LBL-12478

MASTER

CONF-816624--6

Lawrence Berkeley Laboratory

UNIVERSITY OF CALIFORNIA, BERKELEY

EARTH SCIENCES DIVISION

To be presented at the 22nd U.S. Symposium on Rock Mechanics, Massachusetts Institute of Technology, Cambridge, MA, June 29-July 2, 1981

HYDRAULIC FRACTURING AND OVERCORING STRESS MEASUREMENTS IN A DEEP BOREHOLE AT THE STRIPA TEST MINE, SWEDEN

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March 1981

Prepared for the U.S. Department of Energy under Contract W-7405-ENG-48

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Thomas Doe[1]
Kjell Ingevald, Lars Strindell[2]
Berzalel Haimson[3]
Hans Carlsson[4]

[1]Lawrence Berkeley Laboratory
University of California
Berkeley, California 94720

[2]Swedish State Power Board
Stockholm, Sweden

[3]University of Wisconsin-Madison
Madison, Wisconsin

[4]Swedish Nuclear Fuel Supply Company
Stockholm, Sweden

DISCLAIMER

INTRODUCTION

The importance of in situ stress measurements in the prediction of the behavior of underground openings is widely recognized, yet the methods for obtaining reliable values for these stresses have long been the object of controversy. Overcoring methods--besides having generally been limited to holes of tens of meters in length-- have been questioned as to the cause of data scatter, roles of residual stresses, and appropriate scale over which to measure strains. Hydraulic fracturing, the only alternative for deep measurements has gained increased acceptance, yet questions remain particularly in the areas of non-coincidence of the borehole with one of the principal stresses, role of tensile strength in data interpretation, and determination of fracture orientation away from the borehole.

Recently, the Swedish State Power Board has developed a workable method of performing overcoring measurements in holes hundreds of meters in length. With the need to obtain in situ stress values for nuclear waste disposal simulation experiments at the Stripa Mine in central Sweden, we undertook the task of running both the Power Board's Leeman triaxial cell and hydraulic fracturing in the same hole. This paper is a report of the results of the comparative measurements.

The borehole for the stress measurements was located approximately 250m north of the experimental test area at the 348m level of the mine (Figure 1). The borehole and the test facilties are entirely in medium grained granite. The mine had produced iron ore from stratabound layers in leptite (interbedded metasedimentary and metavolcanic rocks of Precambrian age) which had been intruded by the granite. The contact follows the northeast trend of the outcrops in Figure 1, and dips at about forty-five degrees to the southeast. Whereas the iron ore follows the trend of the beds and the contact, the mined out areas are confined roughly to a northeast striking, southeast dipping slab.

The total depth of the stress measurement hole was 381 meters and it was drilled with a 76mm (3 inch) diameter core barrel. Overcoring measurements were carried out in groups of four to five measurements each at roughly one hundred meter intervals in the hole. After completion of most of the drilling, hydraulic fracturing measurements were carried out at approximately 50 meter intervals with additional measurements below 300m at the approximate depth of the test facility.

OVERCORING MEASUREMENTS

The Swedish State Power Board stress measurement system is a modification of the Leeman triaxial cell, which measures the complete state of stress from a single borehole. The procedures used in the overcoring by the Swedish State Power Board at Stripa are the same as those described in Hiltscher and others (1979). As with most overcoring techniques, a pilot bore of a few decimeters in length is drilled at the end of the main borehole. A strain cell consisting of three, three-component strain gage rosettes is cemented to the wall of the pilot hole. After a reading of the strain gage outputs, the strain cell is overcored by a drill with a larger diameter bit. For the Power Board strain cell the pilot bore and main borehole diameters are 38 mm and 76 mm respectively. The

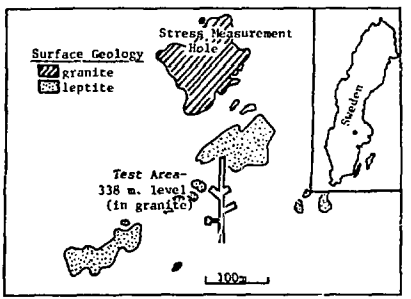

Figure 1. Location of stress measurement hole relative to test area in mine; inset shows location of Stripa Mine.

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change in the strain cell readings after the cell is overcored and brought to the surface are then used to calculate the in situ stresses using elastic formulae for borehole deformation (Leeman, 1971). The overcoring procedures are shown schematically in Figure 2. After the cores containing the overcored triaxial cells are removed from the hole, laboratory measurements of the response of the strain gages to axial and biaxial loading are used both the determine the modulus of the rock and to verify the effectiveness of the strain gage bonds.

OVERCORING RESULTS

Seventeen successful measurements were made with the Leeman cell technique. The orientation and magnitude of the principal stresses are shown in equal area projections in Figure 3. Although the magnitudes of the stresses measured show a fair amount of scatter, there are strong consistencies of orientation between measurements made at the same depth level. The source of the scatter in the magnitude data is not clear. However, confirmation of the strain gage bonds in laboratory tests of the cores suggests that the source is not in the instrumentation. Rather, the data scatter may reflect grain size effects or an elastic anisotropy which may not be consistent between cores.

The orientation data for the 110m level measurements shows a consistent northwest-southeast orientation for the minimum stress, which is tensile in two measurements; the other two principal stresses do not show a consistent orientation. At 200m depth, the

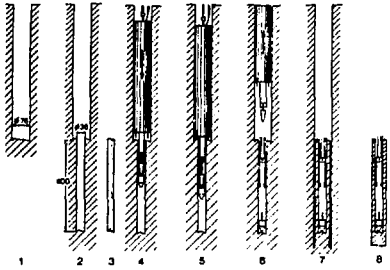

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Figure 2. Procedures for overcoring stress measurements: (1)main 76mm borehole, (2) drilling 38mm pilot hole, (3) appraisal of rock quality, (4) positioning of stress probe before setting gages, (5) cementing gages and recording initial strain readings, (6) release of stain gage carrier and removal of probe, (7) overcoring, (8) removal of strain gaged core from hole and second strain gage reading (from Hiltscher et al., 1979)

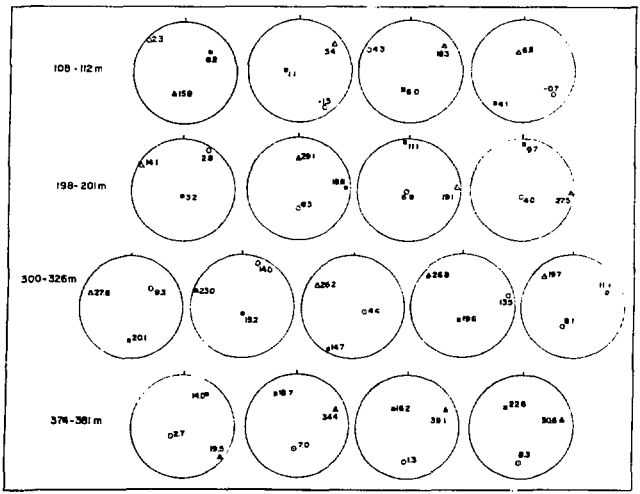

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Figure 3. Lower hemisphere equal area plots of principal stress data from overcoring; magnitudes in MPa; σ_1- triangles, σ_2- squares, σ_3- circles

principal stresses are close to horizontal and vertical, and maximum principal stress, σ_1 is oriented east-west to west-northwest. At 300m the maximum principal stress is oriented northwest and nearly horizontal, however, the other two principal stresses are rotated from the vertical and horizontal. At 380m the maximum stress is oriented west to west-southwest, and the minimum stress is skewed from the vertical towards the direction of the mine openings.

Figure 4 is a plot of the maximum and minimum horizontal stress, σ_{Hmax} and σ_{Hmin}, and the vertical stress versus depth. Values shown are the averages for each depth grouping. As with the principal stresses, the maximum stress is oriented east-west to northwest in all but the shallowest tests. The vertical stress is measured as 1.5 to 2.5 times the weight of the overburden except at 380m where the measured vertical stress is close to the calculated stress.

Preliminary finite element anaylses have been made by Chan et al. (1981) to determine the possible effect of the mine openings on the measured stresses. Although the analysis is very conservative in removing the entire orebody, the calculations are consistent with the measurements in showing only a slight effect on the stresses due to the mine at 200m and a rotation of the minimum stress towards the mine at greater depth. In addition, the calculations show an increase in the vertical stress over the lithostatic, an effect decreasing below 300m depth.

Although the orientation of the Leeman cell probe is done by magnetic compass, the orebody is not felt to have affected the measurement. This view is based on the correspondence of the data obtained on hole orientation from the borehole compass used in the impression packer work in conjunction with the hydraulic fracturing. These data were consistent with the survey data obtained using the Fotobor, a non-magnetic survey tool.

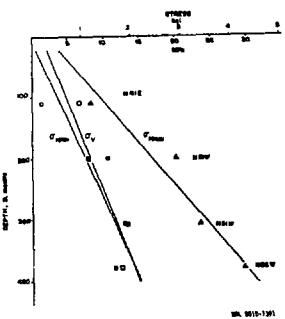

Figure 4. Variation in horizontal and vertical stress magnitudes with depth as determined by overcoring; data points are averages of each depth group; σ_{Hmax} - triangles, σ_{Hmin} -circles, σ_V - squares.

HYDRAULIC FRACTURING EQUIPMENT AND PROCEDURES

The hydraulic fracturing of the borehole was performed using a packer assembly with a straddled interval of two feet (0.6 meters). Mounted above the packer assembly was a water tight housing containing two pressure transducers, one to monitor the pressure in the injection test zone, the other to monitor the packer pressure. The downhole pressures were monitored continuously at the surface. The packers were inflated through hydraulic hoses strapped to the steel tubing along with the electrical cable. The fracturing was done with clean water using a positive displacement pump modified with an accumulator to remove pressure surges. The complete system is shown schematically in Figure 5.

Once the packers had been lowered to the desired test zone, the packers were inflated to a pressure exceeding the expected breakdown pressure. For most tests this pressure was about 2500 psi (17 MPa). Originally we had expected to set the packers at a lower pressure and rely on the pressure increase in the zone between the packers to act on the ends of the packers and increase the pressure of the seal. This interaction of the packer pressure and the injection zone pressure has been obser ved by some workers (Kim and Smith, in press) engaged in hydraulic fracturing work for stress measurement. We had observed this phenomenon while testing in a 3 inch casing our original packer system. However after the discovery of leaks in the packers forced us to fall back on an alternate packer system, we found the packer pressure was not at all affected by the pressure in the test zone.

Once the packers were set, the pressure was raised in the test zone at a rate of 2000 psi (14 MPa) / minute until fracturing was indicated by an increase in the pumping rate. As soon as fracturing was felt to have occurred, the manifold was shut in and the pressure was monitored for two to four minutes. This procedure was repeated two to four additional times to observe second ary breakdown pressures, and to obtain additional shut in pressure values. One test involving slowly pumping up the test zone to the pressure where the fracture would just

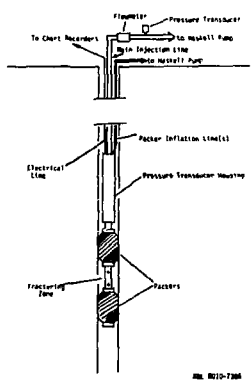

Figure 5. Schematic of hydraulic fracturing equipment.

begin to open was run at each test level to further set the value of the shut in pressure.

IMPRESSION PACKER PROCEDURES AND EQUIPMENT

Once the fracture experiments were completed, impressions of the test zone were carried out to determine the orientation of the fractures. The impression packer equipment consisted of a 1 7/8 inch (48mm) packer element with a removable sleeve coated with a soft rubber capable of extrusion into the fracture under pressure. The packer was placed below a 1 3/8 (35mm) inch, single shot, magnetic borehole survey compass modified to screw directly into the packer end cap. This entire assembly was attached to a wireline and inflated pneumatically through the same hydraulic tubing used in the fracturing experiment. The packer was hoisted by a wireline hoist which included a cable counter to assure proper depth of emplacement. The deepest test zones required about two hours for completion of the impression. The impression packer system is shown schematically in Figure 6.

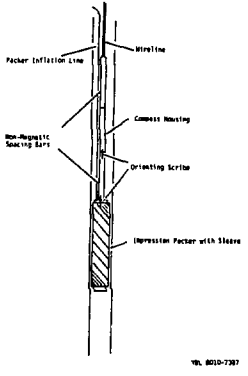

Figure 6. Schematic of wireline impression packer system.

FRACTURING DATA ANALYSIS

The hydrofracturing data were interpreted using both the first breakdown technique of Haimson and Fairhurst (1968)

$$\sigma_{Hmax} = 3 P_{si} - P_{b1} + T - P \qquad (1)$$

where P_{si} = shut in pressure = σ_{Hmin}
P_{b1} = first breakdown
T = hydrofracture tensile strength
P = pore pressure

and the second breakdown technique of Bredehoeft et al. (1976)

$$\sigma_{Hmax} = 3 P_{si} - P_{b2} - P. \qquad (2)$$

where P_{b2} = second breakdown pressure.

The breakdown pressures were readily determined from the pressure-time records (Fig. 7), however, the shut-in pressures required some subjectivity. Previous analysis of hydrofracture data have used such criteria as the value to which pressure rapidly drops after breakdown or the stable pressure value reached a few minutes after breakdown has occurred. Both of these conditions are affected by intrumentation factors, such as the speed of the chart recorder, or fracture boundary conditions, such as intersection of another fracture. The approach used in selecting shut in pressures in this work is based on the pressure pulse permeability test in single fractures (Wang, et al., 1977). Considering the post breakdown pressure behavior as a test where the fracture aperture is large at pressures greater than the in situ stress and smaller where pressures are less, then the pressure-time record should behave as a two-stage pulse test. The shut in pressure could then be taken as a sharp break in semi-logarithmic plot of the pressure versus time. Shut in pressures obtained by this technique were in excellent agreement with the pressures required to open the fracture by slow pumping.

Hydrofracture tensile strength tests were run on forty samples of core from the test zones to obtain tensile strength values for use in data analysis by eq. 1. Half of the samples contained 1/4 inch (7mm) holes and the other half 1/2 inch (13mm) holes. The strengths were stronly size dependent as the smaller hole diameter yielded strength averaging 2355 psi (16MPa) and the larger gave 1860 psi (13 MPa). Presence of calcite or epidote healed fractures lowered these values by about 25%.

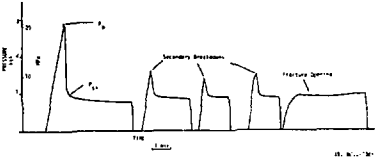

Figure 7. Typical pressure-time record for hydraulic fracture (test 17, 304.9m depth)

One can extrapolate the tensile strength data to the 3 inch (76mm) diameter size used in the field using a fracture mechanics approach similar to that of Abou-Sayed et al. (1978). For a sample under no radial load, Abou-Sayed's breakdown equation (1978, eq. 17) reduces to

$$P_b = \frac{K_{1c}}{F(L/r)\sqrt{\pi L}} \qquad (3)$$

K_{1c} = critical stress intensity
L = crack length
r = hole radius
F(L/r) = stress intensity coefficient (From Paris and Sih, 1965, Table 7)

Given that grain size of the Stripa granite as 1mm to 5mm, (Olkiewicz et al. 1979), and assuming that failure will occur on the larger grain boundary cracks, we can use an L of 5mm (.25 inch). The ratio of the tensile strengths for two borehole diameters can readily be obtained from equation 3 as

$$\frac{Pb \text{ at } r1}{Pb \text{ at } r_2} = \frac{F(L/r2)}{F(L/r_1)} \qquad (4)$$

Given the average strength of 2355 psi (16 MPa) for the 0.25 inch (7mm) holes, equation 4 gives a strength of 1950 psi (13 MPa) for the 0.5 inch (13mm) holes and 1280 psi (9 MPa) for the 3 inch holes in the field.. The calculated strength for the larger laboratory test holes agrees well with the observed strengths. The 3 inch holes strengths are calculated as 54% of the 0.25 inch holes; Rummel and Jung (1975) observed that the ratio in their tests in limestone was about 45%. Whereas the failure should be expected to occur on the largest flaws available rather than the average sized flaw, one might expect tensile strengths in the larger holes to be smaller than those calculated above. Tensile tests on large cores containing 3 inch holes are planned.

The field tensile strength, which has been *defined as the difference between the first and second* breakdown pressures varied from 350 (2.4 MPa) to 1400 psi (9.7 MPa) and averaging 750 psi (5.2 MPa). The "field" values are considerable lower than the extrapolated laboratory values thus resulting in a lower calculation of σ_{Hmax}. Table 1 gives the data for calculation methods based on both eq. 1 and eq. 2. The laboratory based values are shown as a function of depth in Figure 8.

In comparing the two methods of hydrofracture analysis, it should be noted that the second breakdown technique is only applicable where the ratio of the maximum to minimum horizontal stress is less than two, as for greater stress differences the second breakdown would be less than the shut in pressure. Furthermore, fracture opening pressure depends on the total load on the fracture wall (J. Noorishad, personal communication) and not just the borehole pressure. The opening of the fracture depends on the transient pressure distribution along the fracture wall and may thus reflect pumping rate as well as stress conditions.

Table 1. Hydrofracture Stress Results *

No.	Depth m	STRESS, psi (MPa) First Breakdown σ_{Hmax}	Second Breakdown σ_{Hmax}	σ_{Hmin}	σ_v
1	28.5	1140 (7.9)	265 (1.8)	240 (1.7)	105 (0.7)
4	52.2	760 (12.1)	960 (6.6)	600 (4.1)	195 (1.3)
5	101.2	2050 (14.1)	1490 (10.3)	890 (6.1)	380 (2.6)
6	153.2	2000 (13.8)	1490 (10.3)	817 (5.6)	570 (3.9)
7	203.8	1370 (9.4)	1300 (9.0)	939 (6.5)	760 (5.2)
19	201.4	2330 (16.1)	1680 (11.6)	1070 (10.4)	750 (5.2)
8	251.6	3230 (22.3)	2730 (18.8)	1790 (12.3)	940 (6.5)
18	279.6	2610 (18.0)	2480 (17.1)	1560 (10.8)	1040(10.1)
17	304.9	1510 (10.4)	1850 (12.8)	1390 (9.6)	1140 (7.8)
9	308.6	2480 (17.1)	1980 (13.7)	1320 (9.1)	1150 (7.9)
10	318.1	2600 (17.9)	2310 (15.9)	1700 (11.7)	1190 (8.1)
16	325.8	3800 (26.2)	3340 (23.0)	1980 (13.7)	1215 (8.4)
11	328.6	3070 (21.2)	2720 (18.8)	1620 (11.2)	1230 (8.5)
13	356.7	3300 (22.8)	2530 (17.4)	1790 (12.3)	1330 (9.1)
14	367.1	2580 (17.8)	2080 (14.4)	1540 (10.6)	1370 (9.4)
15	369.7	3210 (22.1)	2640 (18.2)	1780 (12.3)	1380 (9.5)

* based on T=1100 psi

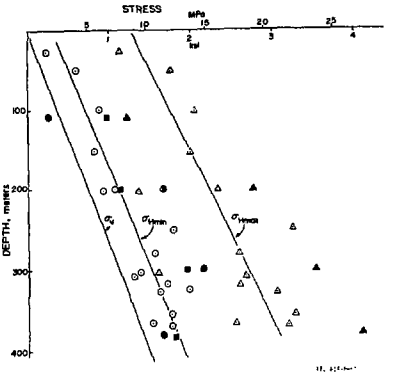

Figure 8. Variation in horizontal and vertical stress with depth. Hydrofracture values are open, overcoring are solid. Hydrofracture values based on first breakdown. σ_{Hmax}-triangles, σ_{Hmin}-circles, σ_v-squares

RESULTS -- IMPRESSION PACKER WORK

Successful impressions were obtained for only nine of the tests. Although this is as many impressions as have usually been taken in hydrofracture experiments, *a larger number of results would have been more* satisfying. The failed impressions generally appeared to be the result of uneven inflation of the packer. The problem of the uneven inflation of the impression packer may be solvable by using elements that are both shorter in length and larger in diameter. Although our original aim in designing the wireline impression packer system was to minimize the size to avoid potential problems getting hung up in the hole, our experience with running the wireline system in high quality rock has given us greater confidence in using larger packers.

Despite the lost impressions, the impressions that were successful show a reasonably consistent WNW trend to the fracture strikes. The stereographic projection of the fracture planes in Figure 9 shows a set of fractures with a mean strike of N 65 W, with a standard deviation of 28 degrees.

COMPARISON OF STRESS DATA BY OVERCORING AND HYDRAULIC FRACTURING

Table 2 gives the interpolations of the horizontal and vertical stresses at the depth of the test facility based on the overcoring results and both methods of hydrofracture interpretation. The overcoring results agree well with the hydrofracturing in both the magnitudes of the minimum horizontal stress and in the orientation of the maximum horizontal stress. The overcoring results are higher than those calculated by either breakdown pressure technique (Figure 8). The first breakdown data, being higher, agrees better with the overcoring results. The vertical stress values in excess of the overburden weight may indicate that the overcoring values are too high; however the

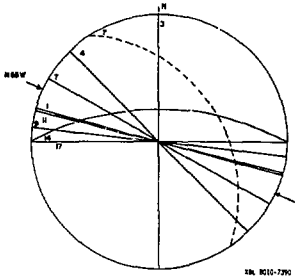

Figure 3. Lower hemisphere stereographic projection of planes of hydrofractures. Numbers refer to tests (see Table 1).

the preliminary calculations of Chan et al (1981) show that the mine openings may cause an increase in vertical stress at the shallow depths where the overcoring results are also the highest.

The area of closest agreement between the stress measurement techniques is in the northwest to east-west orientation of the maximum stress. This direction is consistent with previous data collected by the Power Board (Hiltscher et al, 1979) and N. Hast (1969).

In conclusion, the overcoring and hydraulic fracturing results give comparable data with respect to magnitudes of horizontal stresses and to orientation. In holes where principal stress may not be coincident with borehole axes, the hydraulic fracturing may yield mainly the stresses normal to the hole rather than principal stresses. Although major questions remain both as cause of the scatter in the over coring data and the question of tensile strength in hydrofracture data analysis, the overall agreement of the methods would indicate that both the overcoring and hydraulic fracturing provide data applicable to practical problems in the field.

Table 2. Interpolated In Situ Stress Values at the Depth of the Test Facility (318m in the stress borehole)

Method	σ_{Hmax}	σ_{Hmin}	σ_v*
Overcoring	3670 (25.3)	1650 (11.7)	1750 (12.0)
Hydrofracture (1st Breakdown)	2740 (18.9)	1300 (9.0)	1040 (7.2)
Hydrofracture (2nd Breakdown)	2390 (16.5)	1300 (9.0)	1040 (7.2)

*Hydrofracture values calculated from weight of overburden.

ACKNOWLEDGEMENTS

Special thanks are due to M. Moebus of LBL and P.A.Halen of the Stripa Mine for help in conducting the field work in Sweden. T. Chan of LBL graciously shared unpublished model results. J. Ratigan of U.C. Berkeley provided considerable help in conducting the tensile strength measurements. P. Nelson provided a helpful review of the manuscript.

This work was supported by the Assistant Secretary for Nuclear Energy, Office of Waste Isolation of the U.S. Department of Energy under contract number W-7405-ENG-48. Funding for the project was administered by the Office of Nuclear Waste Isolation at the Battelle Memorial Institute.

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