magnitude and the maximum horizontal stress orientations should be compared. Then the maximum stress magnitudes should be compared, considering the possible risks that some of the assumptions for the methods may not be fulfilled. For example Non-elastic strains, caused by microcracking, may over-estimate the stress magnitude for the overcoring and if the stress anisotropy ratio is larger than 3, the calculation of the major horizontal stress may under-estimate the stress magnitude for the hydraulic fracturing method.

- The results from the overcoring measurements scatter significantly. This is due to the feldspar crystals that are too large for the used strain gauges. However, the actual measurements strengthen the rule of thumb that an average of 3–5 successful tests provides an acceptable design value.

- The measurement of the p-wave velocity on core samples in various directions perpendicular to the core has proven to be a quick and reliable method to indicate anisotropic or non-elastic behaviour. Laboratory testing is however required to determine the degree of anisotropy or sample disturbance.

- It may be able to determine design values at large depth with an uncertainty within 15–20%, as long as the site conditions are in reasonable agreement with the assumptions for the used stress measurement method.

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Appendix 1

Table. Summary of stages of the drilling of the vertical borehole.

Moment	Time period	Comments
Establishment of the drilling machine	2 - 4 April	Trouble: Aligning and concrete slab and also anchorage of frame to poor rock
Aligning the machine	9 - 10 April	
Drilling, gluing of casing and re-gluing	11 April, 17 April	Drilling 2,0 m with ϕ 131 mm
Establishment of HQ3 drilling	18 - 19 April	
Waiting time for equipment	23 - 24 April	Inner tube rod
Drilling HQ3	25 - 26 April	12.75 m + 18.23 m
Waiting time for equipment	27 April	Coring bit
Drilling HQ3	28 April - 1 May	14.77 m + 20.83 m +24.04 m + 11.83 m
Drilling HQ3 and measuring of borehole curvature	7 May	2.55 m
Stress measurement AECL, level -450 m	8 - 12 May	6 overcorings, totally 1.01 m
Drilling HQ3	12 - 14 May	5.72 m +12.01 m + 2.26 m
Stress measurements AECL, level -470 m	14 - 18 May	8 overcorings + 2 commenced, totally 1.32 m
Dismounting of equipment at vertical hole	18 May	
Pressure build up test	22 May	
Bips logging	7 June	
No activity in the hole	June - September	
temperature measuring	4 October - 10 October	At level -400 m
Hydraulic fracturing	11 - 12 October	Between level -448.5 to -466.5 m
Dismounting of equipment at vertical hole	12 October	

Appendix 2

Table. Summary of stages of the drilling of the horisontal borehole.

Moment	Time period	Comments
Establishment and aligning of the drilling machine and gluing of casing	19 - 20 May (18.5 hours shift)	Drilling 2.12 m with ϕ 131 mm
Establishment of HQ3 drilling	20 May	
Drilling HQ3	21 - 22 May	21.01 m + 5.49 m
Stress measurements AECL	28 - 31 May	8 overcorings, totally 2.34 m (0.3 m due to fracture)
Stress measurements SwedPower, including measuring of borehole curvature	31 May - 4 June	3 overcorings, totally 2.41 m
Waiting time for equipment	5 June	Pilot coring bit
Stress measurements SwedPower	6 June	1 overcoring, totally 2.59 m (1.0 m due to fracture)
Dismounting of equipment	6 - 7 June	
Bips logging	7 June	
No activity in the hole	June - September	
Hydraulic fracturing	9 - 10 October	Between hole length 21,5 to 32,0 m
Dismounting of equipment at horizontal hole	10 October	

Appendix 3

Overcoring Rock Stress Determinations Using the Deep Doorstopper Gauge System in Boreholes KA2599G01 and KF0093A01 at the Äspö Hard Rock Laboratory

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2001 July 25

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Overcoring Rock Stress Determinations Using the Deep Doorstopper Gauge System in Boreholes KA2599G01 and KF0093A01 at the Äspö Hard Rock Laboratory

1.0 Introduction

AECL was contracted by SKB to conduct in situ rock stress measurements with the Deep Doorstopper Gauge System (DDGS) in Boreholes KA2599G01 and KF0093A01 at the Äspö Hard Rock Laboratory near Oskarshamn Sweden. Testing was performed under the terms and conditions specified in SKB Purchase Order 4744 of 2001 March 09.

2.0 Field Testing

In Borehole KA2599G01 it was intended that three repeatable overcore tests were to be obtained at about 110-m-depth in and three repeatable overcore tests were to be obtained about twenty metres deeper. The sub-vertical borehole was collared near the –340 m shaft access level beside the main access tunnel and was plunging 80° at the borehole collar. The borehole azimuth at the test depths was about 325°. Testing in borehole KA2599G01 took place between May 7 and May 18 2001. The DDGS was used in this borehole.

The overcore testing in Borehole KF0093A01 was to involve three repeatable overcore tests conducted at about 30-m-depth. The borehole was collared near the base of the main access tunnel below the -450-m shaft access level. The borehole was plunging -3° at the borehole collar, and the azimuth was 130°. Testing in borehole KF0093A01 took place between May 28 and May 31 2001. Because this borehole was sub-horizontal, with minimal head pressures, the doorstopper gauges were installed attached to the Intelligent Acquisition Module in a low-pressure aluminum housing. Installation was accomplished using quick-connect installation rods. Orientation was set using a down-hole electronic clinometer, with Gauge 3 of the Doorstopper Gauge rosette being installed vertically.

3.0 DDGS Overcore Testing in KA2599G01

DDGS tests were attempted at five locations between 107.29 m and 107.95 m, and three successful tests were obtained. Only one successful test was obtained at the deeper test zone in KA2599G01, despite attempting tests at eight locations between 128.11 m and 129.28 m. Section 3.1 will discuss the test results. A discussion on the various causes of test failures is contained in Section 5.0.

A summary of the DDGS overcore tests attempted is contained in Table 1. In two instances (107.79 and 128.45 m) more than one test installation was performed because there was a trigger failure initially, which prevented the release of the Deep Doorstopper Gauge in its high-pressure housing. In these cases it was possible to reflatten the borehole bottom and attempt a subsequent installation.

Test Depth (m)	Date	Remarks
107.29	2001 May 09	Successful Test
107.44	2001 May 10	Data Logger Improperly Programmed
107.63	2001 May 10	Glue Failure
107.79	2001 May 11	Trigger Failure
107.79	2001 May 11	Successful Test
107.95	2001 May 12	Successful Test
128.11	2001 May 14	Glue Failure
128.28	2001 May 15	Successful Test
128.45	2001 May 15	Trigger Failure
128.45	2001 May 16	Trigger Failure
128.45	2001 May 16	Rock Fragments at Borehole Bottom
128.61	2001 May 16	Broken Battery Clip in Data Logger
128.78	2001 May 17	Glue Failure
128.97	2001 May 17	Glue Failure
129.13	2001 May 17	Glue Failure
129.28	2001 May 18	Glue Failure

Table 1. Summary of DDGS Overcore Test Depths for Borehole KA2599G01

The strain versus time plots for the four successful tests are shown in Figures 1 to 4.

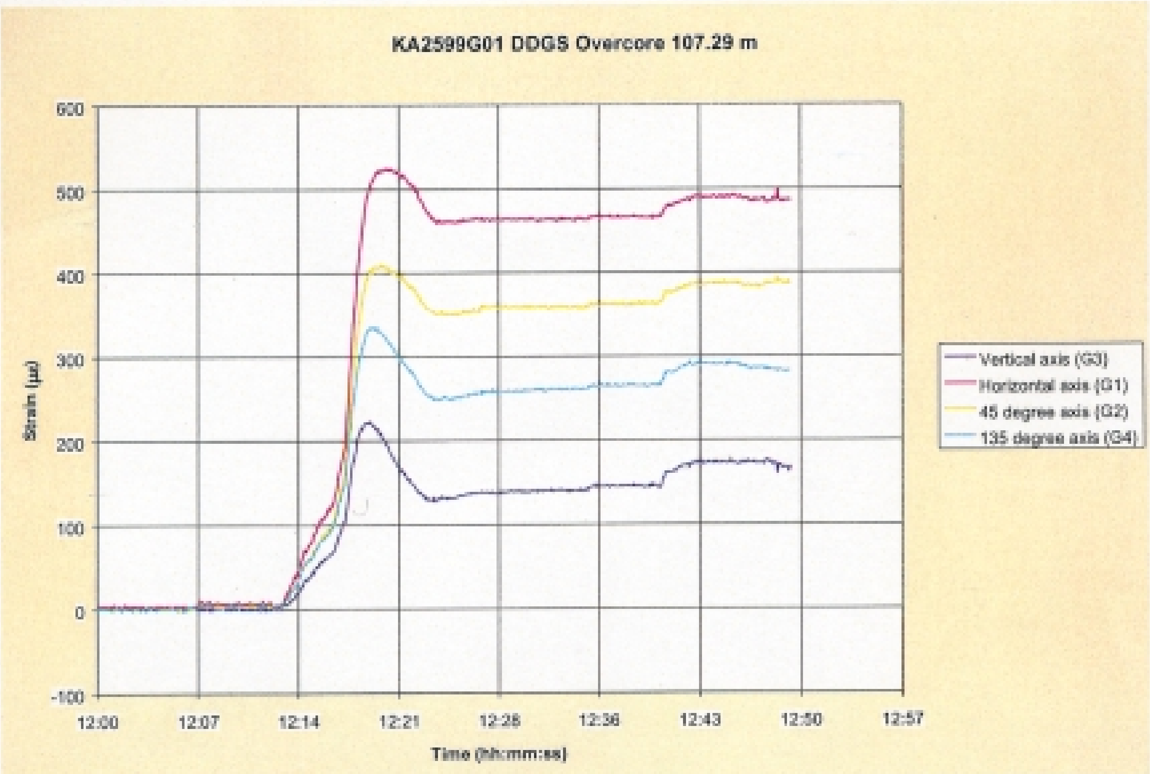


Figure 1. Overcore Test Data for KA2599G01 107.29 m.

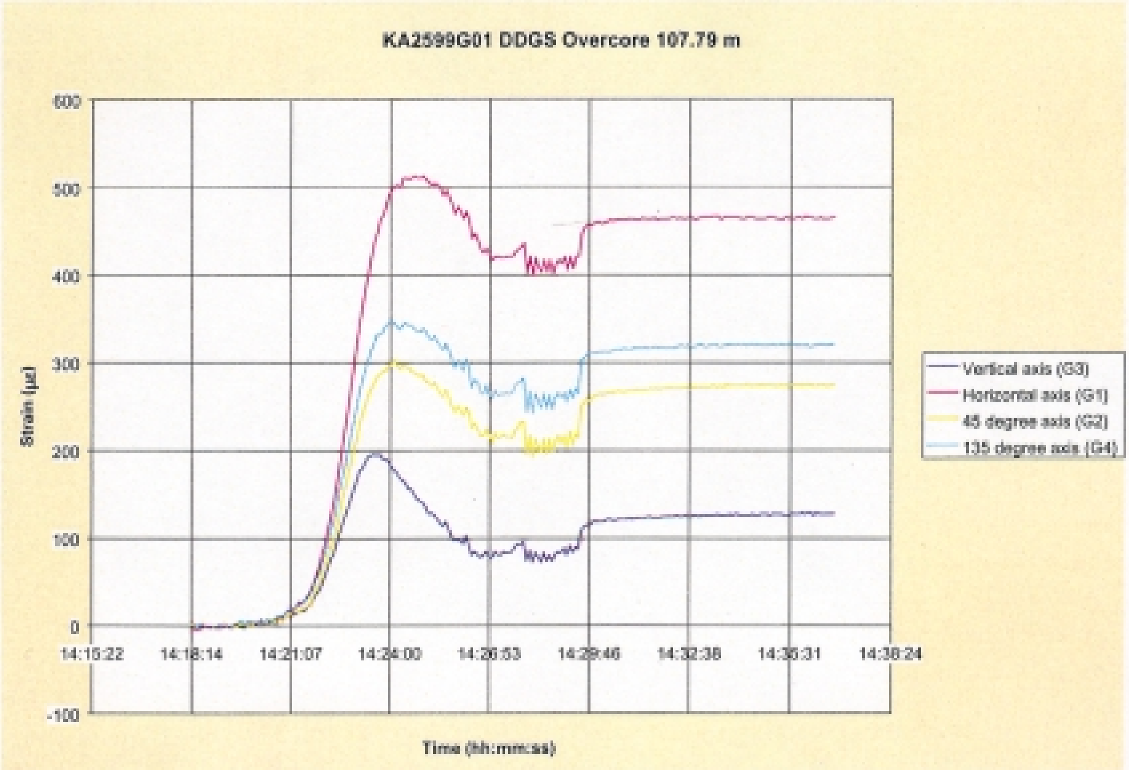


Figure 2. Overcore Test Data for KA2599G01 107.79 m.

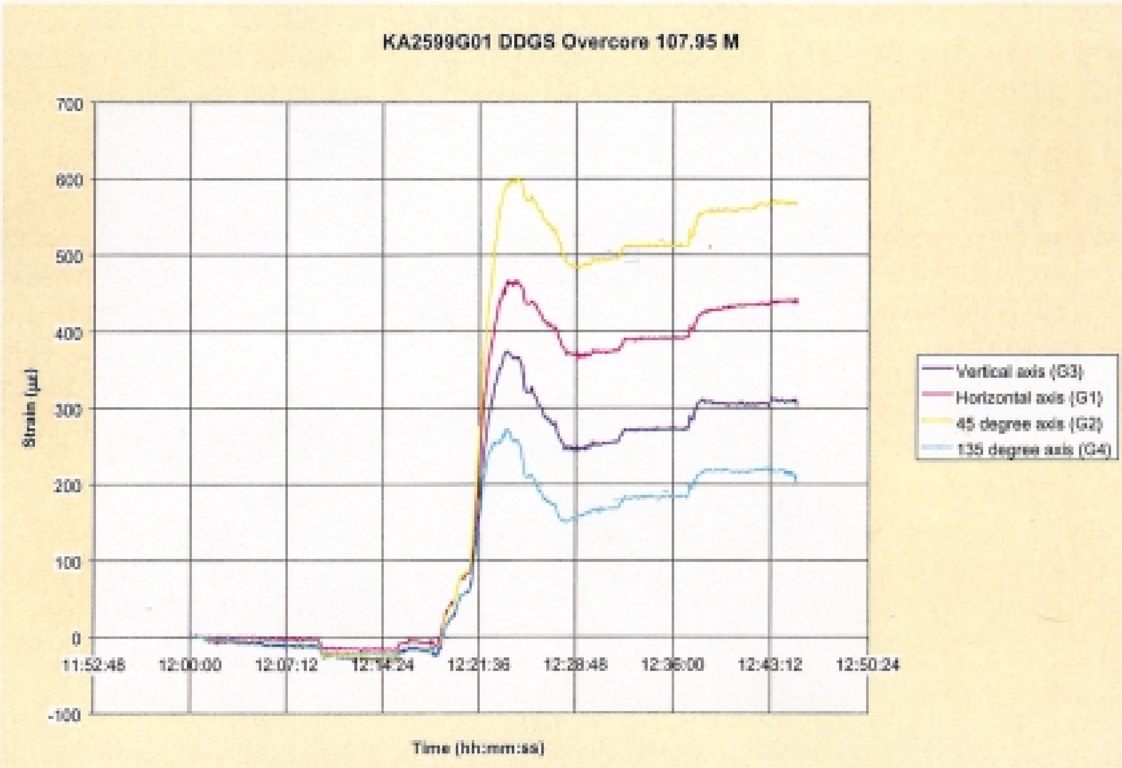


Figure 3. Overcore Test Data for KA2599G01 107.95 m.

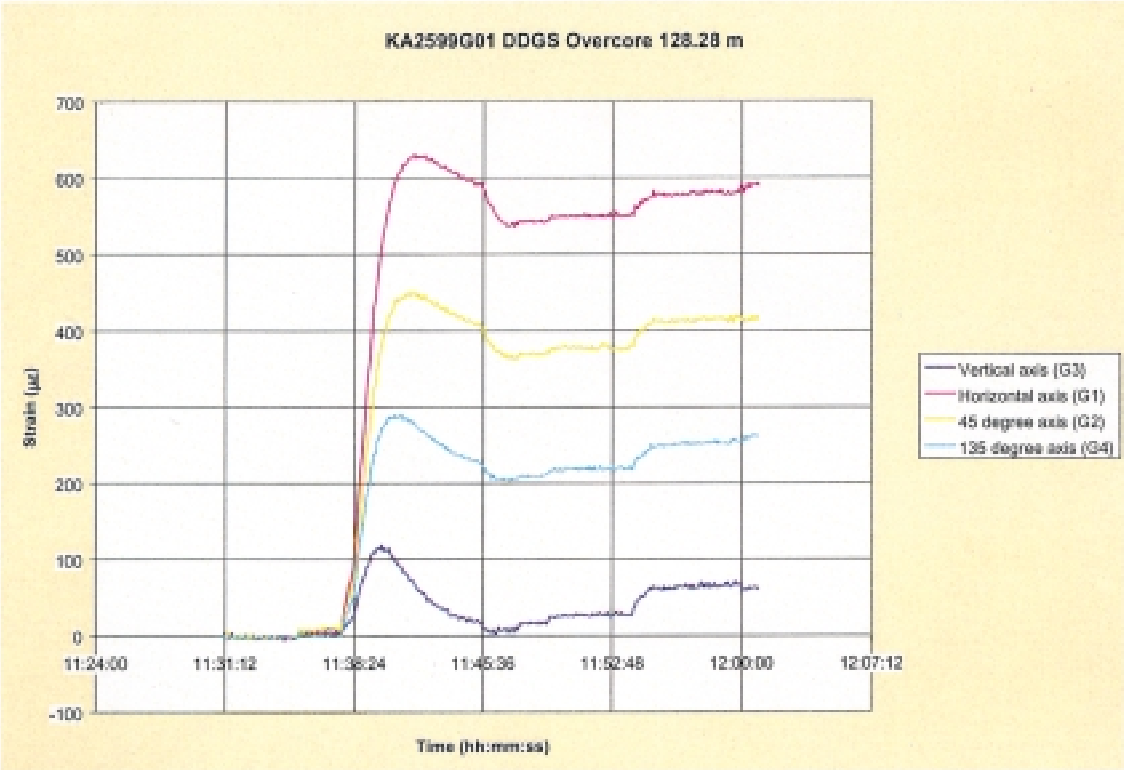


Figure 4. Overcore Test Data for KA2599G01 128.28 m.

The pattern of these four tests is relatively consistent, with the maximum stable final strain in each test varying from about 450 to 550 με, and the minimum stable final strain varying from 30 to 180 με. This would indicate a relatively large variation between the maximum and minimum principal stresses in the plane of measurement (assuming that the rock is relatively isotropic).

Biaxial Pressure Tests were conducted on each test specimen, using a hydraulically actuated Hoek cell. Unfortunately, the signal conditioner for the pressure transducer failed after the first test. Subsequent tests were conducted without the pressure transducer, producing time versus pressure (read from a pressure gauge) data, which were then converted manually to strain versus pressure and ultimately modulus versus pressure. Modulus versus pressure plots for the four tests in KA2599G01 are shown in Figures 5 to 8.

The modulus was variable from one test to another. In some of the tests, the IAM was quite unstable. These tests were repeated with no improvement. There appeared to be a problem with the power supply and/ or grounding as we experienced small shocks and internal "humming/ vibration" from all of the three IAMs. This problem is discussed in Section 5.0.

A summary of the measured moduli is contained in Table 2. The modulus ranges from an average of 63 GPa at 107.95 m to 88 GPa at 128.8 m. The test samples from 107.29 m

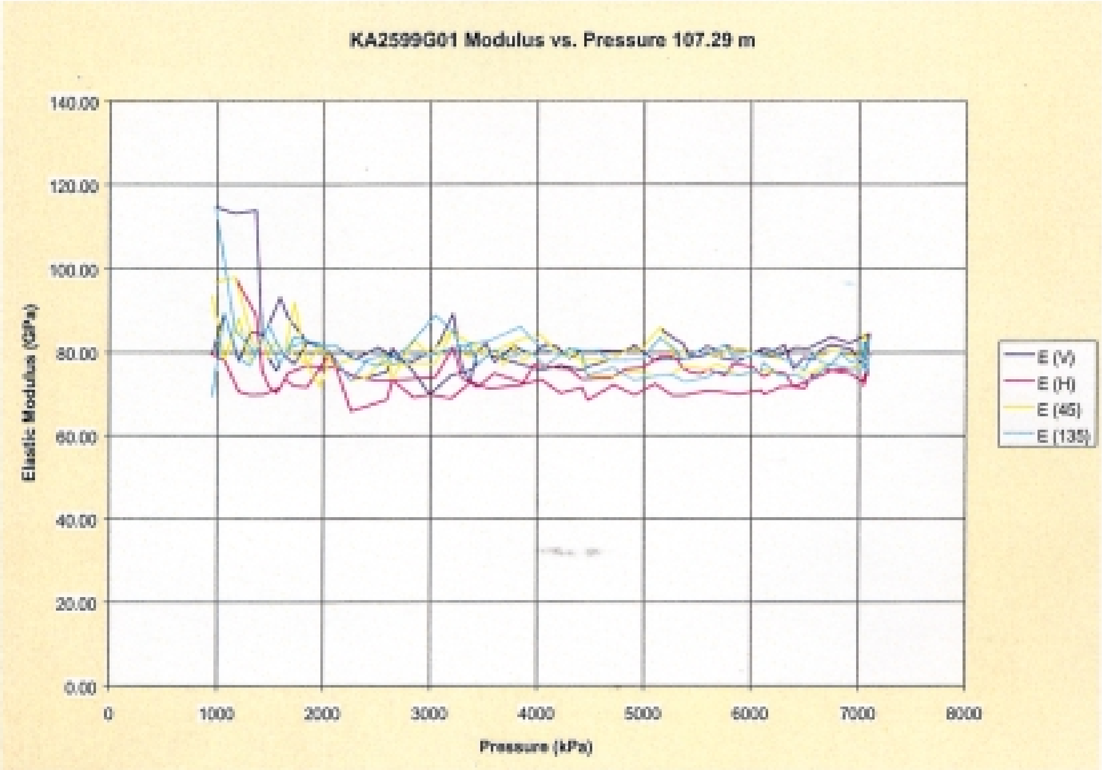


Figure 5. Biaxial Pressure Test Results for KA2599G01 107.29 m

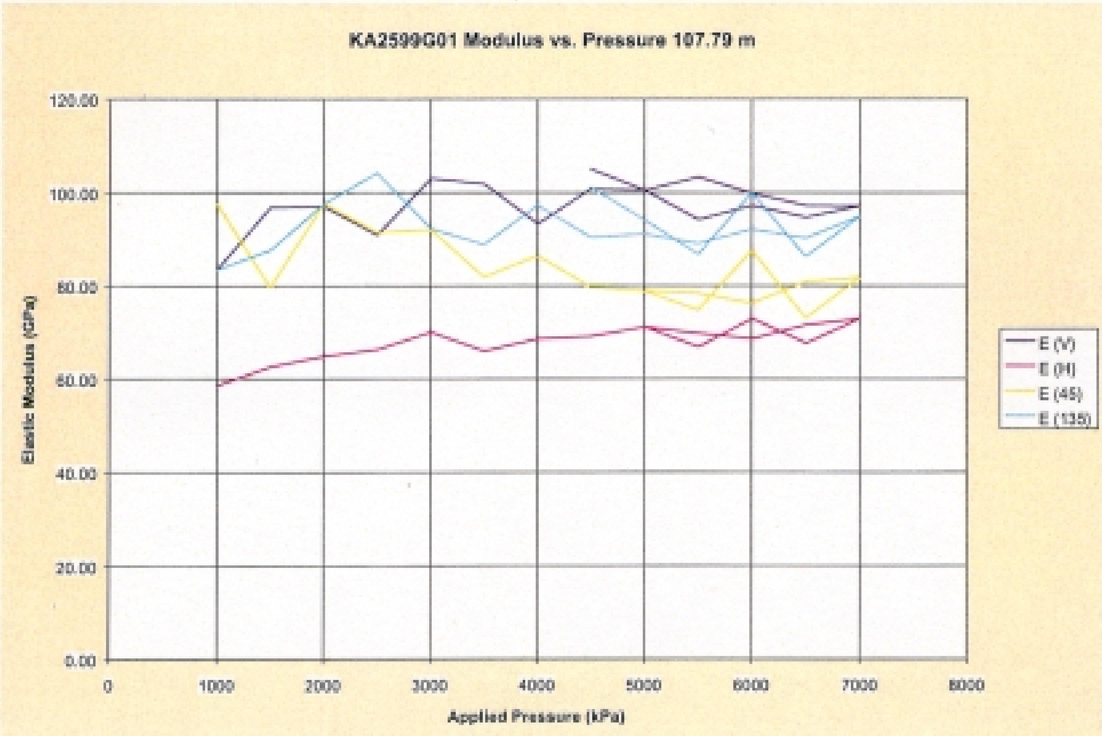


Figure 6. Biaxial Pressure Test Results for KA2599G01 107.79 m

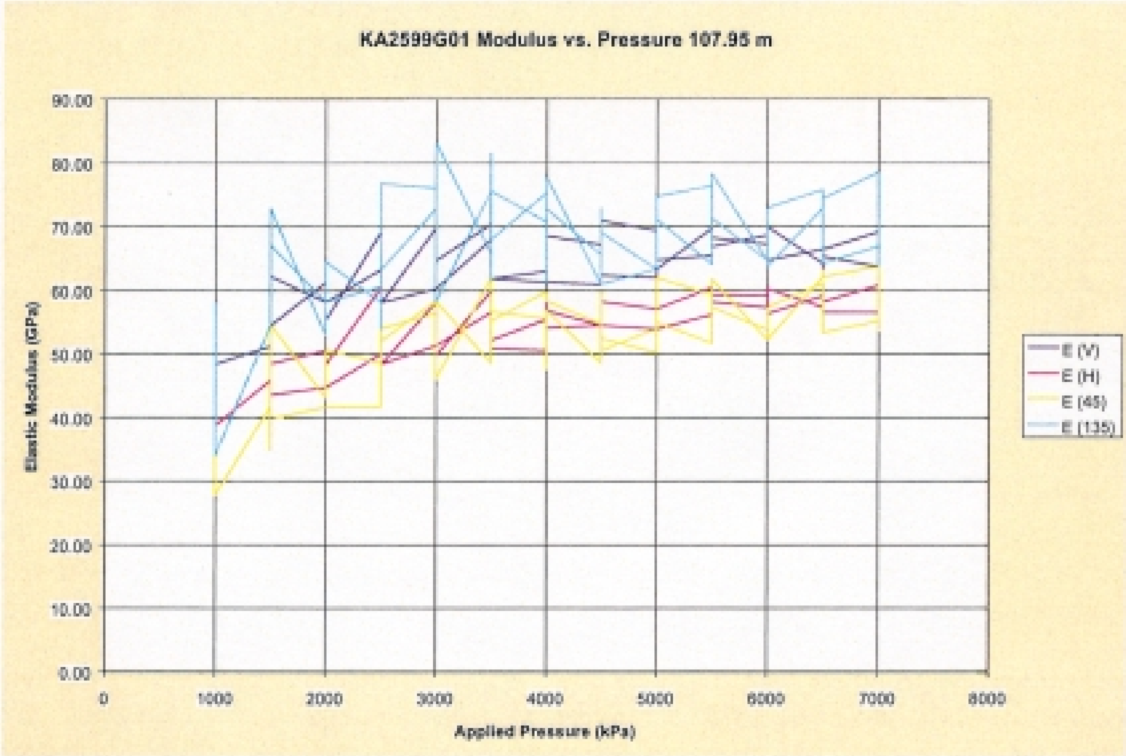


Figure 7. Biaxial Pressure Test Results for KA2599G01 107.95 m

Figure 8. Biaxial Pressure Test Results for KA2599G01 128.28 m

Depth (m)	E Gauge 3 (V) (GPa)	E Gauge 1 (H) (GPa)	E Gauge 2 (45) (GPa)	E Gauge 4 (135) (GPa)	E Ave. (GPa)	Anisotropy
107.29	80	72.5	77	75	76	Low
107.79	94	68	76	92	83	Moderate
107.95	66	59	57	69	63	Low
128.28	57	120	87	*	88	High

*Gauge 4 was not functioning for this test. The average is of the remaining gauges.

Table 2. Summary of measured elastic moduli for Borehole KA2599G01

and 107.95 m exhibited fairly isotropic behaviour, while the test at 107.79 m was moderately anisotropic and the test at 128.28 m was highly anisotropic.

Although there is a significant variation in the average moduli and degree of anisotropy between the four tests, there was no difference in the overcore samples visible to the naked eye.

The calculation of in situ stresses in the plane perpendicular to the borehole axis was done using the methodology outlined in Thompson and Martino, 2000. The strain values were obtained from the overcore data. The modulus used for each test was the average as determined from the biaxial pressure test on that overcore sample. The Poisson's Ratio was assumed to be 0.25, and the stress perpendicular to the plane of the gauges was calculated from the weight of overburden.

The calculated principal stresses in the plane perpendicular to the test borehole axis are contained in Table 3.

Test Depth	P (MPa)	Q (MPa)	σ_z (MPa)	Azimuth of P
107.29 m	37.0	22.3	12.1	111°
107.79 m	39.1	22.8	12.1	133°
107.95 m	34.1	21.6	12.1	135°
128.28 m	44.6	20.4	12.3	122°

Table 3. Summary of Calculated Principal Stresses for Borehole KA2599G01

4.0 DDGS Overcore Testing in KF0093A01

DDGS tests were attempted at eight locations between 28.87 m and 31.05 m, and three successful tests were obtained in the final three attempts. A discussion on the various causes of test failures is contained in Section 5.0.

A summary of the DDGS overcore test attempts is contained in Table 4.

Test Depth (m)	Date	Remarks
28.87	2001 May 28	Glue Failure (HBM X-60)
29.25	2001 May 28	Glue Failure (HBM X-60); broken IAM battery clip
29.42	2001 May 29	Glue Failure (Versilok; switch to fresh drill water)
29.92	2001 May 29	IAM failed to download data (Good glue bond)
30.07	2001 May 30	Dimple on borehole bottom caused gauge to break
30.23	2001 May 30	Successful Test
30.89	2001 May 30	Successful Test
31.05	2001 May 31	Successful Test

Table 4. Summary of DDGS Overcore Test Depths for Borehole KF0093A01

The strain versus time plots for the three successful tests are shown in Figures 9 to 11.

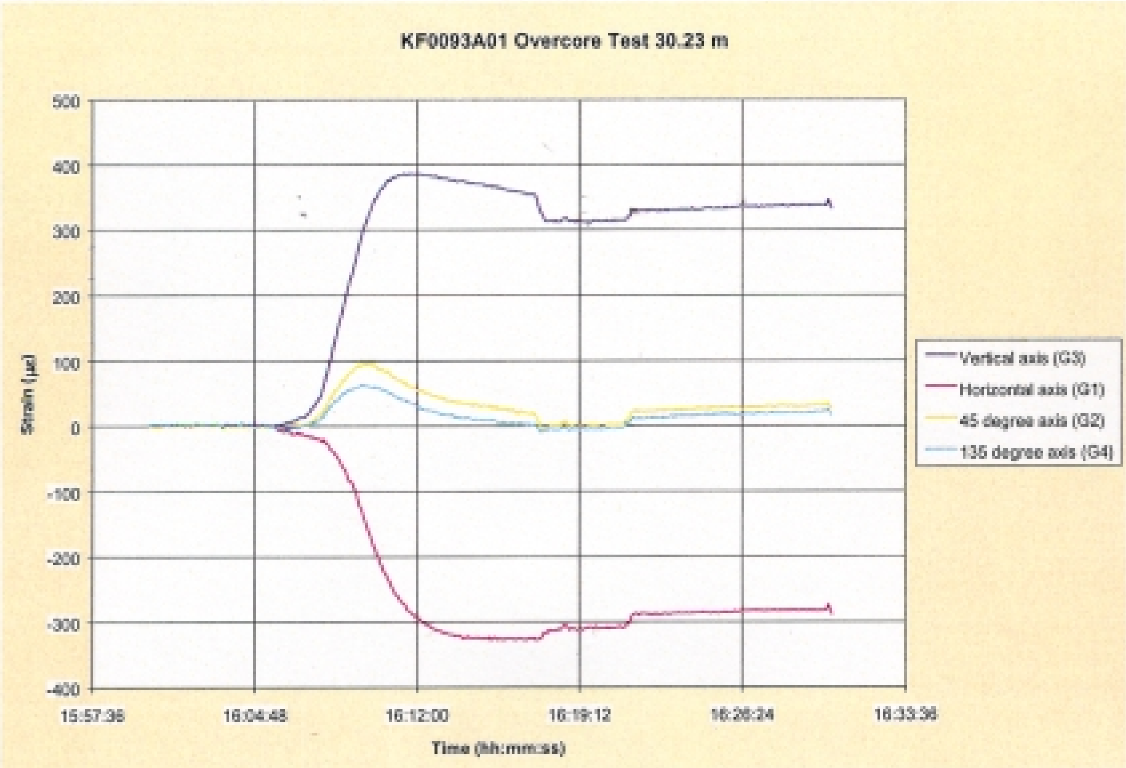


Figure 9. Overcore Test Data for KF0093A01 30.23 m.

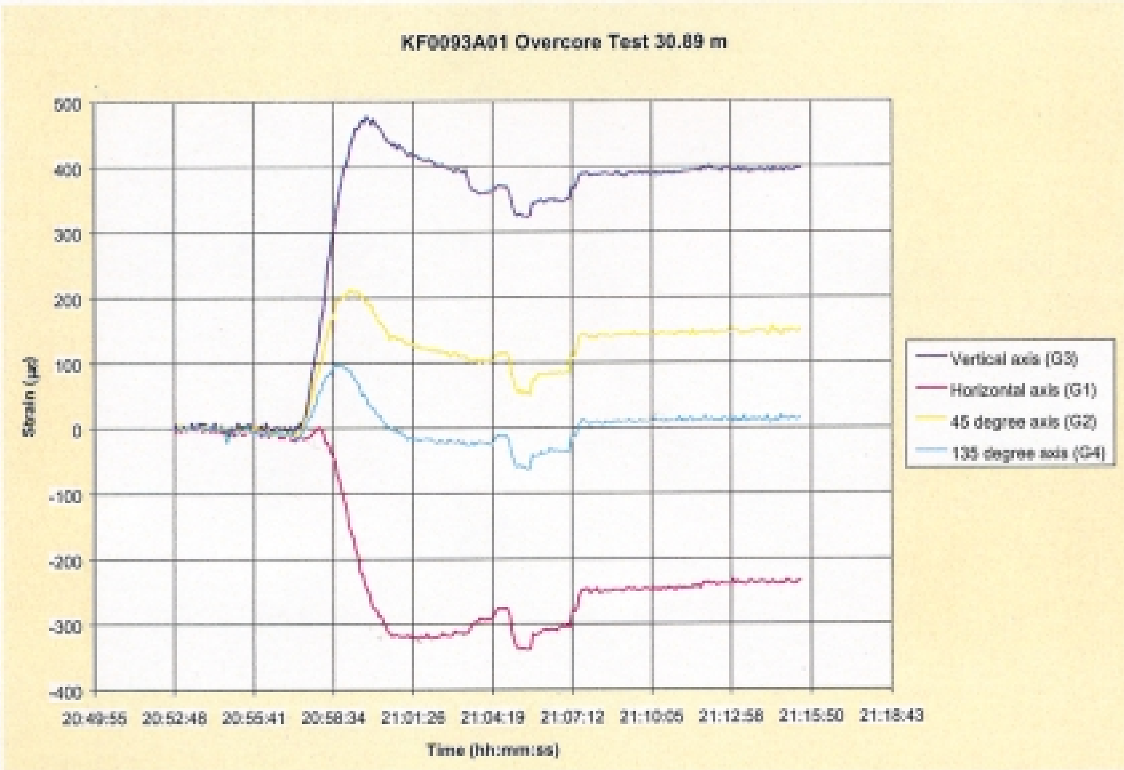


Figure 10. Overcore Test Data for KF0093A01 30.89 m.

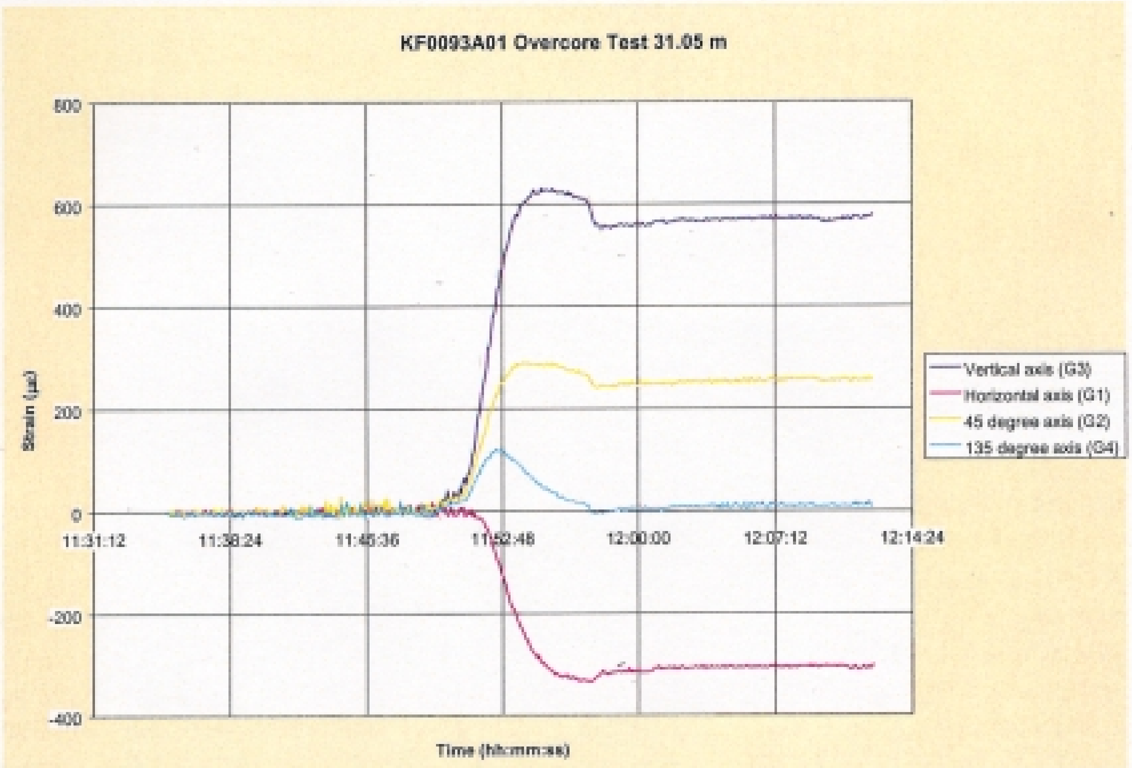


Figure 11. Overcore Test Data for KF0093A01 31.05 m.

The pattern of these three tests is both consistent and unusual, with positive maximum stable final strains in the vertical direction of 325 to 560 $\mu\varepsilon$, and negative maximum stable final strains in the horizontal direction of –240 to –310 $\mu\varepsilon$. This would tend to indicate relatively high vertical stress magnitudes and relatively low horizontal stress magnitudes.

Biaxial Pressure Tests were conducted on each test specimen, using a hydraulically actuated Hoek cell. Because the signal conditioner for the pressure transducer had failed earlier, the tests were conducted without the pressure transducer, producing time versus pressure (read from a pressure gauge) data, which were then converted manually to strain versus pressure and ultimately modulus versus pressure. Modulus versus pressure plots for the three tests in KF0093A01 are shown in Figures 12 to 14.

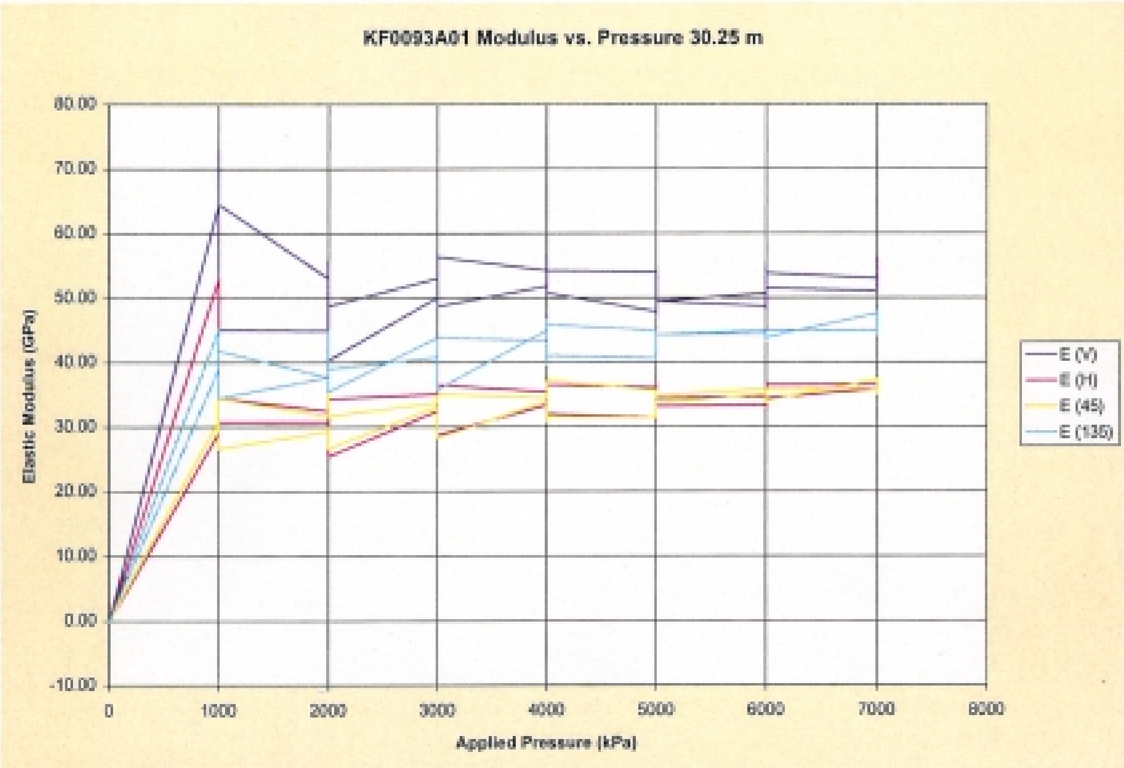


Figure 12. Biaxial Pressure Test Results for KF0093A01 30.25 m.

As seen in Borehole KA2599G01, the modulus was variable from one test to another, although a consistent pattern of anisotropy was evident, with the rock generally stiffer in the vertical/135° direction and less stiff in the horizontal/45° direction. Again in some of the tests, the IAM was quite unstable. Two of the tests (30.23 m and 31.05 m) were repeated with some improvement. The problem with the power supply and/ or grounding experienced in testing at KA2599G01 repeated as we experienced small shocks and internal "humming/ vibration" from all of the IAMs. This problem is discussed in Section 5.0.

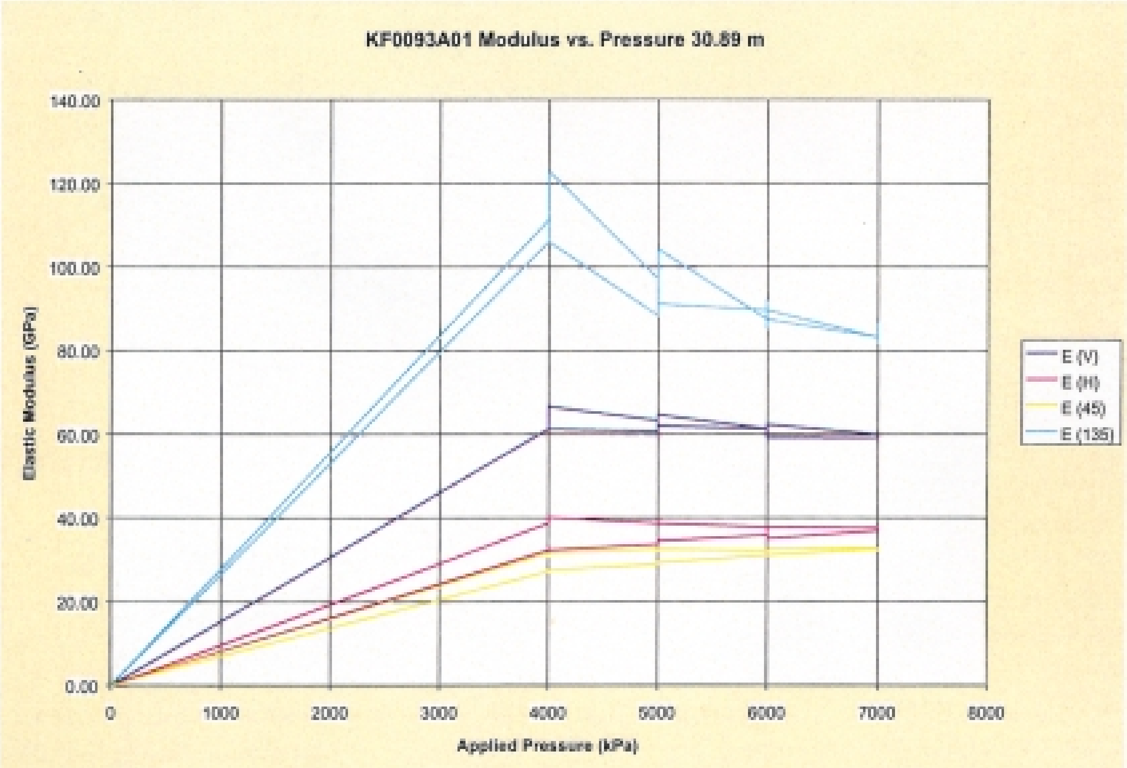


Figure 13. Biaxial Pressure Test Results for KF0093A01 30.89 m.

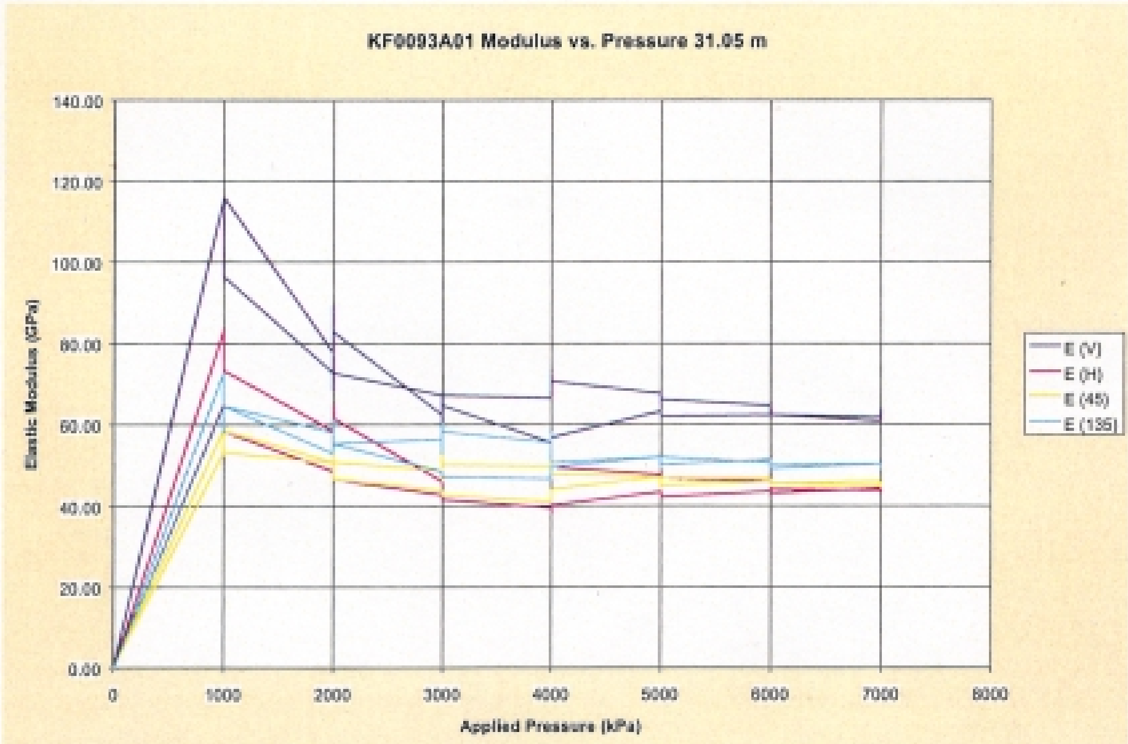


Figure 14. Biaxial Pressure Test Results for KF0093A01 31.05 m.

A summary of the measured moduli is contained in Table 5. The average modulus for the three tests was 42, 53 and 50 GPa from shallowest to deepest. This is considerably lower than the moduli values measured in Borehole KA2599G01, suggesting that the rock has suffered non-elastic damage during the overcoring process. The unusual shape of the overcore plots lends credibility to this possibility.

Depth (m)	E Gauge 3 (V) (GPa)	E Gauge 1 (H) (GPa)	E Gauge 2 (45) (GPa)	E Gauge 4 (135) (GPa)	E Ave. (GPa)	Anisotropy
30.23	52	36	36	45	42	High
30.89	60	37	32	83	53	High
107.95	61	44	45	50	50	High

Table 5. Summary of measured elastic moduli for Borehole KF0093A01

The calculation of in situ stresses in the plane perpendicular to the borehole axis was done using the methodology outlined in Thompson and Martino, 2000. The strain values were obtained from the overcore data. The modulus used for each test was the average as determined from the biaxial pressure test on that overcore sample. The Poisson's Ratio was assumed to be 0.25, and the stress perpendicular to the plane of the gauges was the average of the P values determined from the measurements in Borehole KA2599G01 (38.7 MPa), since the azimuth of Borehole KF0093A01 was chosen to be roughly parallel to the direction of P as determined in the first series of measurements.

The calculated principal stresses in the plane perpendicular to the test borehole axis are contained in Table 6.

Test Depth	P (MPa)	Q (MPa)	σ_z (MPa)	Direction of P*
30.23 m	27.0	12.2	38.7	0°
30.89 m	32.6	12.7	38.7	-6°
31.05 m	38.1	12.2	38.7	-8°

*Clockwise from vertical

Table 6. Summary of Calculated Principal Stresses for Borehole KF0093A01

These results are relatively consistent except for the measured range in the value of P (27.0 to 38.1 MPa). The calculated value of P is 12.2 to 12.7 MPa and the direction of P, as one would expect from the recorded strains, is essentially vertical. However these results are difficult to explain. At first glance, it would appear that the instrument must have been incorrectly installed by 90,° meaning that the vertical and horizontal responses are reversed. The calculated Q values are virtually the same as the calculated weight of overburden, suggesting that the direction of P should be horizontal, as one would expect from the tests in Borehole KA2599G01. This possibility has been examined thoroughly, and we are confident that this is not the case. In all three successful overcore tests the measured vertical strains were positive (indicating expansion) and the measured horizontal strains were negative (indicating contraction).

The only plausible explanation for the strange results would be extreme anisotropy in the rock fabric. The overcore sample obtained in the final overcore test in Borehole KA2599G01 at 128.28 m depth (Table 2) exhibited a high degree of anisotropy ranging from 57 GPa to 120 GPa. If one looks at the direction of this anisotropy (using the orientation data from the overcore test), the direction of maximum horizontal stiffness is approximately parallel to Borehole KF0093A01, while the direction of minimum horizontal stiffness is approximately perpendicular to it. This is confirmed by the tests in Borehole KF0093A01, where the vertical stiffness is much greater than the horizontal stiffness in all three overcore samples (Table 5). If micro cracking of the rock were the reason for this anisotropy, one would expect the microcracks to be sub-vertical and parallel to Borehole KF0093A01. If one assumes that the microcracks are slightly open in situ, then it can be seen that as the stresses are relieved during overcoring, the cracks could close up resulting in measured contraction in the horizontal direction.

It is therefore concluded that the results presented in Table 6 are not valid since the calculations assume homogeneous isotropic linear elastic material. A better understanding of the material properties and a solution that takes anisotropy into account would be necessary to estimate the in situ stress conditions from the measured strain data.

5.0 Problems Encountered

The testing success rate was lower than anticipated for several reasons, the main one being glue failure. This various causes of failure are discussed in this section.

IAM-related Failures

In KA2599G01 at 107.44 m, the IAM monitoring period was accidentally set to 5 minutes instead of 5 hours. The IAM monitored strains for 5 minutes after the 1.5-hour waiting period. As a result, no strain data were recorded during the overcore test. This was an unfortunate error, since it appears that everything else about the test was good, including the drilling, rock conditions, glue bond, and the operation of the IAM. However, a lesson was learned and the error wasn't repeated.

In KA2599G01 at 128.61 m, the IAM failed to download collected overcore data. The cause was determined to be a faulty battery connector. Only one of two 9-v batteries had been connected to the IAM. As a result, battery voltage fell too low and all data was lost. The clip was repaired and the problem did not repeat.

In KF0093A01 at 29.92 m, the IAM (#011) failed to download data to the computer after what appeared to be a successful test. The reason for this was not known. A different IAM (#007) was used successfully for the next three tests. IAM #007 failed during preparation for the test at 31.05 m (a transistor overheated), so IAM #11 was used, and this time was used to download data from a successful test.

Glue Failures

Six tests in KA2599G01 occurred as a result of glue failure using the Versilok glue that has been used with a high success rate at the URL. Three glue failures occurred in KF0093A01, two using HBM X-60, a fast setting strain gauge cement, and one using Versilok. The two failures of the HBM X-60 were attributed to its fast setting time. We were unable to install the Doorstopper Gauge fast enough at 29-m-depth, to prevent the glue from starting to set before the gauge was installed. It was then decided to try the Versilok glue again.

The most obvious difference between the testing environment at Äspö compared to the URL is the difference in drill water/ groundwater chemistry between the two sites. At the URL, freshwater is used for overcore drilling operations, whereas at Äspö, saline water with a composition similar to the groundwater is used. It was postulated that the salt might have an adverse effect on glue performance. It wasn't possible to do anything about the presence of saline water in KA2599G01, since there was a large quantity of saline groundwater flowing into the borehole, however in KF0093A01, a switch to freshwater was made after the first three tests had experienced glue failures. No more glue failures occurred in KF0093A01 after the switch to freshwater, leaving us to conclude that the saline conditions were the cause of the glue failures, however this didn't explain why four successful tests had been achieved in KA2599G01 in saline water conditions.

On our return to Canada, a series of laboratory tests were conducted at the URL to confirm the theory that glue failures were the result of saline conditions. Surprisingly, the tests showed no significant difference in glue failure rates, or bond strengths, between glue used in freshwater conditions and glue used in saline water conditions. Therefore the cause of the high glue failure rate is unknown. The glue used was from a relatively new shipment, and two different batches were used in case the first had somehow become contaminated.

Trigger Failures

Three trigger failures occurred in KA2599G01, once at 107.79 m and twice at 128.45 m. This failure type was attributed to a difference in diamond bit design between the Dimatec/ Longyear type used at the URL and a Craelius version used at Äspö. The former both have a tapered lower inner surface that smoothly transitions to the final bit kerf. The gently tapered surface serves to both centralize the DDGS as it passes through the bit and also acts as the surface on which the trigger rod is actuated. The Craelius design utilizes internal waterways that commence in a sharply angled ring several centimetres above the lower tapered bit bottom.

It appears that in at least one of the first three test attempts, the base of the DDGS installation tool, contacted the sharp angled ring inside the bit, which left a sharp impression on the base of the installation tool. In the test at 107.79 m, this impression locked on the angled ring, preventing the DDGS installation tool from reaching the bottom of the bit and releasing the trigger. When this happened, the weight of the DDGS installation tool sliding sleeve and overshot bent the trigger and slide rods in the installation tool and prevented installation of the DDGS, which was retrieved when the installation tool was pulled up about two hours after installation.

It was then that we realized what had occurred. The borehole bottom was repolished to remove any glue, the trigger and slide rods were straightened and adjusted, the base of the DDGS installation tool was filed flat to remove the indentation and the inside of the diamond bit was machined to make it more similar to the Dimatec/ Longyear style.

This solved the problem for the next four tests, three of which were successful, but the problem then resurfaced for the two tests at 128.45 m. It was determined that the repair conducted after the failure at 107.79 m had been imperfect, leaving the slide and trigger rods slightly misshapen. Despite many successful dry runs of the trigger operation on surface, the first repair proved unsuccessful as the trigger failed to operate a third time. It was repaired a third time with no repeat of the trigger problem, although no further successful tests were achieved in KA2599G01.

Drilling-related Failures

In general the diamond drilling operations were very good, however two of the tests failed as a result of drilling operations, one in each borehole. In KA2599G01 at 128.45 m, rock fragments on the borehole bottom prevented a successful test. Again, the Craelius bit design may have contributed since the waterways on the bit face are very wide compared to the Longyear/ Dimatec design used at the URL. This makes it easier for rock fragments to pass through a waterway to the flattened borehole base.

In KF0093A01 at 30.07 m, a dimple had formed at the centre of the flattened borehole base as a result of wear at the centre of the bit. This dimple sheared the strain gauges when the DDGS was installed on the borehole bottom.

Rock-related Failures

When planning and proposing the work, we felt that most, if not all, test failures we would experience at Äspö would be related to rock conditions, such as installing the DDGS across a fracture in the rock. Surprisingly not a single failure could be attributed to conditions in the rock, although as reported in Section 4.0, anisotropy in the rock, presumably resulting from vertically oriented microcracks, made standard interpretation of the test results impossible.

Electrical Problems

Throughout the course of the work, a number of problems were identified that seemed to be related to the electrical supply or the electromagnetic environment. Several pieces of equipment failed despite being carefully checked and calibrated prior to leaving the URL. One of these, the power supply for the biaxial pressure test pressure transducer was unfortunate in that it compromised the biaxial pressure tests and made data reduction more time-consuming than normal.

We experienced electrical shocks from the 220 – 120 v transformer as well as the circuits of the IAM. In addition, the IAMs had a noticeable oscillation and "hum" when connected through the laptop power supply. This vibration had never been experienced previously and did not occur after we returned the equipment to Canada. The stability of the IAMs was quite good when they were used remotely on battery power down-hole during the overcoring tests, but when connected through the laptop power supply for the biaxial pressure tests, they were very unstable, producing poor results in terms of "noise". In retrospect, we should have tried to conduct the biaxial tests by using the laptop computer batteries to see if the tests were more stable. Unfortunately we did not think of trying this at the time, as we suspected the problem was related to electromagnetic interference in the tunnel itself.

The electrical power was checked by an SKB electrical engineer and deemed to be okay while we were there, but we have never experienced similar problems in the past, even when using the same equipment and transformers in Hungary to convert 220 to 120 v. At one point, a transistor on IAM #007 heated up and burned, causing the IAM to fail. It was repaired after being returned to the URL. We lost the ability to communicate with IAM #008 while working at Äspö and are still trying to repair it. Our electronic technician believes that the symptoms we describe may relate to a grounding problem in the AC power supply. The moist salty environment may also be a contributing factor.

6.0 Conclusions

The DDGS was applied to determine in situ rock stress conditions in Boreholes KA2599G01 and KF0093A01 at the Äspö Hard Rock Laboratory near Oskarshamn Sweden. Four successful overcore tests were conducted in sub-vertical Borehole KA2599G01, and three successful overcore tests were conducted in sub-horizontal Borehole KF0093A01. 7 of 24 tests attempted were successful, for a test success rate of 29%, which is considerably lower than anticipated. 9 of the failures were the result of glue problems, 3 were the result of problems with the IAM datalogger, 3 were the result of trigger malfunction (related to a different diamond bit design), and 2 were drilling related. Tests conducted at the URL indicate that the saline water conditions were not the cause of the glue failures. The reason for these is not presently known.

The successful tests provided consistent results in both boreholes, but standard isotropic linearly elastic analysis produced results (particularly in Borehole KF0093A01) that are clearly incorrect. In Borehole KF0093A01 calculated maximum principal stress magnitudes range from 27.0 to 38.1 MPa in the vertical direction, and calculated minor stress magnitudes range from 12.2 to 12.7 MPa in the horizontal direction, whereas calculated vertical stress magnitudes (from the weight of overburden) should be about 12.3 MPa, and the horizontal stress magnitude in the direction perpendicular to the borehole axis should be considerably higher.

The only plausible explanation for the strange results would be extreme anisotropy in the rock fabric. The overcore sample obtained in the final overcore test in Borehole KA2599G01 at 128.28 m depth exhibited a high degree of anisotropy ranging from 57 GPa to 120 GPa. If one looks at the direction of this anisotropy (using the orientation data from the overcore test), The direction of maximum horizontal stiffness is approximately parallel to Borehole KF0093A01, while the direction of minimum horizontal stiffness is approximately perpendicular to it. This is confirmed by the tests in Borehole KF0093A01, where the vertical stiffness is much greater than the horizontal stiffness in all three overcore samples. If microcracking of the rock were the reason for this anisotropy, one would expect the microcracks to be sub-vertical and parallel to Borehole KF0093A01. If one assumes that the microcracks are slightly open in situ, then it can be seen that as the stresses are relieved during overcoring, the cracks could close up resulting in measured contraction in the horizontal direction, as we observed in these tests.

It is therefore concluded that the calculated results are not valid since the calculations assume homogeneous isotropic linear elastic material. A better understanding of the material properties and a solution that takes anisotropy into account would be necessary to better estimate the in situ stress conditions from the measured strain data.

Appendix 4

PM
on
3D OVERCORING ROCK STRESS MEASUREMENTS IN BOREHOLE KF0093A01 AT THE ÄSPÖ HRL

For comparison to the 2D AECL Deep Doorstopper Method

PM prepared by Hans Klasson and Stig Andersson
June, 2001

ABSTRACT

The PM presents briefly the results from three-dimensional overcoring rock stress measurements conducted in borehole KF0093A01 in the Äspö Hard Rock Laboratory. The borehole was drilled into undisturbed rock at 450 m level. The orientation of the hole is 2 degrees upwards from the horizontal in the direction 310 degrees according to the local Äspö coordinate system.

Firstly, measurements were conducted using the two-dimensional AECL Deep Doorstopper Method. For comparison to that technique, SwedPower were contracted to perform three-dimensional overcoring measurements from hole depth 30 m and on. The measurement plan aimed at gathering three reliable results as quickly as possible.

SwedPower conducted four tests between 32.14 m and 35.38 m hole depth. Three of these were successful with results displaying good redundancy.

The results indicate high stresses in the vertical direction, however not as high as the maximum horizontal stress, σ_H. The biaxial test results are scattered. Generally, the results yield a lower value for Young's modulus than expected for the rock at Äspö. The discrepancy in the results from individual gages located at 120 degree angles on the same overcore sample, give reason to suspect rock anisotropy .

On the average, the results as interpreted for the virgin stress field around borehole KF0093A01 are:

σ_1 = 29 MPa, σ_2 = 14,5 MPa, and σ_3 = 9 MPa.

None of the principal stresses is vertical. σ_1 trends 309º (Äspö local system) and plunges 31º.

σ_H = 25 MPa directed at 125º (Äspö local system).
σ_h = 10 MPa.
σ_v = 18 MPa

1 INTRODUCTION

This PM presents, in brief, the results from overcoring stress measurements in borehole KF0093A01 in the Äspö Hard Rock Laboratory (HRL). The field stress measurements started on May 31 and was completed on June 6, 2001.

The objective with the stress measurements was to:

- Provide three sets of three-dimensional stress data for comparison to the data gathered previously using a two-dimensional overcoring technique in the same borehole and in one vertical borehole drilled from above into the same section of the rock mass.

2 FIELD WORK

Four test attempts between 32.14 and 35.38 m hole depth resulted in three successful results. Test attempt no. 3, at 33.23 m, failed due to breaking of the core during overcoring. The borehole had penetrated into a section of rock containing more feldspar. In order to reach better rock the borehole was extended 2 m before attempting the fourth and last test. Still located in feldspar-rich rock, test no. 4 was successful.

The measurement work was stalled for 1,5 days during the field period. Malfunctioning of the drilling machine caused part of the delay. Severe wearing of the pilot hole drill bits when penetrating into feldspar-rich quartzite rock also put a temporary halt to the measurements.

Borehole KF0093A01 is located at the 450 m level in the Äspö HRL. The borehole is drilled in the direction 310º according to the local Äspö coordinate system. The dip of the hole is 2º up relatively to the horizontal. Drilling was executed using wireline technique. For the 3D overcoring measurements the dimension was WL76.

3 RESULTS

3.1 GENERAL TEST DATA

Table 3-1 summarizes the general information from the test.

The strain gauge response curves registered during the overcoring process are presented in Appendix A.

Table 3-1. *General test data from measurements in borehole KF0093A01.*

Measuring point No.	Hole depth (m)	Comment	Incl. in Evaluation
1	32.14	Test OK	Yes
2	32.70	Test OK	Yes
3	33.23	Unstable	No
4	35.38	Test OK	Yes

Note: Hole depth calculated from the tunnel wall.

3.2 BIAXIAL TESTING

The overcore rock samples from test no 1, 2 and no. 4 were suitable for biaxial testing. The gage response curves from the tests are given in Appendix B. Table 3-2 shows the values of E and ν as interpreted from the biaxial tests.

The elastic parameters are usually determined using the secant method from the unloading part of the biaxial testing curves. The test pressure interval during biaxial pressurization is normally 0-10 MPa. The overcore rock samples from the WL76 borehole (KF0093A01) are slightly slimmer than cores produced using a conventional Craelius T2-76 core barrel for which the biaxial cell is intended. Thus, the WL-overcore sample had to be taped at the openings of the biaxial cell in order to be pressurized without having oil leaking out from the cell. As the tape would still leak oil at high pressures, the loading cycle was stopped at the test pressure
8 MPa.

Table 3-2. *Results from biaxial tests on overcore rock samples from borehole KF0093A01.*

Measuring point No.	Hole depth (m)	Young's modulus, E [GPa]	Poisson's ratio, ν
1	32.14	51	0.19
2	32.70	60	0.23
4	35.38	51	0.19
Average:		***54***	***0.20***

Note: Hole depth calculated from the tunnel wall.

Biaxial testing proved good bonding between gages and the rock. For measuring point no. 2, gage no. 8 did not respond well during the test, Appendix B. Thus, the rosette comprising gage no. 8 was discarded in the analysis of the biaxial test results.

Studying the biaxial tests response curves, Appendix B, the unloading cycle, for most gage rosettes, produces results corresponding to a softer rock than does the loading cycle. This cannot be accredited to the tape only, as the gages are located in the middle of the biaxial test chamber, as far away from the taped parts of the rock sample ends as possible. Moreover, data from the unloading cycle are less condensed. For evaluation purposes secant data from the loading cycle have been used in the stress calculations.

Overall, the E-values given by the biaxial tests are lower than expected when studying previous tests on rock samples form the Äspö HRL. The results from the biaxial tests, Appendix B, are not distinct for neither E, nor ν. Depending on individual gage rosette, located at 120º angles on the overcore cylinder, secant E-values taken from the loading cycle and processed in the stress analysis vary between 41 GPa and 74 GPa. The variance is large and could be an indicator of rock anisotropy.

3.3 PRIMARY STRESS FIELD

The results from borehole KF0093A01 are supposed to represent the virgin (undisturbed) stress field at depths around 450-460 m. Tables 3-3 through 3-5 show the results in figures whereas a graphical presentation of the principal stress orientations is given in Figure 3-1. All orientations are given in the Äspö local coordinate system.

Table 3-3. *Primary stress field, borehole KF0093A01: Principal stress magnitudes as determined by overcoring.*

Measuring point No.	Hole Depth (m)	σ_1 (MPa)	σ_2 (MPa)	σ_3 (MPa)
1	32.14	32.5	13.8	8.7
2	32.70	36.0	17.7	8.9
4	35.38	23.2	14.2	6.9
460 m lev. ave.		***29.8***	***14.8***	***9.4***

The average magnitudes for the primary stress field have been obtained by transformation of all applicable results to one common coordinate system, and then solving the average stress tensor for its eigen values.

Table 3-4. *<u>Primary stress field, borehole KF0093A01</u>: Principal stress orientations as determined by overcoring. Orientations are given as trend/plunge of the stress vectors σ1, σ2 and σ3, respectively.*

Measuring point No.	Hole Depth (m)	σ_1 Trend/pl.	σ_2 Trend/pl.	σ_3 Trend/pl.
1	32.14	307/38	096/48	204/16
2	32.70	310/38	114/51	214/08
4	35.38	308/10	044/30	204/58
460 m lev. ave.		***310/31***	***088/52***	***206/21***

Note: Strike is calculated clockwise from the bearing of the local north of the Äspö local coordinate system (local north is 12º west of true magnetic north). Plunge is defined as being zero in the horizontal plane.

Table 3-5. *<u>Primary stress field, PRT borehole KA3579G</u>: The horizontal - and vertical stress state as determined by overcoring.*

Measuring point No.	Hole Depth (m)	σ_H (MPa)	σ_h (MPa)	σ_v (MPa)	Trend σ_H (º clockwise fr. local north)
1	32.14	25.3	9.2	20.4	123
2	32.70	29.2	9.1	24.3	127
4	35.38	22.8	12.3	9.2	125
460 m lev. ave.		***25.7***	***10.2***	***18.0***	***125***

Mote: Trend is calculated clockwise from the bearing of the local north.

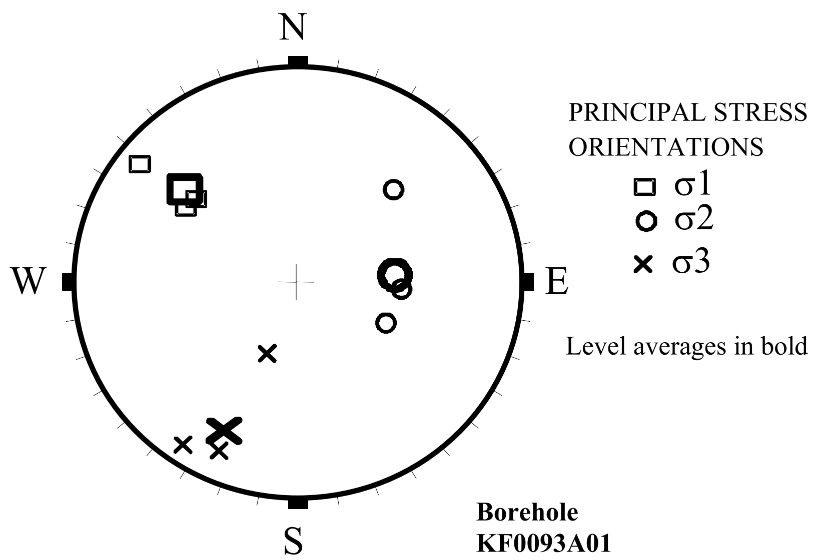


Figure 3-1. *<u>Primary stress field, borehole KF0093A01:</u> Principal stress directions for test points located in undisturbed rock. Lower hemisphere, schematic plot. North refers to local north (Äspö x-axis). Data taken from Table 3-4.*

3.4 COMMENTS

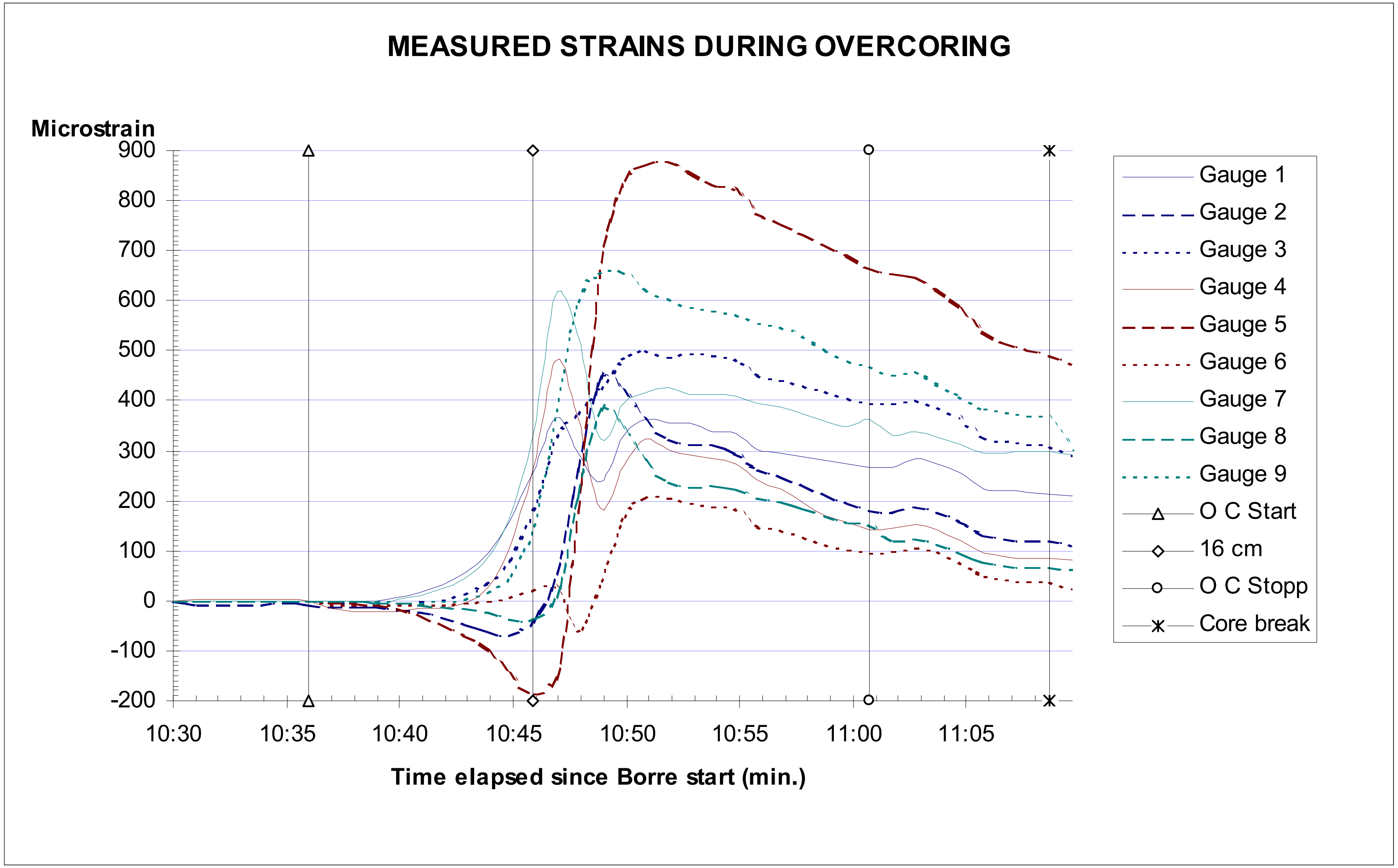
- With a maximum value around 36 MPa in test no. 2, σ_1 averages around 30 MPa. The relationship σ_1/σ_2 is roughly 2.0 whereas σ_1/σ_3 is around 3.0.
- The principal stresses are neither horizontal, nor vertical. σ_1 trends 309º with a dip around 30º, Table 3-4 and Figure 3-1.
- Average magnitudes in the vertical- and horizontal plane are: σ_H = 26 MPa, σ_h = 10 MPa, and σ_v = 18 MPa, Table 3-5. It is noted that the magnitude of σ_v is 40% higher than the stress corresponding to the overburden pressure.
- The direction of σ_H is uniform, Table 3-5 at 125º. Transformed with respect to magnetic north the results yield a NW-SE direction for the maximum horizontal stress.
- The biaxial test results are indistinct. Values on Young's modulus are generally in the lower region of the interval for E found for core samples from the Äspö HRL. The variance between E-values from gages located 120º apart on the same overcore sample is hard to explain. The results could indicate rock anisotropy and it is suggested that this should be further investigated.

Appendix 4-A:

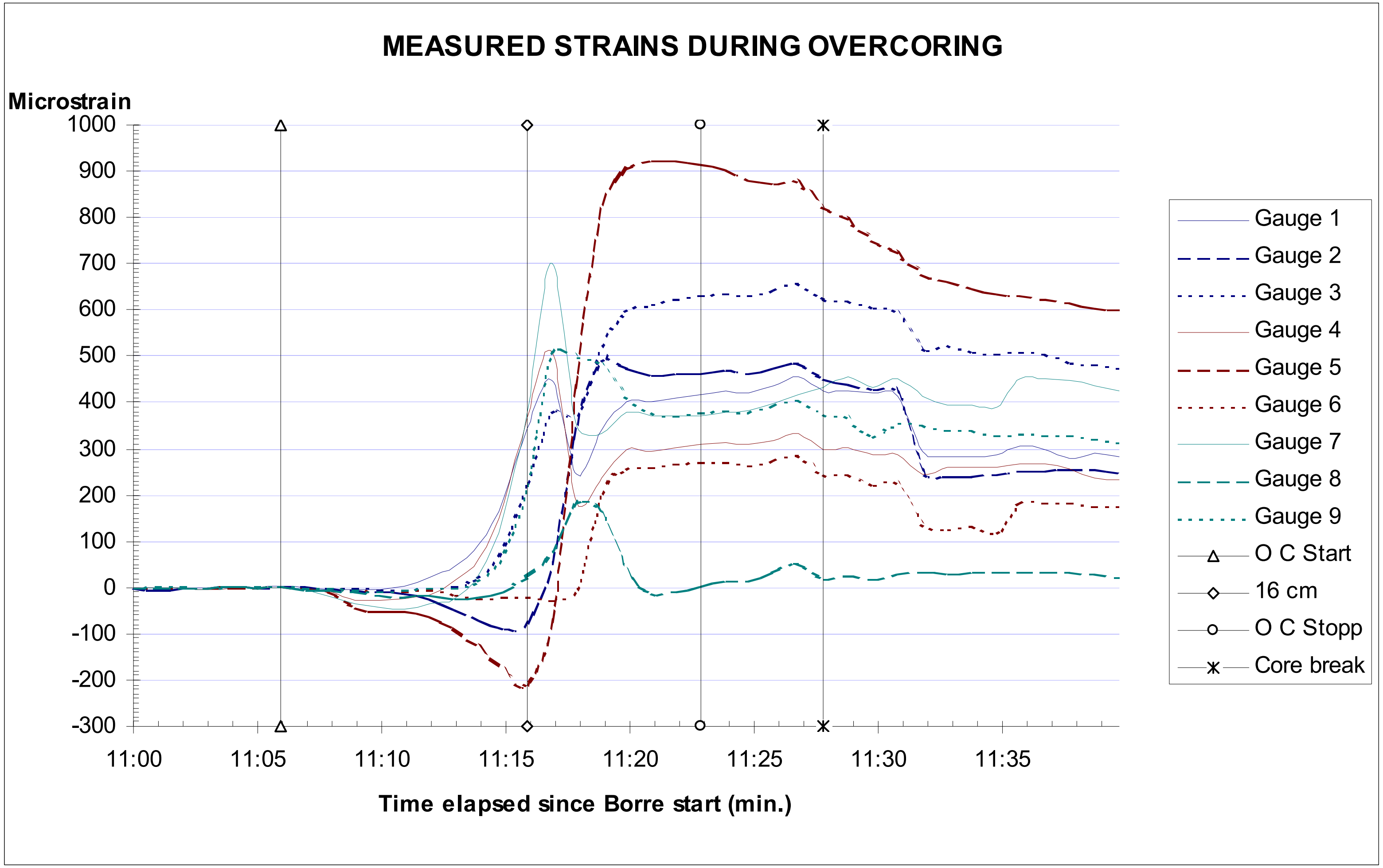
Registered strains during overcoring

Borehole KF0093A01, Äspö HRL

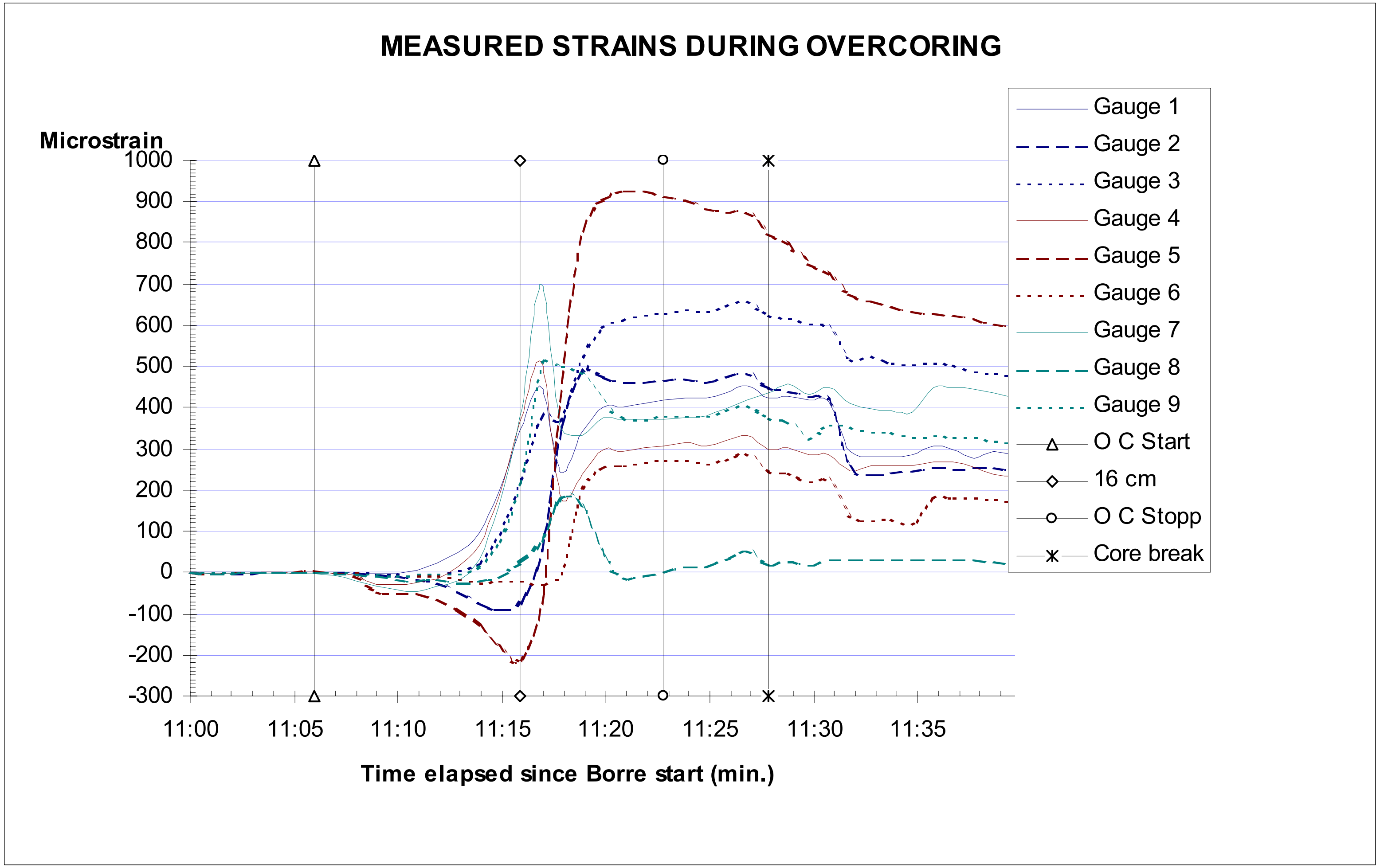
(4 pp)



Measuring point 1, hole depth 32,14 m, borehole KF0093A01, Äspö HRL



Measuring point 2, hole depth 32,70 m, borehole KF0093A01, Äspö HRL



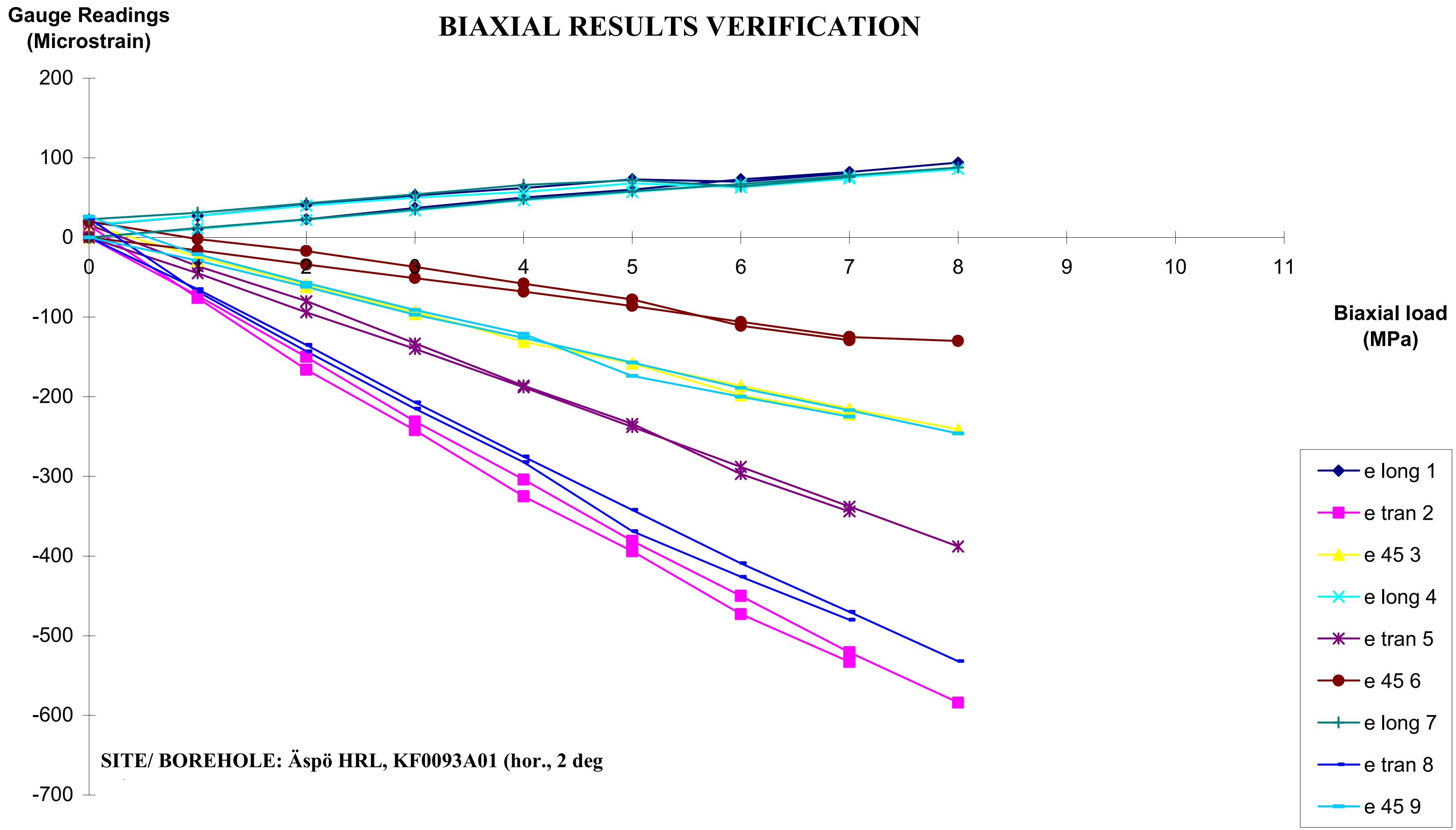
Measuring point 4, hole depth 35,38 m, borehole KF0093A01, Äspö HRL

Appendix 4-B

Biaxial tests

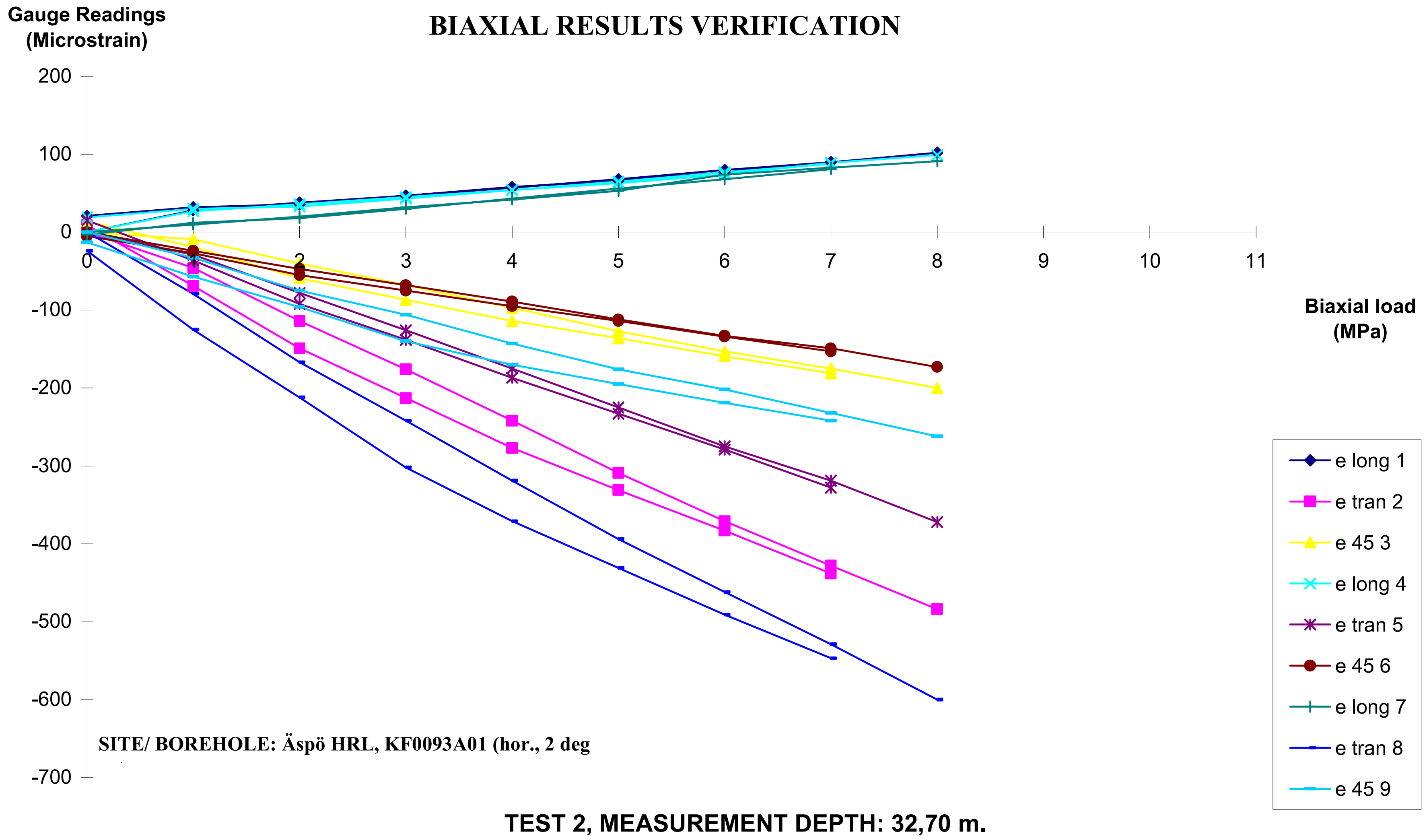
Borehole KF0093A01, Äspö HRL

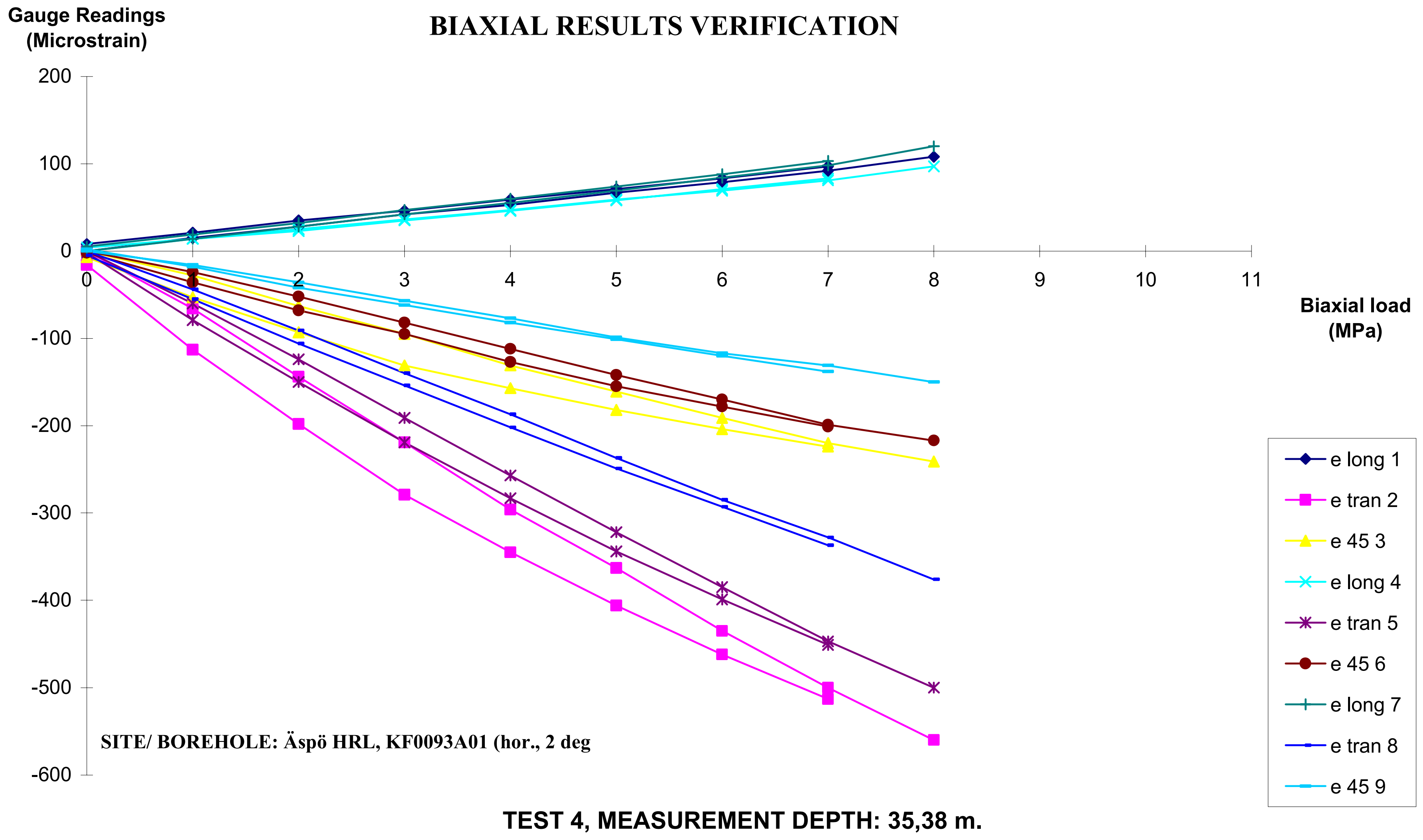
(4 pp)



TEST 1, MEASUREMENT DEPTH: 32,14 m.

PROJECT NO: 1457400




Gauge Readings
(Microstrain)
BIAXIAL RESULTS VERIFICATION
200
100
0
-100
-200
-300
-400
-500
-600
0
2
3
4
5
6
7
8
9
10
11
Biaxial load
(MPa)
e long 1
e tran 2
e 45 3
e long 4
e tran 5
e 45 6
e long 7
e tran 8
e 45 9
SITE/ BOREHOLE: Äspö HRL, KF0093A01 (hor., 2 deg
TEST 4, MEASUREMENT DEPTH: 35,38 m.

Appendix 5

Laboratory testing of anisotropic elastic properties of rocks from the boreholes KF0093A01 and KA2599G01 at the Äspö Hard Rock Laboratory

Contents

1 Introduction

In connection with field tests of the AECL Deep Doorstopper Gauge System at the Äspö Hard Rock Laboratory, the degree of anisotropy of the rock has been brought forward as a reason for seemingly strange results. To investigate this, rock core material from the sub-vertical borehole KA2599G01 and the sub-horizontal KF0093A01 has been tested at the Rock Mechanics laboratory of SINTEF Civil and Environmental Engineering, Trondheim, Norway.

2 Test procedure

From 61 mm cores from the boreholes provided by SKF, 21 mm cores have been drilled out. From the vertical hole, cores have been drilled out parallel with and perpendicular to the apparent maximum horizontal stress, and in the vertical direction (parallel with the borehole axis).

From the horizontal hole, cores have been drilled out perpendicular to the borehole axis in the horizontal and vertical directions, and in the horizontal direction parallel with the borehole axis (which is parallel with the apparent maximum horizontal stress).

From the core material series of five specimens have been prepared. The length / diameter ratio of the specimens is approximately 2.5.

The specimens have been uniaxially loaded up to 50 MPa and unloaded. Axial and tangential strains have been recorded by strain gauge rosettes glued diametrically in pairs to each specimen. Strains have been recorded manually by a standard strain gauge bridge, and stress- strain curves have been plotted.

Young's modulus and Poisson's ratio are given as secant values at 50 MPa.

3 Test results

The test results are presented in table 1 and table 2 in the attached Test report, together with the stress – strain curves.

In the vertical hole KA2599G01 the following average values were obtained:

$E_{parallel}$ = 69.4 ± 4.4 GPa, E_{normal} = 65.9 ±3.2 GPa, E_{axis} = 73.5±11.0 GPa

$\nu_{parallel}$ = 0.24 ±0.03 ν_{normal} = 0.19±0.02 ν_{axis} = 0.28±0.08

i.e. fairly isotropic conditions for a rock material. The stress-strain curves also indicate fairly linear behaviour.

In the horizontal hole KF0093A01 the following average values were obtained:

E_{vert} = 58.7±8.5 GPa, E_{hor} = 42.8±23.6 GPa, E_{ax} = 62.5±2.2 GPa

ν_{vert} = 0.23±0.03 ν_{hor} = 0.27±0.03 ν_{ax} = 0.21±0.04

i.e. there is apparently a pronounced anisotropy horisontally perpendicular to the borehole. However, the standard deviation of E that direction is also very high compared with the other directions. A visual inspection of the actual 61 mm core shows that the core contains a lot of large feldspar crystals up to 2o mm in diameter, sometimes with visible cracks between crystals. This may cause large differences in the measured elastic properties, which actually are not caused by real anisotropy but rather because the rock is in-homogenous.

The stress-strain curves in this case are more curvilinear and show more hysteresis than the others.

Regardless of this, the anisotropy measured can not be regarded as extreme.

Comparing Young's modulus in the two holes gives the following relation:

E_{vert} = 58.7 GPa corresponds to E_{axis} = 73.5 GPa

E_{hor} = 42.5 GPa corresponds to E_{normal} = 65.9 GPa

Average: 55.0 GPa 69.6

This shows the same tendency as the biaxial cell results from the stress measurement programme, i.e. the average Young's modulus in the horizontal hole is lower than in the vertical hole.

Appendix 5A

TELEFAKS

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Telefaks: 73 59 47 78

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Telefon: 73 59 46 00
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Foretaksregisteret: NO 948 007 029 MVA

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2002-01-16	Fortrolig	trondel@civil.sintef.no

- [ ] INNHOLDET I DENNE TELEFAKSEN ER AV FORMELL ART, OG VIL BLI BEKREFTET MED BREV
- [x] INNHOLDET I DENNE TELEFAKSEN ER AV FORMELL ART, MEN VIL IKKE BLI BEKREFTET MED BREV
- [ ] INNHOLDET I DENNE TELEFAKSEN GJELDER RUTINEMESSIG INFORMASJONSUTVEKSLING DEN ER IKKE AV FORMELL KARAKTER, OG IKKE BINDENDE FOR SINTEF

Det ble mye feil forige gang. Det ble en kommafeil i regnearket mitt. Sender tabellcne fra rapporten som viser dybden på hvert "specimen no".

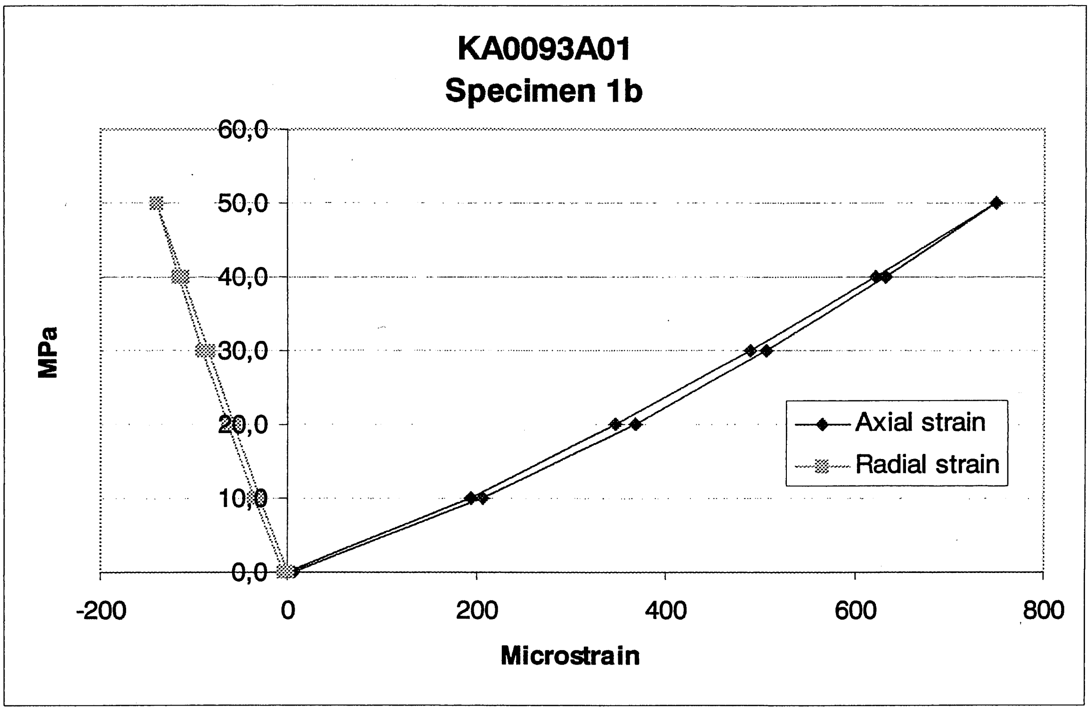
Specimen no	Young´s-modulus [GPa]	Poisson´s ratio	Depth [m]
1	67,2	0,189	27,9
2	54,9	0,257	27,9
3	45,9	0,237	28,0
4	62,7	0,216	28,0
5	63,0	0,269	28,1
Average	58,7	0,234	
Std.deviation	8,5	0,032	
6	35,3	0,253	28,2
7	81,7	0,242	28,2
8	28,7	0,303	28,3
9	21,5	0,246	28,3
10	46,8	0,291	28,3
Average	42,8	0,267	
Std.deviation	23,6	0,028	
A	61,6	0,221	28,4
B	64,6	0,175	28,5
C	64,9	0,267	28,5
D	60,4	0,172	28,6
E	60,7	0,191	28,6
Average	62,5	0,205	
Std.deviation	2,2	0,040	

Spesimen no.	Young`s-modulus [GPa]	Poissons ratio	Depth [m]
11	74,7	0,237	112,2
12	66,4	0,262	112,3
13	63,8	0,257	112,3
14	70,0	0,190	112,4
15	72,0	0,272	112,4
Average	69,4	0,244	
Std. deviation	4,4	0,033	
16	65,5	0,218	112,5
17	61,1	0,167	112,5
18	69,5	0,193	112,5
19	65,1	0,217	112,6
20	68,1	0,177	112,6
Average	65,9	0,194	
Std. deviation	3,2	0,023	
21	77,0	0,271	112,6
22	56,2	0,362	112,7
23	71,9	0,233	112,7
24	86,4	0,172	112,8
25	76,2	0,368	112,9
Average	73,5	0,281	
Std. deviation	11,0	0,084	

Med vennlig hilsen
for SINTEF Bygg og miljø
Berg og geoteknikk

Trond Larsen

Trond Larsen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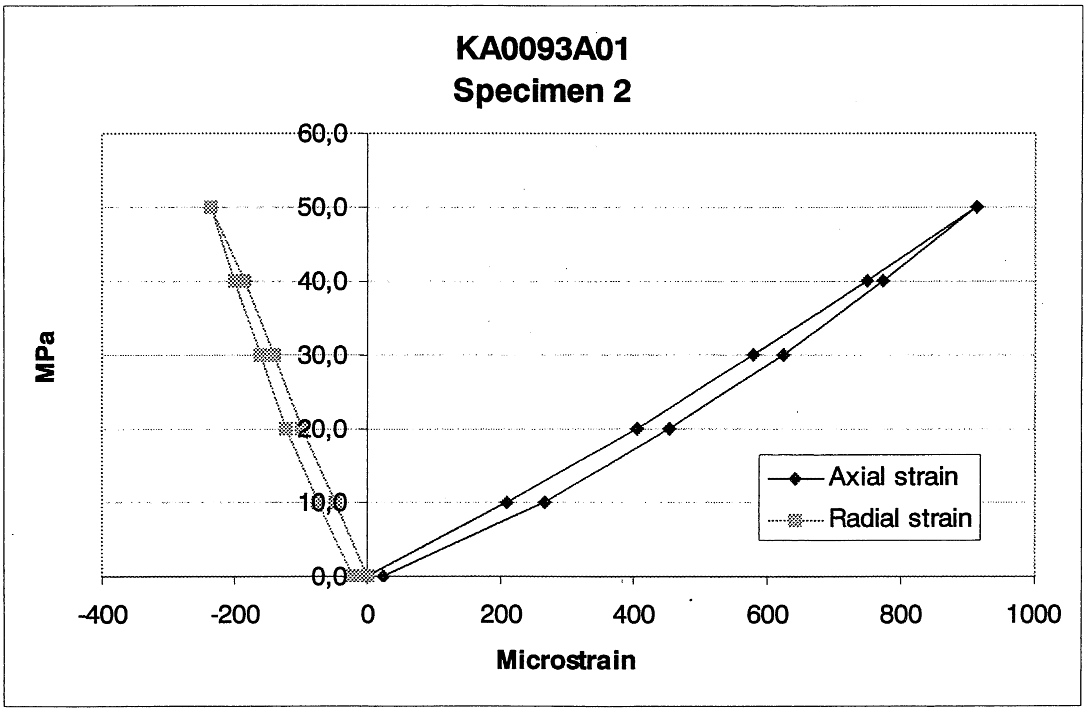
Radial strain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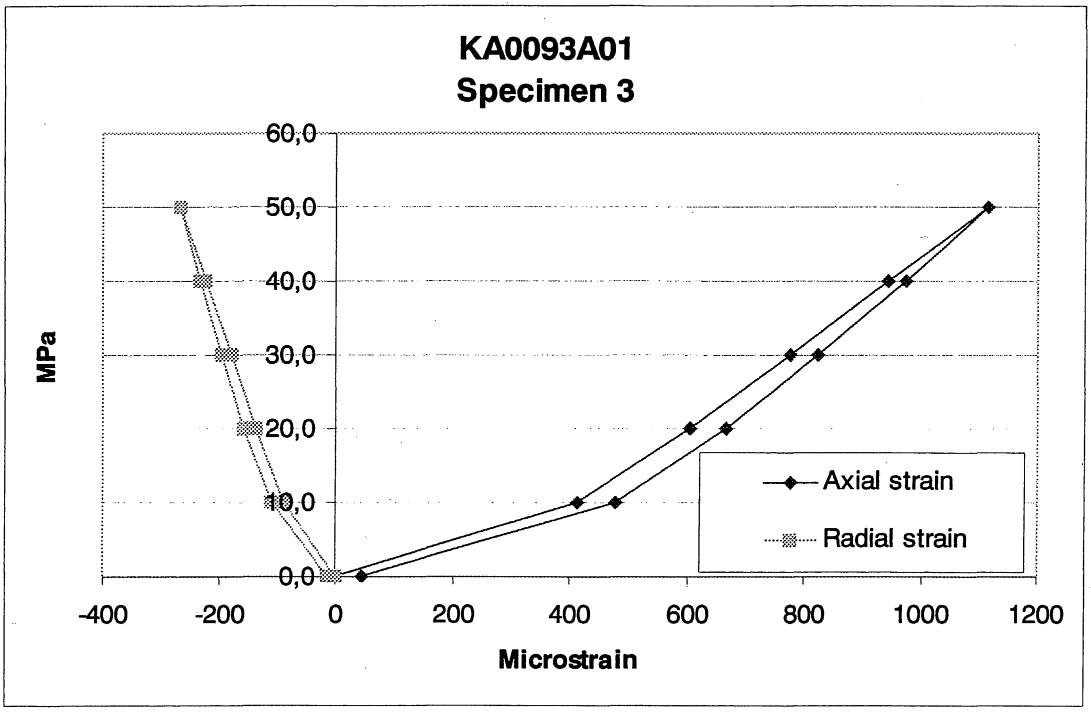
Radial strain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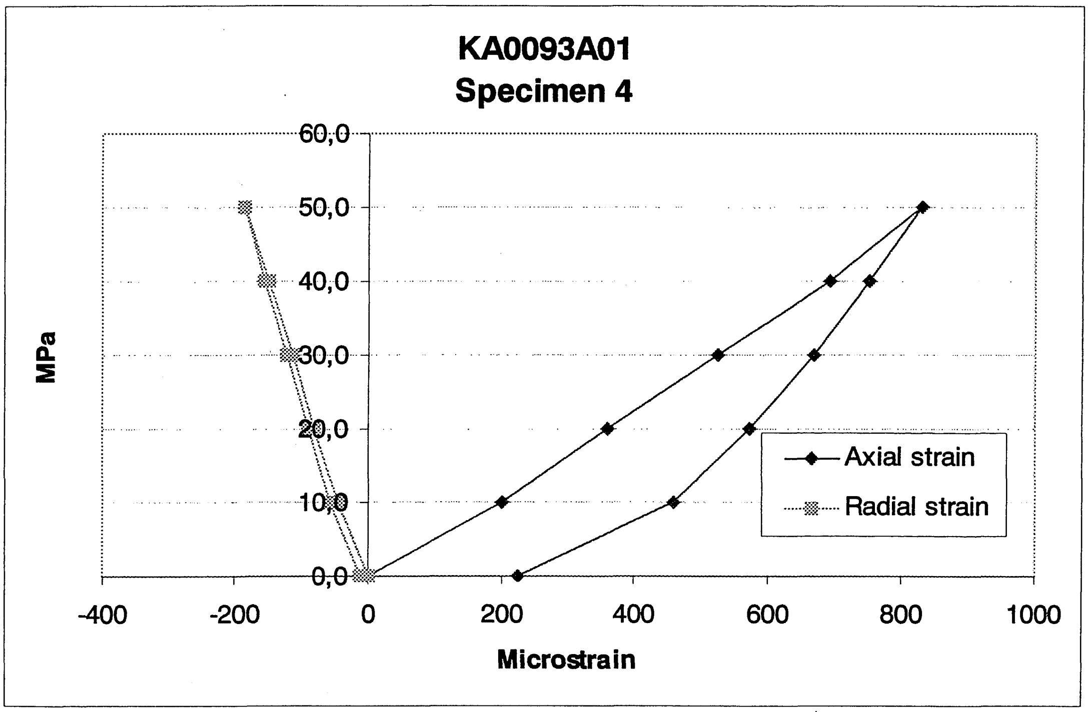
Microstrain
Axial strain
Radial strain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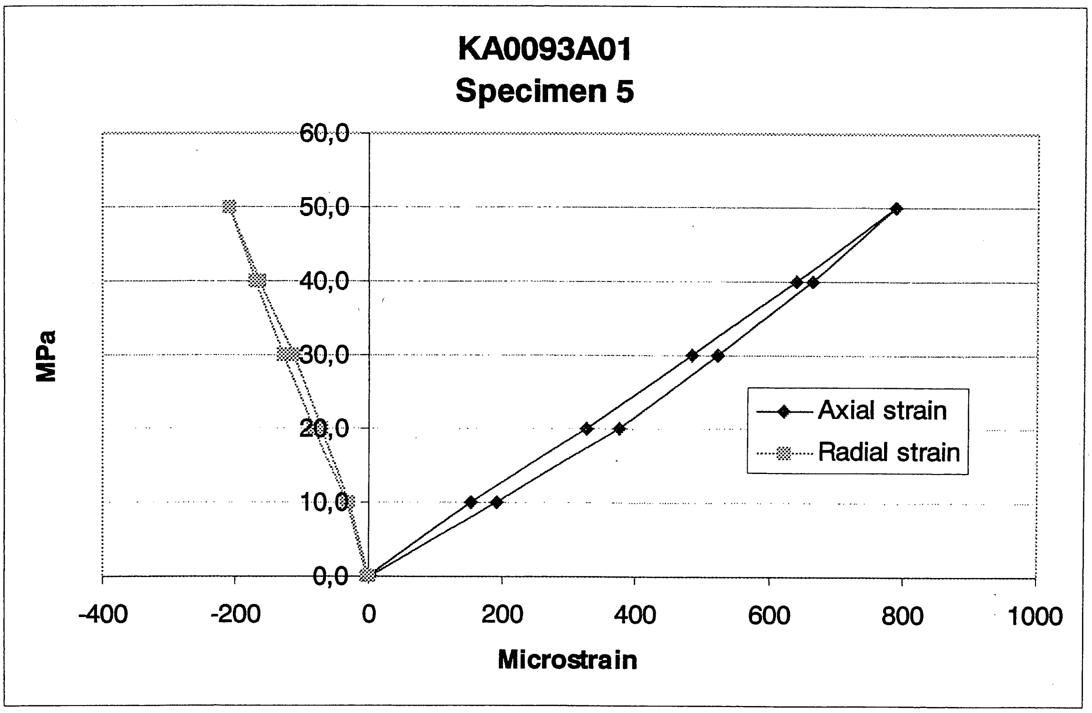
Radial strain

SINTEF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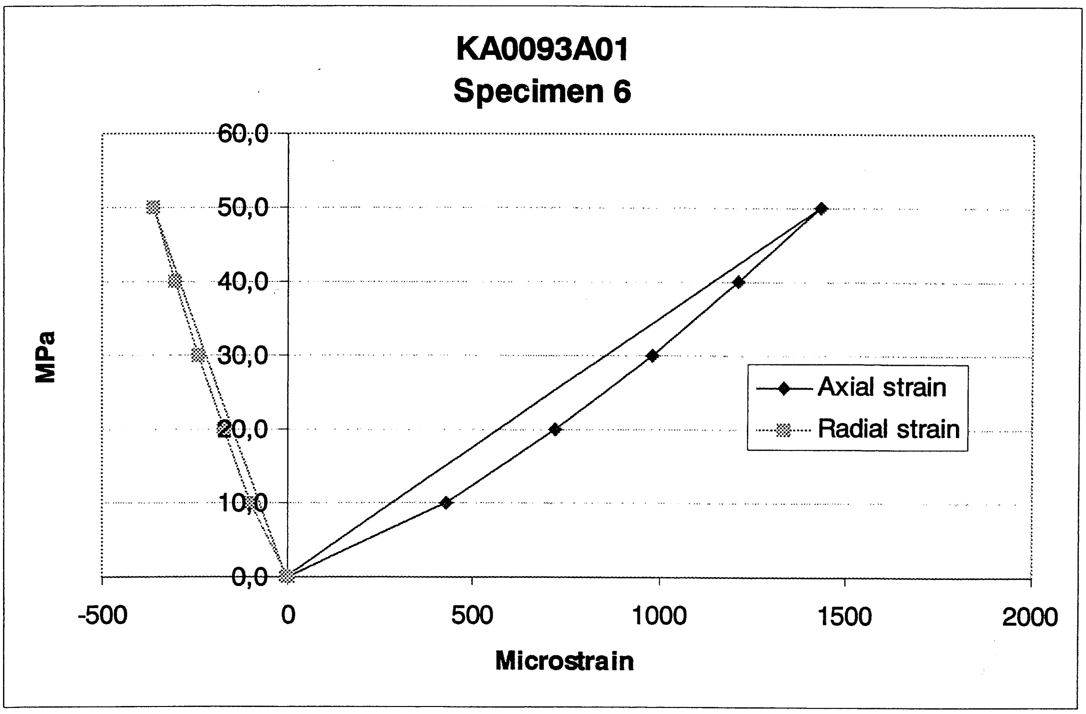
Radial strain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2000
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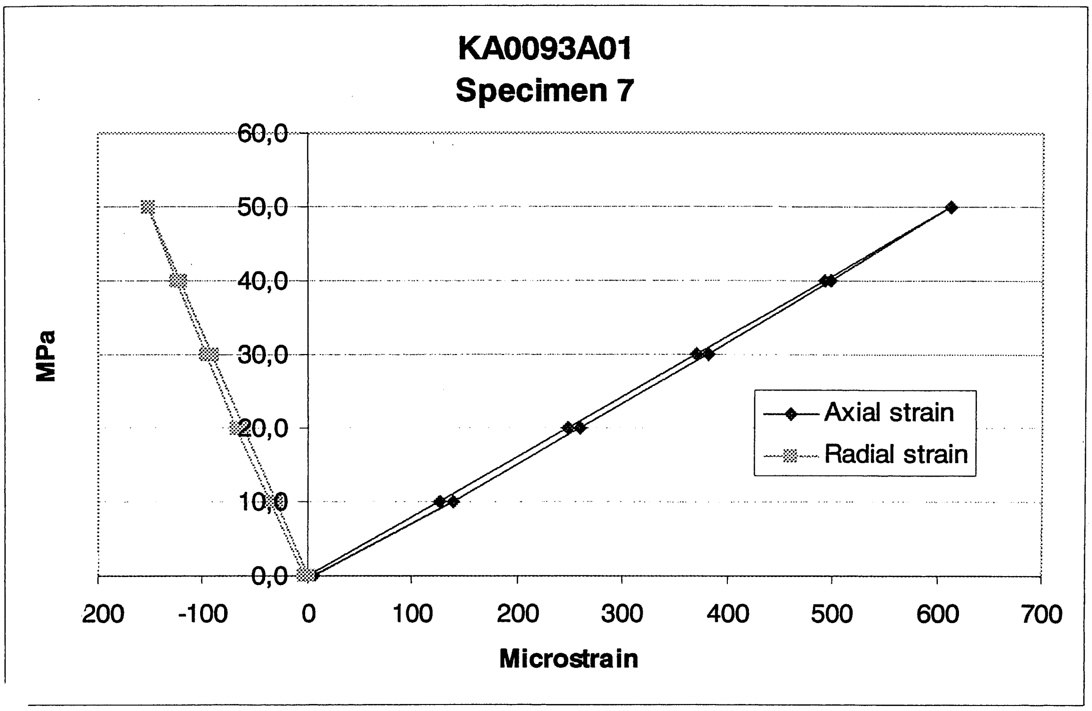
Radial strain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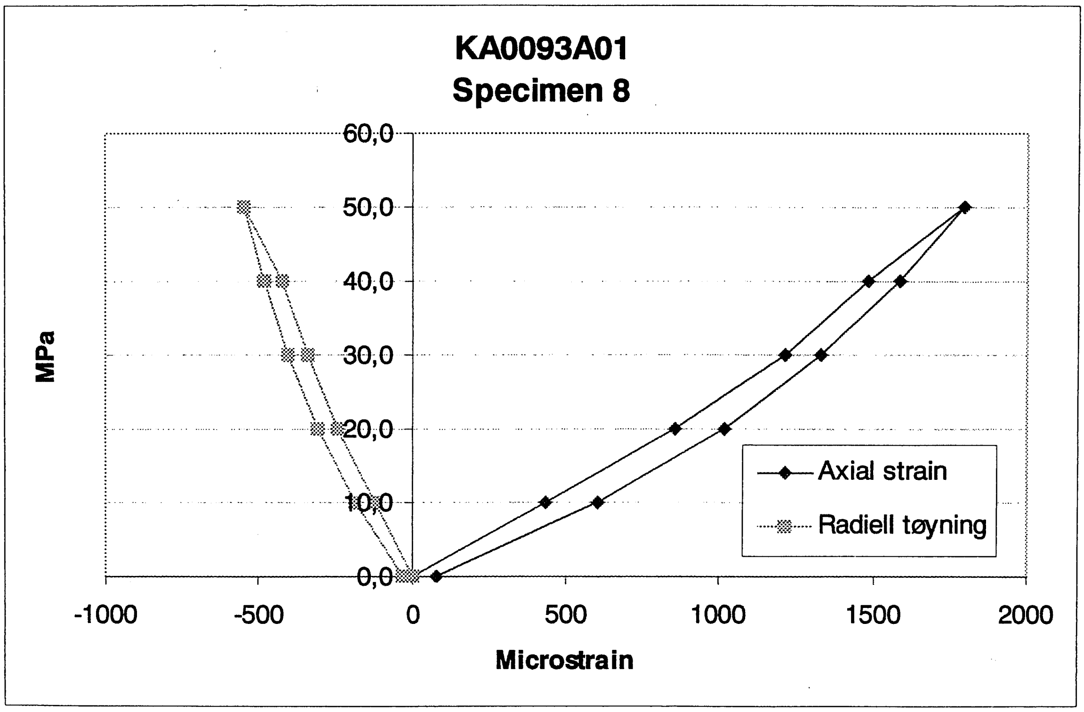
Axial strain
Radial strain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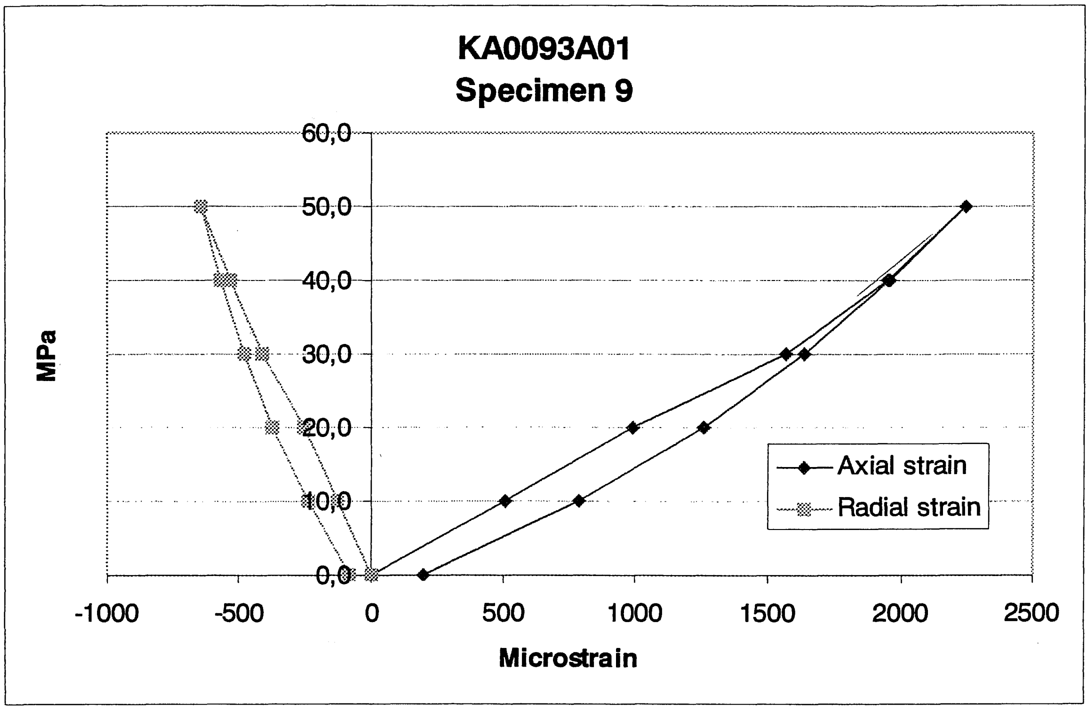
Microstrain
Axial strain
Radiell tøyning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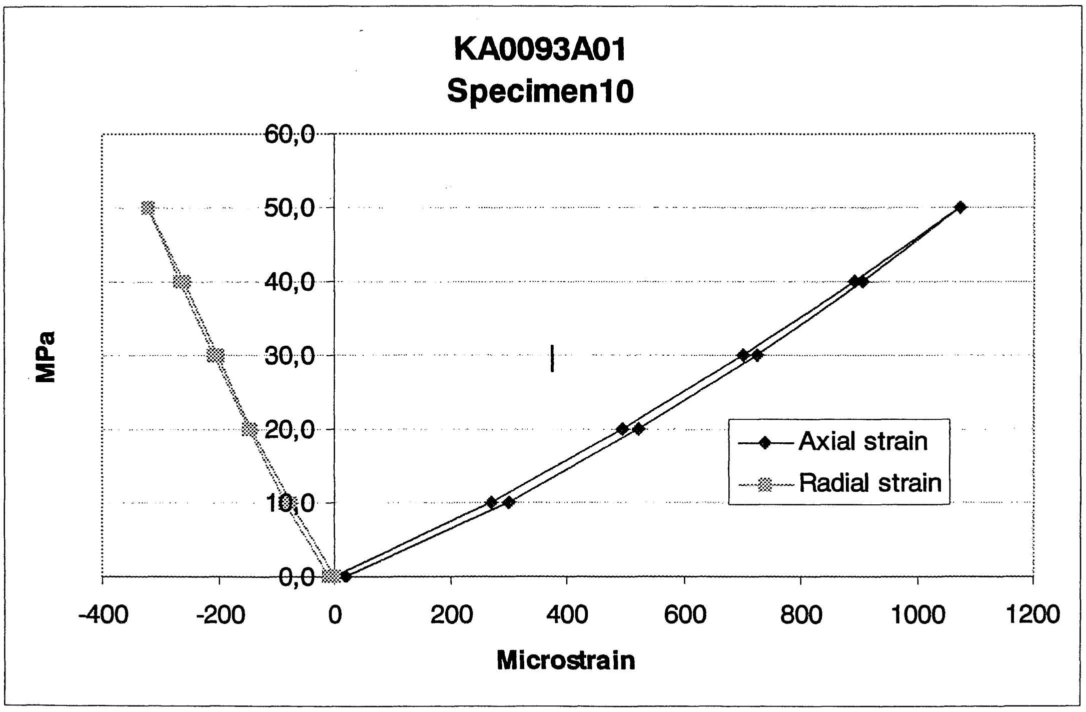
Microstrain
Axial strain
Radial strain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800
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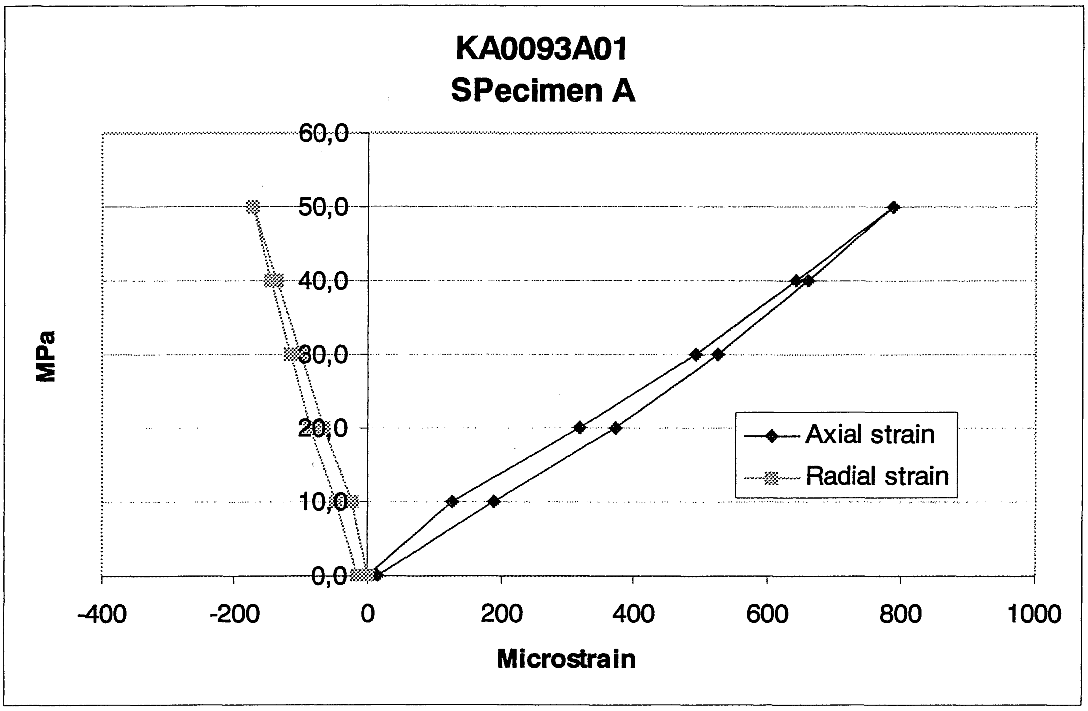
Microstrain
Axial strain
Radial strain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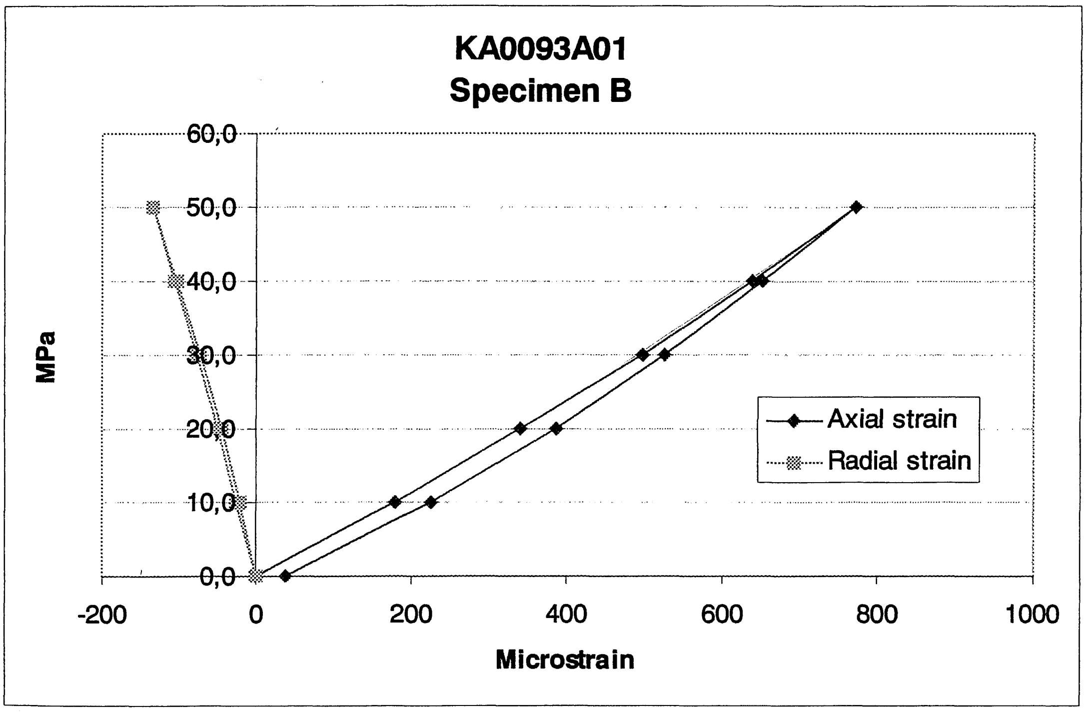
Microstrain
Axial strain
Radial strain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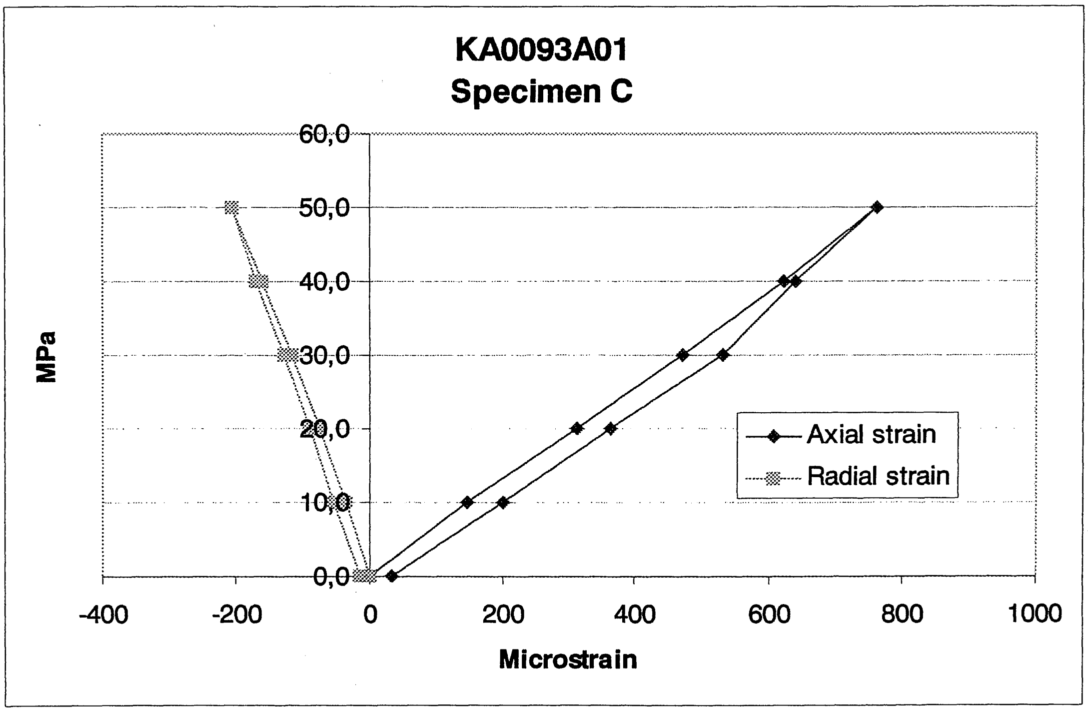
Radial strain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600
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1000
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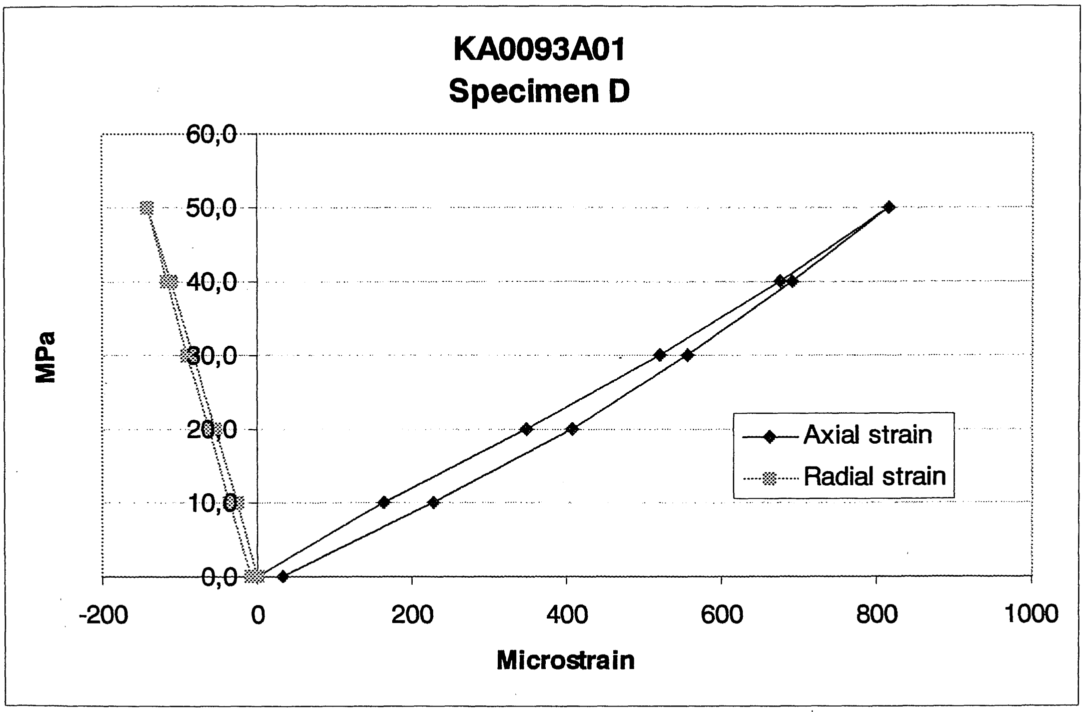
Axial strain
Radial strain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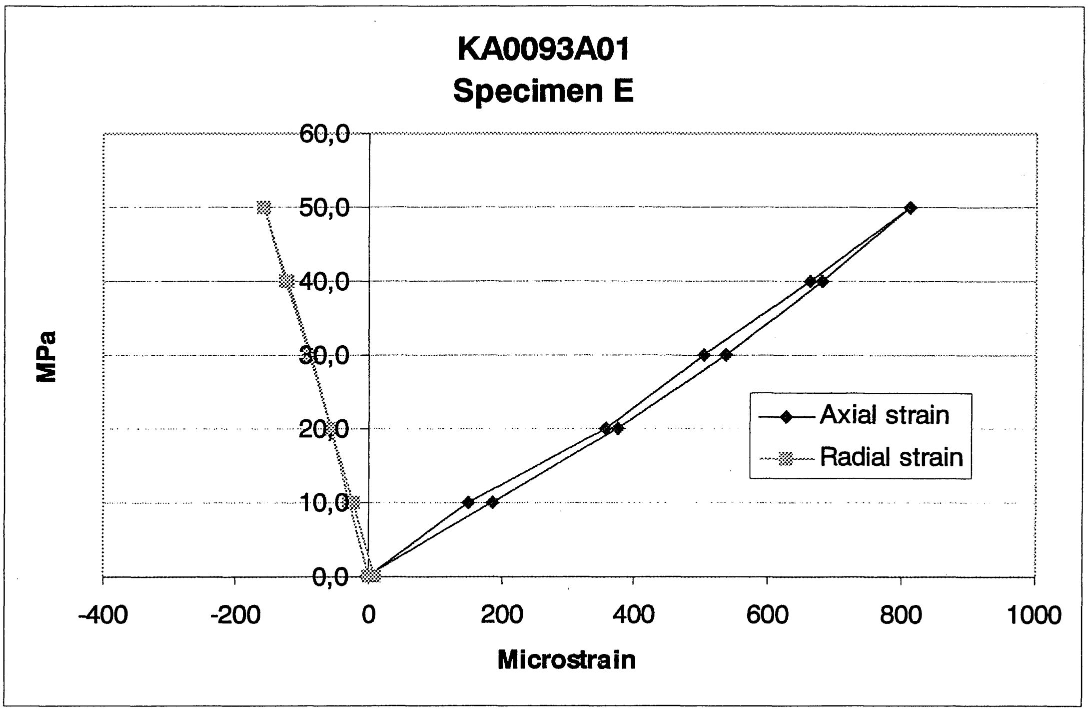
Radial strain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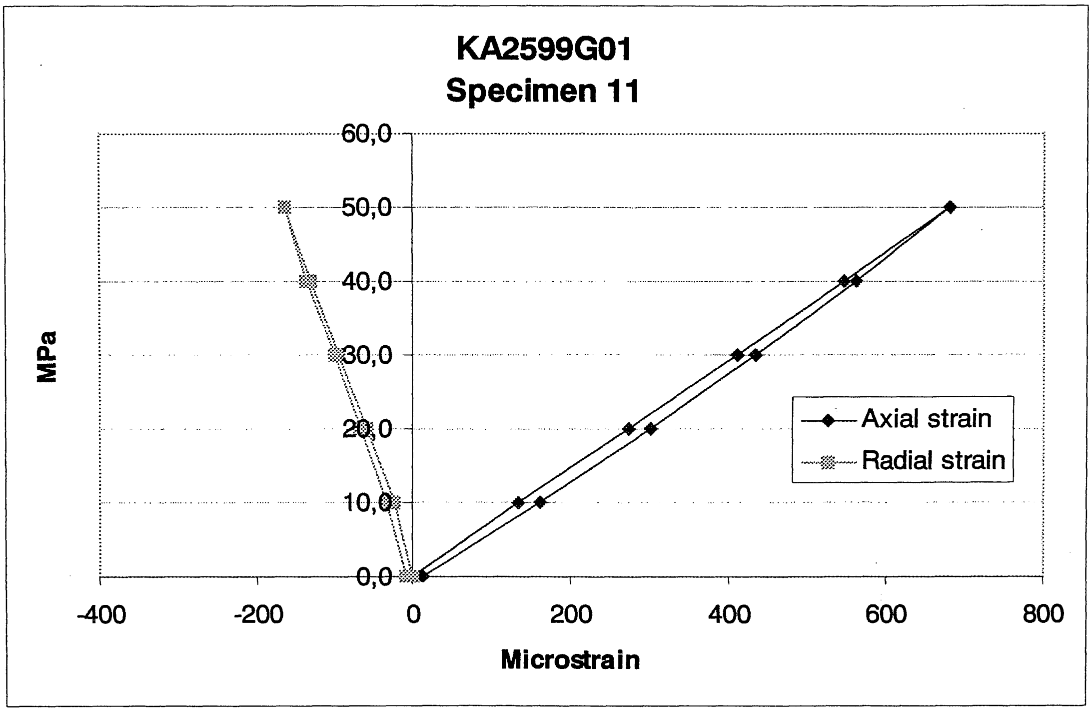
Radial strain



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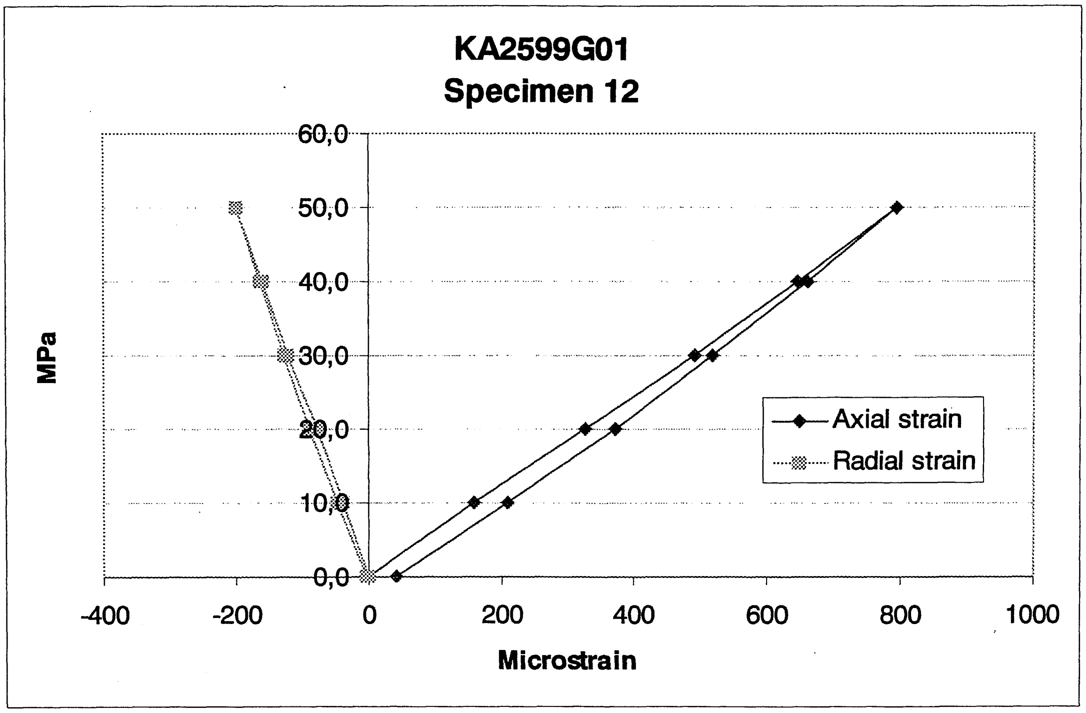
Radial strain



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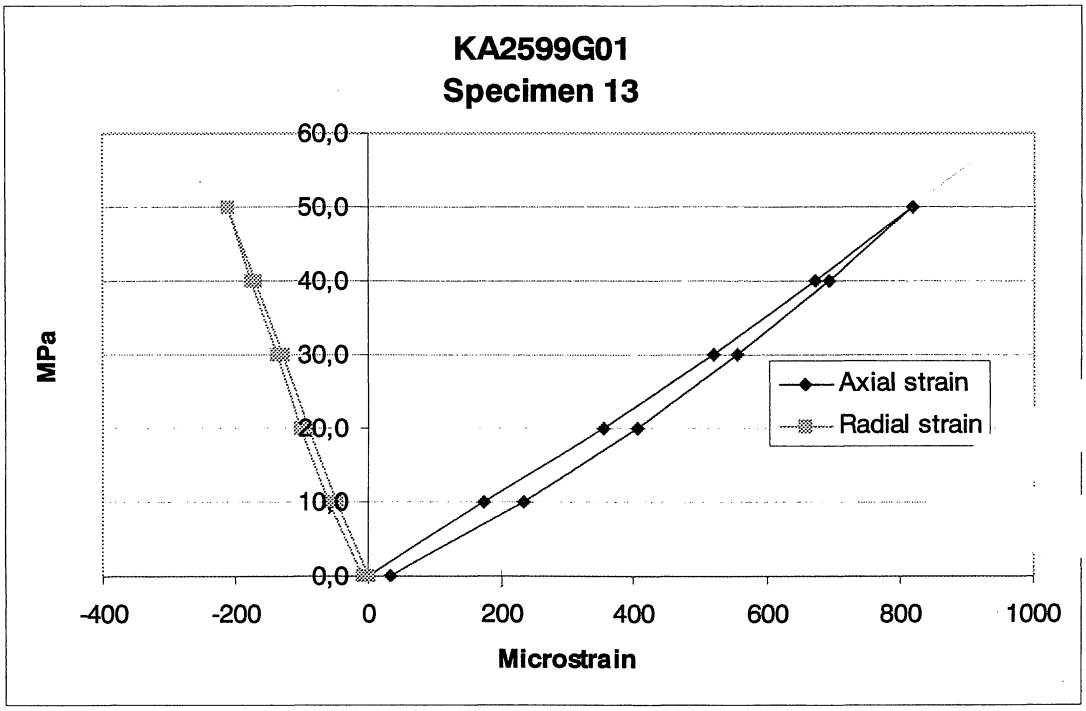

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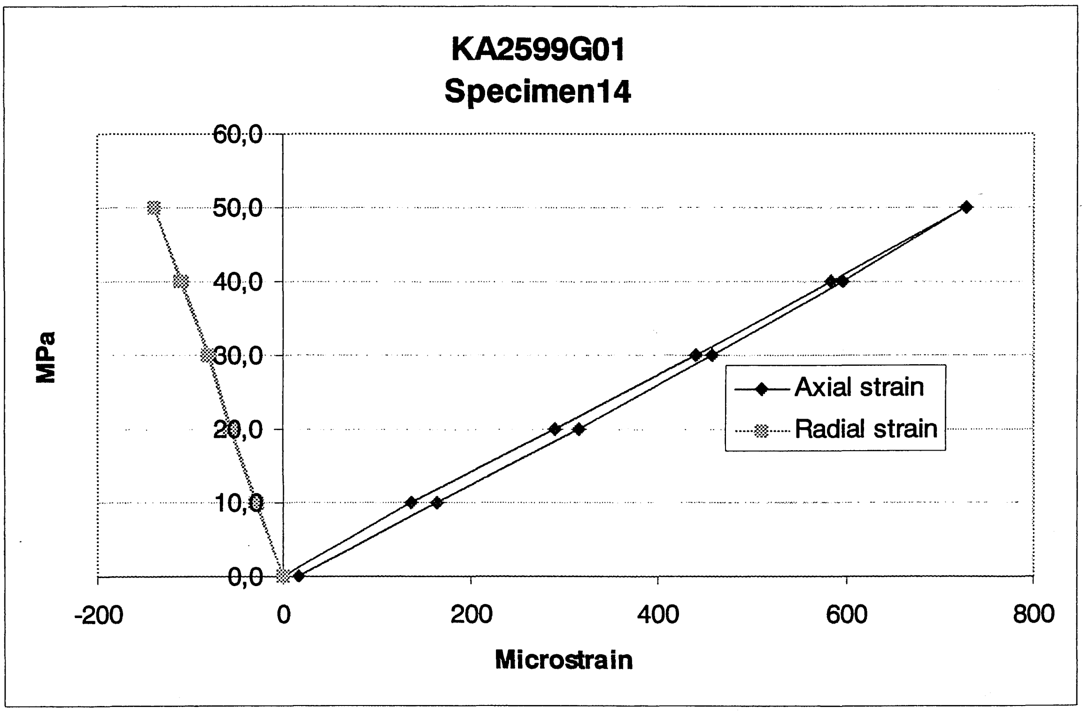
Radial strain



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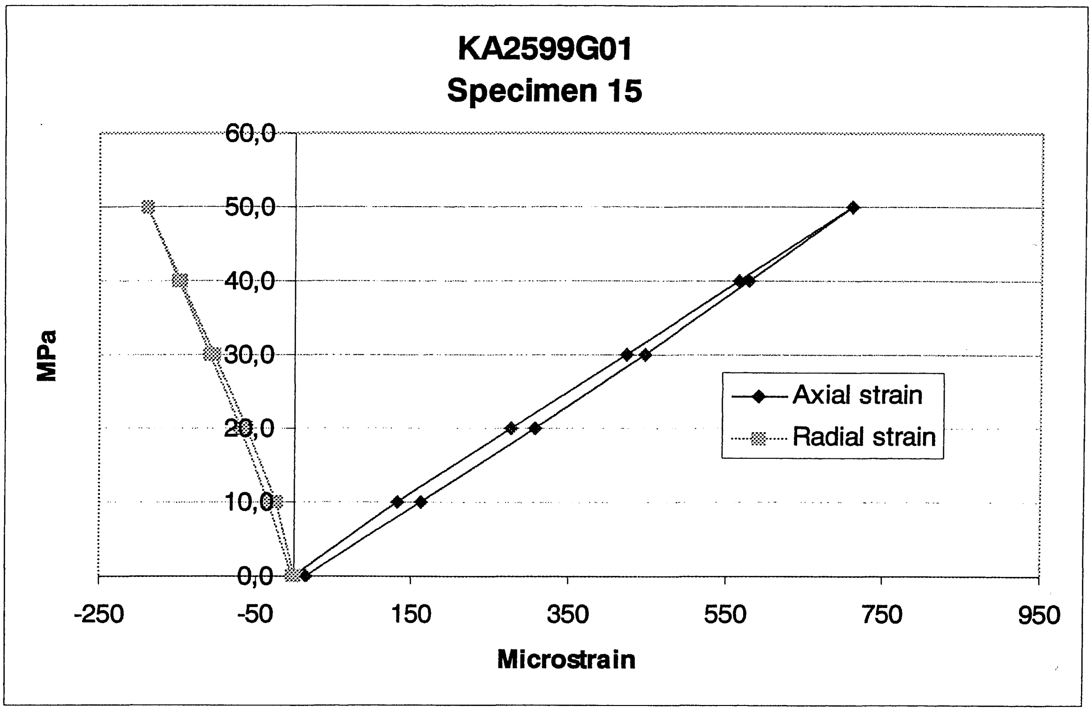
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Microstrain
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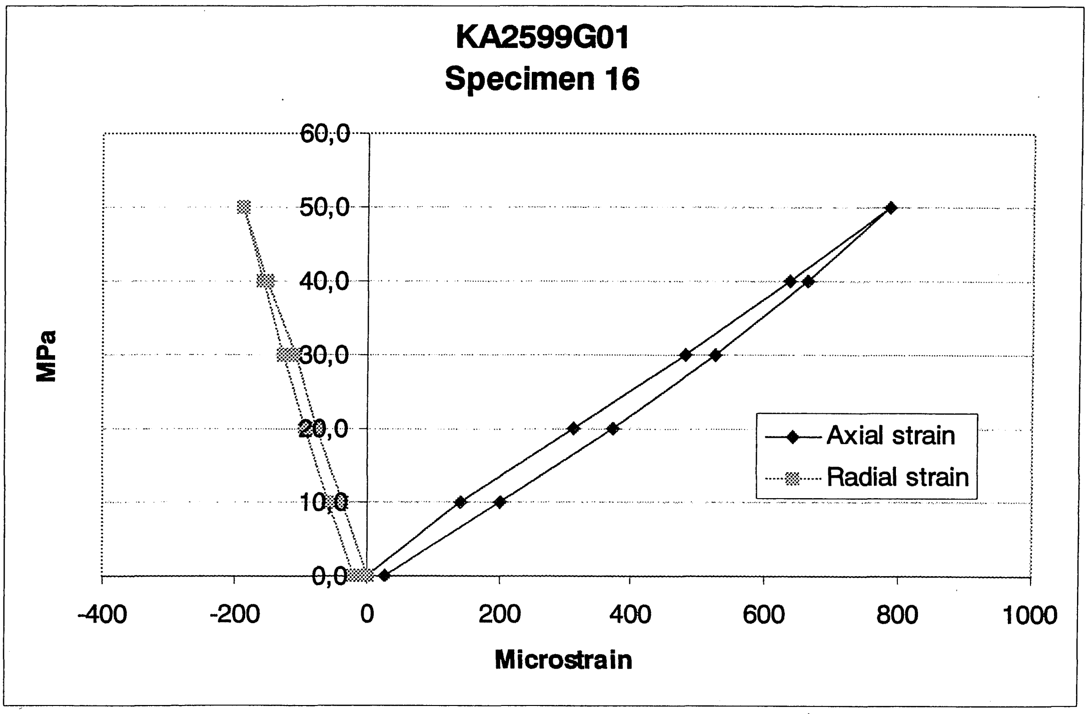
Radial strain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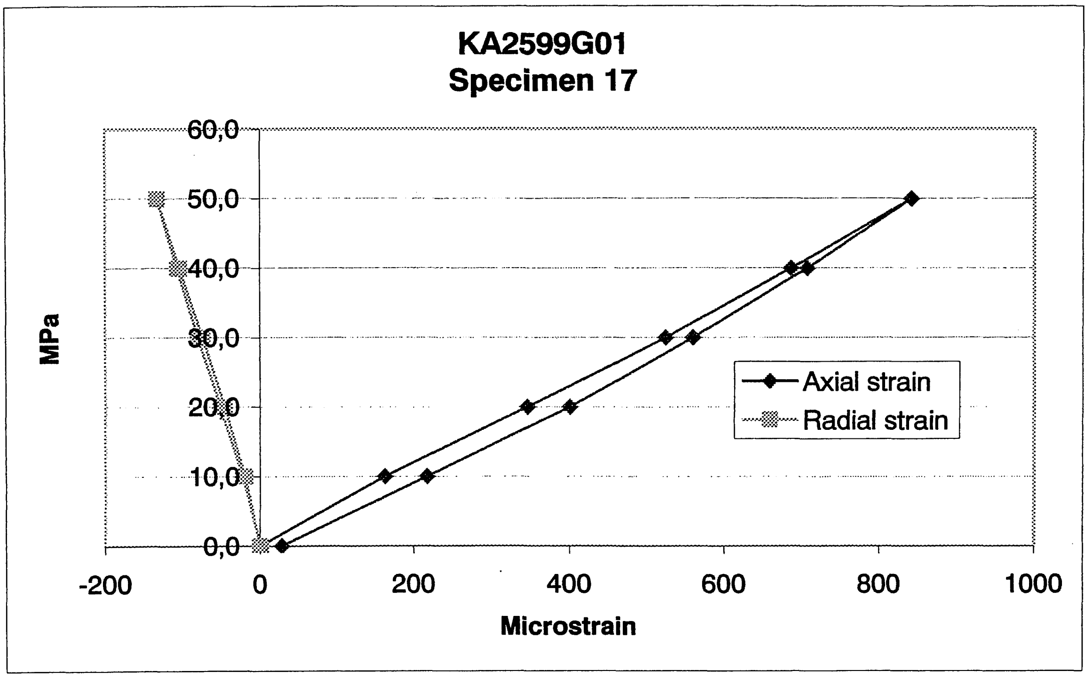
Microstrain
Axial strain
Radial strain



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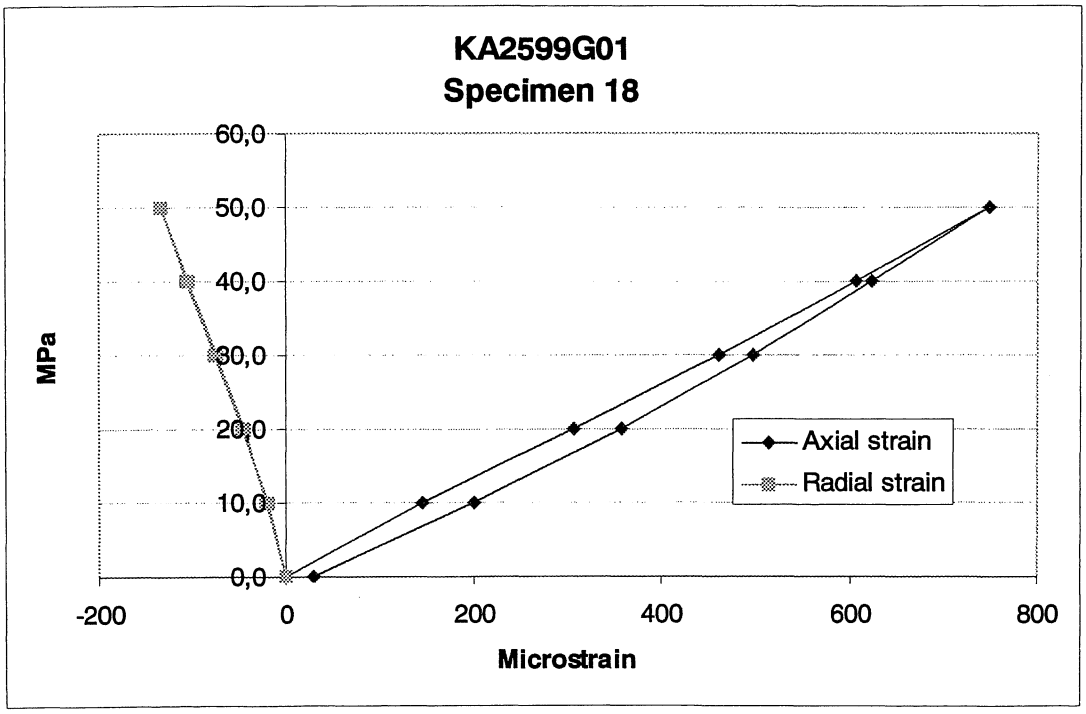
600
800
1000
Microstrain
Axial strain
Radial strain



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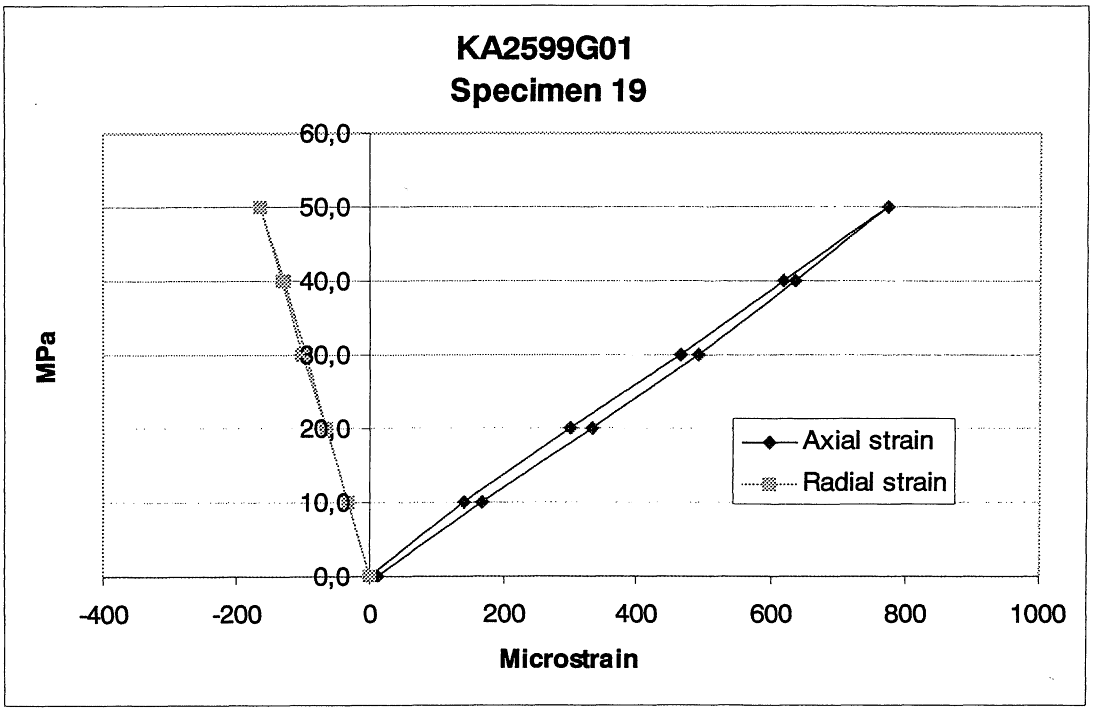
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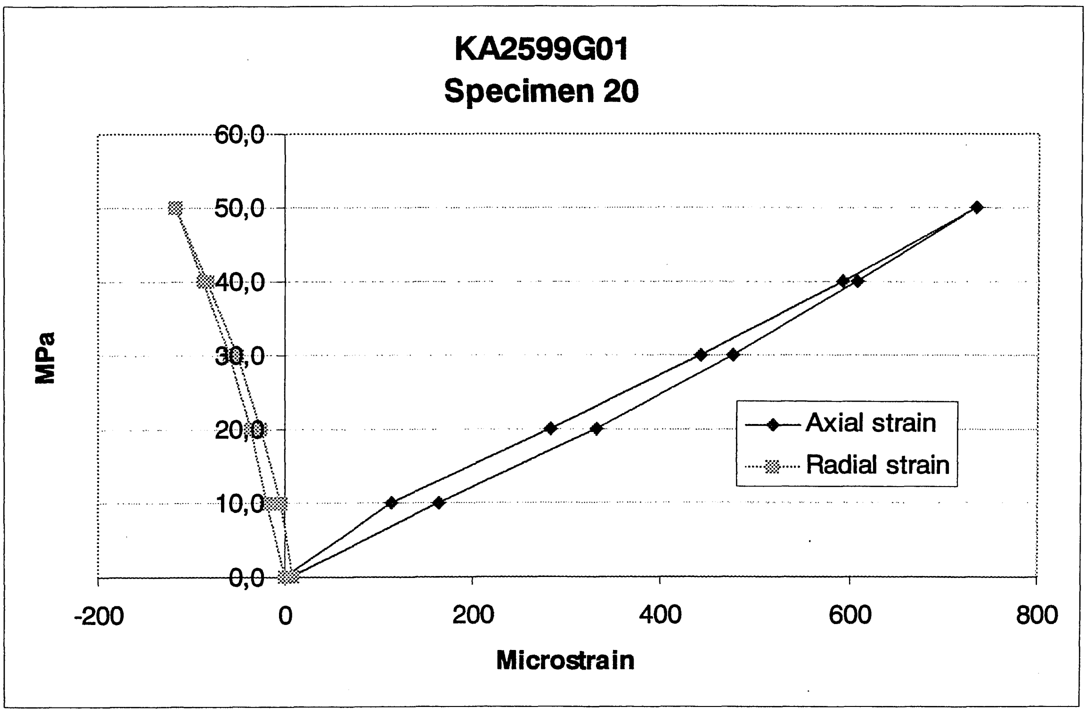
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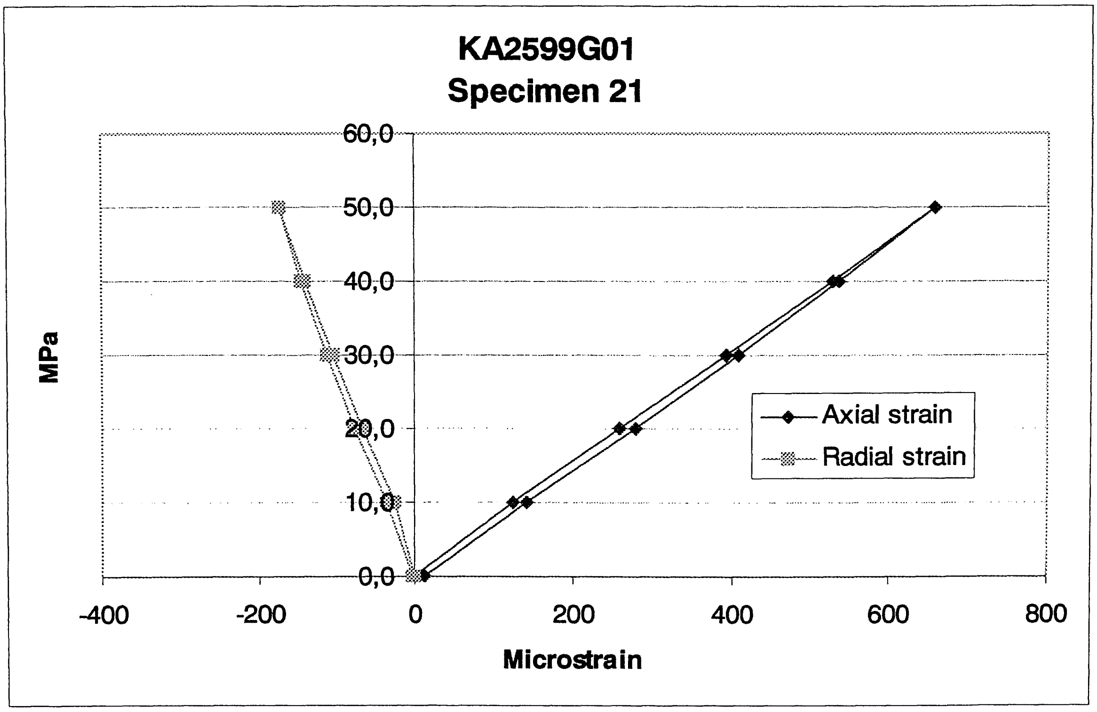
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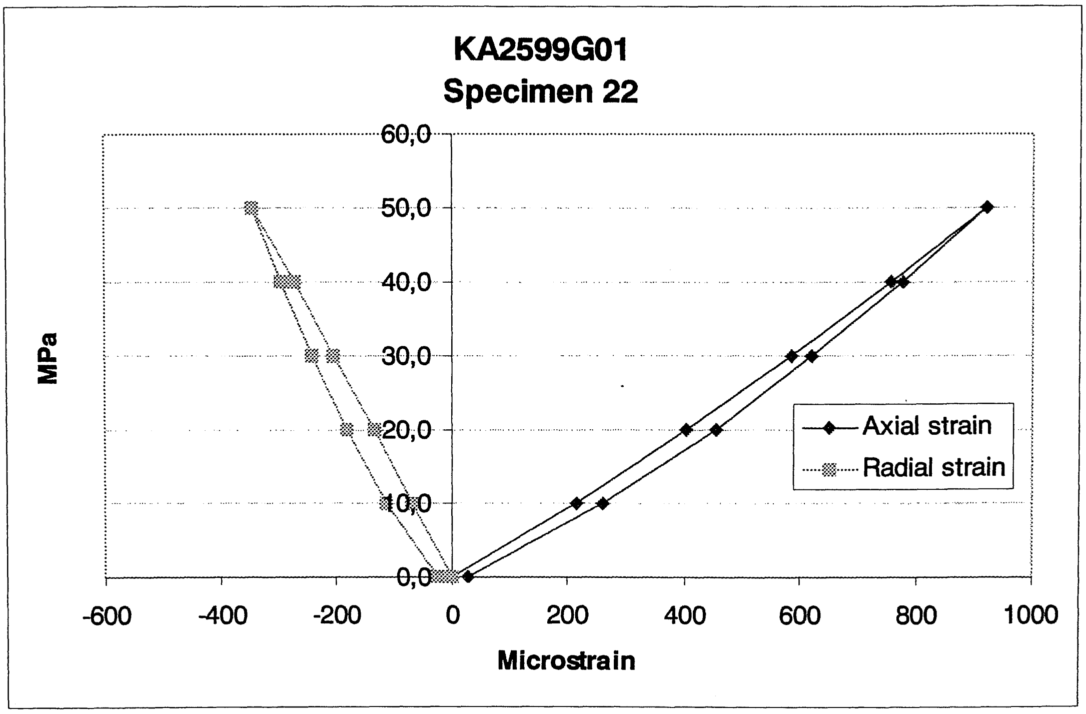
Radial strain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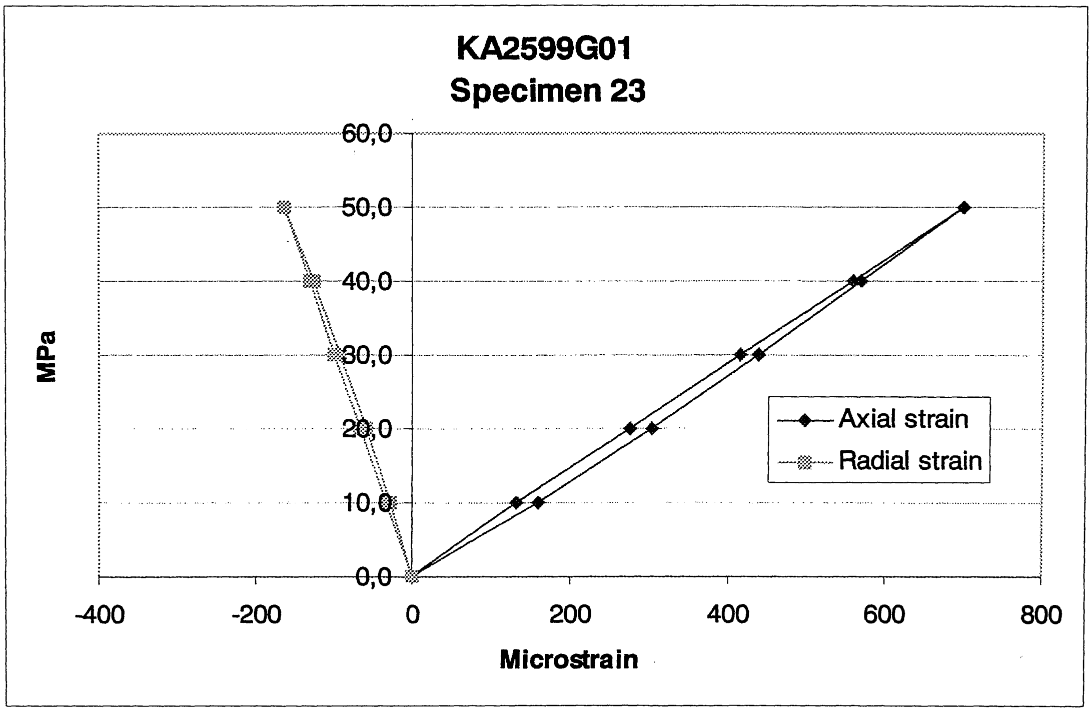
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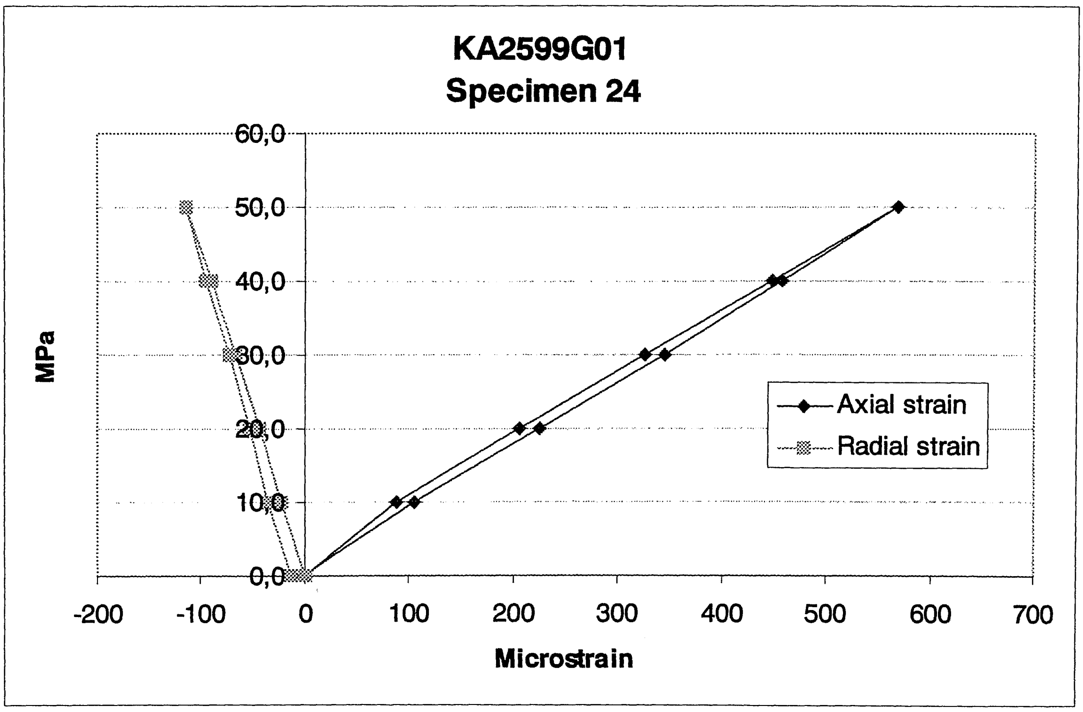
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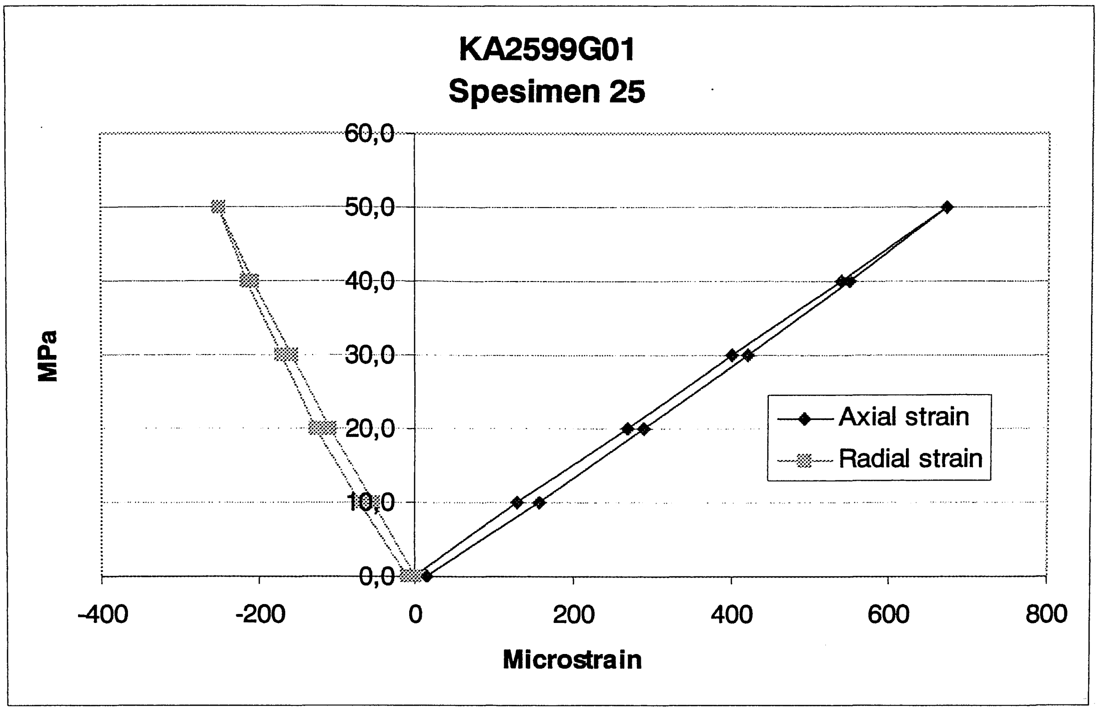
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0
200
400
600
800
Microstrain
Axial strain
Radial strain

Appendix 6

Evaluation of AECL Deep Doorstopper Gauge System tests at the Äspö Hard Rock Laboratory

Arne Myrvang

SINTEF Civil and Environmental Engineering

Rock and Soil Mechanics

Trondheim

Summary

SKB as during May and June 2001 tested the AECL Deep Doorstopper Gauge System (DDGS) for rock stress measurements. Tests were made in the near vertical, water filled borehole KA2599G01 at depth between 107.29 m and 129.28 m, and in the sub-horizontal borehole KF0093A01 at depths between 28.87 m and 31.05 m. The success rate was quite low, and SKF has come up with a number of questions concerning different aspects that may be of concern for the general quality and reliability of the measuring results. This report gives an independent evaluation of the measurements and the stress determination.

Some general comments on the DDGS and doorstopper measurements are given. The importance of the strain invariants in connection with the applied strain gauge rosettes is addressed. The strain invariants are the sums of any perpendicular strain, and according to elastic theory, all strain invariants must be equal. In the practical, doorstopper case, the sum of the 0° and the 90° strains, and the 45° and 135° strains should be equal. This holds very well for the technically successful tests in both boreholes, indicating fairly reliable readings, and also fairly isotropic conditions in the plane of measurement.

The existence of the borehole itself will give stress concentrations at the borehole bottom due to stresses both perpendicular to and parallel with the borehole. This requires corrections of the strains recorded at the borehole bottom. The stress concentration factors are functions of Poisson's ratio of the rock material. Therefore a reliable determination of Poisson's ratio also should be carried out in connection with the final stress calculations. Of particular importance is the effect of the stress parallel with the borehole. In the horizontal borehole case, this explains that quite large negative strains are recorded.

Control measurements with 3D overcoring (and to some extent hydraulic fracturing) indicate that the technically successful DDGS measurements give reasonably reliable results.

Biaxial cell determination of Young's modulus using the overcored doorstopper as strain measurement device will give acceptable values of E, and also of ν provided uniaxial loading in the laboratory. A major issue in this connection and in the field is the quality of the glue bond. This will always be a problem, as gluing under water always will be a most difficult task, and requires continuous attention.

In general the DDGS concept may have a potential as a future alternative for in situ rock stress measurements in deep, water filled boreholes. However, the achievements during practical field testing bear evidence of the technique still being in a development stage, and also to some extent lack of experience. This gives unacceptable high failure rates.

Contents

1 Introduction

SKB has during May and June 2001 tested the AECL's Deep Doorstopper Gauge System (DDGS) for rock stress determination at the Äspö Hard Rock Laboratory. The practical measurements were carried out by a measuring group from the Underground Research Laboratory of the Atomic Energy of Canada Ltd (AECL) in co-operation with a Swedish core drilling contractor. Tests were made in the near vertical hole KA2599G01 and in the sub-horizontal hole KF0093A01. Following the measurement campaign, SKB has come up with a number of questions concerning different aspects that may be of concern for the general quality and reliability of the measuring results. The author of this report has been asked to give an independent evaluation of the measurements and the stress determination, SKB Order 5671 of 2001-10-16.

2 Basis for the evaluation

During a kick-off meeting in Stockholm on 30 October 2001 the following material was handed over:

- Overcoring Rock Stress Determinations Using the Deep Doorstopper Gauge System in Boreholes KA2599G01 and KF0093A01 at the Äspö Hard Rock Laboratory. Measurements report, Underground Research Laboratory, AECL, 2001-07-25 (Ref. 1)

- Application of the deep doorstopper gauge system to deep in situ rock stress determinations. Report no: 06819-REP-01200-10019-R00, March 2000. Ontario Power Generation (Ref. 2)

- PM on 3D overcoring rock stress measurements in borehole KF0093A01 at the Äspö HRL For comparison to the 2D Deep Doorstopper Method. SwedPower report of June 2001 (Ref. 3)

- Hydrofrac / hydraulic injection stress measurements in the Äspö Hard Rock Laboratory Borehole nos. KA2599 and KF0093A01. Preliminary test report MESY Gmbh, 2001-10-25 (Ref. 4)

- SKB Geological logs from Boreholes KA2599G01 and KF0093A01 (Ref. 5)

- Bergspänningsmätning och termisk test i ∅96 mm borrhål. SKB Project plan 2001-02-09 (Ref. 6).

3 General comments on the DDGS and doorstopper measurements

The doorstopper principle for in situ rock stress measurements was originally developed by Leeman, South Africa in the 1960ies, and has since been used in different versions by several organisations throughout the world, including the Norwegian University of Science and Technology and SINTEF, which have used self developed doorstoppers since 1966. The standard doorstopper can normally only be used in drained boreholes on the dry, flattened end of a borehole. The DDGS version is in principle identical with other versions, but major improvements are the cable free in-hole data-logging system IAM -Intelligent Acquisition Module, the wireline installing system permitting installation in deep boreholes without retraction of drilling rods, and the strain gauge glue permitting application in deep, water-filled boreholes. In addition, the DDGS is also supplied with a thermistor, which continuously records the temperature close to the strain gauge.

The main element of the doorstopper is the strain gauge rosette. In the DDGS a standard four gauge 45° apart configuration is used (0°, 90°, 45°, and 135°). This is the normal configuration used in most modern doorstopper versions (In the past, a three gauge 45° configuration was used). An important feature of the four gauge configuration is that the strain invariants must be equal. The strain invariants are the sums of any two perpendicular strains, and according to elastic theory, all strain invariants must be equal. In the practical doorstopper case, the sum of the 0° and the 90° strains, and the 45° and 135° strains should be equal:

$$\varepsilon_0 + \varepsilon_{90} = \varepsilon_{45} + \varepsilon_{135}$$

This is a very simple and reliable procedure to check if a doorstopper overcoring result is technically successful.

The standard procedure for stress calculations from measured strains is in principle to use standard strain gauge rosette expressions derived from elastic theory to determine the (secondary) principal strains and their directions in the plane of the borehole bottom, and then using Hooke's law to calculate the principal stresses. Linear elastic behaviour of the rock is normally assumed, (although more sophisticated tranverse isotropic solutions exist). In the Äspö DDGS case, linear elastic, homogenous rock is assumed.

Due to the existence of the borehole itself, stress concentrations will occur at the borehole bottom due to the rock stresses both perpendicular to and parallel with the borehole. The relation between the in situ far stress field and the stresses at the borehole bottom may be expressed by the following general equations:

$$\bar{\sigma}_x = a\,\sigma_x + b\,\sigma_y + c\,\sigma_z$$

$$\bar{\sigma}_y = a\,\sigma_y + b\,\sigma_x + c\,\sigma_z$$

$$\bar{\tau}_{xy} = (a - b)\,\tau_{xy}$$

where σ_x, σ_y, σ_z and τ_{xy} are the in situ far field stresses. The z – axis is parallel with the borehole axis.

In addition, water pressure p_o will also influence the results through a fourth factor. Over the years, a number of stress concentration factors have been derived by different researchers with somewhat different values. However, in most cases the differences are not too critical, and in all cases they will vary with the Poisson's ratio of the rock material.

The factor b is either zero or very small and are normally for engineering purposes set as zero.

For Poisson's ratio approximately 0.25, typical values for a and c may be a = 1.25 – 1.30, and c = - 0.60 – 0.70

In the DDGS case, stress concentration factors according to Leite is used (see Ref. 2, Attachment E). This also includes a factor for water pressure. The stress concentration factors are presumed to be included in the computer code for stress determination.

It is important to note the pronounced influence from the stress acting parallel with the borehole. Accordingly, it is very important to know the correct value of that stress to obtain proper results. This is probably the most difficult challenge in connection with doorstopper measurements in general, and the use of the normal doorstopper should therefore, after the author's opinion, be limited to cases where the stress parallel with the borehole is known to be zero or very low. Typical cases are measurements of tangential stresses in the immediate roof or walls of rock chambers (including shotcreate or concrete support), and stress profiles through relatively slender rock pillars.

During practical doorstopper measurements the preparation of the borehole bottom is crucial. A common phenomenon is the formation of a "dimple" in the centre of the bottom. This is normally formed because a diamond is lost in the centre of the flat diamond bit. Several cases of this during tests in Canada are mentioned in Ref. 2, and one case is reported in Ref. 1 during the Äspö campaign. The DDGS procedure also includes a final polishing of the bottom. The author of this report believes that this is unnecessary and may even have harmful effects on the bond between the doorstopper and the rock. The slightly rougher surface created by the diamond bit gives a far better adhesion. In a vertical hole, this might also make the bond less sensitive to potential fine mud remnants at the bottom. The surface of the strain gauge rosette should also be slightly roughened by using fine sandpaper to improve the adhesion even more.

(To avoid dimples, SINTEF uses a specially designed flat diamond bit with a special ex-centric, circular insert covering the bit centre. By rotating the insert a few degrees before every flattening, there will always be a "fresh" diamond in the centre, and formation of a dimple very seldom occurs).

Gluing of strain gauges under wet conditions / under water will always be difficult. In the DDGS case a glue known as Versilok is applied (+ the HBT X60, a well known, high quality strain gauge glue). Comprehensive tests has been carried out on Lac du Bonnet grey granite, and the results as presented in Ref. 2 are apparently very convincing. However, it is the author's experience that even if a glue is perfect for one type of rock, this may not be the case with other rock types. One reason for this may be that the glue may have different affinity to different mineral surfaces.

4 Analysis of DDGS measurements at Äspö

The results are presented in Ref. 1. The different tables and graphs will not reprinted in this report. The reader is kindly requested to refer to Ref. 1 if necessary.

4.1 The sub-vertical borehole KA2599G01

The test hole was drilled from the -340 m level, plunging 80° with borehole azimuth at the test depth of approximately 325°. DDGS tests were carried out at five locations between borehole depths 107.29 m and 107.95, and three were technically successful.

Eight tests were performed between 128.11 m and 129.28 m with only one technically successful.

The overcore strain recording graphs are presented in fig. 1, 2, 3 and 4 in Ref. 1. From some reason, no attempt has been done in Ref. 1 to carry out the invariant check. From the recorded strain graphs the author has manually picked out the different strains from the stable part of the graphs and checked the invariants. The results are presented in table 1:

Table 4-1: Strain invariant check of the DDGS tests in Borehole KA2599G01.

	Depth 107.29	**Depth 107.79**	**Depth 107.95**	**Depth 128.28**
ε_v	173	129	310	65
ε_h	491	466	445	580
Sum	**664**	**595**	**765**	**645**
ε_{45}	391	272	570	410
ε_{135}	291	319	220	250
Sum	**682**	**589**	**790**	**660**

From the table it will be seen that the strain invariant checks in all measuring points show a very good compatibility. This again indicates that the technical quality of the measurements as such are good, and that the rock probably also is fairly isotropic in the measuring plane in each of the test locations.

To investigate the anisotropy more closely, SINTEF has carried out laboratory tests on 21 mm

cores drilled out of 61 mm cores from the test holes (separate report). Young's modulus of cores drilled out parallel with and normal to the apparent maximum horizontal stress and parallel with borehole axis (vertical) has been determined. This gave the following results:

$E_{parallel}$ = 69.4 ± 6.3% GPa, E_{normal} = 65.9 ±4.9% GPa, and E_{axis} = 73.5 ±15% GPa

I.e. fairly isotropic conditions for a rock. The stress - strain curves also indicate fairly linear behaviour.

The values of the magnitudes and directions of the maximum and minimum horizontal stresses from location to location is fairly consistent with average values 38.7 MPa and 21.8 MPa respectively. The direction of the maximum horizontal stress N125 E coincides fairly well with previous measurements at Äspö.

Based on this, in may be concluded that the successful tests seem to give reasonably reliable results.

After the DDGS tests Mesy GmbH has performed hydraulic fracturing tests in the same borehole (Ref. 4). The direction of the maximum horizontal stress coincides very well with the DDGS value with an average of N 128 E. However, the magnitudes are only half the values of the DDGS values with averages 21.8 MPa and 11 MPa respectively.

The reason for this is difficult to explain (see also under analyses of borehole KF0093A01).

4.2 The sub-horizontal borehole KF0093A01

The test hole was drilled from the -450 level with azimuth N 130 E, which is the apparent direction of the maximum horizontal stress in the area. The hole is drilled slightly upwards (3°). Eight DDGS tests were carried out between 28.87 m and 31.05, but only the last three tests were technically successful. The X60 glue was used for the two first tests. This glue requires 100% dry conditions and will fail when used on wet surfaces. The use in this case should therefore have been omitted.

The successful overcore strain recording graphs are presented in fig. 9, 10 and 11 in Ref. 1.

The author has again manually carried out the strain invariant check based upon the stable part of the graphs. The results are presented in table 2:

Table 4-2: Strain invariant check of the DDGS tests in borehole KF0093A01

	Depth 30.23	**Depth 30.89**	**Depth 31.05**
ε_v	337	397	563
ε_h	-285	-230	-296
Sum	**52**	**167**	**267**
ε_{45}	23	144	259
ε_{135}	17	17	7
Sum	**40**	**161**	**266**

Again it will be seen that that the strain invariant checks show good compatibility in all measuring points, indicating that the technical quality of the measurements is fairly good, and also that the rock in the plane of measurement probably is fairly isotropic.

The strain sets in the three test points are fairly consistent, showing an increasing trend.

A somewhat surprising result is the high negative (tensile) values of the horizontal strains. This is not uncommon in connection with doorstopper measurements in slender vertical rock pillars, but will at first sight look strange within a solid rock mass. It is likely that this is connected to the high stress parallel with the borehole.

To illustrate this, a simple manual calculation of the stresses has been done for the test at 30.23 m, assuming Young's modulus E= 42 GPa and Poisson's ratio ν =0.25 as in Ref. 1. , and stress concentration factors a = 1.25 and c = - 0.65. This gives the following results with and without correction for the axial stress:

Without correction for axial stress: σ_v = 13 MPa σ_h = - 10 MPa

With correction for axial stress 38.7 MPa: σ_v = 33 MPa σ_h = 10 MPa

(Please observe that the calculated stress values are approximate values only)

The values computed in Ref. 1 are : σ_v = 27 MPa σ_h = 12.2 MPa

This clearly shows the importance of the axial stress parallel with the borehole, and that the measured doorstopper strains are not unrealistic.

The gravity vertical stress at the test site is approximately 12 MPa. This could indicate that the orientation of the strain gauge rosette is 90° wrong as the computed vertical stresses in Ref. 1 are 12.2, 12,7 and 12.2 respectively. However, this was thoroughly checked by the test crew after the tests and can be ruled out.

To check the doorstopper results, SwedePower has carried out 3-D overcoring tests in the same hole between 32.14 m and 35.38 m (Ref. 3). The two closest tests to the DDGS tests also showed much higher vertical stress than the gravity stress with 20.4 MPa and 24.3 MPa respectively, while the third test approximately 3 m further in gave a vertical stress of 9.2 MPa, i.e. fairly close to the gravity stress. The minimum horizontal stresses , which are all approximately perpendicular to the borehole axis, are 9.1 MPa, 9.2 MPa and 12.3 MPa respectively, i.e. fairly close to doorstopper values. This may indicate a local stress anomaly in the test area. The geological core log (Ref. 5) shows that there is a an inclusion of fine grained granite in the diorite in the middle of the test area, and this may at least be part of the explanation.

In general, it is the author's experience that distinct rock type borders may give considerable disturbances both in magnitude and directions of the stresses.

Mesy GmbH has also carried out hydraulic fracturing tests in borehole KF0093A01.

The interpretation (preliminary) done in Ref. 4 is confusing. It is not clear to the author of this report how the two horizontal stresses can be determined from hydraulic fracturing in a horizontal hole. According to the steroplot in fig. 5 in Ref. 4, the majority of the induced fractures are oriented more or less vertically. This indicates that the maximum stress in a plane normal to the borehole axis is more or less vertical. If this is correct, the hydraulic fracturing show the same pattern as the two overcoring techniques has indicated, i.e. a vertical stress anomaly.

In Ref. 1 extreme anisotropy in the rock fabric is suggested as one of the explanations for the "strange results" (i.e. the large negative strains). The analysis above indicates that the results are not strange when compared with the 3-D measurements, and taking the high stress parallel with the borehole into consideration.

SINTEF has also carried out laboratory tests on 21 mm cores drilled out of 61 mm cores from this test hole (separate report). Young's modulus of cores drilled in the vertical, horizontal, and axial directions has been determined.

This gave the following results:

$E_v = 58.7 \pm 14.4\%$ GPa, $E_h = 42.8 \pm 55\%$ GPa and $E_{aks} = 64 \pm 3.5\%$ GPa

I.e. there is apparently a pronounced anisotropy horisontally perpendicular to the borehole. However, the standard deviation in that direction is also very high compared with the other directions. A visual inspection of the actual 61 mm core shows that the core contains a lot of large feldspar crystals up to 20 mm in diameter, sometimes with visible cracks between crystals. This may cause large differences in the measured elastic properties, which actually is not caused by real anisotropy but rather that the rock is in-homogenous.

The stress-strain curves in this case are more curvilinear and show larger hysteresis than the others.

Regardless of this, the anisotropy measured is not extreme, and can hardly give "strange results" as indicated in Ref. 1.

Compared with the results from Borehole KA2599G01 the following relations exists:

E_v = 58.7 GPa corresponds to E_{axis} = 73.5 GPa

E_h = 42.8 GPa corresponds to E_{normal} = 65.9 GPa

$\underline{E_{aks}}$ = 64.0 GPa corresponds to $E_{parallel}$ = 69.4 GPa

Average: 55.0 GPa 69.6

This shows the same tendency as the biaxial cell results, i.e. the average Young's modulus in the horizontal hole is lower than the one in the vertical hole.

5 Determination of elastic constants -biaxial cell testing

Biaxial cell testing has for decades been used to determine elastic parameters in connection with overcoring. The procedures used in connection with the DDGS with an aluminium extension of the core are appropriate, and should give reasonably reliable values for Young's modulus. However, Poisson's ratio cannot be determined directly. As shown above, the different stress concentration factors are dependant upon Poisson's ratio. In a case with high stress parallel with the borehole (as with the horizontal hole) it is much more important to have correct values of Poisson's ratio than in a case with moderate axial stress. The Poisson's ratio should therefore in this case be determined in the laboratory by uniaxial loading of the core with the doorstopper attached, as described in Ref. 2.

During practical measurements it is always a chance that the glue layer between the strain gauge and the rock get different thickness, and sometimes also variation of thickness over the doorstopper surface. This means that the strains recorded during overcoring and also during biaxial testing may be too low if the glue layer is too thick. This will again result in a too high Young's modulus. This may be the case in the vertical hole at 128.28 m, where a value of 120 GPa has been determined in one of the directions. This is clearly an unrealistic high value.

6 General impression of the DDGS as used at Äspö

It is the author's opinion that the DDGS has a potential as a future alternative method for in situ rock stress measurements in deep boreholes.. However, the present achievements during practical testing bear evidence of the technique still being in a development stage, and also to some extent lack of experience. This results in unacceptable high failure rates.

A major concern will always be the quality of the gluing, as has been shown also in the Äpsö case. In this connection the preparation technique used for flattening the borehole bottom should be improved, and further glue testing on different rock types is recommended.

7 Test report (Ref. 1) conclusions

The conclusion in Ref. 1 is that the calculated results achieved during the test programme are not valid. The author of this report does not agree. As shown in the analysis above, the technically successful measurements are reasonably reliable, provided the high stress parallel with the borehole is taken into consideration.

Laboratory tests carried out by SINTEF show that the degree of anisotropy is relatively modest. However, the rock may locally be quite in- homogenous, and this may highly influence the elastic constants.

In general it must be remembered that “rock is rock”, i.e. there will always be smaller or larger discrepancies between ideal conditions and conditions in real rock mass. This often will call for interpretations based upon a good knowledge of the local geological conditions.

8 Conclusion

In general DDGS concept has a potential as a future alternative for in situ rock stress measurements in deep, water filled boreholes. However, the present achievements during practical field testing bear evidence of the technique still being in a development stage, and also to some extent lack of experience. This gives unacceptable high failure rates.

Apart from mechanical / electronic malfunctioning, the main problem is the quality of the glue bond between the strain gauge rosette and the rock. This will always be a problem, as gluing under wet conditions is extremely difficult regardless. The preparation of the borehole bottom is in this connection a major challenge.

The technically successful tests in the vertical and horizontal holes seem to give reasonably reliable results as long as the influence of the stress acting along the borehole axis is understood and corrected for.

Due to the existence of the borehole itself, stress concentrations will occur at the borehole bottom, both because of stresses perpendicular to and parallel with the borehole.

The stress concentration factors are a function of Poisson's ratio of the rock material. Particularly when the stress along the borehole is high, it is therefore important to determine both Young's modulus and Poisson's ratio properly.

The directions of the maximum and minimum horizontal stresses as determined by the successful DDGS measurements in the vertical hole coincides quite well with previous measurements and later control measurements with 3D overcoring and hydraulic fracturing.

To determine the orientation of the horizontal hole from the DDGS results in the vertical hole should therefore in this case be appropriate.

The determination of Young's modulus by using a biaxial cell as described in Ref. 2 is considered appropriate. However, Poisson's ratio should also be determined by uniaxial loading in the laboratory. The thickness of the glue may vary during practical field testing. This should be checked visually after all tests are finished by knocking the doorstopper loose, and then inspect the bond.

The measurement report (Ref. 1) concludes that the test results are invalid due to extreme anisotropy. Later laboratory tests carried out by SINTEF, and also by and large the biaxial tests, reveal that the rock is not extremely anisotropic. Most specimens also show a reasonable linear elastic behaviour. However, visual inspection of cores from the site show large feldspar crystals in some of the cores. This makes the rock locally in-homogenous, rather than anisotropic. The author of this report therefore does not agree in the conclusion drawn in Ref. 1.

Appendix 7

Stress disturbance within rock due to irregular contact area at a fracture surface

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Introduction

The present study proposes a method to estimate the stress disturbance within rock due to irregular contact area at a fracture surface.
Rock fractures are often assumed to be planes existing between rock blocks. The contact between the blocks is assumed to be evenly applied on the fracture surface. In reality, rock fractures are characterised at small scale by their roughness (mm scale) and waviness (cm to m scale).
These variations in the fracture surface make that the contact between the blocks is not constant over the area but behaves more like discontinuous contact areas.
Rock stress applied to the fracture planes are concentrated to these contact areas and relaxed where there's no contact.

Model set-up

The figure below illustrates how the contact areas and relaxation areas of a real fracture are modelled with positive and negative loads applied normally on a half-space medium.

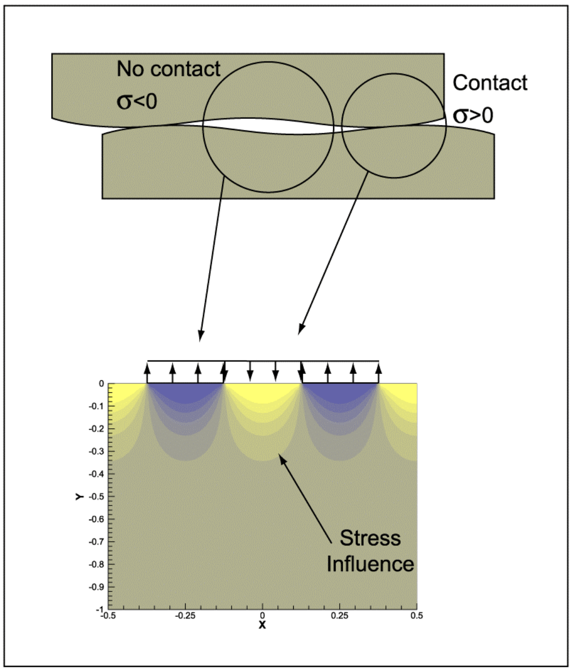


Figure 1. Natural fracture and model.

A two dimensional model is used to calculate the influence of the contact area loads on the stress distribution within the rock.
The stress within a semi-infinite, homogeneous, isotropic mass, with a linear stress-strain relationship, due to a strip load on the surface, were determined by Boussinesq in 1885 (Lambe & Whitman, 1969). The vertical stress at a given vertical and horizontal distances from the load were given.

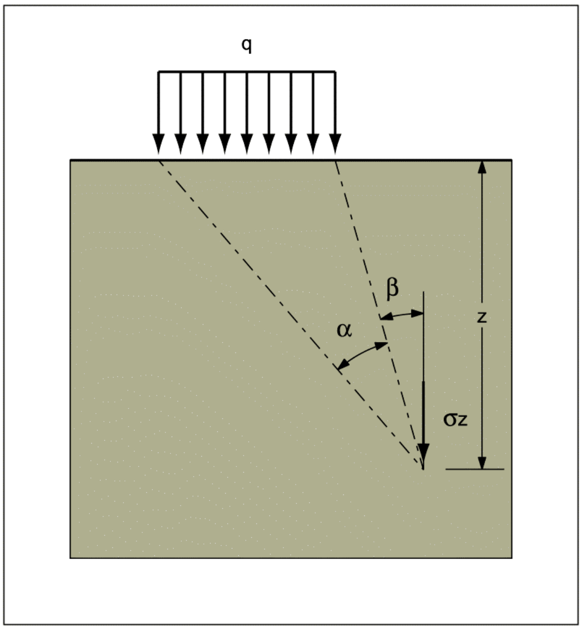


Figure 2. Boussinesq vertical stress solution for a strip load

The vertical stress σ_z at a given point due to a uniform pressure q on a strip area is given in terms of angle α and β as:

$$\sigma_z = \frac{q}{\pi}\left(\alpha + \sin\alpha\cos(\alpha + 2\beta)\right)$$

The principle of superposition is used for equally distant contact areas of constant size over the fracture surface.
The vertical stress under a contact area is given by the superimposition of positive and negative influence of the contact area and fracture voids. The model size is chosen large enough for the results not to be affected by the boundary effects.
Finally, the overburden stress calculated at 455m is added to the vertical stress under the contact area.

It has been found that the disturbance distance from the fracture plane is not sensitive to the ratio (contact area size) / (distance between contacts). For the present study, a constant ratio of 0.5 has been set. I.e. the size of the contact areas and the fracture voids are kept equal.

On the other hand, the disturbance distance from the fracture plane has been found sensitive to the distance between the contact areas. This distance is designed by λ. This represents in fact the waviness of a slightly sheared fracture.

For a given distance from the fracture surface, one notices that the stress increase is maximum for a point exactly under a contact area. The vertical stress at this point is calculated and its magnitude is compared to the vertical stress that would exist in this point if there was no fracture (in our case this magnitude is equal to the overburden of the rock).

The parameter *k* is defined as:

$$k = \frac{\textit{vertical stress under a contact area}}{\textit{vertical stress due solely to the overburden}}$$

k reflects the stress intensification due to the presence of the fracture.
k is calculated for different values of λ and different distances from the fracture plane. The results are presented in the section below.

Results and discussion

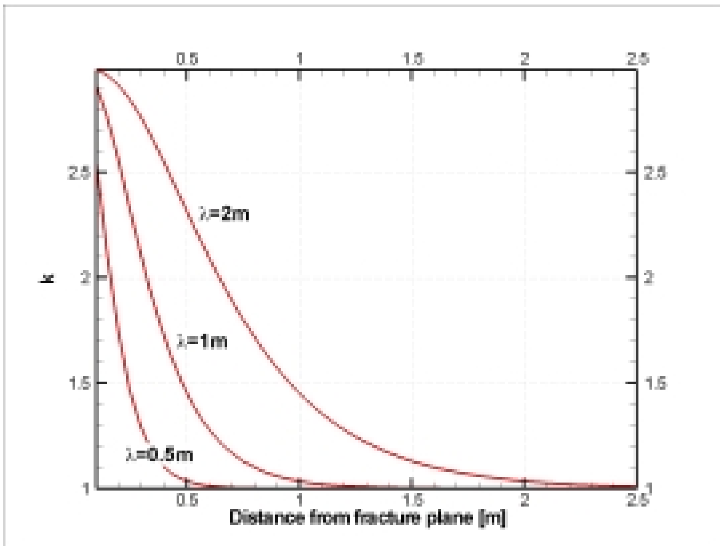


Figure 3. Vertical stress intensification factor vs. distance from the fracture plane under a contact area.

The figure above shows the vertical stress intensification factor as a function of the distance from the fracture plane under a contact area. The stress intensification factor is calculated for three different distances λ between contact areas, namely 0.5m, 1m and 2m. The size of the contact areas and the fracture voids are kept equal.

For a distance of 0.5m under a contact area, the vertical stress intensification factor is almost equal to 1 for a distance between contact points of 0.5m. This means that the influence of discontinuous fracture contacts becomes negligible for distances from the fracture greater than 0.5m.
If the distance between contact points is 1m, the vertical stress intensification factor is of 1.5 at a distance of 0.5m from the fracture plane and drops to a value of almost 1 at 1m.
If the distance between contact points is 2m, the vertical stress intensification factor is of 2.3 at a distance of 0.5m from the fracture plane and drops to a value of almost 1 at 2m.

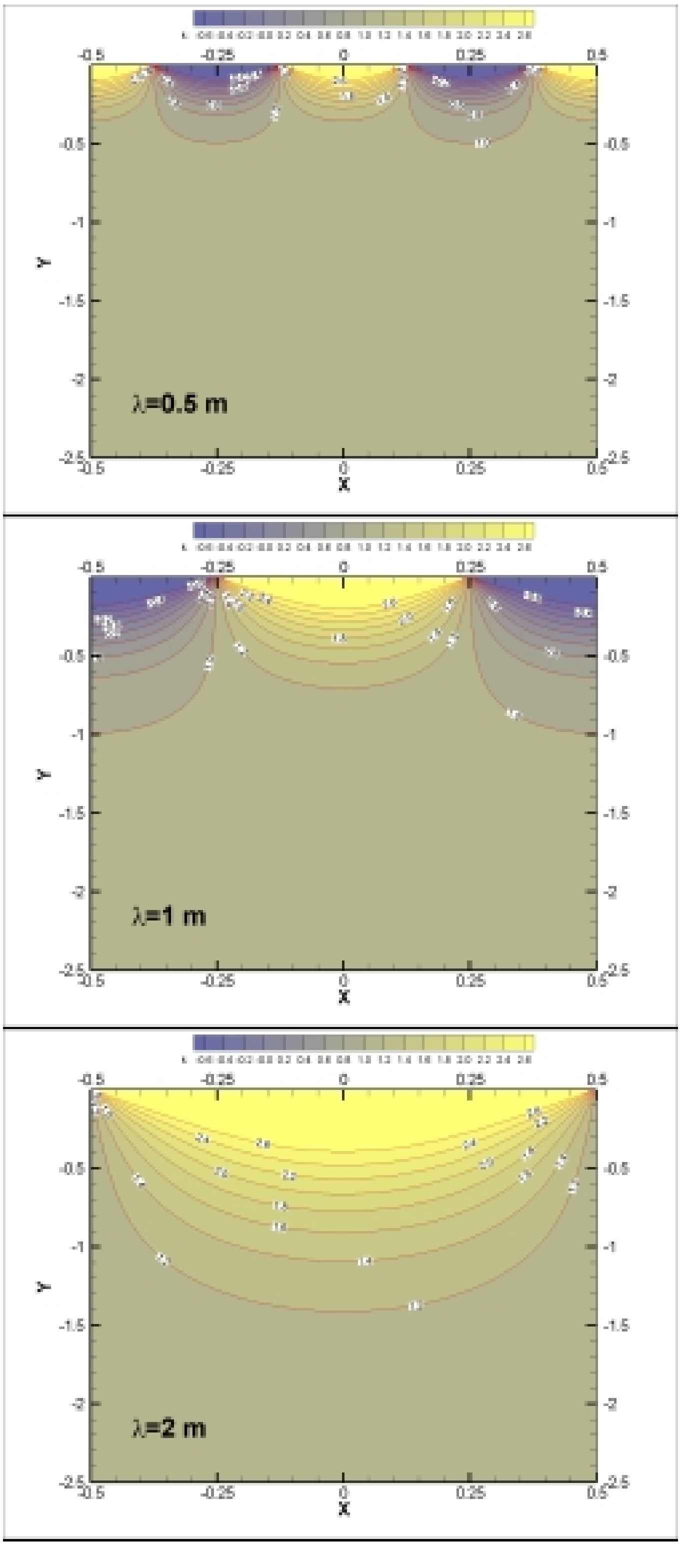
The results show that the greater the distance between the contacts, the deeper the influence of the contacts in terms of vertical stress.

Conclusion

Stress measurements carried out in a location at the vicinity of a rock fracture may be influenced by the fact that the contact between the two fracture walls is not continuous. It provokes local variation in the stress field. The study showed that this influence could be positive under a contact area. The stress can in some case be increased of a factor 2 or 3 depending on the distance to the fracture. The closer to the fracture, the greater the disturbance.

Likewise, the stress magnitude can be reduced by a factor 2 or 3, for a same distance, but for measurement points closer to fracture voids. Fracture voids create local stress release.

As a thumb rule, one can expect the effect of discontinuous fracture contact to become negligible for distances from the fracture plane that are greater than the waviness of the fracture.



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Appendix 8

A NUMERICAL STUDY (EXAMINE3D) OF THE STRESSES AROUND A BORE HOLE BOTTOM IN AN OVERCORING PROCESS

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1 INTRODUCTION

1.1 BACKGROUND

In Äspö Hard Rock Laboratory (HRL) a project has been performed with the aim to test and compare three stress measurement methods. The three methods are a 2-D overcoring method (called DDGS), a 3-D overcoring methods (called Borre Probe) and hydraulic fracturing. Higher stress magnitudes have been observed in the results from the DDGS tests compared to the other methods and former stress measurement results.

Deeper investigations have been carried out on the properties of the rock, heterogeneity, geological structure and an auditing on the DDGS strain recording. Non of these investigations could directly answer the questions why the DDGS tests gave higher stress magnitudes.

The DDGS strain gauges are applied on the borehole bottom before the overcoring. During the overcoring the strain in the borehole bottom is recorded continuously. The determination of the stresses is based on the difference in strain before and after the overdrilling of the bottom core.

The borehole rise stress concentrations, especially around the hole bottom. The DDGS gauges are glued directly on the borehole bottom and therefore measure disturbed strains and stresses. During the analysis of the strains and the stresses the disturbed stresses are corrected to the actual stresses. The correlation is performed with help of empirical stress concentration factors, see for example Amadei & Stephansson (1997).

Using a numerical analysis, with estimated in-situ stresses and rock properties, the strains can be calculated. The calculated strains could be compare to the DDGS measured strains. The stress concentration factors could also be calculated based on the numerical analysis and compared with the DDGS measured strains.

1.2 OBJECTIVE

The objective with the numerical analysis is to compare the numerically calculated strains with the DDGS measured strains and compare the stress concentration factors based on the numerical analysis with the one used in the DDGS analysis.

1.3 DESCRIPTION OF THE WORK

The numerical analysis is performed using a 3D boundary element program, called Examine3D. The main steps in the data analysis is:

1. Determination of the input data

2. Application of the borehole (loaded state at the hole bottom)

3. 17 mm slit/overcoring (unloaded state at the hole bottom)

The numerical analysis is performed both for the vertical and the horizontal borehole. The geometry and calculation is the same for both boreholes expect that the stresses are rotated.

2 RESULTS

2.1 STEP 1: INPUT DATA

Table 1 The input data for the numerical analysis:

Elastic properties		Field stresses			Strength parameters		
Young's modulus	Poisson ratio	σ_V	σ_H	σ_h	Tensile	Cohesion	Friction angle
70 000 GPa	0.25	30 MPa	20 MPa	15 MPa	15 MPa	30 MPa	49º

2.2 STEP 2: STRESSES AROUND THE BORE HOLE BOTTOM, AT LOADED STATE

The results from the numerical analysis is stress vectors close to the bore hole bottom. These vectors are projected on the plane of the borehole bottom. Each vector is then divided into the components in the σ_H and σ_h direction in the vertical hole and into the σ_V and σ_h direction in the horizontal. Table 2 shows the average stress components along lines parallel to the σ_H, σ_h and σ_V directions respectively. The lines are 40 mm long and centred on the origin of the borehole.

Table 2 The tangential stresses close to the bore hole bottom, 20 mm from the hole origin.

Bore hole	**Profile**	σ_H	σ_h
Vertical	average along σ_H	17.5 MPa	32.6 MPa
Vertical	average along σ_h	17.8 MPa	31.5 MPa
Vertical	**average along both lines**	**17.7 MPa**	**32.1 MPa**

Bore hole	**Profile**	σ_h	σ_V
Horizontal	average along σ_V	2.7 MPa	-9.2 MPa
Horizontal	average along σ_h	2.2 MPa	-10.1 MPa
Horizontal	**average along both lines**	**2.5 MPa**	**-9.7 MPa**

2.3 STEP 3: AT UNLOADED STATE

After the slit/overcoring, the numerical analyses gave zero or nearly zero stresses close to the former borehole bottom, which was expected.

3 ANALYSIS

3.1 NUMERICAL CALCULATIONS

To analyse the numerical strains the equations for Doorstopper isotropic solutions are used (Amadei & Stephansson, 1997). The strains are based on the numerical stresses, see table 2. Table 3 and 4 show the strains from the numerical model (Examine3D) and the measured strains from the DDGS tests.

Table 3 The numerical strains (Examine3D) and the measured strains (DDGS test) for the vertical hole, normalised to the corresponding stresse in the same directions.

Measured or calculated	ε_h (με)	ε_H (με)	ε_h/σ_h (με/MPa)	ε_H/σ_H (με/MPa)
DDGS, average	132	468	5.9	12.7
Examine3D	138	395	6.9	13.2

Table 4 The numerical strains (Examine3D) and the measured strains (DDGS test) for the horizontal hole, normalised to the corresponding stresse in the same directions.

Measured or calculated	ε_V (με)	ε_h (με)	ε_V/σ_V (με/MPa)	ε_h/σ_h (με/MPa)
DDGS, average	407	- 307	12.5	-24.8
Examine3D	70	- 148	4.7	-7.4

Further, a comparison has been carried out between the used stress concentration factors in the measured analysis (DDGS test) and in the numerical calculation. The factors from the measured analysis are based on results presented in PM 3 and the numerically analysed factors are based on the results in table 2 together with know relationship (Amedi & Stephansson, 1997). Table 5 shows the calculated stress concentration factors.

Table 5 The calculated stress concentration factors, based on the DDGS test and numerical method.

Measured or calculated	**Factor a**	**Factor b**	**Factor c**
DDGS	1.35	-0.05	-0.65
Examine3D, vertical hole	1.44	0	-0.74
Examine3D, horizontal hole	2.44	0	-1.54

4 CONCLUSIONS

There is a good correspondence of the results between the DDGS test and the numerical model in the vertical borehole, see table 3 and 5. However, in the horizontal hole the discrepancy between the DDGS test and the numerical method is large, see table 4 and 5.

The explanation could be that the horizontal hole is influenced by some external factor, like fractures in the vicinity, inhomogeneous material, micro fractures etc.

Appendix 9

MIKROSPRICKOR I BORRKÄRNOR

Andreas Eitzenberger, Avd för Bergmekanik, Luleå tekniska universitet

SAMMANFATTNING

Vid spänningsmätningar utförda vid laboratoriet i Äspö, med doorstoppermetoden, erhölls töjningar och därmed spänningar, som var orealistiskt stora. Töjningarna antogs vara orsakade av mikrosprickor som skapats på grund av spänningskoncentrationer vid borrhålsbotten. För att undersöka om så var fallet, startades en utredning där tunnslip karterades med hjälp av mikroskop. Denna rapport presenterar arbetet och de observationer som gjordes vid karteringen. Vid undersökningen observerades mikrosprickor i berget under borrhålsbotten. Undersökningen visade att det fanns mikrosprickor som var parallella med borrhålsbotten. Dessa sprickor skulle kunna orsaka stora töjningar. Det fanns dessutom sprickor som hade en föredragen orientering. Detta verifierar den anisotropi som observerats i tidigare undersökningar med diametrala mätningar av p-vågens hastighet. Med de undersökningar som gjorts så går det ej att avgöra om de stora töjningarna har orsakats av mikrosprickor. Det som går att säga är att mikrosprickor finns vid borrhålsbotten men vad som skapat dem eller om de påverkar spänningsmätningarna går ej att avgöra.

SUMMARY

During stress measurements performed at the Äspö HRL, using the Doorstopper method, unrealistically high strains and consequently stresses were obtained. The strains were supposed to be caused by microcracks created by stress concentrations at the drill hole bottom. To investigate if this was the case, a microscopy investigation was done. This report presents the work and observations done during investigation. Microcracks were observed in the rock under the drill hole bottom. The investigation showed the existence of microcracks parallel to the drill hole bottom. These cracks could cause large strains. There were also mikrocracks that had a preferred orientation. This verifies the anisotropy observed in earlier investigations using diametrical speed measurements of P-wave velocity. It is not possible to determine whether the large strains have been caused by microcracks from the investigations that have been made. It is evident that there are microcracks but their cause and whether they affect the strain measurements can not be determined.

1 INLEDNING

Vid spänningsmätningar utförda vid laboratoriet i Äspö, med doorstoppermetoden, erhölls töjningar och därmed spänningar, som vid jämförelse med andra spänningsmätningsmetoder, var orealistiskt stora (Christiansson och Janson, 2002). En möjlig orsak till de stora töjningarna tros vara mikrosprickbildning orsakad av höga spänningskoncentrationer som uppstått vid borrhålsbotten. Ett annat problem var att många mätningar misslyckades, dels på grund av att utrustningen fallerade (t.ex. ingen signal) men även av att limfogen mellan givaren och borrhålsbotten inte höll.

Under vintern 2002 genomfördes därför en studie där man genom att mäta p-vågens hastighet diametralt på borrkärnor skulle undersöka om det fanns en koncentration av sprickor nära ändytan på vilken doorstoppern var limmad. Från resultaten av mätningarna gick det inte att avgöra om så vara fallet, däremot kunde man se att berget vara svagt anisotropt (Eitzenberger, 2002).

Syftet med undersökningen var att se om mikrosprickor fanns vid borrhålsbotten (ändytan på kärnan) och om dessa mikrosprickor vara anledningen till de oväntade resultaten (höga spänningar). Genom att tillverka tunnslip av kärnornas ändyta kan eventuella mikrosprickor studeras med hjälp av mikroskop

2 UNDERSÖKNING

2.1 Angreppssätt

De undersökningar som gjorts har utförts med hjälp av mikroskop. Vid undersökningarna har sprickornas antal, deras orienteringar samt längder karterats. Detta har utförts för att skapa en enklare bild av hur sprickorna är utbredda i kärnan. Då den förenklade bilden genererats är det möjligt att diskutera och dra slutsatser från den information som erhållits.

2.2 Metod

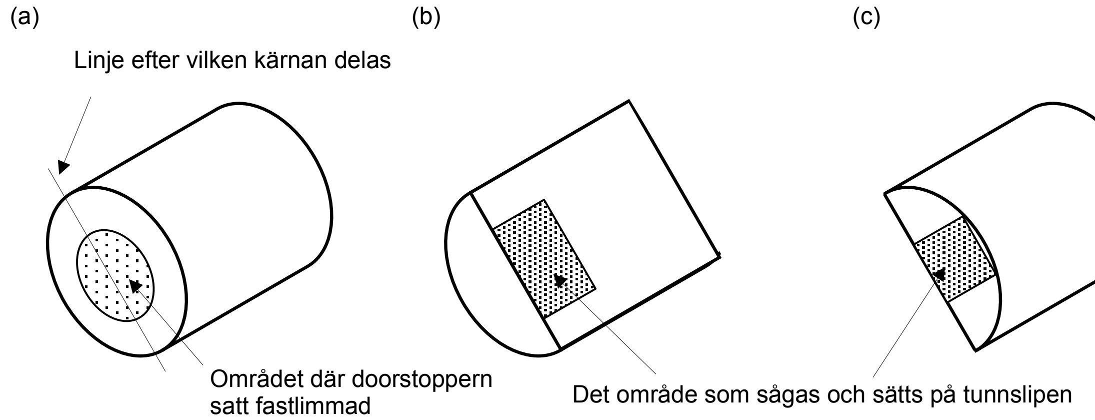
Mikroskopet är av märket Ortholux (II Pol BK) och har en förstoring mellan 50 och 200 gånger. Vid undersökningarna har en förstoring på 50 ggr använts. Tunnslipen består av en glasskiva på vilken en tunn sektion av berg limmas fast. Sektionen vakuumimpregneras samt behandlas med fluorescerande medel. Det fluorescerande medlet fyller håligheter som sprickor och porer, vilka då de belyses med ultraviolett ljus kan identifieras. Tunnslipen undersöktes i mikroskop och olika egenskaper för sprickorna dokumenterades.

2.2.2 Kärnor

Fyra kärnor valdes ut med utgångspunkt från de resultat som erhållits vid undersökningarna gjorda med diametrala p-vågsmätningar på borrkärnorna (Eitzenberger, 2002). Kärnor där högsta graden av anisotropi erhållits (för de olika borhålen) valdes ut. Anledningen var att det då var möjligt att samtidigt studera om diametral mätning av p-vågens hastighet kunde upptäcka anisotropi orsakad av sprickor med en föredragen orientering. Två av kärnorna, 29.92 och 30.89, är från det horisontella borrhålet (KF0093A01) och två kärnor, 107.95 och 128.28, är från det vertikala borrhålet (KA2599G01). För en noggrannare beskrivning av kärnorna se Eitzenberger (2002).

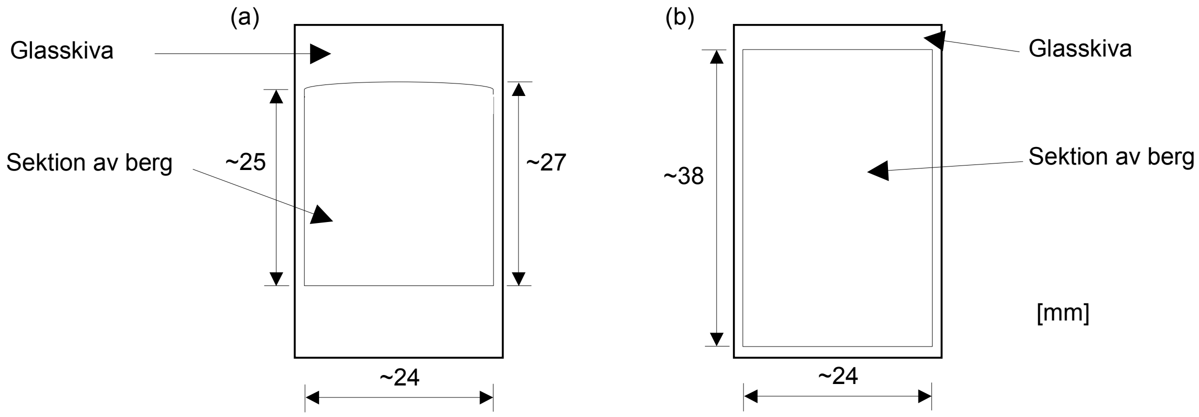
2.2.3 Tunnslip

För att kunna såga ut tunnslip från kärnorna var man tvungen att ta bort doorstoppern. För att inte skada ändytan på kärnan, på vilken doorstoppern var fastlimmad, sågades doorstoppern bort och resterna som blev kvar slipades bort. Detta gjordes för att inte generera nya sprickor. Därefter kapades kärnan på mitten (se Figur 1a). Från den kapade ytan, som var närmast ändytan på vilken doorstoppern har varit placerad (vinkelrätt ändytan), sågades ett tunnslip, se Figur 1b. På ändytan av den andra halvan (parallellt ändytan) sågades ytterliggare ett tunnslip, se Figur 1c. Linjen, efter viken kärnan kapades, var vinkelrätt mot den högsta observerade p-vågshastigheten, d.v.s. parallellt med den lägsta p-vågshastigheten.



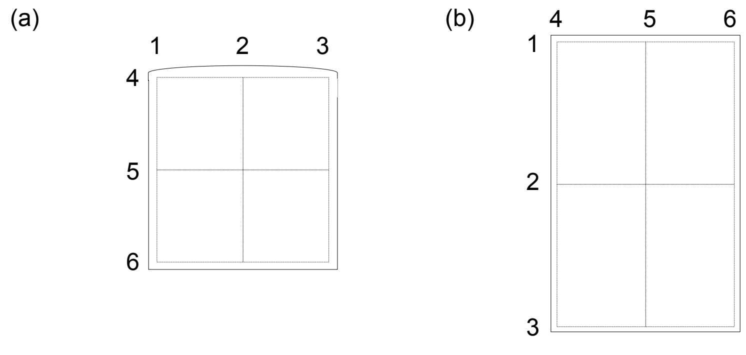
Figur 1. Proceduren för att erhålla tunnslip från borrkärnorna.

Från de fyra borrkärnorna erhölls totalt åtta tunnslip; fyra vinkelrätt ändytan och fyra parallellt med ändytan (se Figur 2).



Figur 2. Mått på tunnslipen (den yta som är berg). (a) parallellt ändytan, (b) vinkelrätt ändytan.

På tunnslipen placerades OH-blad på vilken streckade linjer var markerade (se Figur 3). Längs med dessa linjer kontrollerades sprickorna med avseende på; antal, antal i viss orientering, längdintervall och medellängd. På tunnslipen tagna parallellt med ändytan så är linje 6 närmast centrum av kärnan medan linje 4 är närmast kanten på kärnan. För tunnslipen tagna vinkelrätt ändytan så är det endera linjen 4 eller 6 som är närmast ändytan, vilken som är närmast beror på hur tunnslipen har preparerats.

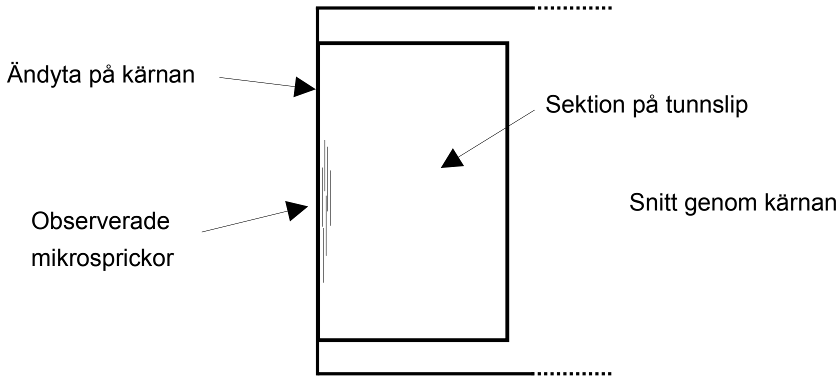


Figur 3. Linjerna efter vilka sprickorna karterades. (a) parallellt ändytan, (b) vinkelrätt ändytan

3 OBSERVATIONER

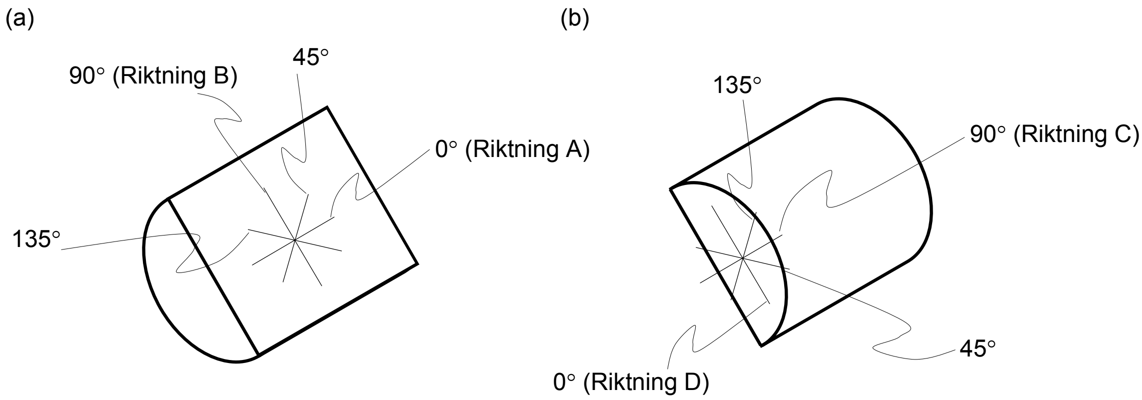
På de tunnslip, som visar den sektion av kärnan som är vinkelrätt mot ändytan, kan man på två (av fyra) tunnslip observera mikrosprickor som är parallella med ändytan, se Figur 4. Dessa mikrosprickor är relativt långa > 2 mm och finns bara inom ett område på ca. 1 – 2 mm från ändytan.

Dessa sprickor kan inte detekteras vid diametral mätning av p-vågens hastighet. Dels så är sprickorna parallella med p-vågens propageringsriktning, och påverkar därmed inte dess hastighet, dessutom är området för litet för att kunna identifieras av vågorna; vågen skulle bara passera där det inte finns några sprickor.

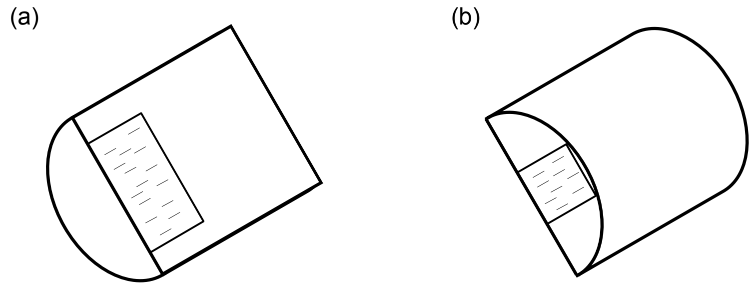


Figur 4. Observerade sprickor vid ändytan på kärnorna 30.89 och 107.95 (ej skalenlig figur).

På sex av de åtta tunnslip som undersökts (från kärnorna; 29.92, 30.89 och 128.28) observerades en tydlig trend på orienteringen av sprickorna. På den sektion som var vinkelrätt mot ändytan var mer än 31 % (31, 42 och 32 %) av sprickorna orienterade i riktning A (Figur 5a). På sektionen som var parallell med ändytan var 28 % eller mer (28, 44 och 30 %) av sprickorna orienterade i riktning C (Figur 5b). Om man illustrerar de dominerande sprickorna, så som de skulle se ut om de var i kärnan, så ser man att det är samma sprickor som observerats på de olika tunnslipen (se Figur 6).



Figur 5. Sprickornas orientering i förhållande till kärnan, jmf med Figur 1. (a) sektion kapad vinkelrätt ändytan, (b) sektion kapad parallellt ändytan (ej skalenlig figur).



Figur 6. *Insättning av de dominerande orienteringarna på sprickorna. (a) de vågräta sprickorna observerade på tunnslipen som visar berget vinkelrätt ändytan, (b) de lodräta sprickorna observerade på tunnslipen som visar berget parallellt ändytan (ej skalenlig figur).*

Om vi antar att sprickorna har formen av cirkulära skivor, där endast kanten på skivan har observerats vid karteringen, samt att vi vet att kärnorna kapats vinkelrätt mot den orientering i vilken den högsta hastigheten var observerad, så ser man att sprickornas orientering stämmer överens med det observerade beteendet hos p-vågshastigheten. Den högsta hastigheten är observerad parallellt med sprickorna (parallellt med skivan) och den lägsta hastigheten är observerad vinkelrätt mot sprickorna (vinkelrätt mot skivan). När p-vågen propagerar parallellt med sprickor påverkas hastigheten ej, men då vågen propagerar vinkelrätt mot sprickor så sjunker hastigheten (Paterson, 1978). Således är det möjligt att upptäcka anisotropi, orsakad av sprickor med en föredragen orientering, med hjälp av diametral mätning av p-vågens hastighet.

På den sjunde och åttonde tunnslipen (en kärna, 107.95) går det inte att se någon tydlig trend förutom på sektionen som är parallell med ändytan; där har 61 % av sprickorna orienteringen 135°. Denna orientering sammanfaller inte med någon av de sprickor som observerades på sektionen som är vinkelrätt mot ändytan. Kärnan 107.95 var den kärna som uppvisade minst anisotropi vid mätningarna med p-våg (Eitzenberger, 2002). Detta kan vara anledningen till att sprickorna inte har någon föredragen orientering; det finns helt enkelt ingen anisotropi i kärnan.

De mikrosprickor som observerats kan vara skapade av olika situationer. De kan vara orsakade av spänningskoncentrationer som uppstår vid borrningen av borrhålet samt slipningen av borrhållsbotten för att givaren ska kunna limmas fast. Att spänningskoncentrationer uppstår då berget är anisotropt har påvisats av bl.a. (Rahn, 1984). En annan anledning till att mikrosprickor finns i kärnan är att de skapats då kärnan flyttats från inspänt spänningstillstånd (*in situ*), där spänningen är ett tiotal MPa, till ett tillstånd där endast lufttrycket trycker på kärnan. Denna förflyttning genererar sprickor, speciellt om bergarten är hård t.ex. granit (Kowallis and Wang, 1983). Ett tredje alternativ är att mikrosprickorna redan fanns i kärnorna innan det ens hade börjat borras i området. Mikrosprickorna kan ha uppkommit vid stelnandet av bergmassan eller då området belastades av inlandsisen.

Genom den undersökning vi gjort går det ej att avgöra vad som orsakat de mikrosprickor som observerats. Det kan vara ett av de ovan nämnda alternativen som orsakat sprickorna eller en kombination av två av alternativen eller så är det alla tre alternativ som varit involverade. Det troliga är att alla de ovannämnda alternativen varit med i processen, men hur stor del av sprickorna som varje alternativ kan ha orsakat går inte att avgöra.

4 SLUTSATSER

Med hjälp av de mätningar och analyser som gjorts inom ramen för denna studie, så går det inte att avgöra om sprickorna är orsakade av spänningskoncentrationen (skapad av den plana ytan i borrhålet), på grund av överförandet från inspänt till icke inspänt tillstånd eller om de rentav var där innan man ens började att borra.

Mikrosprickor parallella med ändytan observerades på två av fyra tunnslip. Mikrosprickorna var relativt långa men var bara observerade närmast ytan. Dessa sprickor skulle kunna orsaka störningar på spänningsmätningarna, men inget definitivt går att säga om så är fallet eller ej.

Mikrosprickor med föredragen orientering observerades. Dess orientering sammanföll med resultaten från p-vågsmätningarna som gjorts tidigare. Det medför att anisotropi, orsakad av mikrosprickor med en föredragen orientering, kan detekteras med diametral mätning av p-vågens hastighet.

ANSLAGSGIVARE OCH TACK

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APPENDIX

Tabell A1. Sprickdata för 29.92 parallellt ändytan.

Linje	1	2	3	4	5	6
Totalt antal sprickor	15	26	17	27	20	23
90° (Riktning C)	3	7	3	8	8	10
0° (Riktning D)	4	2	3	1	0	2
45°	2	5	4	6	4	4
135°	3	7	3	3	2	1
Längre än 1 mm	2	5	8	7	6	1
Mellan 0.5 och 1 mm	5	8	7	9	8	14
Kortare än 0.5 mm	8	13	2	11	6	8
Medellängd	0.6	0.6	1.2	1.0	1.2	0.7
Medellängd långa (>1 mm)	1.8	1.5	1.8	2.4	2.2	2.1

Tabell A2. Sprickdata för 29.92 vinkelrätt ändytan.

Linje	1	2	3	4*	5	6
Totalt antal sprickor	4	9	8	21	28	24
90° (Riktning B)	2	1	1	2	2	0
0° (Riktning A)	0	0	1	10	10	9
45°	0	0	0	2	2	2
135°	1	0	0	2	5	1
Längre än 1 mm	0	2	2	2	7	3
Mellan 0.5 och 1 mm	2	3	3	7	9	6
Kortare än 0.5 mm	2	4	3	12	12	15
Medellängd	0.5	0.9	0.8	0.6	0.8	0.8
Medellängd långa (>1 mm)	-	2.5	2.1	1.6	1.8	2.1

*) Linjen som är närmast ändytan.

Tabell A3. Sprickdata för 30.89 parallellt ändytan.

Linje	1	2	3	4	5	6
Totalt antal sprickor	16	16	20	14	20	16
90° (Riktning C)	5	4	2	5	7	6
0° (Riktning D)	3	1	2	0	2	0
45°	2	3	4	3	1	5
135°	1	1	3	3	3	2
Längre än 1 mm	7	7	7	3	8	3
Mellan 0.5 och 1 mm	2	4	7	4	7	6
Kortare än 0.5 mm	7	5	6	7	5	7
Medellängd	1.1	1.3	1.0	0.9	1.3	0.9
Medellängd långa (>1 mm)	2.1	2.4	1.7	2.1	2.2	1.8

Tabell A4. Sprickdata för 30.89 vinkelrätt ändytan.

Linje	1	2	3	4*	5	6
Totalt antal sprickor	9	15	17	22	23	21
90° (Riktning B)	2	3	5	1	3	4
0° (Riktning A)	1	2	2	8	12	8
45°	2	6	1	0	3	1
135°	2	0	0	3	2	5
Längre än 1 mm	0	5	6	5	10	5
Mellan 0.5 och 1 mm	3	5	6	4	7	9
Kortare än 0.5 mm	6	5	5	13	6	7
Medellängd	0.5	1.4	0.9	0.8	1.8	0.9
Medellängd långa (>1 mm)	-	2.7	1.5	1.7	3.3	1.6

*) Linjen som är närmast ändytan.

Observationer: Långa och grova sprickor utmed linje 5, sprickor som är parallella med ändytan (linje 4).

Tabell A5. Sprickdata för 107.95 parallellt ändytan.

Linje	1	2	3	4	5	6
Totalt antal sprickor	13	9	13	10	9	8
90° (Riktning C)	1	0	1	4	0	0
0° (Riktning D)	1	0	2	1	0	0
45°	0	0	0	1	1	2
135°	6	7	9	3	7	6
Längre än 1 mm	4	4	5	5	1	6
Mellan 0.5 och 1 mm	5	3	6	1	3	2
Kortare än 0.5 mm	4	2	2	4	5	0
Medellängd	0.8[1]	1.1[1]	1.1	1.1	1.4	1.9
Medellängd långa (>1 mm)	1.8[1]	2.4[1]	1.8	1.6	7.6	2.3

1) Två långa sprickor som inte är medräknade.

Observationer: Grova sprickor utmed linje 6, två långa sprickor som inte är medräknade

Tabell A6. Sprickdata för 107.95 vinkelrätt ändytan.

Linje	1	2	3	4	5	6*
Totalt antal sprickor	8	14	9	22	11	24
90° (Riktning B)	2	6	2	2	1	10
0° (Riktning A)	0	3	0	6	7	4
45°	2	2	3	2	0	1
135°	3	1	4	8	1	3
Längre än 1 mm	0	4	3	7	4	9
Mellan 0.5 och 1 mm	2	6	1	5	2	6
Kortare än 0.5 mm	6	4	5	10	5	9
Medellängd	0.5	1.0	0.9	1.0[2]	0.9[2]	1.0[2]
Medellängd långa (>1 mm)	-	2.0	1.8	1.6[2]	1.4[2]	1.4[2]

*) Linjen som är närmast ändytan.

2) Lång spricka som inte är medräknad.

Observationer: En del grova sprickor av vilka några går tvärs över hela tvärsnittet, sprickor som är parallella med ändytan (linje 6).

Tabell A7. Sprickdata för 128.28 parallellt ändytan.

Linje	1	2	3	4	5	6
Totalt antal sprickor	3	12	8	13	6	6
90° (Riktning C)	2	3	2	9	3	2
0° (Riktning D)	0	0	1	0	0	0
45°	0	1	1	1	1	1
135°	0	1	0	0	0	0
Längre än 1 mm	0	4	0	7	2	2
Mellan 0.5 och 1 mm	2	5	2	4	1	2
Kortare än 0.5 mm	1	3	6	2	3	2
Medellängd	0.8	1.0	0.4	1.3	0.9	1.3
Medellängd långa (>1 mm)	-	1.7	-	1.8	1.6	2.5

Tabell A8. Sprickdata för 128.28 vinkelrätt ändytan.

Linje	1	2	3	4*	5	6
Totalt antal sprickor	5	8	4	19	28	25
90° (Riktning B)	0	0	0	0	0	0
0° (Riktning A)	1	3	1	11	12	9
45°	0	0	2	3	5	5
135°	0	1	0	2	0	2
Längre än 1 mm	2	2	1	4	6	7
Mellan 0.5 och 1 mm	2	1	1	8	8	7
Kortare än 0.5 mm	1	5	2	7	14	11
Medellängd	1.4	0.8	0.8	0.9	0.8	0.9
Medellängd långa (>1 mm)	2.3	2.1	1.8	2.1	1.6	1.7

*) Linjen som är närmast ändytan.