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GEOLOGICAL SURVEY

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WORKSHOP XVII

WORKSHOP ON HYDRAULIC FRACTURING STRESS MEASUREMENTS

VOLUME I

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MENLO PARK, CALIFORNIA
1982
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In just the past few years the use of the hydraulic fracturing technique for making in-situ stress measurements has become widespread around the world. In convening the Workshop on Hydraulic Fracturing Stress Measurements, it was our aim to bring together active investigators to comprehensively discuss their experiences with the method. The workshop was held in December, 1981 in Monterey, California and attended by forty investigators from eight counties.

The workshop proved to be a successful forum for discussion of case histories, data interpretation techniques, technological advances, and overall progress and problems with the method. A high point of the workshop was the good comparison demonstrated in many cases between hydraulic fracturing stress measurements and other methods. Hydraulic fracture orientations at the wellbore seem to agree quite well with the $S_h$ direction implied by geologic data, earthquake focal mechanisms, and strain relief methods used at depth. Moreover, several investigators reported that very similar stress magnitudes were determined with hydraulic fracturing and overcoring methods at several locales.

In determining the maximum horizontal principal stress, $S_h$, several investigators reported success in using fracture reopening pressures rather than breakdown pressures. Although it is not always feasible to use the fracture reopening method, and errors could be introduced if significant fluid penetration into a fracture occurs before it reopens, the groups using the method reported internally consistent results and they were able to derive values of tensile strength that agreed well with laboratory values.

The most straightforward determination that can be made from hydraulic fracturing is that of the minimum horizontal principal stress, $S_h$. The shut-in pressure is customarily used as a measure of $S_h$ and these measurements are usually found to be quite consistent and reliable. At shallow depth, where the least principal stress is often vertical, it is sometimes possible to determine both $S_h$ and the vertical stress, $S_v$, because the hydraulic fracture "rolls-over" into a horizontal plane as it propagates. Several investigators reported that in some cases the shut-in pressure slightly decreases as the fracture propagates and it was generally agreed that the minimum shut-in pressure should be used as a measure of $S_h$ as long as the fracture did not "roll-over," intersect a pre-existing fracture, or breakout around the packers. A few investigators pointed out that picking accurate shut-in pressures is sometimes difficult and several semi-log plotting techniques were proposed for increasing the accuracy of the shut-in pressure determination. It was also proposed that pumping at very low pumping rates and using the pumping pressure rather than the shut-in pressure is a good way to determine $S_h$ if shut-in pressures are not distinct.
We believe that the papers included in this report give the reader a nearly comprehensive overview of the current research related to hydraulic fracturing stress measurements. However, because of the rapidly expanding use of the method, it was clearly recognized that the state-of-the-art is still evolving. It was frequently suggested that workshops of this type should be held every few years.

We would like to thank William F. Brace, Charles Fairhurst, D. Ian Gough, and Fritz Rummel for serving as session chairpersons. The meeting was convened under the auspices of the U.S. Geological Survey and the U.S. National Committee for Rock Mechanics. These reports will later appear as a special publication of the USNC/RM. We would like to thank Jessie Reeves and Barbara Charronat for their help in planning the excellent accommodations for the meeting.

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TOPIC I

HYDRAULIC FRACTURING CASE HISTORIES AND INTERPRETATION TECHNIQUES

MODERATOR - F. RUMMEL
Hydraulic Fracturing Stress Measurements along the Eastern Boundary of the SW-German Block

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Abstract

During the last decade more than 50 in situ stress measurements have been carried out in Central Europe by various researchers in Austria, France, Italy, Switzerland and the Federal Republic of Germany. Various stress measuring techniques have been applied. A list of references and locations of all those measurements is presented.

Among these measurements are about 100 new hydraulic fracturing experiments conducted in 19 boreholes at 15 locations situated along the eastern boundary of the SW-German block. The depth of the measurements ranges from about 40 to 450 m. The experiments were carried out with a new wireline straddle packer system, which allows experiments in boreholes to a maximum depth of about 1000 m. Technical details of this system as well as the interpretation method used to derive crustal stresses are described. For two locations, a deep borehole into sediments as well as a granite test site, the measurements are presented with full details including hydraulic fracturing pressure plots and stress data. All other stress data are summarized to obtain a general stress-depth relation for the specific area around the eastern boundary of the SW-German block. If the averaging method over such a large area as well as an extrapolation to greater depth is acceptable, the stress data would explain the present tectonic stability of this area compared to the western block boundary.
1. Introduction

The SW-German block is considered as a major tectonic block unit in Central Europe. Its southern boundary is given by the main thrust fault of the Northern Alps, the western boundary consists of the Upper Rhine graben structure and its northern continuation in the Hessian graben, and the eastern boundary is formed by the NW-SE oriented Franconian line and the Donau fault which separate the block from the outcropping crystalline basement of the Bohemian and Thuringian massifs (Fig. 1).

![Diagram of tectonic units in Central Europe and major horizontal stress direction derived from earthquake data.](image)

Fig. 1: Tectonic units in Central Europe and major horizontal stress direction derived from earthquake data.

Seismo-tectonic active are mainly regions within the SW corner of the block (Swabian Alb and Hohenzollern Graben) and the Rhine graben system. Latter consists of the Upper Rhine Valley in the South, and curves to NW intersecting the Hercynian block of the Rhenish massif and following the Lower Rhine embayment in the north. Earthquake fault plane solutions for this regions indicate an approximately NNW-SSE direction of horizontal compression (Ahorner 1975, Bonjer 1979), which explains sinistral shear motion in the Upper Rhine graben parallel to the graben axis and the extensional tectonic features in the Rhenish massif as well as present rifting in the Lower Rhine embayment (Illies and Greiner, 1979). In comparison, the eastern boundary of the block is tectonically inactive if we neglect the seismic activity further to the south in NE-Italy and northern Yugoslavia.
To obtain further information on the presently active tectonic stress field in and around the SW-German block, numerous direct in-situ stress measurements have been conducted during the last decade. The geographical distribution of the relevant test sites are shown in Fig. 2, references are given in Table 1. Most of the test sites are located along the western block boundary. In most cases, the stress data are derived from shallow overcoring doorstopper measurements, some from flat jack measurements at the surface, the rest from hydraulic fracturing experiments in deep boreholes. Although the scatter is considerable the data indicate a N-S trend of the direction of maximum horizontal compression in Central Europe.

Here, we only focus on the results of hydraulic fracturing stress measurements in 19 deep boreholes which are distributed in the eastern part, along the eastern boundary and north of the SW-German block. The measurements reach to a maximum depth of about 450 m allowing to speculate on the stress-depth relation. In addition, we present specific technical details of the hydrofrac system used as well as information on pressure-data interpretation to derive principal stresses.

2. The Bochum Hydrofrac Stress Measuring Technique

2.1 Hydrofrac System

Starting in 1973 with hydraulic fracturing stress measurements in Germany in shallow boreholes (Rummel and Jung, 1975), successively a new wireline hydrofrac stress measuring system has been developed at the Ruhr-University Bochum. The system at present permits to carry out measurements in boreholes to a maximum depth of about 1000 m by university staff, only. The system is easily portable with a mini-truck. No drilling equipment such as drilling rods or a drill-tower are necessary. A similar system is used for tests in dry boreholes in deep mines. It was tested in a 3000 m deep Indian gold mine, where packer pressures up to 1 kb were needed for hydraulic insulation of the injection interval during breakdown operations.

A schematic view of the deep hydrofrac system is shown in Fig. 3. It consists of the double straddle-packer, a heavy tripot, a motor-driven winch mounted on a 1-ton trailer, a 7-conductor logging cable and a high pressure steel reinforced rubber hose, an air-driven hydraulic high pressure pump and the pressure monitoring unit. A suitable air-compressor to activate the hydraulic pump (capacity 3 to 5 m³ min⁻¹) usually is rented on-site.

*The results from underground hydrofrac measurements carried out in two deep mines (800 m) NE of the block are presently neglected, since they seem to be considerably affected by the mine structure (Rummel and Heuser 1981, Rummel 1981).
Fig. 2: Locations of stress measurements in Central Europe until 1982 (see Table 1 for details).
Table 1: Locations of stress measurements in Central Europe until 1982 (see Fig. 2)

<table>
<thead>
<tr>
<th>No.</th>
<th>Location</th>
<th>Coordinates</th>
<th>Method</th>
<th>No. of tests</th>
<th>Reference</th>
</tr>
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<td>1</td>
<td>Eldagsen (FRG)</td>
<td>52° 9' 7° 17'</td>
<td>Oc/44</td>
<td></td>
<td>II. 82</td>
</tr>
<tr>
<td>2</td>
<td>Hildesheim I (FRG)</td>
<td>52° 7.1' 9° 52.6'</td>
<td>HF/10</td>
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<td>Ru. 82</td>
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<tr>
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<td>Bielefeld (FRG)</td>
<td>51° 57.3' 8° 39.1'</td>
<td>HF/5</td>
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<td>5</td>
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<td>51° 53.2' 10° 14.7'</td>
<td>HF/2</td>
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<td>this paper</td>
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<td>Harz II (FRG)</td>
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<td>Dornap (FRG)</td>
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<td>OC</td>
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<td>Hess. Bergland II (FRG)</td>
<td>50° 56.3' 9° 50.8'</td>
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<td>Schlitz (FRG)</td>
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<td>Longitude</td>
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<td>Falkenberg I</td>
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in 3 boreholes

**Method:** HF hydraulic fracturing, OC overcoring, FL flat jack test.

**Location:** A Austria, Ch Switzerland, F France, FRG Fed. Rep. Germany, I Italy.

The straddle-packer consists of two 1 m long nylon reinforced rubber packers (Schmidt-Kranz Company, 3421 Zorge/FRG), which shorten about 10 percent axially at 20 percent radial extension under unconfined conditions, and the injection unit between. It is in most of our applications about 70 cm long and contains the injection pipe and the packer pressure transmission line. Schematic cross-sections of the tool are given in Fig. 4, both for packer pressurization and injection into the frac-interval. The unit is based on a unitized construction principle which allows fast assembly as well as the use of different size packers, presently for boreholes with diameters of 76 to 80, 95 to 100 and 118 to 125 mm.

Generally, only one pressure line is used in the borehole. To switch from packer pressurization to injection of the frac interval and vice versa, a push-pull valve on top of the straddle packer tool is activated by releasing or applying tension to the borehole cable. Schematic
cross-sections of the valve are given in Fig. 5 which demonstrate both injection positions. To permit deflation of the packers after the test in partly dry boreholes, a pressure release valve is mounted on top of the push-pull valve in the packer pressure line. It closes during high pumping rate packer inflation and opens as soon as an adjustable minimum packer pressure after venting the packer line on the surface is reached. A schematic cross-sectional view of the release valve is presented in Fig. 6 both, during packer inflation and deflation.

Fig. 4: Double straddle packer unit, system Ruhr University Bochum.
- a) fluid flow for packer pressurization
- b) fluid flow for injection into the test interval.
The pressure is measured down-hole by an integrated amplifier fluid-pressure gauge (Burster Precision Technique, Typ 821.8), which is located within the cable head on top of the push-pull valve (Fig. 7).

Fig. 5: Push-pull valve at the top of the straddle packer for both packer pressurization (a) and injection (b).

The 12 mm O.D/8 mm I.D. high pressure hose (Argus, Typ 1st) is connected to the 7-conductor borehole cable (9.5 mm O.D, USS Amergraph No. 7-H-37-SB, strength 5.7 tons) at intervals of about 25 m in order to avoid too much tension in the hose by its own weight.

Pressurization is achieved by a double-acting air-pressure-driven hydraulic pump with a maximum pressure of 1 kb and a pumping rate of 5 l·min⁻¹ at an air-pressure supply of 5 m³·min⁻¹ (Schmidt and Kranz Company, type HD-GW 100). The fluid injection rate is measured on surface by a flow meter (0 to 2 GPM, 3500 psi, model TMRA, Euromatic Machine and Oil Comp., London, U.K.).

Pressure and flow rate data are monitored on a strip-chart recorder (paper speed 20 mm·min⁻¹) and stored on tape (Teac RG 1 tape recorder).

Frac-orientations are obtained using an impression packer (same packer as described above, Fig. 8) in connection with a magnetic single shot unit. In general, the impression packer is pressurized with 100 to 200 bars over a period of 30 to 60 minutes.
Fig. 6: Pressure release valve for deflation of packers in 'dry' boreholes.

Fig. 7: Cable head including the fluid pressure transducer with amplifier.
2.2 Testing Procedure

As far as possible intact borehole sections without macroscopic joints are selected as test intervals by core inspection and analysis of available logging data. Packers are set to a pressure of 100 to 250 bars depending on depth and wall rock quality. Then, to examine the test interval for open joints, the interval pressure is increased by 10 to 30 bars instantaneously. The following pressure drop is observed for about 5 minutes, which allows to estimate the rock mass 'permeability' in the test interval.

The fracturing operation starts after the pore pressure has reached its original value, with maximum pumping rate. Immediately after frac generation (break-down) fluid injection is interrupted (shut-in) and the pressure decrease is observed. After venting the interval, the frac is extended during several refrac tests at maximum pumping rate. In general, a total water volume of about 10 to 20 liters is injected. Injected volumes and back flow after venting are carefully monitored. Finally, the interval pressure is increased either at constant pressurization rate \( \dot{p} = \text{constant} \) and observing the flow rate \( q \), or at constant intermediate flow rate \( q = \text{const.} \) and observing the pressure increase, or step-wise in 10 bar intervals and observing the pressure drop (permeability test). Either of these tests may yield a good additional measure for the shut-in pressure equivalent to the normal stress acting across the frac-plane.

2.3 Data Analysis

In general, principal stresses in this study are derived from hydraulic fracturing pressure data using the classical concept first suggested by Hubbert and Willis (1957). The tensile strength \( T \) in the standard equation is assumed to be equivalent to the in-situ hydraulic fracturing tensile strength, which is given by the difference between the breakdown pressure and the final refrac pressure to re-open the induced fracture, \( P_c - P_R \). In sediments the pore pressure \( P_0 \) is assumed to correspond to the pressure given by the natural water column in the borehole (\( P_0 = 0.1 \cdot (z - z_0) \); \( P_0 \) in bars, \( z, z_0 \) in m). In crystalline rock (eclogite, granite) the pore pressure in the rock at depth is neglected. Otherwise pressure data interpretation leads to erroneous and unmeaningful results. In all cases, the overburden pressure is taken as the principal vertical stress, \( S_v = \rho gz \). At shallow depths \( (z < 50 \text{ m}) \) fracture propagation generally occurred horizontally, although in most cases vertical fractures were induced, so that two shut-in pressure values could be used to calculate the horizontal stresses as well as to check the overburden stress (first suggested by Rummel and Jung, 1975).

In cases, where pre-existing vertical joints not aligned with the major horizontal principal stress \( S_H \) were re-opened, the principal horizontal stresses were estimated following a suggestion by Cornet (1979):

\[
P_R = (S_H + S_h) - 2(S_H - S_v) \cos 2(\theta_o - \theta) \\
P_{si} = 1/2(S_H + S_v) - 1/2(S_H - S_v) \cos 2(\theta_o - \theta)
\]

\( \theta_o - \theta \) is the angle between the direction of \( S_H \) and the frac plane,
and $S_h$ is the minor horizontal stress. The three unknowns $S_H$, $S_h$ and the direction of $S_H$ may be estimated if joints of different azimuths can be tested.

If laboratory fracture mechanical data on rock cores were available, a fracture mechanics approach was used for in-situ pressure data interpretation. A first quantitative simple application is given by Rummel and Winter (1982), which yields the following relation for the extension of vertical fractures:

$$P_c = \frac{1}{h^*} \left( \frac{K_{IC}}{\sqrt{R}} - S_h^* f^* - S_h^* g^* \right)$$  \hspace{1cm} (2)

Here, $K_{IC}$ is the fracture toughness of the rock derived from laboratory studies, $R$ is the borehole radius and $h^*$, $f^*$ and $g^*$ are dimensionless functions only depending on the normalized fracture length $a/R$. They are given in Fig. 8 assuming that the fluid pressure is constant within the fracture and is equal to the pressure in the injection interval. Then, for zero external stresses the term

$$T = \frac{K_{IC}}{h^* \sqrt{R}}$$  \hspace{1cm} (3)

corresponds to the hydraulic fracturing tensile strength. Thus, the major principal horizontal stress $S_H$ is given by the relation

$$S_H = -\frac{h^*}{f^*} \left( P_R + \frac{g^*}{h^*} S_h \right)$$  \hspace{1cm} (4)

assuming $S_h$ to correspond to the shut-in pressure. The appropriate values for $h^*$, $g^*$ and $f^*$ can be estimated from the observed in-situ strength data and fracture mechanics tests on the core material (fracture toughness and laboratory hydrofrac data from mini-cores tested under various confining pressures (for details see Rummel and Winter, 1982)).

![Fig. 8: Dimensionless functions $f^*$, $g^*$ and $h^*$ for a fracture mechanics analysis of hydraulic fracture propagation (see eq. (2),(3),(4).](image-url)
3. Stress Field at the Eastern Boundary of the SW-German Block

Since 1978 more than 100 successful hydraulic fracturing experiments have been carried out in 19 boreholes located in the eastern part of the SW-German block, on its eastern boundary and north of the block (No. 2, 3, 4, 5, 6, 11, 13, 15, 23, 24, 25, 26, 29-31, in Fig. 2 and Table 1). Most of the locations are distributed along a NS profile of about 200 km from Hannover in the north to Würzburg in the south. The depths of the boreholes range from about 100 m to about 800 m. The boreholes intersect mesozoic and paleozoic sediments (cretaceous limestones, triassic and permian sandstones and siltstones), except the boreholes Weißenstein (No. 24) and Falkenberg (No. 29-31) in the east, which are drilled into an eclogite body and into granite. The depths of the test intervals range from about 40 m to a maximum depth of about 450 m.

As representative examples, in the following the details of measurements at only two locations are presented. A complete summary of all hydrofrac stress measurements is given elsewhere (Rummel et al., 1982).

3.1 Stress Measurements Spessart I

The borehole Spessart I (No. 23 in Table 1) is located on the northern boundary of the Spessart mountains. Drilling was terminated at a depth of 607 m. The upper part of 348 m was cased. The open-hole section had a borehole diameter of 96 mm. Due to borehole collapse at about 450 m, fracturing experiments were only possible within the depth-interval from 350 to 450 m. At this depth the borehole intersects fairly intact and uniform permian sandstones and siltstones (Zechsteinformation). The natural water level was at 135 m below surface.

Within this interval 7 fracs were induced. Breakdown pressures ranged between 131 and 191 bars, the final refrac pressures between 93 and 108 bars, indicating an in-situ hydrofrac tensile strength \( T = P_c - P_r \) between 38 and 97 bars (Table 2). The shut-in pressures were almost constant at about 90 bars (79 to 93 bars). Typical pressure-time plots from two experiments are given in Fig. 9. They indicate extremely low permeability of the wall rock in the test interval prior to fracturing (P-test), a sharp pressure drop after breakdown and immediate shut-in (F-test), as well as distinct shut-in pressures after repeated pumping. After venting the test interval, the pressure immediately increases again if venting is interrupted, due to fluid back-flow from the induced fracture. Generally, 80 percent of the injected water was recovered. The final test with slow constant pumping rate clearly demonstrates the critical pressure to reopen the frac and to keep it open against the acting normal stress across the frac plane.

The evaluation of the pressure data yields average horizontal principal stresses of \( S_h = 125 \) bar and \( S_h = 84 \) bar, compared to an assumed vertical stress of \( S_v = 104 \) bar at a mean depth of 425 m (\( \rho = 2.5 \text{ g.cm}^{-3} \)). The stress data are given in Table 2 and in Fig. 10. According to the frac orientation obtained from the impression packer measurements, maximum horizontal compression (\( S_h \)) is acting N 156° (Fig. 11, Table 2). This average value does not include the azimuth of a vertical fracture oriented N 31° E, which was observed during test No. 1 at a depth of 370.6 m. The relatively low breakdown pressure value (\( P_c = 149 \) bar) and the relatively high shut-in pressure (\( P_{si} = 93 \) bar) suggest that
Fig. 9: Pressure-time plots during hydrofrac tests No. 4 and 6 (Table 2) in borehole Spessart I, P initial permeability test, F frac test, RF refrac test. Pressure values are pressures above hydrostatic.
Table 2: Hydraulic fracturing pressure data and principal stresses from frac experiments in borehole Spessart I

| No. of test | depth (m) | $P_c$ (bar) | $P_R$ (bar) | $P_{si}$ (bar) | $P_o$ (bar) | $S_v^{1)}$ (bar) | $S_H$ (bar) | $S_h$ (bar) | $\theta^{2)}$ (degree) | $\alpha^{3)}$ (degree) |
|------------|-----------|-------------|-------------|---------------|-------------|----------------|-------------|-------------|----------------|----------------|----------------|
| 1          | 370.6     | 149         | -           | 93            | 24          | 91             | (114)       | -           | 31             | 90             |
| 2          | 375.4     | -           | 96          | 86            | 24          | 92             | 138         | 86          | 163            | 85             |
| 3          | 385.4     | 173         | 108         | 86            | 25          | 94.5           | 125         | 86          | 147            | 90             |
| 4          | 413.4     | 191         | 96          | 86            | 28          | 101.5          | 132         | 86          | 161            | 90             |
| 5          | 427.2     | 131         | 93          | 83            | 29          | 104.5          | 127         | 83          | 163            | -68            |
| 6          | 443.4     | 184         | 87          | 97            | 31          | 108.5          | 119         | 79          | -              | -              |
| 7          | 448.4     | 187         | 95          | 83            | 31          | 110            | 123         | 83          | 144            | 90             |

$P_c$: breakdown pressure, $P_R$: refrac pressure, $P_{si}$: shut-in pressure, $P_o$: pore pressure, $S_v$: overburden pressure, $S_H$, $S_h$: major and minor horizontal principal stresses, $\theta$: frac azimuth, $\alpha$: inclination of frac-plane.

1) $S_v = \rho g z$, $\rho = 2.5 \text{ g cm}^{-3}$

2) frac azimuth in degrees N $\theta^\circ$ E

3) frac inclination with respect to the borehole axis.
Fig. 10: Principal horizontal stresses $S_H(\square)$ and $S_h(\bigcirc)$ and vertical stress $S_V(\rho_H = 2.5$ g.cm$^{-3}$) in borehole Spessart I.

Fig. 11: Frac-orientation and direction of maximum horizontal compression $S_H$, respectively, in borehole Spessart I.
3.2 Stress Measurements in the Falkenberg Granite Massif, NE Bavaria

In the Falkenberg Granite massif, NE Bavaria, hydraulic fracturing experiments were carried out in 5 boreholes to a depth of 300 m. Three of the boreholes (No. 29 in Table 1) were drilled for a hot-dry-rock geothermal experiment on one test site, two additional older drill holes (No. 30 and 31 in Table 1) were available from uranium ore prospection. The diameters of the fully core-drilled boreholes were 76, 96 and 132 mm, respectively. From the core material, geophysical logs and televiewer inspection full information was available on the existing joint pattern. To a depth of 150 m horizontal joints dominate due to stress relief by isostatic uplift and subsequent erosion. The vertical joints at greater depth belong to 4 joint systems, which also can be observed in numerous surface outcrops over the entire region. The rock itself is a coarse-grained granite with large potassium feldspar phenocrysts predominantly oriented horizontally within a medium-grained groundmass of quartz, plagioclase and mica. In the two ore prospection boreholes the granite is partly altered.

Totally 27 frac experiments were conducted. Three typical pressure-time plots are presented in Fig. 12. They demonstrate a distinct breakdown, but at low pumping rates (Fig. 12 a, b) only poorly defined shut-in pressures. In comparison, the shut-in pressure at a high pumping rate (190 liters per minute) is clearly defined (Fig. 12 c). This result is in perfect agreement to other tests performed in crystalline rocks (e.g. Weißenstein, Münchberger Gneiss mass, No. 24 in Table 1).

A list of the pressure data observed is given in Table 3. Generally, the breakdown occurs at pressures between 110 and 160 bars. The exceptionally high breakdown pressure of 203 bars in test 16 certainly is due to the high pumping rate of 190 liters per minute (Jung, 1980). The refrac pressures vary between 50 and 120 bars yielding a hydraulic fracturing in-situ tensile strength between 60 and 80 bars. Lower tensile strength data indicate the existence of latent joints which could not be detected by televiewer observation or from core inspection. Shut-in pressures in average ranged from 40 to 50 bars with only little increase with depth. As mentioned above they were usually poorly defined. Pressure data in the two ore prospection boreholes were generally slightly less due to the alteration of the granite. Particularly, this is true for the breakdown pressure and refrac pressure data.

For the estimation of the principal stress field data the pore pressure in the granite is neglected. Assuming the pore pressure to be equivalent to the pressure of the water column above depth, as done for boreholes in sediments, leads to unmeaningful results. A similar result is obtained for other tests in crystalline rocks (e.g. borehole Nr. 24, Table 1). The principal stress data are summarized in Table 4 and presented graphically in Fig. 13. The data indicate that the calculated vertical stress \((g = 2.65 \text{ g-cm}^{-3})\) is the least principal stress to a depth of about 150 m. This result would explain the preferred existence of horizontal stress relief joints at shallow depth above 150 m. At greater depth the minor principal horizontal stresses systematically are below the overburden stress, while the major horizontal principal stresses generally are above the vertical stress. At a depth of 300 m the major horizontal principal stress is nearly equal to the vertical stress.
Most of the induced fractures initiated as vertical fractures in average oriented N 115° (Fig. 14). Deviations from this average orientation are considered in the stress estimation using the formulae suggested by Cornet (1979). A detailed analysis is given by Rummel and Alheid (1979). This average frac orientation corresponds to orientation of the induced macro-frac in borehole HB4a (Test No. 16 in Table 3, 253.5 m), which was located by acoustic emissions during frac extension (Leydecker, 1981) and confirmed by intersecting the frac-plane by drilling and subsequent fluid circulation experiments (Jung 1980).

Fig. 12a:
Fig. 12: Pressure-time plots in test 2 (12a, NB1, 119.6 m), test 12 (12b, NB3, 230 m) and test 16 during the generation of a macro-fracture with a pumping rate of 190 l·min⁻¹ (12c, HB4a, 253.5 m).
Fig. 13: Principal stresses in the Falkenberg Granite massif ($S_H(\Theta), S_V(\Theta)$).

Fig. 14: Frac-orientation in borehole No. NB3 (○ vertical fracs, ⋆ inclined fracs).
Table 3: Pressure data during hydraulic fracturing tests in boreholes in the Falkenberg Granit massif (No. NB1, NB3 and HB4 correspond to No. 29, No. LII 16 and LV 17 correspond to No. 30 and 31 in Table 1).

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Table 4: Principal stresses $S_{V}$, $S_{H}$ and $S_{\phi}$ ($\rho = 2.65 \text{ g/cm}^3$) in-situ tensile strength $T$, and strike ($\theta$) and dip angle ($\alpha$) of the induced fracs.

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3.3 Horizontal Stresses as a Function of Depth

Although the borehole locations are distributed over a large geographical area with different geological units, and the fracturing experiments were carried out in various rock formations, an attempt is made to derive a general depth relation for the active stress field at the eastern boundary of the SW-German block. For this purpose all horizontal principal stress data so far measured in 19 boreholes are plotted versus depth. The results are presented in Fig. 15, 16, 17 and 18.

If we only neglect the stress values obtained in borehole Spessart III (No. 26, borehole was located on the steep slope towards the Main river valley) the minor horizontal stresses $S_h$ (Fig. 15) follow an approximately linear depth relation of the form

$$S_h = S_{h0} + \frac{dS_h}{dz} \cdot z$$

with $S_{h0} = 9$ bars at the surface and a gradient of 0.21 bar $\cdot$ m$^{-1}$. The correlation coefficient for this relation is $k = 0.9$. This indicates that $S_h$ is slightly greater than the vertical stress at shallow depth and becomes the least principal stress at a depth of about 200 m. This general result agrees perfectly with the results obtained for any single location where continuous data over the total depth range exist, such as in the Falkenberg case.

The values of the major principal stresses $S_H$ (Fig. 16) generally show a considerable scatter, however, in most cases $S_H$ is above the corresponding value of the vertical stress $S_y$. Again neglecting the Spessart III data and assuming a linear depth relation we obtain the equation

$$S_H = S_{H0} + \frac{dS_H}{dz} \cdot z$$

with $S_{H0} = 8$ bars at the surface and a gradient of 0.34 bar $\cdot$ m$^{-1}$. The correlation coefficient is 0.7.

Thus, for transcurrent vertical faults oriented with an angle $\beta$ with respect to the major principal stress $S_H$, for a depth greater than 200 m the active normal and shear stresses may be estimated by the relations

$$\sigma = 9 + (0.275 + 0.065 \cdot \cos 2\beta) \cdot z$$

$$\tau = 0.065 \cdot z \cdot \sin 2\beta$$

Together with an appropriate instability criterium (e.g. friction) for strike-slip faulting along a favourably oriented fault plane ($\beta = 65^\circ$, $\mu = 0.85$, $\mu$ static friction coefficient) this result would explain the absence of recent tectonic activity in this region, if we assume a natural pore pressure gradient of about 0.1 $\cdot$ z (pressure in bar, z in m). It is recognized that the extrapolation of the results to greater depths as well as the averaging process applied may be questionable until more data from greater depth are available. However, a similar treatment of stress data has been successfully applied to other crustal plates such as the Canadian shield or South Africa (McGarr and Gay, 1978; Rummel, 1978).
Finally, we consider the distribution of the direction of maximum horizontal compression as derived from the frac orientations in 19 boreholes under consideration of the pressure data. As shown in Fig. 17 the scatter of the frac orientations is significant, particularly at shallow crustal depths. However, if Cornet’s suggestion (eq. (1)) is applied the observed frac orientations lead to a rather consistent stress field orientation (Fig. 18). The data demonstrate that the orientation of maximum horizontal compression is about N 150° for all locations within the SW German block, and is N 110° to N 120° for locations situated on its eastern boundary or in the north. The first value is in good agreement with the stress orientation derived from overcoring stress measurements in the western part of the block (Il-lies and Greiner, 1979; Baumann, 1981) as well as with the result of a first hydrofrac experiment in the Hohenzollern graben, SW Germany (Rummel and Jung, 1975) and with seismic data from the tectonically active western block boundary. The same orientation was obtained from flat-jack stress measurements in the eastern part of the Gallic block (Froidevaux et al., 1980) suggesting the existence of a large regional homogeneous stress field in Central and Western Europe, which originates from active plate tectonics in the Alps. In contrast, the slight change in the orientation of the stress field at the eastern boundary of the SW-German block could be explained by alpine orogenetic tectonics in the Karpathian region.

Acknowledgement

The following co-workers participated in the field measurements at various times: H.J. Alheid, J. Baumgartner, R. Czedzak, R. Eckes, C. Frohn, U. Heuser, H. Jütte, B. Kappernagel, G. Möhring, F. Rummel, Fl. Rummel, St. Teufel, Th. Wöhrle, R. Jung. R.B. Winter carried out most of the laboratory work and was involved in the fracture mechanics analysis (both is not explicitly presented here).

The development of all hydrofrac equipment was done in the Mechanics workshop of the Institute of Geophysics under B. Kappernagel.

A detailed report on all hydrofrac experiments in the FRG was recently submitted under contract No. RUB-7084408-82-3 to the Federal Bureau of Geoscience and Resources, Hannover, Bochum 1982.
Fig. 15: Minor horizontal principal stress $S_h$ versus depth from 19 boreholes at 14 locations at the eastern boundary of the SW-German block.
Fig. 16: Major horizontal principal stress $S_H$ versus depth.
Fig. 17: Azimuths of observed fracs versus depth.
Fig. 18: Orientation of maximum horizontal compression along the eastern boundary of the SW-German block as derived from frac tests in 17 boreholes.
Literature


Baumann, H.: Regional stress field and rifting in Western Europe. Tectonophysics, 73, 105-111, 1981


Cornet, F.: Priv. comm., 1979/82 (see also this volume)

Elmohandes: see Baumann 1981


Appendix

Summary of hydrofrac stress data

All stress data are stored in a central data bank at the Ruhr-University, including all data from references given in Table 1. Computer printouts as well as plots can be ordered.

Numbers of locations in the following table correspond to numbers given in Table 1. Borehole signatures correspond to computer data files. Azimuth angles are only given for vertical fractures as obtained from impression packer tests, including pre-existing vertical joints. Results from tests on horizontal or significantly inclined fractures are not included.
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IN SITU STRESS MEASUREMENTS BY MEANS OF
THE HYDRAULIC FRACTURING TECHNIQUE
IN THE KANTO-TOKAI AREA, JAPAN

Hiroaki TSUKAHARA
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Abstract

In situ stresses were obtained by the hydraulic fracturing technique in the Kanto-Tokai area, which has been designated an "area of intensified observation" by the Coordinating Committee for Earthquake Prediction in Japan. From a plate tectonics perspective, this area is one of complex interactive motion between the Pacific, Philippine Sea, and Eurasian plates. Eight wells (two 100 m deep and six 450 m deep) were drilled for stress measurements. About 60 hydrofracturing tests in total were conducted in these wells.

The results about the magnitude of stress show that (1) both the minimum and maximum horizontal compressive stresses increase steadily with depth, (2) at each site the minimum horizontal stresses are greater than vertical stresses (calculated from density) at every depth, (3) the differential stress between the maximum horizontal stress and the vertical stress varies widely from site to site (from 4 to 10 MPa at 400 m depth), (4) the difference between the minimum horizontal stress and the vertical stress is typically rather small (< 2.5 MPa at 400 m depth), (5) the gradient of the minimum stress increase with depth is closely equal to that of vertical stress increase with depth (except for one site), and (6) two sites, Okabe and Nishiizu, located on either side of the Suruga trough have relatively high differential stresses, approximately 10 MPa. The phenomenon of (6) may be related to the high potential of crustal activity around the Suruga trough, where the rupture zone of an impending great Tokai earthquake is presumed to be.
Distribution of directions of the maximum compressive stress in the area is summarized by using results from various methods; in situ stress measurements by the hydrofracturing technique and the overcoring method, focal mechanisms of shallow earthquakes, geological survey of active faults, Quaternary cinder cone alignments, and Quaternary dike trends. The different types of data are mostly consistent with each other. Distribution of these stress directions indicates that the Kanto-Tokai area could be divided into some "stress provinces" where stress directions appear almost uniform. Stress directions in most of the stress provinces are well explained in terms of interactive motion of the three plates. The maximum compressive stress direction in the northern part of the Kanto district is ENE-WSW, which is attributed to the interaction of the Eurasian plate and the Pacific plate. The stress direction in the southern part of the Kanto district and the west-side of Suruga bay (southeastern part of the Tokai district) is NW-SE, which is mainly ruled by interaction of the Eurasian plate and the Philippine Sea plate. The direction in the east-side of Suruga bay, that is, the western part of the Izu Peninsula, is N-S, which is explained by downward bending of the Philippine Sea plate.

The maximum compressive stress directions are compared with the maximum compressive strain directions determined from geodetic survey over the last 50-80 years. The stress provinces with high seismic activity in the upper crust show good agreement between compression directions determined geodetically and the maximum compressive in situ stress directions. The stress provinces where these two directions do not agree well are seismically inactive. These phenomena are explained by considering stress increase with increasing strain accumulation. When two directions do not agree well, the increase in strain does not work effectively to increase the differential stress. In the extreme case, if the angle between two directions is larger than 45°, the differential stress decreases even if strain is increasing. Therefore, it is important for earthquake prediction to detect the orientation difference between the maximum in situ stress and maximum strain rate.

1. Introduction

Hydraulic fracturing as a method of in situ stress measurement has been developed in the last 10 years. Now, it offers one of the best opportunities to determine both the orientation and magnitude of stress to depths exceeding a few kilometers. As is well-known, earthquakes occur when rocks cannot sustain the increasing stress. Therefore, stress data at depth is one of the most important items of information for earthquake prediction research.

The first project on in situ stress measurements for earthquake prediction using the hydraulic fracturing technique in Japan was begun in 1976 at the National Research Center for Disaster Prevention. Some ten measurements were planned in the Kanto-Tokai area, which is the "area of
intensified observation" designated by the Coordinating Committee for Earthquake Prediction in Japan. The first successful stress measurements were made in two 100 m deep wells in 1978 (Tsukahara et al., 1978a, b). Subsequently, measurements in several boreholes of a depth of 450 m have been made (Tsukahara et al., 1980, 81, and Ikeda and Takahashi, 1981). After the stress measurements, seismometers were installed at the bottom of some of the wells so that they could be used as observation wells for microearthquakes.

This report presents the results obtained from two 100 m deep wells and six 450 m deep wells in the Kanto-Tokai area and also presents some interpretations about the magnitude of stress, differential stress, stress direction and some relations between in situ stress and seismic activity.

2. Experimental sites and geological setting

The locations of the wells are shown in a geologic and tectonic map (Fig.1), which shows the complicated geological setting of this area. The Kanto-Tokai area stands at the junction of three plates; the Philippine Sea, Pacific, and Eurasian plates, which is illustrated in Fig. 1 (insert). The Philippine Sea plate is considered to be moving northwestward and subducting at the Suruga and Sagami troughs under the Eurasian plate, and the Pacific plate is moving westward and subducting at the Japan trench.

Some faults active during Quaternary time are distributed in the Miura Peninsula and the Boso Peninsula, and particularly in the Izu Peninsula. In the Izu Peninsula high seismic activity and abnormal uplift have been observed for several decades. In the Kanto plains around Tokyo, only a small number of active faults have been detected. It is thought that many faults could be covered by thick accumulations.

Fig. 2 shows vertical cross sections through the experimental sites. The azimuth of each cross section was chosen to indicate maximum topographic relief. This figure shows that the effect of topographic reliefs on the stress distribution at the measurement sites of Yokosuka (Y), Choshi (C) and Nakaminato (NA) are negligible. The stresses measured at the Okabe (M, K and OK), Nishiizu (N) and Futtsu (F) wells may be affected in some degree by the topographic reliefs. In this paper, we basically neglect the topographic effect. That is, we assume one principal stress is vertical and it is only due to the weight of the immediate overburden.

Okabe wells (M, K and OK)

Three wells were drilled into mudstone and sandstone with some altered clayey beds of Paleogene age (the Setogawa Group); two 100 m deep wells (K and M) and a 450 m deep well (OK). Measurements were made in sandstone beds. Stress measurements in the K and M wells were made in 1978 and in the OK well, in 1981. These three wells are situated in a valley trending
north-south, and are located about 35 km west of the Suruga trough (Fig. 1), which may soon be the site of a large earthquake (M 8) based on the recurrence time and geodetic survey data, and about 10 km west of the Itoigawa-Shizuoka Tectonic Line (Fig. 1) which is one of the major tectonic lines in the Japanese Islands. The OK well is located 10 m and 4 km south of K and M, respectively.

Nishiizu well (N)

A 450 m deep well was drilled into indurated tuffaceous sandstone of Miocene age (the Nishina Formation in the Yugashima Group), which contains some altered clayey beds. All measurements were made in consolidated tuffaceous sandstone. The Izu Peninsula has been one of the most vigorous districts in crustal activity in Japan since 1974 (M=6.9, the Izu-Hanto-Oki Earthquake). The well is located 15 km north of the fault of the 1974 earthquake and 20 km east of the Suruga trough. The nearest active fault (left-lateral strike-slip) to the site is located 5 km southeast of the well. The well is located in a valley trending southwest-northeast.

Yokosuka well (Y)

A 450 m deep well was drilled into mudstone of Early Miocene age (the Morito Formation in the Hayama Group). The mudstone had many pre-existing fractures. However, hydrofracturing data were obtained at three depths in competent but highly jointed rock. The site is approximately 35 km north-east of the Sagami trough, which contains the hypocenter of the Kanto Earthquake, 1923 (M=7.9), and 400 m northeast of the nearest active fault which is a right-lateral strike-slip fault (see Fig. 2).

Futtsu well (F)

A 450 m deep well was drilled into sandstone of Late Miocene age (the Amatsu Formation in the Miura Group). The rocks had few pre-existing fractures. The site is 4 km south of the nearest active fault, which is a dip-slip with right-lateral fault.

Choshi well (C)

A 450 m deep well was drilled into shale containing interbedded clayey rocks. Lower Cretaceous rocks crop out on the surface around this site, which is composed mainly of well-cemented sandstone. However, the rocks in the well were composed of shale and clayey beds and contained many joints. Therefore, we could not detect any new cracks originated by hydraulic fracturing. Rocks of this age in this area are bordered on the west by an extinct fault and are narrowly distributed along the coastline. Exposures of the rocks are confined to such a small area that they cannot be shown in Fig. 1. Although the rocks crop out narrowly on the land, they are estimated to extend largely from the coast to the sea floor.
A 450 m deep well was drilled into sandstone of Late Cretaceous age (the Hiraiso Formation in the Nakaminato Group). Rocks were composed of well-cemented sandstone. Rocks of this age in this area are also bordered on the west by another extinct fault and exposures of these rocks are confined to a narrow area. Although they are not shown in Fig. 1, they are estimated to extend to the sea floor.

3. Field operations

Our field equipment is schematically illustrated in Fig. 3. The straddled interval subjected to high pressure for hydraulic fracturing was 2.2 m long. Fracturing was carried out by pumping water into the straddled interval between two inflated packers at constant rates which varied between 5 and 100 l/min. The pressure and flow rate of the fluid was measured simultaneously in the hydraulic line on the surface, and in some experiments a downhole pressure transducer with logging cable was also used for pressure measurement. All of the data were recorded by both a multipen chart recorder and a magnetic tape recorder.

The azimuth of the hydraulic fracture (which indicates the direction of the maximum horizontal compressive stress, Hubbert and Willis, 1957) was detected by an ultrasonic borehole televiwer (Zemanek et al., 1969) and/or an impression packer (Anderson and Stahl, 1967).

4. Field data and stress calculations

A total of about 60 hydraulic fracturing tests were conducted in the 8 wells. The number of measurements attempted in each well was limited by the well wall condition estimated from various logging data and from a borehole televiwer picture.

The following relationships were used in calculating the principal horizontal stresses (Bredehoeft et al., 1976, Zoback et al., 1977, and Haimson, 1978).

\[
S_{Hmin} = \frac{P_s}{S} \quad (1)
\]

\[
S_{Hmax} = 3P_s - P_b - P_p \quad (2)
\]

where \(S_{Hmax}\) and \(S_{Hmin}\) are the maximum and minimum horizontal compressive stresses (compression is positive), respectively, and \(P_s\), \(P_b\), \(P_i\) and \(P_p\) are the reopening pressure of the hydraulic fracture, the instantaneous shut in pressure (the pressure necessary merely to keep the fracture open), and the normally existing pore pressure in the rock, respectively. We take the value of \(P_p\) as hydrostatic pressure because the water table was near the surface at all the sites.

A typical pressure-time record (hydrofracture at 248 m in the Naka-
minato well) is given in Fig. 4. Pumping was started at point A and was continued at a constant rate. Pumping was stopped at point B, and the well was shut in to obtain the instantaneous shut in pressure \( P_1 \). Four pressurization cycles are shown, which yielded reopening pressures \( P_{rb} \) as well as repeated \( P_s \) values. Instantaneous shut in pressures and reopening pressures were simply determined from inflection points in pressure-time records. In cases where multiple pressurizations of a zone produce multiple values of the instantaneous shut in pressure and/or the reopening pressure, an averaged value was taken after omitting greatly deviating values from other data.

The horizontal stresses \( (S_{Hmax}, S_{Hmin}) \) were calculated based on the relationships (1) and (2) by using \( P_s \) and \( P_{rb} \) values. The vertical compressive stress (lithostatic pressure) \( S_v \) was calculated based on rock density (taken from logging data and/or core) of each well;

\[
S_v = 9.8 \times 10^{-3} D \cdot h
\]

where \( S_v \) is the vertical stress in MPa, \( D \) is the density in g/cm\(^3\) and \( h \) is the depth in m.

Typical pictures of created cracks observed by the televiewer and obtained by the impression packer are shown in Figs. 5 and 6. Fig. 5 shows a borehole televiewer record of clear hydraulic fractures from a depth of 263 m in the Nishiizu well. Fig. 6 shows an impression packer with clear hydraulic fractures from 225 m in the Okabe well. The orientation of the impression packer was measured by using a compass in non-magnetic drillpipe, which is placed just above the upper packer. We have not succeeded in detecting the same fracture by both techniques.

Calculated stresses \( (S_{Hmax}, S_{Hmin}, S_v) \), and \( S_{Hmax} \) directions are summarized in Table 1 and presented in Fig. 7. The data which indicate small values of tensile strength \( (=P_{rb} - P_{O}) \), showing reopening of a pre-existing fracture, are also plotted all together. However, the data which show a greatly different azimuth of the fracture from the average value obtained from the same well are not plotted in the figure. In the case of N343, N423 and N436, hydraulic fractures were made by holding constant high water pressure in the straddled interval. Therefore, we did not measure the prevailing breakdown pressure. The prevailing breakdown pressure is somewhat greater than the holding pressure due to decrease of strength with increase of the pore pressure in rocks. However, we do not need to evaluate the decrease \( (a, b \text{ and } c \text{ in } P_{O} \text{ column of N343, N423 and N436 in Table 1}) \) because \( P_{O} \) is not used for calculation for stress values. In Fig. 7 solid straight lines fitting the \( S_{Hmax} \) and \( S_{Hmin} \) data were determined by the least squares method, except for Yokosuka. As there are only three scattered data in the case of the Yokosuka site, the least squares method was applied to the data under the condition of the straight line with the same gradient as the lithostatic pressure.
5. Discussion of results

(1) Magnitude of stress

As shown in Fig. 7 both the minimum and maximum horizontal stresses generally increase with depth, which is the same phenomenon as those reported in many other stress measurements.

Differential stresses of $S_{H_{max}}$ and $S_{H_{min}}$ from lithostatic pressure ($S_v$) as a function of depth are given in Fig. 8. The left-hand figure reveals that the minimum horizontal stresses are greater than lithostatic pressure at every depth at each site. However, the difference between the minimum horizontal stress and lithostatic pressure is rather small (<2.5 MPa at a depth of 400 m for all measurements except for Yokosuka) and the difference is almost constant regardless of the depth (except for Nishiizu).

The differential stress between the maximum horizontal stress and the lithostatic pressure varies widely from site to site, which is shown in the right-hand figure in Fig. 8. Because lithostatic pressures (vertical stresses) are the minimum stress for all depths and sites, this figure represents the variations of the maximum differential stresses with depth. The figure shows that Okabe, Nakaminato and Nishiizu have relatively large differential stresses; around 10 MPa at 400 m. The Okabe and Nishiizu sites are located near the Suruga trough, which is supposed as mentioned in a previous section to be one of the most critical areas for a next great earthquake. The relatively high differential stress level at these two sites may suggest high potential for crustal activity around the Suruga trough area.

(2) Fault type and stress condition around the sites

Nishiizu site

Focal mechanisms of shallow (<15 km) earthquakes and active faults near the site are strike-slip type faults. Therefore, relative magnitude of the stresses should be $S_{H_{max}} > S_v > S_{H_{min}}$. We can estimate the stress magnitude at depth from extrapolation of measured data to deeper parts. As shown in Fig. 7, the relative magnitude of $S_{H_{min}}$ to $S_v$ at Nishiizu well changes at about 600 m, and that of $S_{H_{max}}$ to $S_v$ seems also change at a depth of several kilometers. Therefore, at depths between 600 m and several kilometers, the relative magnitude of the stresses is consistent with that estimated from the shallow earthquakes and active faults mentioned above. However, the relative magnitude is inconsistent in the crust deeper than several kilometers. This disagreement indicates that the straight line on the data of $S_{H_{max}}$ cannot be used for extrapolation deeper than several kilometers.
Yokosuka and Futtsu sites

Although there is no data on the focal mechanism of a shallow earthquake in the vicinity of these two sites, there are some active faults near these sites as mentioned in a previous section.

Active faults near the Yokosuka well are right-lateral strike-slip faults. Therefore, the stress condition should be $S_{Hmax} > S_v > S_{Hmin}$. The relative magnitude of $S_{Hmin}$ to $S_v$ is different from the measured data; $S_{Hmin} > S_v$.

Active faults near the Futtsu well are complicated; dip-slip (both normal and reverse movements can be traced but current movement is not detectable) with right-lateral strike-slip faults. Therefore, we cannot discuss stress condition in connection with fault analysis.

Other sites

There is no data to be compared with our data now.

(3) Stress orientation: Local stress orientations estimated from various methods

Fig. 9 shows the maximum compressive stress directions measured by various methods; active fault analyses, Quaternary cinder cone alignments (Nakamura, 1975), Quaternary dike trends (Nakano et al., 1980), focal mechanism solutions of shallow (in the upper crust) earthquakes (Abe, 1974, Japan. Meteor. Agen., 1978, and Ishibashi, 1980), in situ stress measurements on the surface by the overcoring method (Geol. Surv. Japan, 1980 and Koide et al., 1981), and the hydraulic fracturing data. It is demonstrated that the stress directions measured by various methods are compatible with each other locally, but the stress direction is not entirely uniform through the Kanto-Tokai area.

The stress directions estimated from the dike trends, focal mechanisms of earthquakes, and cinder cone alignments indicate the near-surface stress condition in the crust. Dike trends and cinder cone alignments have formed for about a million year, while earthquake mechanisms and the in situ stress measurement data show the current stress state. It is noteworthy that most data on stress direction are compatible even though they are for different periods within the Quaternary and the methods sample different depths in the crust.

(4) Stress orientation: Relation to the plate tectonic model

The distribution of measured stress direction seems to be complex for such a small area. However, the stress state is rather understandable when the relative motion of the Pacific, Philippine Sea, and Eurasian plates is taken into account. We interpret the observed stress direction.
in terms of the interaction among the plates in the following discussion.

Our view of current stress orientations in the upper crust is illustrated in Fig. 10 (the central figure) by dashed lines. We divided this area into 6 "stress provinces", where stress directions appear almost uniform, as proposed by Zoback and Zoback (1980). Basically, we believe that stress in the southern part of this area is controlled by the north-westward movement of the Philippine Sea plate, and that in the northern part is controlled by the westward movement of the Pacific plate relative to the Eurasian plate (see upper left-hand corner of Fig. 10).

We describe each stress province as follows. The stress state in area R is ruled directly by the NW movement of the Philippine Sea plate, and all of the directions measured in this area are compatible with the NW movement. The stress directions in areas Q and U are quite different from that in area R although areas Q and U adjoin R. As proposed by Nakamura (1980) to explain the stress direction variation in the Izu Peninsula, downward bending of the Philippine Sea plate at the Suruga trough seems to be the most probable process for giving rise to area Q with its different stress direction. This process is illustrated schematically in Fig. 10, lower left. The magnitude of the NW component of the stress decreases with the downward bending near the Suruga trough, and the maximum stress direction becomes parallel to the Suruga trough in area Q. This process cannot explain the stress directions in area U unless the plate extends beneath the land area.

The upper crust of area P has enough distance not disturbed by the downward bending, and the stress in this area is dominated by the EW compression by the Philippine Sea plate. Area S will stand on equal ground with area P from the tectonic viewpoint as shown in Fig. 10, lower right. That is, the upper crust of area S should be compressed in a N45°W direction like that of area P. The direction at the Futtsu well shows good agreement with this interpretation, but that at the Yokosuka well does not show good agreement. As explained later, this discrepancy may be due to local stress disturbance by an active fault near the measurement site (distance of only 400 m).

The stress state in area T is mainly controlled by the westward motion of the Pacific plate.

(5) Relation among in situ stress, geodetic strain and seismic activity

The maximum compressive strain directions observed by geodetic survey are illustrated in Fig. 11 by dotted lines (accumulated strain more than 10° of the maximum shear strain, Nakane and Fujii, 1979 and Dambara, 1980) with the maximum compressive stress directions. The periods of strain accumulation are shown in the same figure (insert). We adopt results calculated by using the data over as long a period as possible. However, in the case of areas B and E, in order to avoid the local strain disturbance accompanying large earthquakes, we indicate the results cal-
culated by using the data surveyed immediately after two large earthquakes (the 1923 Kanto earthquake, M=7.9, and the 1930 Kita-Izu earthquake, M=7.0) in each site.

Areas C and D show strain accumulation less than $10^{-5}$ of maximum shear strain and show no systematic preferred orientation of strain.

It is clearly shown in Fig. 11 that the stress direction and accumulated strain direction do not always agree. Close agreement in stress provinces P and R, and disagreement in U and S are shown.

We attribute the disagreement in areas U and S to a long-term after-effect of the 1923 Kanto earthquake which occurred along the Sagami trough (near the intersection of the trough and the coastline). The area suffering from crustal deformation in the earthquake (Matsuda et al., 1978) corresponds to areas U and S. Although the geodetic strain measurement is made on the surface, the strain will distribute almost uniformly to depths of several hundreds of meters where our stress measurements have been made. The amount of the maximum shear strain in U and S is less than $4\times10^{-5}$, and this is equivalent to maximum shear stress of 0.6 MPa under the conditions of Young's modulus of 1.6x10^9 MPa (rocks from 150 m depth of the Futtsu well) and elastic deformation only. This stress is small relative to the measured maximum differential stresses. The small magnitude of stress estimated from the strain also suggests that strain accumulation started in 1923 as a long-term after-effect of the earthquake has hardly affected the in situ stress direction yet.

In the case of Yokosuka, the site is located at a distance of only 400 m from an active fault. Stress accumulated in remote ages has been released gradually with plastic deformation around the fault. Therefore, it is supposed that stress around this site has been accumulated, concentrated and amplified for a short period, and the stress direction measured near the fault is closer to the compressive strain direction. On the other hand, stress direction estimated from the active faults near Yokosuka seems to represent the stress direction in the upper crust because of their large dimensions. Therefore, the direction estimated from the faults is reasonably compatible with the NW movement of the Philippine Sea plate. From the facts described above, the current maximum stress direction is concluded to be N45°W in the upper crust in areas S and U regardless of the different maximum compressive strain direction.

Areas P, Q and R show high seismic activity, and areas S, T and U are seismically inactive in the upper crust. In areas P and R, the strain accumulation is continued in the same direction as the maximum stress, while in areas S and U the compressive strain direction is different from the in situ stress direction. That is, the region where the strain accumulation goes in the same direction as that of in situ stress is seismically active because the in situ differential stress increases with
strain accumulation. On the other hand, the region where the current strain accumulation direction disagrees with the in situ stress direction is seismically inactive in the upper crust because the strain accumulation does not work efficiently to increase the in situ differential stress. If the angle between the direction of the maximum compressive in situ stress and that of accumulating strain is larger than 45°, the in situ differential stress will even decrease as strain increases. It is important for earthquake prediction to detect the orientation difference between the in situ stress and currently accumulating strain.

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Fig. 2 Vertical cross-sections and depths at which stresses were obtained. The azimuth of each cross-section is chosen to indicate the maximum topographic relief.
Fig. 3 Equipment for hydraulic fracturing. 1: Clock, 2: Magnetic tape recorder, 3: Multipen chart recorder, 4: Water tank, 5: Flow rate meter, 6: Water pump, 7: Pressure transducer, 8: Casing pipe (16 cm, 50 m), 9: Inflatable packers, 10: Fluid outlet, and 11: Downhole Pressure transducer with a hydrophone.
Fig. 4 Typical records of water pressure and flow rate vs. time plot (at 248 m in the Nakaminato well). Pumping was maintained in each injection cycle between points A and B. $P_b^o$: Breakdown pressure, $P_b^r$: Reopening pressure, and $P_s$: Instantaneous shut in pressure.
Fig. 5 Comparison of ultrasonic borehole televiewer log taken before hydraulic fracturing (left-hand picture) with that taken after hydraulic fracturing (right-hand), which shows new vertical cracks created by hydraulic fracturing. Nishiizu, 263 m depth.
Fig. 6 Newly created cracks printed on the impression packer for the 225 m deep experiment at the Okabe site.
Fig. 7 Stress variations with depth. The linear variation of the lithostatic pressures which are calculated from density log data and/or core density are also shown for reference. M, K and OK: Okabe, N: Nishiizu, Y: Yokosuka, F: Futtsu, C: Choshi, and NA: Nakaminato.
Fig. 8 Differential stresses of $S_{H\text{max}}$ and $S_{H\text{min}}$ from lithostatic pressure ($S_V$). M, K, and OK: Okabe, N: Nishiizu, Y: Yokosuka, F: Futtsu, C: Choshi, and NA: Nakaminato.
Fig. 10 Simplified current stress directions in the upper crust. Dashed lines indicate directions of the maximum horizontal compressive stresses. Arrows show the relative movement of the Philippine Sea plate and the Pacific plate with respect to the Eurasian plate.
HORIZONTAL STRAIN
PERIODS
A 1885-1974
B 1924/26-1974/75
C 1893-1976/78
D 1899/1900-1979
E 1931-1973/75

Fig. 11 The maximum compressive stress directions and the maximum compressive strain directions. 1 - 6: The same legend as on Fig. 9, 7: Area strained less than $10^{-5}$ of maximum horizontal shear strain (Geograph. Surv. Inst., 1979 and 80), and 8: Area strained more than $10^{-5}$ of the maximum shear strain (Nakane and Fujii, 1979, and Dambara, 1980). Dotted lines show the direction of the maximum compressive strain accumulated since two big earthquakes in this area (the 1923 Kanto earthquake, M=7.9, and the 1930 Kita-Izu earthquake, M=7.0). Periods of geodetic survey for each area are given in the inserted figure.
EXPERIENCE WITH HYDRAULIC FRACTURING AS A MEANS OF ESTIMATING IN SITU STRESS IN AUSTRALIAN COAL BASIN SEDIMENTS

BY

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1. INTRODUCTION

The Australian coal industry has for some time expressed the need for a method of measuring in situ stress in coal basin sediments, from the surface, to depths up to 500 m or more. This information is considered of importance for the early planning of colliery layouts and the choice of optimum sites for major entries. Overcoring techniques can be used to measure stress from underground openings once these have been excavated. This, however, obviously precludes use of the information in the early planning phase of project evaluation. After consideration of various alternatives CSIRO Division of Applied Geomechanics decided, in 1975, to undertake a programme of evaluation and/or development of the hydraulic fracturing technique as a means of estimating the in situ stress field at depth (up to 1000 m from the surface) in normal exploration size holes (maximum 75 mm diameter) in typical coal measure roof and floor rocks (not in the coal itself).

Effort concentrated initially on a programme of laboratory work which revealed a number of fundamental aspects of the technique which might be relevant to field applications. The work described here, however, represents the results to date of an ongoing field evaluation programme, in which a very pragmatic point of view is being taken. In this programme the technique is being applied to various situations, employing the simplest possible approach to interpretation consistent with obtaining reasonable results. The opportunity is taken, wherever possible, to compare the results obtained from hydraulic fracturing with the results obtained from overcoring at the same site. In these situations, the results obtained from overcoring are considered as 'standard values'. The reliability of overcoring has been established through considerable experience.

The particular overcoring technique employed in most instances is the CSIRO 'Hollow Inclusion' cell, in which a thin plastic annulus containing several strain gauges is glued into a pilot hole and then overcored using a larger diameter trepanning bit. Strain changes measured during overcoring are related to the pre-existing triaxial stress tensor in proximity to the cell.

The overall aim of the evaluation programme is to develop, through practical experience rather than necessarily through fundamental theoretical considerations, a universal operating procedure and interpretive approach that can be applied by the type of personnel involved in coal mine exploration projects, without specialist supervision. For this reason, complicated experimental procedures and sophisticated interpretations are specifically avoided. On the way through the evaluation programme a number of problems are being encountered. In some cases it has been found possible to either circumvent or allow for these problems in the operating procedure or the method of interpretation. In other instances problems have been highlighted for further attention and the extent of their potential influence on the usefulness of the technique outlined.
2. EQUIPMENT DEVELOPMENT

In order to pursue the evaluation programme it has been necessary to develop a range of original experimental equipment. Three distinct phases of development can be distinguished. The first of these involved the development of a set of portable equipment suitable for use in underground coal mines. The equipment had to be compact, manually handable and intrinsically safe. The equipment is shown in Figure 1. The inflatable straddle packer is based on the use of commercially available seals, $\frac{1}{3}$ m long and designed for operation in B size holes (approx. 60 mm diameter). The sealed-off test section is also $\frac{1}{3}$ m long. The impression packer is also based on a commercially available element and is long enough (1 m) to cover the full extent of the test zone (two seals and sealed-off section). Both packers are located in the hole on simple, tubular installation rods. Separate, flexible, hydraulic hoses allow independent pressurisation of the seals and the sealed-off test section. This means that while hydraulic oil is normally used for seal pressurisation, any fluid desired can be employed in the test section. Separate hand pumps are used for the seals and the sealed-off test section. The pressure in the test section is monitored continuously throughout testing on a clockwork, rotary pressure recorder (Figure 2).

The compact nature of the fracturing tool means that testing has been facilitated in situations where only limited extents of suitable rock types are available. The normal experimental procedure with this equipment involves initially inflating the seals to a pre-determined pressure and then pressurising the test section at a constant rate while keeping the seals at a marginally higher pressure (approximately 1 MPa). This requires two operators. Pressure synchronisation is achieved by means of matched pressure gauges, one in each pressure line. Experimental control in this fashion is quite satisfactory. Pressurisation is continued until crack initiation occurs at which time pumping is ceased to allow an initial shut-in pressure to be determined. Pumping is then continued through several cycles of repressurisation.

The second phase of development involved the design and construction of a fracturing tool for surface operation. After initial experience was gained with commercially available equipment, the equipment in Figures 3 and 4 was contrived. In principle this tool is identical to the original underground equipment. The commercially available sealing elements are 1 m long and designed for operation in N size holes (approx. 75 mm diameter). The sealed-off test section is of adjustable length, from approximately 1 m to 3 m, to suit the particular test horizons available. Separate pressurisation of the seals (hydraulic oil) and the sealed-off test section is possible. A flexible hydraulic hose is used for seal inflation. The tool incorporates down-hole pressure transducers for monitoring both the seal pressure and the pressure in the sealed-off test section during testing. The tool also incorporates a solenoid valve to allow the static head pressure to be relieved from the seals after each test. This allows multiple tests to be conducted without having to bring the tool to the surface. A multi-core electrical cable connects the transducers and solenoid valve to the surface. The impression packer used in conjunction with the tool is similar to that used for the underground work except that it is fitted with a remote reading, flux-valve orientation system to facilitate impression orientation. The impression packer is also fitted with a solenoid valve, like the fracture tool.
To date this tool has been used to limited depth (maximum approx. 120 m) by employing conventional AW size diamond drill rods to locate it in holes. The rods are modified to allow them to be used as a high pressure conduit through which the sealed-off test section can be pressurised with any desired fluid (water to date). Pressurisation of both the seals and the test section is by means of an electric/hydraulic pump, employing flow control valves to permit regulation of the differential pressure between the seals and the test section. Both pressures (down-hole) are recorded throughout testing on a two channel potentiometric chart recorder. The experimental procedure used to date has been essentially the same as that employed for the underground work.

The third phase of development is currently underway. It involves the development of an 'endless tubing unit' (Figures 5 and 6) to allow the surface tool and impression packer to be lowered and raised quickly to prospective test horizons, without the time consuming and uncertain use of drill rods. This will facilitate testing from the surface to depths of up to 1000 m. The unit can potentially 'run' tools at up to approximately 60 m/min. though it has not been used to date. The unit is self-contained with a comprehensive range of pumping capability, pressure and flow rate recording and ancillaries.

3. SUMMARY OF EXPERIENCE

The work described in this section was conducted at a number of sites representing a range of conditions encountered in Australian coal basins. The exception was the testing conducted at Lancefield, near Melbourne Victoria which was conducted in a granite outcrop. This site was used essentially as a proving ground for equipment development but yielded interesting results in its own right. Work at this site was conducted in relatively shallow holes (10 metres) drilled from the surface. The secondary principal stress components in the horizontal plane were measured independently by overcoring from the surface using the U.S.B.M. borehole deformation gauge. Fracturing at all other sites was conducted either exclusively from underground (depth up to 450 m) using the previously described portable equipment in holes up to 20 metres deep drilled into the roof or floor from openings, or a combination of underground and surface testing, the latter in holes up to 120 metres deep. At all sites, except Lancefield, the full triaxial stress tensor was measured independently, in proximity to the location of fracturing, by overcoring from underground openings using the CSIRO Hollow Inclusion cell. Testing, other than at Lancefield, was conducted predominantly in various grades of sandstone using either water (tests designated W), hydraulic oil (test designated O) or hydrapol (tests designated H) as the fracturing fluid. Some testing was also conducted in shales. All testing at all sites was conducted in vertical holes. In all, about 100 tests have been conducted to date at seven sites. The results of 30 tests have been drawn on here to exemplify selected facets of experience.

TAHMMOOR

Table 1 is a summary of results obtained from a series of tests, conducted from underground, at the Tahmoo Colliery, 100 km south-west of Sydney, N.S.W. The tests were carried out in a fine grained sandstone unit, using hydraulic oil as the fracturing fluid. Impression packer images revealed that induced cracks were located within the sealed-off test sections and that these cracks all exhibited definite signs of rotation from near axial (vertical) toward transverse (horizontal) at their extremities.
Figure 7 is a typical pressure record obtained during this series of tests. The horizontal orientations of the axial portion of the cracks obtained from the tests in Table 1 are summarised in Figure 8. The average orientation shows good agreement with the orientation of the major, near horizontal, principal stress component obtained from overcoring. The scatter in the orientation can be readily attributed to the error associated with the manual indexing of installation rods used to obtain the orientation of the impression packer.

A series of laboratory miniature fracturing tests were conducted on core recovered from the test hole, to estimate the appropriate strength to use for analysis of results. Lengths of core were prepared with a central axial hole sealed at one end, and with a fluid inlet at the other end to permit internal pressurisation. The samples were all confined externally (radially and axially) to simulate the stress field anticipated to exist in situ. Internal pressurisation was conducted at the same rate and with the same fluid as was used for the field tests. The average strength so determined was used in conjunction with the mean instantaneous shut-in pressure (estimate of minor stress component in horizontal plane \( \sigma_2^1 \)) and corresponding crack initiation pressures to make estimates of the magnitude of the major stress component in the horizontal plane \( (\sigma_1^1) \), employing the simple elastic solutions for impermeable materials \((3 \times \text{instantaneous shut-in [crack initiation - strength]})\). The resulting average values of the magnitudes of the minor and major stress components in the horizontal plane show good agreement with the magnitudes of the corresponding, near horizontal, principal stress components obtained from overcoring.

Values of instantaneous shut-in pressure, recorded in Table 1, were obtained from the pressure records by means of tangent intersection analysis*, which gives a lower estimate than the tangent divergence analysis*. From a purely pragmatic point of view, the authors' experience has shown that the tangent intersection method of analysis gives estimates of \( \sigma_2^1 \) which are generally closer to the values obtained by overcoring than are the corresponding estimates of \( \sigma_2^1 \) obtained from the tangent divergence method of analysis. It was observed that the tangent intersection analysis estimate of \( \sigma_2^1 \) for the first pressurisation cycle was of the same approximate magnitude as the tangent divergence analysis estimate for subsequent pressurisation cycles and that the two estimates tended to converge.

* "Tangent intersection" refers to the point of intersection between the tangent to the pressure curve immediately after crack initiation, and the tangent to the long term pressure curve.

* "Tangent divergence" describes the point of inflection at which the pressure curve diverges from its tangent immediately following crack initiation.
Table 2 is a summary of the results obtained at the Lancefield site mentioned previously. Two tests were carried out in an apparently uniform granite using water as the fracturing fluid. Impression packer images revealed that the induced cracks in this instance appeared to originate from under one or the other of the inflatable seals and extend only to a limited extent into the sealed-off test sections. There was no evidence of crack rotation on the impression packer images. Figure 9 is an abridged pressure record for one of the tests. One interesting feature of this particular record is the relative slowness of the pressure drop-off after crack initiation (cf Figure 7). This appears to be a characteristic of situations in which crack initiation presumably occurs under a seal and is noticed only when fluid can leak from the test section, past the seal and into the crack. This form of behaviour was noticed in many other instances discussed later. In the case of the test depicted in Figure 9, a definite phase of test section depressurisation and repressurisation was included to allow an estimate to be made of the crack re-opening pressure. The average horizontal orientation of the cracks obtained at this site shows reasonable agreement with the orientation of the major secondary principal stress component in the horizontal plane obtained from overcoring (Fig. 10).

One feature of the results summarised in Table 2, compared with those in Table 1, is the much greater range in the instantaneous shut-in pressures for repeated cycles of pressurisation. This can possibly be attributed to the increasing degree of fracture fluid access to the developing crack as the crack extends lengthwise after each pressurisation, with commensurate decrease in the pressure loss associated with the fluid having to leak past the seal. This implies that the best available estimate of the magnitude of the minor stress component in the horizontal plane, \( \sigma_2^{1} \) when crack initiation occurs under a seal, would be made by considering the instantaneous shut-in pressure after several cycles of repressurisation. Certainly in the case of the results summarised in Table 2, best agreement with the magnitude of the minor secondary principal stress component in the horizontal plane obtained from overcoring was for the longer term shut-in pressure (using the tangent intersection method), rather than the initial instantaneous shut-in pressure.

A series of laboratory tests, identical in principle to those described previously, was conducted to determine the range of strength values of the granite in the test zones (including the seal locations) appropriate for analysis of results. The mean strength, as well as the upper and lower limit values, were used in conjunction with the long term instantaneous shut-in pressures and corresponding crack initiation pressures (in this case the peak seal pressures) to make lower bound, upper bound and mean estimates of the magnitude of the major stress component in the horizontal plane (\( \sigma_1^{1} \)) (Table 2). The simple elastic solution for impermeable materials was employed as previously. The average mean value estimate of this component shows reasonable agreement with the magnitude of the major secondary principal stress component in the horizontal plane obtained from overcoring. The average range from lower bound to upper bound estimate was acceptably small. A separate estimate of the magnitude of the major stress component was made for each test by employing the crack re-opening pressure to replace the term 'crack initiation - strength' used in the previous analysis. The results
of this analysis are included in Table 2. In this instance the average estimate was considerably less than that made by using the first method of analysis and also considerably less than the magnitude of the major secondary principal stress component obtained from overcoring.

The magnitude of the crack re-opening pressure was approximately the same regardless of whether the tangent intersection or the tangent divergence point on the curve was adopted*. This was generally the case at all sites where crack re-opening pressure was recorded. Furthermore, the shape of the curves after re-opening appeared always to assume a form consistent with the measured stress regime. For example, at Lancefield where the major stress component was approximately three times the minor stress component in the horizontal plane (i.e. \( \sigma_1 \approx 3\sigma_2 \) and hence \( 3\sigma_2 - \sigma_1 = 0 \)), the borehole pressure continued to rise after re-opening had occurred. However, at the Moura test site (see later) where \( \sigma_1 \approx 2\sigma_2 \) and hence \( 3\sigma_2 - \sigma_1 \approx \sigma_2 \), the pressure curve tended to flatten out after re-opening.

**MOURA**

Table 3 represents a summary of an extensive series of tests conducted both from underground (depth approximately 100 m) and from the surface (hole depth 85 m) at the Moura mine, 200 km south-west of Rockhampton, Queensland. The tests were conducted in a densely cemented, medium grained sandstone using water (surface tests) hydraulic oil and hydrapol (underground tests) as the fracturing fluids. Impression packer images revealed that in all instances (underground and surface) cracks apparently developed from under seal locations and extended into the sealed-off test sections. Some images showed signs that the cracks may have been consistent with high angle (to horizontal) joint planes, at least over part of their extent. Although there was no obvious evidence of jointing (open or incipient) in the core corresponding to any of the test zones when it was inspected prior to testing, high angle jointing was a common feature of the area and was encountered in all holes drilled for the test work. In the cases of cracks suspected of being influenced by jointing, the impression packer images showed a tendency for axially (vertically) extending cracks to 'blend into' high angle planes crossing through the sealed-off test section. In other cases there was no evidence of crack rotation, the essentially linear, vertical crack traces apparently terminating at the limit of lengthwise crack development established by the extent of pumping.

The horizontal orientations of the axial cracks for the tests contained in Table 3 are summarised in Figure 11. There is very good agreement between the orientation of the cracks produced by surface testing and the orientation of the major, near horizontal principal stress component obtained from overcoring. There is also acceptable agreement in the case of the cracks produced by underground testing, particularly for those tests in which the possible influence of jointing could be reasonably ruled out. In the latter instances the orientations clustered very tightly. In most respects the test pressure records

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* With respect to the determination of crack re-opening pressure; "tangent intersection" refers to the point of intersection of the tangents to the pressure curve before and after crack re-opening, and "tangent divergence refers to the point at which the pressure curve diverges from its tangent prior to crack re-opening.
obtained for the work at Moura were similar to those obtained at Lancefield. There was the characteristic relatively slow drop-off of pressure after crack initiation described previously, and the generally considerable range of instantaneous shut-in pressures for multiple pressurisation cycles noticed at Lancefield. Tangent divergence analysis estimates of instantaneous shut-in pressure were much higher than the corresponding values of horizontal principal stress obtained from overcoring. Tangent intersection estimates on second and subsequent loading cycles were closer to the overcoring estimates. As at Lancefield, an attempt was made in each test to establish the crack re-opening pressure by de-pressurising between pressurisation cycles.

The results in Table 3 indicate the effect of test fluid viscosity on the ability to reliably estimate the magnitude of the minor stress component in the horizontal plane (\(\sigma_2^1\)) from the long-term instantaneous shut-in pressure, when crack initiation under a seal is suspected. The closest approximation to the magnitude of the minor, near horizontal, principal stress component obtained from overcoring was for the tests conducted with the lowest viscosity fluid (water), with progressively increasing disparities for the tests conducted with higher viscosity fluids (oil, hydrapol). While the agreement was acceptable, for practical purposes, for the tests conducted with water and oil (especially considering the relatively severe influence of experimental errors at the absolute stress levels involved) the disparity in the case of the tests conducted with hydrapol was excessive. It was observed that the relative differences between the tangent intersection and tangent divergence estimates for instantaneous shut-in pressure, \(\sigma_2^1\), decreased as the viscosity of the test fluid increased.

The tests conducted with hydrapol produced the most consistent crack orientation, free of any apparent influence of jointing. This may have been coincidental (at least two cracks produced using oil showed the same traits) or it may have been a direct result of the higher viscosity fluid not being able to penetrate into incipient joint planes, as may have occurred for water and oil. Re-pressurisation some months later, of the test zones originally tested with hydrapol, using hydraulic oil, produced instantaneous shut-in pressures and crack re-opening pressures consistent with the results obtained using oil as the only test medium. Testing using a combination of fluids such as this may represent a practical means of obtaining usable results in a situation where jointing may otherwise adversely effect the outcome of testing.

Since no reliable estimates were made of the peak seal pressure at crack initiation for any of the tests conducted at Moura, and since crack initiation was believed to occur universally under the seals, it was not possible to estimate the magnitude of the major stress component in the horizontal plane (\(\sigma_1^1\)) using the first method described previously. Attempts to estimate the magnitude of this component using crack re-opening pressures, for the tests conducted with water and oil, gave only fair agreement with the magnitude of the major, near horizontal, principal stress component obtained from overcoring (much better agreement for the tests conducted with water than oil). Errors in this instance can be substantially accounted for by errors associated with the estimation of the magnitude of the minor stress component rather than any fundamental misconception in using the crack re-opening pressure.
Table 4 represents a summary of a series of tests conducted from the surface (hole depth 120 m) at the Wambo mine, in the Hunter Valley area of N.S.W. approximately 200 km north-west of Sydney. The tests were carried out in a moderately permeable, medium grained sandstone using water as the fracturing fluid. Impression packer images revealed that all the induced cracks were axial, linear (i.e. no sign of rotation) and, while apparently extending into the region of the seals, were extensively developed in the sealed-off test section in each instance. Figure 12 is an abridged pressure record for one of the tests. One immediately obvious feature of this record is the relatively rapid drop-off of pressure following crack initiation, when compared with the record in Figure 9. This can probably be attributed to crack initiation commencing under the seals in the test section contemporaneously with, if not preceding, the sealing process. Another factor that points to this is the relatively small range in instantaneous shut-in pressure between the initial value and the value after multiple re-pressurisation cycles for each test. The horizontal orientations of the axial (vertical) cracks for two of the tests contained in Table 4 are summarised in Figure 13. In this case there appears to be excellent agreement with the spatial orientation of the near horizontal components of the stress field obtained from overcoring some distance away but in the same relative location in the rock sequence as the fracture tests.

It was not possible, at this particular site, to make direct comparison between magnitudes of the stress components estimated from the fracturing tests and determined independently by overcoring, because of the possibility that the overcoring measurements were made at a site relatively destressed as a result of mining activity and topography when compared with that used for the fracturing tests. What was concentrated on at this site was the potential reliability of using crack re-opening pressure as a means of estimating the magnitude of the major stress component. To establish this, a series of laboratory miniature fracturing tests (as described previously) was conducted on core covering the full extent of the test zones (including seal locations) for two of the tests contained in Table 4. This allowed 95% confidence limits to be established statistically for the material strength, under conditions comparable to the respective field test (fracturing fluid, rate of pressurisation, degree of confinement) in each case. These results are included in Table 4. It can be seen that in each instance, there is excellent agreement between the mean laboratory determined strength and the corresponding field estimated strength based on the difference between the crack initiation pressure in the sealed-off test section and the crack re-opening pressure. This would seem to lend credence to the use of crack re-opening pressure in the analysis of results.

The tests summarised in Table 4 were all conducted at different rates of pressurisation. This possibly accounts for what appears to be a statistically significant difference in apparent strength for the two tests mentioned above, (W2 and W3) which were conducted in what appeared by core inspection to be identical material. To take this aspect further, a second series of laboratory miniature fracturing tests was conducted on a number of representative core samples selected from the general rock unit in which the field tests were carried out. These tests were conducted at various rates of pressurisation covering the range employed for the field tests. The results are summarised in Figure 14. Also included in Figure 14 are the estimates of strength
obtained from the field tests contained in Table 4. Figure 14 reveals what appears to be a systematic variation in strength with rate of pressurisation, applying to both the laboratory and field situation. This can probably be attributed to the effect of fluid penetration on apparent strength. At the slower rates of pressurisation, penetration presumably extends further causing a relatively lower apparent strength than at faster pressurisation rates. Figure 14 highlights a potential problem when analysing results using laboratory determined strength, in that unless laboratory testing is carefully matched to field test conditions, serious errors may be introduced into the analysis to determine the magnitude of the major stress component. It appears that the use of crack re-opening pressure may be a way of overcoming this potential problem regardless of whether or not fluid penetration occurs.

WALLSEND BOREHOLE

Table 5 represents a very brief resume of some results of particular interest selected from a comprehensive programme of work carried out, from underground (depth approx. 200 m), at the Wallsend Borehole Colliery in the Newcastle area of NSW. The tests were conducted in a particularly strong, fine-grained sandstone, using hydraulic oil as the fracturing fluid. Impression packer images revealed that in most instances short, rotating cracks within the sealed-off test section were induced. These were similar in nature to the cracks produced in the testing at Tahmooor. The horizontal orientations of the axial traces of the cracks obtained at any one site (five sites tested) showed close agreement. It was not possible, however, to compare in absolute terms the orientation found from fracturing with the orientation of the near horizontal stress component obtained from overcoring at one site since, for practical reasons, the overcoring was conducted in the roof of the opening and the fracturing in the floor. There was reason to believe that the stress field may have been horizontally re-orientated in the roof due to local deformation mechanisms.

The most interesting feature of the results of the four tests summarised in Table 5 was the dramatic change in instantaneous shut-in pressure, in each instance, from the initial value immediately following crack initiation to subsequent values following further pressurisation cycles. In the case of tests 1 and 2 (pressure records, Figs. 15 and 16 respectively) the pressure records were similar to that shown in Figure 7 for the tests at Tahmooor (sharp pressure drop after crack initiation) except for the marked change in shut-in pressure which was not a feature of the Tahmooor results. It is postulated that this behaviour arises as a result of the tendency for crack rotation to occur (as noted on the impression packer images) from an initial, predominantly axial (vertical) trend toward a predominantly transverse (horizontal) trend as crack propagation proceeds. In the case of tests 1 and 2 it is believed that the crack development in each case was initially halted while the crack was still predominantly axial (shut-in pressure representative of horizontal stress field) and that after subsequent repressurisation the crack development in each case became predominantly transverse (shut-in pressure representative of vertical stress). In the case of tests 3 and 4 (pressure records Figs. 17 and 18 respectively) it is postulated that the transition from axial to transverse occurred so rapidly that total transition occurred
before the cracks had stabilised after initiation. In each instance there is a characteristic 'kink' in the pressure record during pressure 'drop-off' which can be imagined to correspond to a transient initial shut-in pressure representative of the horizontal stress field.

The validity of the crack rotation postulate can be examined to some extent by comparing the average initial shut-in pressure (estimate of \( \sigma^2 \)) for the four tests in Table 5 with the average magnitude of the near horizontal principal stress components obtained from overcoring in the general region of the fracture tests. As can be seen, the agreement is very good, as is the agreement between the magnitude of the vertical stress obtained from overcoring and the long term shut-in pressure to which all tests appear to be converging (estimate of \( \sigma^3 \)). Similar crack rotational behaviour apparently occurred during the Tahmoores tests. In this case, however, the absence of a marked change in shut-in pressure can probably be attributed to the fact that the vertical stress component and the minor horizontal stress component were of approximately equal magnitude, unlike the situation at Wallsend Borehole.

The implications of this mode of behaviour are potentially serious for the use of hydraulic fracturing to measure in situ stress in some situations. On the evidence of work at Tahmoores and Wallsend Borehole the situation of most concern would appear to be a brittle rock type in a relatively high stress environment (particularly with a large imbalance between horizontal and vertical components), which is probably the situation of greatest practical interest. It seems that it may be possible to overcome the problem by ensuring that pumping is stopped the instant that crack initiation occurs, so that crack development is not taken to the stage where all record of the horizontal stress field is lost. This practice would be contrary to much of the earlier world experience.

NATTAI NORTH

Table 6 represents a selection of results from work carried out, from underground (depth approx. 300 m), at the Nattai North Colliery, approximately 150 km south-west of Sydney, NSW. The tests were conducted in a variety of rock types (shale, fine sandstone with sub-vertical cemented joints and very coarse permeable sandstone) with two different fracturing fluids (hydraulic oil and hydrapol). The results in Table 6 were selected to highlight some of the problems encountered when conducting fracturing experiments in coal measure rocks.

The first part of Table 6 is a summary of results obtained at two sites when testing a horizontally laminated shale, using oil as the fracturing fluid. In all cases, impression packer images revealed that cracks were transverse (horizontal) and were located within the sealed-off test sections. A typical pressure record is shown in Figure 19. This record is similar in all essential respects to records obtained when inducing vertical cracks within the sealed-off test section. The relatively small range of instantaneous shut-in pressures in each case is symptomatic of a stable crack. The average shut-in pressure at each site (estimate of \( \sigma^3 \)) agrees very well with the respective near vertical stress component magnitude obtained from overcoring at that site.
A series of laboratory miniature fracturing tests was conducted on the core recovered from the test zones, using the approach described previously. These tests yielded an average strength across bedding (horizontal) of 3.5 MPa and normal to bedding (vertical) of 16 MPa, a ratio of 0.22 which has been found common for many laminated coal measure rocks. By employing the laboratory value of strength across bedding in conjunction with the average crack initiation pressure for the four tests and the average indicated vertical stress magnitude, it is possible to postulate that the hydraulic fracture tool used for the tests must have applied a maximum axial tensile stress concentration to the wall of the hole in the sealed-off test section of at least 0.6 times the radial pressure. This value may be somewhat on the high side since it is quite likely that there would be a bedding plane of significantly less than average strength situated strategically in the sealed-off test section of each test. If a limiting value of zero strength is considered, the axial stress concentration reduces to approximately 0.3, a value consistent with theoretical predictions. While an axial stress concentration of such magnitude would not cause problems in materials having approximately isotropic strength properties (such as the sandstones involved in most of the tests related) for a wide range of imaginable stress fields, it is obviously of significance when dealing with strongly anisotropic materials such as shale.

The overall significance of the axial stress concentration lies in the implication that it may be impossible to induce axial cracks, in vertical holes, in horizontally laminated rocks when using this type of tool. This would therefore preclude the possibility of obtaining information on the important horizontal stress field, and limit the usefulness of the technique to making estimates of the vertical stress, which is generally unimportant in practical terms. This has been the author's experience in a range of laminated rock types in a variety of stress field situations from vertical stress approaching zero (near surface) to approximately hydrostatic (such as in the case of Nattai North where average horizontal stress magnitude is approximately equal to the vertical stress). Since laminated rock types make up an important proportion of most coal basins and, since situations will inevitably arise in which the technique will need to be used in such rock types, the authors consider the problem of overcoming transverse crack initiation of great importance. Two lines of development in this regard are currently being pursued. The first of these involves the use of an axially reinforced brittle grout column to line the test hole in the selected test zone. The rationale behind this is that during pressurisation the axial reinforcement will inhibit the development of a transverse crack in the rock, at least one that communicates with the pressurised test section, but will not interfere with the free development of an axial crack at a location dictated by the prevailing stress field. The second concept is to construct a tool capable of applying a positive axial compression to the wall of the hole in the test section, during radial pressurisation, with a view to counteracting the axial tensile stress concentration produced.

The second part of Table 6 is a summary of results obtained at one site when testing a fine grained sandstone with sub-vertical cemented joints, using oil as the fracturing fluid. Impression packer images revealed that in each case a sub-vertical crack had developed within the sealed-off test section, corresponding to the location of a joint obvious from core inspection. A typical pressure record is shown in Figure 20.
This record shows a distinctly different form of pressure drop-off from those in which new crack initiation occurred (Fig. 7). There was a tendency for shut-in pressure to decrease with successive pressurisation cycles, but not to the same extent as in the case of cracks originating under seals (see earlier). Since the horizontal orientation (strike) of the joints was approximately parallel to the orientation of the major, near horizontal, principal stress component obtained from overcoring at the site, it was possible to compare the magnitude of the minor, near horizontal, principal stress component obtained from overcoring with the average shut-in pressure used as an estimate of the latter quantity (σ^2). The agreement is reasonable, particularly when considering that relatively small deviations in orientation between the major stress direction and the strike of the joints would mean that the stress acting normal to the plane of the joint would not be the minor stress component but some value in between the minor and major stress component magnitude.

Attempts to estimate the magnitude of the major stress component by using the laboratory determined intact material strength and the 'crack initiation' pressures, employing the simple elastic analysis for impermeable materials, yielded unrealistic results. Since no attempt was made to estimate the 'crack reopening' pressure in either case, it was not possible to take the analysis further. If the 'crack reopening' pressures could have been determined it may have been possible to make some estimate of the magnitude of the major stress component. In any event, it would appear that at least some useful information on the general magnitude of the horizontal stress field can be obtained from the shut-in pressures recorded for tests in vertical holes in which sub-vertical cemented joints occur in the test zones. At least to this extent the problem of sub-vertical joints is not as serious as that of weak bedding planes. It is the authors experience that the problem of joints is also less serious because of their relatively infrequent occurrence, compared with bedding, and the ability to avoid them when selecting test horizons by core inspection. The use of a viscous fracturing fluid, such as hydrapol, to initiate cracking followed by a less viscous fluid to determine shut-in pressure, as employed in the joint affected work at Moura, may prove a way of getting around the problem of jointing if it becomes serious.

The third part of Table 6 is a summary of results obtained at one site, using hydrapol as the fracturing fluid, in a very coarse, permeable sandstone. Earlier attempts to use oil in this material proved fruitless as indicated by the pressure record, Figure 21. The high rate of fluid leakage into the sandstone masked any distinct crack initiation or shut-in pressure. The use of hydrapol as the fracturing fluid meant that a distinct crack initiation could be obtained in this material (pressure record Fig. 22). The cracks induced were shown, from impression packer images, to be transverse (horizontal) and within the sealed-off test section. The sandstone was known, from laboratory testing, to have a considerable anisotropy of strength. This presumably accounts for the transverse crack development. The average long term instantaneous shut-in pressure (estimate of σ^3) showed reasonable agreement with the magnitude of the near vertical principal stress component obtained from overcoring at the site. In all the sites tested, this was the only instance in which a viscous fracturing fluid had to be employed specifically to enable a crack to be initiated. As such, excessive permeability appears to be a relatively rare problem. It would seem, however, that in such cases, viscous fracturing fluids may be employed, and at least the resulting shut-in pressures used to make estimates of some stress component magnitudes.
Table 7 represents a single result obtained from an underground test (depth approx. 400 m) at the Ellalong Colliery, approximately 50 km west of Newcastle, NSW. The test was conducted in a coarse sandstone using hydraulic oil as the fracturing fluid. The test produced a transverse crack (identified from an impression packer image) near the interface between the sealed-off test section and a seal. The occurrence of a transverse crack was considered unusual in what appeared to be an essentially uniform sandstone. The crack was attributed to the occurrence of a carbonaceous parting at the location, representing some degree of anisotropy of strength, in conjunction with the approximately hydrostatic nature of the stress field, as indicated by overcoring, which generally necessitates high borehole pressures in order to initiate a vertical crack. The likelihood of having such a combination of conditions at many test sites lends added import to the need to develop an effective means of overcoming the problem of transverse crack development.
### TABLE 1. SUMMARY OF RESULTS OBTAINED AT TAHHMOOR COLLIERY

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Initial Shut-In MPa</th>
<th>Subsequent Shut-Ins MPa</th>
<th>Range of estimate of $\sigma^1_2$ MPa</th>
<th>Crack Initiation Press. MPa</th>
<th>Mean estimate of $\sigma^1_1$ MPa</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>01</td>
<td>10.8</td>
<td>9.5</td>
<td>9.5-10.8</td>
<td>22</td>
<td>20.8</td>
<td></td>
</tr>
<tr>
<td>02</td>
<td>10.5</td>
<td>8.5</td>
<td>8.5-10.5</td>
<td>21</td>
<td>19.8</td>
<td></td>
</tr>
<tr>
<td>03</td>
<td>10.5</td>
<td>9.2</td>
<td>9.2-10.5</td>
<td>22.5</td>
<td>19.4</td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td></td>
<td>Average mean = 9.8</td>
<td></td>
<td>20.0</td>
<td></td>
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<tr>
<td>Results from overcoring</td>
<td>10.6 MPa</td>
<td></td>
<td>20.5 MPa</td>
<td></td>
<td>Overcoring results for principal components oriented closest to $\sigma^1_2$ and $\sigma^1_1$</td>
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TABLE 2. SUMMARY OF RESULTS OBTAINED AT LANCEFIELD SITE

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<thead>
<tr>
<th>Test No.</th>
<th>Initial Shut-In MPa</th>
<th>Subsequent Shut-Ins MPa</th>
<th>Range of estimate of $\sigma_2^1$ MPa</th>
<th>Crack * Initiation Press. MPa</th>
<th>Best estimate of $\sigma_1^1$ for mean lab strength MPa</th>
<th>Range of estimate of $\sigma_1^1$ for range of lab. strengths MPa</th>
<th>Crack Re-opening Press. MPa</th>
<th>Best estimate of $\sigma_1^1$ from crack re-opening MPa</th>
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<tbody>
<tr>
<td>W1</td>
<td>6.5</td>
<td>4.0</td>
<td>4.0 6.5</td>
<td>12.3</td>
<td>11.0</td>
<td>9.5-13.0</td>
<td>4.6</td>
<td>7.4</td>
<td>Estimates of $\sigma_1^1$ based on range of laboratory strength of 9.8-13.3 MPa with mean strength of 11.3 MPa</td>
</tr>
<tr>
<td>W2</td>
<td>5.6</td>
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<td>3.2 5.6</td>
<td>12.3</td>
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<td>7.1-10.6</td>
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<td>5.7</td>
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<tr>
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<td></td>
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<td>8.3-11.8</td>
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<td>Results from over-coring</td>
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<td></td>
<td></td>
<td>11.0 MPa</td>
<td></td>
<td></td>
<td></td>
<td>Overcoring results for secondary principal components</td>
</tr>
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* Based on seal pressure
### TABLE 3. SUMMARY OF RESULTS OBTAINED AT MOURA MINE

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<th>Test No.</th>
<th>Initial Shut-In MPa</th>
<th>Subsequent Shut-Ins MPa</th>
<th>Range of estimate of $\sigma_2^1$ MPa</th>
<th>Crack Re-opening Press. MPa</th>
<th>Best estimate of $\sigma_1^1$ from crack re-opening MPa</th>
<th>Comments</th>
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<tr>
<td>W1</td>
<td>1.74</td>
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<td>2.22</td>
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<td>Average</td>
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<td>Average best estimate = 1.6</td>
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<td>2.4</td>
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<td>1.90</td>
<td>5.7</td>
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<tr>
<td>Average</td>
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<td></td>
<td>Average best estimate = 1.9</td>
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<td>4.0</td>
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</tr>
<tr>
<td>H1</td>
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<td>-</td>
<td>1.2</td>
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</tr>
<tr>
<td>H2</td>
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<td>5.0</td>
<td>-</td>
<td>1.8</td>
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</tr>
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<td>H3</td>
<td>6.0</td>
<td>6.0</td>
<td>-</td>
<td>2.0</td>
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</tr>
<tr>
<td>Results from over-coring</td>
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<td>1.5 MPa</td>
<td>Estimates of $\sigma_2^1$ and $\sigma_1^1$ not appropriate because of unrealistically high values of shut-in pressures</td>
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<td></td>
<td>Overcoring results for principal components oriented closest to $\sigma_2^1$ and $\sigma_1^1$</td>
<td></td>
</tr>
<tr>
<td>Test No.</td>
<td>Initial Shut-In MPa</td>
<td>Subsequent Shut-Ins MPa</td>
<td>Crack Initiation Press. MPa</td>
<td>Crack Re-opening Press. MPa</td>
<td>Field estimate of strength MPa</td>
<td>Comments</td>
</tr>
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<td>----------</td>
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<td>----------------------------</td>
<td>-----------------------------</td>
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</tr>
<tr>
<td>W1</td>
<td>3.9</td>
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<tr>
<td>W2</td>
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<td>8.4</td>
<td>3.0</td>
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<tr>
<td>W3</td>
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<td>7.0</td>
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<td></td>
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</table>

95% confidence limits for laboratory determined strength at conditions equivalent to field tests

4.5-6.3 MPa (mean 5.4 MPa)

2.6-6.4 MPa (mean 4.5 MPa)
**TABLE 5. SUMMARY OF RESULTS OBTAINED AT WALLSEND BOREHOLE COLLIERY**

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Initial Shut-In MPa</th>
<th>Subsequent Shut-Ins MPa</th>
<th>Comments</th>
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</thead>
<tbody>
<tr>
<td>01</td>
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<td>10.5</td>
<td>6.2-5.5 - 4.8</td>
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<tr>
<td>03</td>
<td>app. 11.0</td>
<td>5.6-5.0 - 4.5</td>
<td></td>
</tr>
<tr>
<td>04</td>
<td>app. 11.0</td>
<td>4.4-4.2 - 4.2</td>
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<tr>
<td>Average</td>
<td>app. 12.0</td>
<td>Converging toward 4.2</td>
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</tr>
<tr>
<td>Results from overcoring</td>
<td>app. 12.0 MPa</td>
<td>app. 4.0 MPa</td>
<td>Overcoring results are for average horizontal stress magnitude and vertical stress component respectively</td>
</tr>
</tbody>
</table>

Average initial shut-in pressure estimate of $\sigma_2^1$
Average long-term shut-in pressure estimate of vertical stress ($\sigma_3^1$)
### TABLE 6. SUMMARY OF RESULTS OBTAINED AT NATTAI NORTH COLLIERY

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Initial Shut-In MPa</th>
<th>Subsequent Shut-Ins MPa</th>
<th>Range of estimate of $\sigma_3^1$ MPa</th>
<th>Crack Initiation Press. MPa</th>
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<tr>
<td>01</td>
<td>6.8</td>
<td>5.6</td>
<td>5.6-6.8</td>
<td>17</td>
<td>Site No. 1</td>
</tr>
<tr>
<td>02</td>
<td>6.4</td>
<td>5.5</td>
<td>5.5-6.4</td>
<td>16</td>
<td></td>
</tr>
<tr>
<td>Results from over-coring</td>
<td></td>
<td></td>
<td>6.4 MPa</td>
<td></td>
<td>Overcoring results for principal component closest to $\sigma_3^1$</td>
</tr>
<tr>
<td>03</td>
<td>5.5</td>
<td>4.8</td>
<td>4.8-5.5</td>
<td>16.3</td>
<td>Site No. 2</td>
</tr>
<tr>
<td>04*</td>
<td>7.5</td>
<td>6.0</td>
<td>6.0-7.5</td>
<td>14.8</td>
<td></td>
</tr>
<tr>
<td>Results from over-coring</td>
<td></td>
<td></td>
<td>5.5 MPa</td>
<td></td>
<td>Overcoring results for principal component oriented closest to $\sigma_3^1$</td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Initial Shut-In MPa</th>
<th>Subsequent Shut-Ins MPa</th>
<th>Range of estimate of $\sigma_2^1$ MPa</th>
<th>Crack** Initiation Press. MPa</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>05</td>
<td>10.5</td>
<td>9.0</td>
<td>9-10.5</td>
<td>17.5</td>
<td>Average laboratory determined strength at conditions equivalent to field tests = 15 MPa for intact material</td>
</tr>
<tr>
<td>06</td>
<td>9.0</td>
<td>8.0</td>
<td>8.0-9.0</td>
<td>15.0</td>
<td></td>
</tr>
<tr>
<td>Results from over-coring</td>
<td></td>
<td></td>
<td>7.1-9.3 MPa</td>
<td></td>
<td>Overcoring results for principal components oriented closest to horizontal plane</td>
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<tr>
<th>Test No.</th>
<th>Initial Shut-In MPa</th>
<th>Subsequent Shut-Ins MPa</th>
<th>Range of estimate of $\sigma_3^1$ MPa</th>
<th>Crack Initiation Press. MPa</th>
<th>Comments</th>
</tr>
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<tr>
<td>H1</td>
<td>8.3</td>
<td>5.5</td>
<td>5.5-8.3</td>
<td>14.5</td>
<td></td>
</tr>
<tr>
<td>H2</td>
<td>7.0</td>
<td>6.3</td>
<td>6.3-7.0</td>
<td>15.0</td>
<td></td>
</tr>
<tr>
<td>Results from over-coring</td>
<td></td>
<td></td>
<td>6.4 MPa</td>
<td></td>
<td>Overcoring results for principal components oriented closest to $\sigma_3^1$</td>
</tr>
</tbody>
</table>

* Result looks atypical ** Actually "joint opening"
TABLE 7. SUMMARY OF RESULTS OBTAINED AT ELLALLONG COLLIERY

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Initial Shut-In MPa</th>
<th>Subsequent Shut-In MPa</th>
<th>Range of estimate of $\sigma_3^{\perp}$ MPa</th>
<th>Crack Initiation Press. MPa</th>
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<tr>
<td>01</td>
<td>7.0</td>
<td>6.5</td>
<td>6.5-7.0</td>
<td>32</td>
<td>Overcoring result for approximate vertical stress</td>
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<td></td>
<td></td>
<td></td>
<td>Approx. 7.0 MPa</td>
<td></td>
<td></td>
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Results from overcoring
Figure 1. Equipment for underground testing, from top: Installation rods, hand pump, hoses, fracturing tool and impression packer. The sonic detector was used in early applications to help in detection of crack initiation, but was not found necessary.
Figure 2. Typical experimental arrangement for underground tests, showing pumps and clockwork pressure recorder.
Figure 3. Fracturing tool for surface use, showing hydraulic hose and instrument cable.

Figure 4. Detail of down-hole pressure transducers and solenoid valve for surface fracture tool.
Figure 5. General arrangement of "Endless Tubing Unit"
Fig. 7.- Typical pressure record obtained during testing at Tahmmoor Colliery, N.S.W.
Figure 6. Detail of injector head on "Endless Tubing Unit".
Fig. 8. - Summary of orientation information obtained during testing at Tahmooar Colliery:
- $\sigma_1'$ is major secondary principal stress component in horizontal plane indicated by fracturing (orientation assumed to correspond to crack orientation)
- $\sigma_1$, $\sigma_2$ are near horizontal principal stress components determined by overcoring.
Fig. 9.— Abridged pressure record obtained during testing at Lancefield site, Vic.
Fig. 10.- Summary of orientation information obtained during testing at Lancefield site:
- $\sigma_1$ is major secondary principal stress component in horizontal plane indicated by fracturing (orientation assumed to correspond to crack orientation)
- $\sigma_1$, $\sigma_2$ are secondary principal stress components determined by overcoring.
Fig. 11.- Summary of orientation information obtained during testing at Moura Mine:

- $\sigma_1$ is major secondary principal stress component in horizontal plane indicated by fracturing (orientation assumed to correspond to crack orientation).
- $\sigma_1$, $\sigma_2$ are near horizontal principal stress components determined by overcoring.
Fig. 12.- Abridged pressure record obtained during testing at Wambo Mine, N.S.W.
Fig. 13.- Summary of orientation information obtained during testing at Wombo Mine:

- $\sigma_1'$ is major secondary principal stress component in horizontal plane indicated by fracturing (orientation assumed to correspond to crack orientation)
- $x$ indicates spatial arrangement of near horizontal principal stress components determined by overcoring.
Fig. 14. - Summary of results of laboratory miniature fracture tests conducted on Wambo rock to determine fracture strength. Field estimates of strength from fracture tests are shown superimposed.
Fig. 15. - Typical pressure record obtained during testing at Wallsend Borehole Colliery when producing a short, rotating crack in test section.
Fig. 16.—Typical pressure record obtained during testing at Wallsend Borehole Colliery when producing a short, rotating crack in test section.
Fig. 17. - Typical pressure record obtained during testing at Wallsend Borehole Colliery when producing a short, rotating crack in test section.
Fig. 18.- Typical pressure record obtained during testing at Wallsend Borehole Colliery when producing a short, rotating crack in test section.
Fig. 19. Typical pressure record obtained during testing at Maltai North Colliery when producing a transverse crack in laminated shale.
Fig. 20. - Typical pressure record obtained during testing at Nattai North Colliery when joint opening occurs.
Fig. 21. - Pressure record obtained when using oil in a very permeable sandstone at Nattai North Colliery.
Fig. 22.- Pressure record obtained when using hydral in a very permeable sandstone at Nattai North Colliery.
The Interpretation of Hydraulic Fracturing Pressure-Time Data for In Situ Stress Determination

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ABSTRACT

In this paper we discuss the manner in which in-situ stresses affect the form of hydraulic fracturing pressure-time records and we suggest a physical basis for interpretation of these data. Hydraulic fracturing pressure-time records can be classified according to the relative magnitudes of the breakdown pressure, $P_b$, the fracture opening pressure, or the pressure at which the already-formed hydraulic fracture opens at the wellbore in subsequent pressurization cycles to accept fluid, $P_b(T = 0)$, and the minimum horizontal principal stress, $S_h$. Type 1 hydraulic fractures occur when $P_b > P_b(T = 0) > S_h$, Type 2 hydraulic fractures occur when $P_b > S_h > P_b(T = 0)$, and Type 3 hydraulic fractures occur when $S_h > P_b > P_b(T = 0)$. The transition from Type 1 to Types 2 and 3 hydraulic fractures corresponds to an increase in the magnitude of $(S_H + P_p)/S_h$, where $S_H$ is the maximum horizontal principal stress and $P_p$ is the pore pressure. The transition from Type 2 to Type 3 hydraulic fractures corresponds either to a further increase in $(S_H + P_p)/S_h$, a decrease in tensile strength, or a combination of both effects. The normal stresses acting on the hydraulic fractures in these three cases are quite similar except for the portion of the hydraulic fracture that is within a few inches of the borehole. This is important because carefully controlled experimental procedures are required for hydraulic fractures of Types 2 and 3 to be correctly identified and their fracture opening pressures to be accurately determined.

The instantaneous shut-in pressure (ISIP) and the downhole pumping pressure are typically observed to decrease slightly with cumulative pumped volume. We suggest that this results from the reduction in magnitude of
viscous pressure losses within the hydraulic fracture as it propagates and that, as the test progresses, the ISIP asymptotically approaches $S_h$. Therefore, it is this approximately stable value of the ISIP that we customarily use as the best measure of $S_h$.

We determine the maximum horizontal principal stress, $S_H$, from the fracture opening pressure, the pore pressure, and $S_h$. The fracture opening pressure is frequently observed to decrease with cumulative pumped volume, in a manner similar to that observed for the ISIP. We suggest that this decrease in fracture opening pressure results from some combination of: 1) incomplete breakdown on the first cycle, 2) infiltration of fluid into the hydraulic fracture during the early stages of borehole repressurization, and 3) an increase in $P_p$ due to the diffusion of fluid into the rock surrounding the borehole during pumping and shut-in. In an attempt to minimize the error due to these effects in $P_b(T = 0)$, and therefore in the computed value of $S_H$, we suggest using the fracture opening pressure from the third cycle for determining $S_H$. This method seems to yield consistent values for $S_H$, even in cases when tensile strength changes markedly in a given well.
INTRODUCTION

Use of the hydraulic fracturing method for making in situ stress determinations at depth has increased dramatically in the past few years. Hydraulic fracturing stress measurements have been used widely to constrain models of crustal dynamics in both seismically active areas and stable continental interiors (Haimson, 1978; Richardson et al., 1979; Zoback and Zoback, 1980), in the design of underground structures such as pumped hydroelectric facilities, nuclear waste repositories, and coal mines (Haimson, 1981; Doe et al., 1982; Enever and Wooltorton, 1982), and in the planning of massive hydraulic fracturing operations to increase the productivity of oil, gas, and geothermal energy reserves (Aamodt and Kuriyagawa, 1982; Bawden, 1982; Gronseth and Kry, 1982).

Determination of in situ stresses using the hydraulic fracturing technique, however, has not proved to be as straightforward as simple theory would predict. Due to the large number of investigators currently employing this method, we feel it useful to present a detailed discussion of common characteristics of the hydraulic fracturing pressure-time data that we have obtained and to provide what we feel is a sound physical basis for the interpretation of these data to determine the principal in situ stresses. In this paper we define three fundamental types of hydraulic fracturing pressure-time histories on the basis of the relative magnitudes of the breakdown pressure, the pressure necessary to open a hydraulic fracture, and the minimum principal stress. We then discuss in detail the relationship between these three types of pressure-time histories and the relative magnitudes of the principal stresses and how the minimum and maximum principal stresses and tensile strength should be determined from the tests. This
discussion draws heavily upon our personal experience in conducting about 60 successful tests at 14 different sites. It should be noted that, because we work primarily in granite using fairly standardized test procedures, we may not address some of the problems encountered by other investigators that are mentioned elsewhere in this volume.

**BASIC TEST PROCEDURE**

In the hydraulic fracturing technique one principal stress is assumed to be parallel to the borehole and the propagation of a hydraulic fracture from a vertical borehole is assumed to be perpendicular to $S_h$, the minimum horizontal principal stress. The assumption that the hydraulic fracture propagates perpendicular to $S_h$ is well supported by both laboratory and theoretical studies (Hubbert and Willis, 1957; Haimson and Fairhurst, 1970; Haimson and Avasthi, 1975). The magnitude of $S_h$, therefore, can be determined from the pressure in the hydraulic fracture immediately after pumping into the well is stopped and the well is shut in. Determination of the maximum horizontal principal stress, $S_H$, requires the assumption of the elastic concentration of stresses around a circular borehole. All of the tests reported here were performed in intact granitic rock and the assumption of elastic behavior near the wellbore is presumed to be valid. In some cases, however, the material around the wellbore clearly cannot support the concentrated stresses and fails in compression, resulting in well elongation (Bell and Gough, 1982). When this occurs, the assumption of elastic behavior near the wellbore is clearly not valid and $S_H$ cannot be determined with the methods described below.

Figure 1 shows a typical hydraulic fracturing pressure-time record from a
well drilled in crystalline rock near the San Andreas fault in central California. The basic procedure we follow in conducting hydraulic fracturing stress measurements is as follows: 1) conduct multiple cycles of fluid injection, increasing the pumping time for each subsequent cycle, 2) pump at the same flow rate on each cycle of the test, and 3) permit "flow-backs" to occur following each injection cycle to allow for drainage of excess fluid pressures from the hydraulic fracture. In this paper we discuss the rationale for this procedure, and the methods we use to determine $S_h$ and $S_H$. It should be noted that all pressure records shown here are from a surface pressure transducer that is affected by a viscous pressure loss in a high pressure hose going to the wellhead. Because the pumping rate is approximately constant on each cycle, the pressure drop in the hose is approximately constant so that on any given cycle the wellhead pressure is several bars less than the pressure recorded during pumping. The downhole pressure can be determined by adding the appropriate hydrostatic pressure to the wellhead pressure. No viscous pressure loss occurs in the wellbore at the low pumping rates used. We also measure pressure directly in the hydrofrac interval with a Kuster gauge but do not present these data here because the records are not amenable to reproduction.

The reader should keep in mind that the shape of a given pressure-time record reflects both the in situ stress field and the procedures used during hydraulic fracturing. In the discussion below, we consider variations in the form of pressure-time histories resulting from changes in the stress field using the test procedures illustrated in Figure 1. We do not consider here the effects of high viscosity fluids (Nolte, 1979), fluid leakage in the drill stem or packer system, or other factors that could change the form of the
pressure-time history.

TYPES OF PRESSURE-TIME RECORDS

Hubbert and Willis (1957) and Haimson and Fairhurst (1967) derived the now classic equation

\[ P_b = 3S_h - S_H - P_p + T \]  

relating the breakdown pressure, or presumed pressure of hydraulic fracture formation, \( P_b \), to the horizontal principal stresses \( S_h \) and \( S_H \), the formation pore pressure, \( P_p \), and the tensile strength, \( T \). Bredehoeft et al. (1976) first suggested that \( S_H \) could be determined without knowledge of \( T \) by using the pressure necessary to open an existing hydraulic fracture, \( P_b(T = 0) \). Setting \( T = 0 \) in (1) we have

\[ P_b(T = 0) = 3S_h - S_H - P_p \]  

as an expression for the borehole pressure necessary to open an existing hydraulic fracture at the wellbore.

Using (1) and (2) we can define three basic types of pressure-time histories (all of which we have observed) based on the relative magnitudes of \( P_b \), \( P_b(T = 0) \), and \( S_h \).

Type 1: Both the breakdown and fracture opening pressures are greater than or equal to \( S_h \). That is, \( P_b > P_b(T = 0) \geq S_h \).
Rearranging (2), we observe that Type 1 pressure-time records occur when

\[
\frac{S_H + P_P}{S_h} < 2
\]  

(3)

Type 2: The breakdown pressure is greater than \(S_h\) but the fracture opening pressure is less. That is, \(P_b > S_H > P_b(T = 0)\). Rearranging (1) and (2) we observe that Type 2 pressure-time records occur when

\[
\frac{S_H + P_P}{S_h} > 2 > \frac{S_H + P_P - T}{S_h}
\]  

(4)

Type 3: Both breakdown and fracture opening pressures are less than \(S_h\). That is, \(S_h > P_b > P_b(T = 0)\). Rearranging (1) we observe that Type 3 pressure-time records occur when

\[
\frac{S_H + P_P - T}{S_h} > 2
\]  

(5)

Pressure-time histories that are illustrative of each of these three types are shown in Figure 2. These data are from wells drilled in the Mojave Desert and have been previously discussed by Zoback et al. (1980).

To provide better physical insight into how the relative magnitudes of the principal stresses result in these three different types of pressure-time histories we calculate the normal stresses on a hydraulic fracture resulting from the elastic stress concentration surrounding a cylindrical borehole (after Hubbert and Willis, 1957). Expressed in polar coordinates, with the center of
the borehole at the origin, the circumferential effective stress $\sigma_\theta$ resulting from a farfield effective stress $\sigma_A$ in a plane perpendicular to the borehole axis is given by

$$\sigma_\theta = \frac{\sigma_A}{2} \left[ 1 + \frac{a^2}{r^2} \right] - \frac{\sigma_A}{2} \left[ 1 + 3 \frac{a^4}{r^4} \right] \cos 2\theta$$

(6)

where $a$ is the borehole radius, $r$ is distance from the center of the hole, and $\theta$ is measured from the direction of applied stress. Decomposing effective stresses into matrix stress and pore pressure components gives

$$\sigma_\theta = S_\theta - P_p$$

$$\sigma_A = S_A - P_p,$$

and superimposing the circumferential stresses resulting from two orthogonal principal stresses, $S_H$ and $S_h$, we obtain an expression for the normal stress $S_N$ acting on a hydraulic fracture:

$$S_N = \left[ \frac{S_H + S_h}{2} - P_p \right] \left[ 1 + \frac{a^2}{r^2} \right] - \left[ \frac{S_H - S_h}{2} \right] \left[ 1 + 3 \frac{a^4}{r^4} \right] + P_p$$

(7)

where the hydraulic fracture is assumed to be planar and perpendicular to $S_h$. Notice that (7) reduces to (2) at $r=a$ as expected.

Using (7) the normal stresses acting on the hydraulic fractures corresponding to the pressure-time histories shown in Figure 2 were computed and are shown in Figure 3. Two important features of Figure 3 should be noted. First, the normal stress on the hydraulic fractures rapidly approaches $S_h$ as the fracture propagates away from the borehole. Second, the stress concentrations corresponding to these three types are quite similar except for that
portion of the hydraulic fracture adjacent to the borehole. The similarity in $S_N$ is important because it requires a carefully controlled hydraulic fracturing experiment (e.g., through constant flow rates, as discussed below) to correctly identify the hydraulic fractures of Types 2 and 3 and thus to accurately determine the fracture opening pressures.

As can be seen from (3), (4), and (5) the transition of hydraulic fractures from Type 1 to Types 2 and 3 corresponds to an increase in the magnitude of $S_H + P_p$ relative to $S_H$. The transition from Type 2 to Type 3 hydraulic fractures, however, corresponds to either a further increase in the ratio $(S_H + P_p)/S_H$, a decrease in $T$, or some combination of both of these effects. Type 1 hydraulic fractures are typical of situations in which the horizontal stress difference is low, while Types 2 and 3 hydraulic fractures are typical of regimes exhibiting a high horizontal stress difference. The transition between these types can occur within a single well. Figure 4 shows the horizontal principal stresses, together with the maximum shear stress (proportional to the horizontal stress difference), for two wells drilled at the same site in the western Mojave Desert (see Zoback et al., 1980). The lower seven of these measurements were made in the Crystallaire well and the pressure-time records from these tests are shown in Figure 5. In the Crystallaire well, a transition occurs from a Type 1 test at 266 m to Type 2 tests at 338 and 561 m, to Type 3 tests at 681 and 751 m, and back to Type 2 tests at 786 and 849 m. In the lower four tests in Figure 5, the peak pressure attained on the first cycle is not significantly higher than that attained on the subsequent cycles. Tests like these, which do not exhibit "classical" breakdown behavior, might be misinterpreted as representing the opening of a preexisting fracture. However, carefully
controlled test procedures (which are discussed below) confirm our interpretation of hydraulic fracture formation on the first cycle and reopening of that fracture on subsequent cycles. Furthermore, evidence that hydraulic fractures were, in fact, generated in these tests is provided by the excellent impression packer result obtained for the fracture at 786 m (Fig. 6).

METHODS FOR DETERMINING $S_h$ AND $S_H$

DETERMINATION OF $S_h$

In nearly all the hydraulic fractures that we have analyzed to date the well bore pressure immediately after pumping has stopped (the instantaneous shut-in pressure, or ISIP) is observed to decrease slightly as fractures are propagated. This phenomenon has also been reported by other investigators (see for example Gronseth and Kry, 1982; Enever and Wooltorton, 1982). Figures 7 and 8 show this decrease in ISIP with total pumped volume for two wells drilled near Monticello Reservoir in South Carolina (see Zoback and Hickman, 1982). As shown, the decrease in the ISIP after about 50 liters have been pumped, and the fracture is extended away from the well bore, is usually quite small.

In addition to the decrease in ISIP with volume, we also customarily observe a decrease in the difference between the downhole pumping pressure (measured immediately before shut-in) and the ISIP. (This pumping pressure is measured downhole and it is not affected by the pressure gradient in the surface hose referred to above.) A plot of the difference between the downhole pumping pressure and the ISIP against the pumped volume, together with a plot of the ISIP against pumped volume, are shown in Figure 9(a) and (b), respectively, for three tests from a 600-m-deep well drilled at Hi Vista.
in the western Mojave Desert (see Hickman et al., 1981). Figure 9(b) shows a decrease in the ISIP with pumped volume similar to that seen in the two Monticello wells. In Figure 9(a) we see that the pumping pressure decreases in a manner similar to, but more rapidly than, the ISIP, such that the difference between the pumping pressure and the ISIP decreases rapidly with time and approaches zero for large pumped volumes. Notice that the intentional reductions in flow rate at the end of the 227- and 539-m fractures prior to shut-in caused further reductions in the magnitude of the downhole pumping pressure relative to the ISIP but resulted in no significant reductions in the ISIP. The importance of this will be discussed later.

Before discussing the significance of these observations let us look briefly at the relationships between \( S_h \), the downhole pumping pressure, and the ISIP. Once the hydraulic fracture has propagated beyond several borehole radii away from the well, the downhole pumping pressure is given by the following expression:

\[
P_{\text{pump}} = S_h + P_V + P_K
\]  

(8)

where \( P_V \) is the viscous pressure loss in the hydraulic fracture during pumping, and \( P_K \) is a small increment of pressure in excess of \( S_h \) necessary to propagate the fracture. Soon after the hydraulic fracture initiates the stress intensity factor at the fracture tip becomes so large (compared to the critical stress intensity factor or fracture toughness) that \( P_K \ll S_h \) and \( P_K \) may be neglected (Zoback and Pollard, 1978). Similarly, when pumping is
stopped, the ISIP is given by

\[
\text{ISIP} = S_h + P_V' + P_K
\]

where \( P_V' \) is the residual viscous pressure loss in the hydrofracture near the borehole immediately after shut-in and

\[
P_V > P_V' > 0
\]

Assuming laminar flow, one can use a parallel plate flow law to gain insight into the functional forms of \( P_V \) and \( P_V' \). For laminar fluid flow between parallel plates the relationship between the volumetric flow rate per fracture height, \( q \), the pressure gradient in the fracture, \( \frac{dP}{dx} \), and the fracture opening, \( d \), is given by Norton and Knapp (1977) and others as

\[
\frac{dP}{dx} = \frac{12\mu q}{d^3}
\]

where \( \mu \) is the dynamic viscosity and \( x \) is defined as positive in the direction of fracture propagation. Roughly speaking, this suggests that both \( P_V \) and \( P_V' \) are proportional to flow rate and inversely proportional to the cube of the hydraulic fracture aperture.

Zoback and Pollard (1978) examined the relationship between the aperture of a hypothetical two-dimensional hydraulic fracture at the borehole wall, or the center wall displacement, and the hydraulic fracture length, \( L \), for the two limiting cases illustrated in Figure 10. In case 1 the fluid pressure
acts only at the center of the fracture (i.e., in the borehole) and acts, in
effect, like a point load. In this case the center wall displacement $D$ is
invariant with fracture length. In case 2 the fluid pressure acts uniformly
throughout the entire length of the fracture and $D$ increases linearly with
fracture length. Obviously, the situation in an actual hydraulic fracture
lies somewhere between these two extremes, so that the center wall
displacement increases with fracture length but in some manner more complex
than the linear case shown.

With these considerations in mind, we can examine what happens to $P_Y$
and $P'_Y$ on successive pumping cycles. From Figure 10 we have seen that,
as pumping into the hydraulic fracture continues and $L$ increases, there will
be a corresponding increase in $D$. Due to the extreme sensitivity of the
pressure gradient in the hydraulic fracture to changes in $D$ indicated by (11),
there are very rapid decreases of the pressure gradient terms $P_Y$ and $P'_Y$.
Thus, with pumped volume (i.e., with increases in $L$ and $D$),

$$P_Y ightarrow P'_Y ightarrow 0$$

(12)

and, from (8), (9), and (10)

$$P_{pump} ightarrow \text{ISIP} ightarrow S_h$$

(13)

This result explains why the ISIP decreases with pumped volume and approaches
the correct value for $S_h$ and why $(P_{pump} - \text{ISIP})$ tends to zero in Figure
9. It also explains the observation in Figure 9 that variations in the flow
rate prior to shut-in (during the later cycles of the 227- and 539-m hydraulic
fractures) resulted in variations in the pumping pressure but not the ISIP; \( P'_v \) had become negligibly small so that ISIP = \( S_h \), while \( P_v \) had not become negligible at the flow rates being used. In this regard, it is interesting to note that in a few cases the pressure gradient term during pumping, \( P_y \), is also of negligible magnitude. This was the case in the 849-m hydraulic fracture at the Crystallaire well (see Fig. 4). In the final cycle of this test the flow rate was increased by approximately 30 with no discernible resultant change in the downhole pumping pressure. Thus, in this case, both \( P_y \) and \( P'_y \) are negligibly small.

In practice, we attempt to measure \( S_h \) using (13). We pump at relatively low flow rates and with a low viscosity fluid (water) to minimize \( P_v \) and \( P'_v \), and we repeatedly pressurize the fracture to insure ISIP values that decrease slowly with pumped volume to an asymptotic value representing \( S_h \).

**DETERMINATION OF \( S_h \)**

After Bredehoeft et al. (1976), we standardly use (2) and \( P_b(T=0) \), or the pressure at which the already-formed fracture opens during repressurization, to determine \( S_h \). That is, we assume

\[
T = P_b - P_b(T=0).
\]  

This approach has two major advantages over the use of laboratory-determined tensile strengths and first-cycle breakdown pressures. First, due to the observed scale dependence of tensile strengths (see Haimson and Rummel, 1982; Ratigan, 1982) there can be considerable uncertainty involved in the extrapolation of laboratory-determined tensile strengths to in situ
conditions. Second, there can be considerable variation in the tensile strength of any rock unit in a given well. Thus, a sufficient number of laboratory-determined values for $T$ from throughout the tested intervals is necessary or considerable error could result in the values determined for $S_H$. One could, of course, obtain and test core samples from all the intervals in a well that were to be hydraulically fractured and use a separate value of $T$ for each interval. Such an extensive coring operation, however, would be unreasonably expensive in most cases.

In theory, the use of secondary breakdown pressures for determination of $S_H$ represents a considerable advancement over conventional methods. However, as discussed in the sections below, extreme care must be taken with the test procedures to determine fracturing opening pressures accurately, and steps must be taken to minimize the effects of processes that could alter the stress concentration around the borehole and thereby negate the use of fracture opening pressures and (2).

Procedure - To determine the fracture opening pressures, we carefully pump at a constant rate, and choose as $P_b(T=0)$ the pressure at which the initial borehole pressurization rate in the later cycles deviates from that established in the first cycle prior to breakdown. This represents the pressure at which the already-formed fracture opens at the borehole to accept fluid. Figure 11 shows how this method was used to obtain the fracture opening pressure for the 787-m hydraulic fracture in the Crystallaire well. It is critical that a constant flow rate be maintained throughout each hydraulic fracturing test, particularly for Type 2 and 3 hydraulic fractures, so that this pressurization rate comparison can be made. In cases where a constant flow rate cannot be maintained from cycle to cycle but can be
maintained on any given cycle, the fracture opening pressure may be approximated as the pressure at which the pressurization curves deviate from a straight line since the borehole pressurization curves prior to breakdown or fracture opening should be linear. Notice that this technique would also have yielded a correct value for the fracture opening pressure in the example shown in Figure 11.

The fracture opening pressures in a given test are typically observed to decrease with successive cycles. This behavior is illustrated in Figure 12 where the initial pressurization curves for the seven cycles from the hydraulic fracture at 97 m in the Monticello 2 well are superimposed. The same behavior is seen in Figures 13 and 14, where the apparent tensile strengths obtained using (14) for tests made in Monticello wells 1 and 2 are plotted against total pumped volume. As can be seen in these figures, the apparent tensile strength sometimes increases abruptly between the second and third pressurization cycles and then continues to increase slowly with pumped volume.

In the sections below we first discuss why the apparent tensile strength sometimes increases abruptly between cycles 2 and 3, and then we discuss reasons for the gradual change with pumping on the subsequent cycles.

Incomplete Breakdown - Ideally, once the hydraulic fracture is formed on the first cycle, there should not be any large changes between the fracture opening pressures on subsequent cycles. However, as mentioned above, on some tests a significant change in fracture opening pressure occurs even between the second and third cycles. We believe that this is due to incomplete breakdown on the first cycle (i.e., T is not reduced to zero) because the well was shut-in immediately after breakdown and before significant fracture
extension could take place (see Fig. 15). Examples of incomplete breakdown are shown at 128 m in Figure 14 and at 961 m in Figure 13. As shown in Figures 13 and 14 there is, in general, very little change in the apparent T between the second and third cycles. At 128 m in Monticello 2 (Fig. 14), and to a lesser extent at 961 m in Monticello 1 (Fig. 13), however, there are significant changes. We think that interpretation of results similar to these in terms of incomplete breakdown is justified because this interpretation routinely yields consistent results. For example, as shown in Figure 15, the magnitude of $S_H$ at 97 and 128 m was interpreted to be quite similar using the third cycle fracture opening pressures even though the breakdown pressures were different due to the differences in tensile strength (Fig. 14). Had the second cycle fracture opening pressures been used, the values of $S_H$ at 97 and 128 m would have differed by about 80 bars even though the $S_h$ values are nearly identical. Moreover, the value of $S_H$ at 128 m would have been less than $S_h$ which is, of course, impossible. Fortunately, incomplete breakdown as dramatic as that seen at 128m in Monticello 2 occurs infrequently, but the extent to which it occurs in other tests is often difficult to judge. For this reason we routinely use third cycle fracture opening pressures to circumvent the possible influence of incomplete breakdowns on the determination of $S_H$. We do not use fracture opening pressures from subsequent cycles because of the small but gradual change in apparent T that accompanies further pumping (Figs. 13 and 14).

Fluid Diffusion - It is important to understand the reasons for the gradual decrease in fracture opening pressure (increase in apparent T) after the third pressurization cycle (see Figs. 13 and 14) even though the magnitude of this effect rarely exceeds 10-20 bars. One explanation for this decrease
might be the diffusion of fluid into the rock surrounding the wellbore. Because the fluid pressure in the wellbore is higher than the pore pressure in the rock surrounding the well during pumping and shut-in, fluid diffusion could raise $P_p$ and, from (2), lower $P_b(T = 0)$. It is difficult to compute the change in pore pressure around the wellbore during a series of pressurization cycles because of the pore pressure increase that occurs during pumping and shut in and the pore pressure decrease that occurs during the flow-back period between cycles. Nevertheless, one can gain insight into the temporal and spatial scales involved in this relatively complex fluid diffusion problem by considering the simple problem of the pore pressure perturbation resulting from a stepwise increase in borehole pressure. Following the solution to the analogous heat flow problem presented by Carslaw and Jaeger (1959, p. 335-336) the change in pore pressure can be estimated for a material having a hydraulic diffusivity of approximately 22 cm$^2$/sec (corresponding to a permeability of $10^{-6}$ Darcies and a porosity of $10^{-3}$; where the contribution to aquifer storage resulting from rock matrix compressibility is neglected). In this case, a stepwise increase in borehole pressure of $H_0$ will increase the pore pressure in the surrounding rock to $0.5 H_0$ at a distance of three borehole radii from the borehole wall in about five minutes. Thus, even when testing in low permeability materials, short pumping and shut-in times and adequate flow-back on the first few cycles is necessary to keep fluid diffusion from significantly affecting $P_p$.

**Fracture Infiltration** - If a hydraulic fracture was fairly permeable at borehole pressures less than $P_b(T = 0)$, fluid would leak into the hydraulic fracture in the early stages of borehole repressurization. The resulting increase in fluid pressure within the hydraulic fracture prior to fracture
opening would lower the fracture opening pressures and invalidate the use of (2) for determining $S_H$. Also, the rate at which fluid leaked into the fracture as the borehole was pressurized would, via Darcy's Law, be a function of borehole pressure. In this case, even with a constant flowrate, the borehole pressurization rate prior to fracture opening would decrease with increasing borehole pressure. We believe that this phenomenon may contribute to the moderate decreases in fracture opening pressures observed on the later cycles of some tests. The borehole pressurization rates prior to fracture opening in these tests become more and more non-linear as the tests progress, and the fracture opening pressures on the later cycles are correspondingly harder to determine. Perhaps this is because the fractures become propped open by rock fragments and other debris and are progressively more permeable after repeated pressurization cycles.

In tests such as those described above significant fluid diffusion and fracture infiltration prior to the third cycles would drastically lower fracture opening pressures and greatly affect both the $S_H$ and apparent $T$ values computed from (2) and (14), respectively. The internal consistency of $S_H$ values computed using fracture opening pressures (Zoback et al, 1980; Enever and Wooltorton, 1982; Haimson and Rummel, 1982; Tsukahara, 1982) suggests that fluid diffusion and fracture infiltration are generally not very important effects in the early cycles because the magnitude of the effect on $P_b(T = 0)$, and therefore on $S_H$, would probably be different for different tests in a given well.

Another argument can be used to demonstrate that fluid diffusion and fracture infiltration are probably not significant problems on the first few pressurization cycles. From (2), any error in $P_b(T = 0)$ resulting from
fluid diffusion or fracture infiltration results in an overestimate of $S_H$ by the same amount. Thus, the error in $P_b(T = 0)$ must, by definition, be less than $S_H - S_h$ and, in some tests, $S_H$ values are computed using $P_b(T = 0)$ which exceed $S_h$ by only a few bars. For example, in the stress measurements made in Monticello 2 at depths less than 210 m (Fig. 15), because of the similarity in magnitudes of $S_H$ and $S_h$, the error in the fracture opening pressure must be less than 10 bars. Interestingly, fracture infiltration does not appear to be a problem here despite the fact that the normal stresses acting across the fracture are relatively low due to the shallow depth of the measurements.

**SUMMARY**

Determination of the in situ principal stresses from hydraulic fracturing data is not as straightforward as simple theory would predict. The arguments presented above provide what we feel are sound physical explanations for some of the subtle complexities often observed in pressure-time records. Based on this, we have presented a scheme for the interpretation of hydraulic fracturing data using our standard field procedure of conducting repeated pressurization cycles of progressively longer duration and pumping at constant flow rates throughout a test. We present arguments suggesting that the stable instantaneous shut-in pressure attained after repeated pressurization cycles should be used as a measure of $S_h$. We have adopted this approach because it minimizes the effects of the viscous pressure loss in the hydraulic fracture on the instantaneous shut-in pressure. We also suggest that the fracture opening pressure observed in the third pressurization cycle is best to use for the determination of $S_H$. We do this to compensate for
the effect of incomplete breakdown on the first cycle, to minimize the effect of fluid diffusion into the rock surrounding the borehole during pumping and shut-in which might alter the stress concentration around the borehole, and to minimize the possible problem of fluid infiltration into the hydraulic fracture prior to fracture opening.

We have also identified three basic types of hydraulic fracturing pressure-time records which are related to the relative magnitudes of the horizontal principal stresses, the pore pressure, and the tensile strength. Identification of these types is important for two reasons: 1) it allows for the recognition and accurate determination of fracture opening pressures in cases where the fracture opening pressure is less than the minimum horizontal principal stress (i.e., Types 2 and 3), and 2) it recognizes hydraulic fractures in which the breakdown pressure is less than or equal to the minimum principal stress (i.e., Type 3) and which might otherwise have been erroneously interpreted as the opening of preexisting natural fractures.
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Zoback, M. D. and S. Hickman, In situ study of the physical mechanisms controlling induced seismicity at Monticello Reservoir, South Carolina, J.


FIGURE CAPTIONS

Figure 1. Surface pressure and flow records from a hydraulic fracture at 185 m in the Limekiln C well, drilled 4 km from the San Andreas fault in central California. Positive flow rate represents injection into the hydrofrac interval; negative flow rate is flow out of the hydrofrac when the surface pressure is vented. The breakdown, fracture opening, and instantaneous shut-in surface pressures (ISIP) are indicated on the record. The small pressure pulse after the fifth cycle results from "choking" the flow-back valve (from Zoback et al., 1980).

Figure 2. Surface pressure and flow records illustrating the three different types of hydraulic fracturing pressure-time histories. These examples are taken from tests conducted in the Mojave 1 and Crystallaire wells, drilled near the San Andreas fault in southern California. These types are defined on the basis of the relative magnitudes of the breakdown and fracture opening pressures and the minimum horizontal principal stress, $S_h$. The calculated surface magnitude of the vertical stress, $S_v$, is shown for comparison. The unusual pressure and flow rate fluctuations following the increase in flow rate at the end of the test at 751 m in the Crystallaire well were caused by the malfunctioning of a high pressure valve at the surface.

Figure 3. The calculated normal stresses acting on hydraulic fractures corresponding to the examples shown in Figure 2. The equations from which these calculations were made are presented in the text and assume the elastic concentration of stress around a
cylindrical borehole that is perpendicular to the two horizontal principal stresses. Notice that the calculated normal stresses rapidly approach the magnitude of the minimum horizontal principal stress, $S_h$, as the fracture propagates away from the borehole, and that the stress concentrations corresponding to these three types are quite similar except for that portion of the hydraulic fracture within a few inches of the borehole.

Figure 4. The horizontal principal stresses and shear stress (resolved onto a plane parallel to the San Andreas fault) in the Crystallaire well. The measurements at depths of 149, 167, and 230 m were made in the Mojave 2 well which is at the same location (from Zoback et al., 1980).

Figure 5. Surface pressure and flow records from hydraulic fractures conducted in the Crystallaire well illustrating the transition with depth of hydraulic fractures from Type 1 through Type 3.

Figure 6. Photographs of impression packer from the hydraulic fracture at 787 m in the Crystallaire well and interpretive line drawing. The line drawing is oriented with respect to magnetic north (from Zoback et al., 1980).

Figure 7. The instantaneous shut-in pressure (ISIP) plotted against cumulative pumped volume for four hydraulic fractures in the Monticello 1 well near Columbia, South Carolina. Notice that the decrease in ISIP with pumped volume, which can be considerable at first, is usually quite small after about 50 liters have been pumped.

Figure 8. The instantaneous shut-in pressure plotted against cumulative pumped volume for four hydraulic fractures in the Monticello 2
well near Columbia, South Carolina. Notice the decrease in ISIP with pumped volume, which can be considerable at first, is usually quite small after about 50 liters have been pumped.

Figure 9. Data from three hydraulic fractures in the Hi Vista well, drilled 32 km from the San Andreas fault in southern California. Shown are a) the difference between the downhole pumping pressure at the end of each cycle and the ISIP against cumulative pumped volume, and b) the ISIP against cumulative pumped volume. Notice that the intentional reductions in flow rate during the later cycles of the tests at 227 and 539 m resulted in considerable reductions in the magnitude of the pumping pressure relative to the ISIP, but caused no significant reductions in the ISIP. Both the pumping pressure and the ISIP are observed to decrease with pumped volume in a manner similar to that seen in Figures 7 and 8.

Figure 10. Center wall displacement as a function of fracture length schematically shown for a two-dimensional fracture with constant internal pressure \( P \) applied over a short central portion of length \( r \) where \( r \ll L \) (case 1) or over the entire fracture length (case 2) (from Zoback and Pollard, 1978).

Figure 11. The beginning of the first and second pressurization cycles from the hydraulic fracture at 787 m in the Crystallaire well. As the flow rate was nearly constant during pressurization and the same on both cycles, the deviation of the pressure buildup curve from a constant rate of pressurization is diagnostic of fracture formation on the first cycle and fracture opening on subsequent cycles (from Zoback et al., 1980).
Figure 12. The beginning of all seven cycles from the hydraulic fracture at 97 m in the Monticello 2 well showing the decrease in fracture opening pressure with each cycle. As in Figure 11, the deviation of the pressure buildup curve from a constant pressurization rate is indicative of fracture formation on the first cycle and fracture opening on subsequent cycles.

Figure 13. Apparent tensile strength, defined as the difference between the breakdown pressure during the first cycle and the fracture opening pressure on subsequent cycles, as a function of cumulative pumped volume for the four hydraulic fractures from the Monticello 1 well. The numbers next to each data point indicate the cycle from which that apparent tensile strength was determined.

Figure 14. Apparent tensile strength, defined as for Figure 13, as a function of cumulative pumped volume for four hydraulic fractures from the Monticello 2 well. The numbers next to each data point indicate the cycle from which that apparent tensile strength was determined. The large difference between the apparent tensile strengths shown for the second and third cycles from the hydraulic fracture at 128 m indicates incomplete breakdown on the first cycle.

Figure 15. Hydraulic fracturing stress measurements as a function of depth in the Monticello 2 well. Dots indicate the magnitude of the least horizontal principal stress, S_h, and the triangles indicate the magnitude of the greatest horizontal principal stress, S_H (from Zoback and Hickman, 1982).
TYPE 1: FRAC OPEN ≥ $S_h$  e.g. Mojave 1: 185 m

TYPE 2: FRAC OPEN < $S_h$  e.g. Crystallaire: 338 m

TYPE 3: FRAC OPEN < $S_h$, BREAKDOWN ≤ $S_h$  e.g. Crystallaire: 751 m

Figure 2
TYPE 1: FRAC OPEN ≥ $S_h$

e.g. Mojave 1: 185 m
Frac Open = 73 bars
$S_h = 56$ bars

TYPE 2: FRAC OPEN < $S_h$

e.g. Crystallaire: 338 m
Frac Open = 63 bars
$S_h = 74$ bars

TYPE 3: FRAC OPEN < $S_h$
BREAKDOWN ≤ $S_h$

e.g. Crystallaire: 751 m
Frac Open = 133 bars
Breakdown = 182 bars
$S_h = 188$ bars

Figure 3
Figure 4

STRESS, BARS

SHEAR STRESS, BARS

DEPTH, METERS

LITHOSTAT
Figure 6
Figure 9
CENTER WALL DISPLACEMENT

CASE 1

CASE 2

Figure 10
Figure 12

Surface Pressure, Bars

Time

Breakdown

1st Cycle

Frac Open

7th Cycle

0 20 sec
MONTICELLO 2

TENSILE STRENGTH, BARS

PUMPED VOLUME, LITERS

Figure 14
Figure 15

STRESS, BARS

DEPTH, KM

LITHOSTAT (ρ = 2.7 g/cm³)

T.D.
INSTANTANEOUS SHUT IN PRESSURE
AND ITS RELATIONSHIP TO THE MINIMUM IN SITU STRESS

BY

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ABSTRACT

In situ stress determinations by hydraulic fracturing require an unambiguous method with which to identify the instantaneous shut in pressure. Results suggest that for low flow rate hydraulic fracturing (<50 l min⁻¹) the instantaneous shut in pressure should be equated with the pressure at the inflection point in the pressure-time record after shut in. Low volume, low rate cyclic pressurization is recommended. The best estimate of the minimum in situ stress is often times the minimum value of the instantaneous shut in pressure after a number of pressurization cycles.

These methods were used to determine minimum in situ stresses in a potential hydrocarbon zone and its bounding formations and thereby assess the containment potential for hydraulic stimulation. A favorable 6 MPa stress contrast was observed. A simplistic prediction of fracture height extension into the bounding formation agreed with the fracture height determined by radioactive logging.
INTRODUCTION

In situ stress determinations by hydraulic fracturing rely upon the assumption that the instantaneous shut in pressure is equal to the stress acting perpendicular to the plane of the induced fracture.

While this assumption can indeed be justified, the examination of many pressure-time records obtained from stress determinations made in the United States and Canada, as well as pressure-time records obtained from controlled laboratory and field experiments, reveals that the instantaneous shut in pressure obtained from multiple pressurizations of a zone do not always have a unique value. It was also found that the shut in pressure from the first pressurization cycle of an interval in many cases significantly overestimates the stress acting perpendicular to the fracture plane.

In principle, the determination of the instantaneous shut in pressure and hence the minimum stress is relatively straightforward. However, in actual practice, this determination can be a highly subjective process. No standardized method exists as to how the instantaneous shut in pressure should be determined. In cases where multiple pressurizations of a zone produce multiple values of the instantaneous shut in pressure, there are no guidelines as to which value should be taken as being the best estimate of the minimum stress.

In most cases, errors in the estimation of the minimum horizontal in situ stress by a few MegaPascals present few, if any, problems for the successful application of the data. However, for one application within the petroleum industry, that of hydraulic fracture containment, errors of a few MegaPascals in the estimation of the minimum in situ stresses acting in the pay zone and in potential barriers can have undesirable economic consequences.

Several investigators have shown theoretically (Simonson et al. (1978), Van Eckelen (1980)) and experimentally, (Warpinski et al. (1980), Teufel and Clark (1981)) that differences in the minimum horizontal in situ stresses between a productive formation and potential barriers are sufficient to inhibit the vertical growth of hydraulic fractures. It has been suggested that the magnitude of this stress difference can be as low as 2-3 MPa, (Warpinski et al. (1981)). Research projects by Esso Resources Canada Limited also suggest that stress contrasts can indeed provide containment for hydraulic fractures.

Operationally, the presence of a few MegaPascal stress contrast between formations is a necessary but not sufficient condition for hydraulic fracture containment. There is typically enough hydraulic power on location during frac treatments to easily overcome a 2-3 MPa stress barrier.
One way to optimize fracturing treatments by maximizing productive fracture area for the volume of fluid pumped is to ensure that hydraulic fractures are effectively contained within the formation being treated. This requires that accurate determinations be made of the minimum in situ stresses acting in the producing formation and in potential barriers and when feasible, to limit the bottomhole treating pressure to those values.

Errors in estimating the minimum stresses in either the producing formation or the potential barriers could easily result in overpressurization of the fracture and undesirable vertical fracture extension or an inefficient fracturing treatment.

**Determination of the Instantaneous Shut In Pressure.**

There is no consensus amongst investigators as to how the instantaneous shut in pressure should be determined so that it provides a reliable estimate of the in situ stress. Hence its determination is somewhat more subjective than is desirable for applications which require accurate values of the minimum in situ stress.

Experience reported in this paper suggests that for stress determinations by low flow rate hydraulic fracturing (<50 l min⁻¹) the instantaneous shut in pressure should be equated with the pressure at the inflection point in the pressure-time record after shut in.

Data was obtained by hydraulically fracturing a 38.10 cm x 38.10 cm x 38.10 cm cube of Charcoal Gray Granite, loaded in bi-axial compression. Stresses were applied using pairs of thin, stainless steel flatjacks. Flatjack pressure was kept constant during each test. Before testing, the specimen was placed in a precisely machined mold and cast in Hydrostone to ensure that opposite and adjacent faces of the cube were parallel and perpendicular to each other, respectively.

Specimen geometry is shown in Figure 1. A vertical borehole, 2.54 cm in diameter, was drilled through the center of the cube and sealed by cementing steel plugs in the top and bottom of the hole with structural epoxy. The interval was pressurized through the top plug. Prior to testing a fracture was propagated from the center hole into two fracture arrest holes located near the edges of the specimen. This was done so that multiple runs could be made on a single block. The internal stress distribution in the specimen containing the fracture arrest holes was modelled using the Displacement-Discontinuity Method. It was found that the fracture arrest holes had no significant influence on the stress distribution in the specimen at distances less than approximately 2.5 borehole diameters.
Two pressure-time records from these tests are shown in Figure 2. A simple graphical technique was used to determine the inflection point and hence the instantaneous shut in pressure. The construction consists of drawing a tangent line to the pressure-time record immediately after shut in. The pressure at which the pressure-time record departs from the tangent line was defined as the instantaneous shut in pressure.

The tangent lines used to determine the instantaneous shut in pressures, along with the stress which was applied perpendicular to the fracture plane are shown in this figure. Tangent lines $t_1$, correspond to the initial portion of the post shut in pressure-time record. Tangent lines $t_2$, corresponds to the slope of the pressure-time record immediately following the deviation of the pressure-time record from tangent lines $t_1$. The intersection of the two tangents is the instantaneous shut in pressure. The determination of a unique instantaneous shut in pressure by methods other than the identification of the inflection point is difficult, if not impossible.

Figure 3 is a plot of the minimum stress determined from the instantaneous shut in pressure vs. the applied minimum stress. Tests were performed with applied minimum stresses ranging from 1.7 MPa to 12.1 MPa. The ratios of the maximum to minimum applied stresses ranged from 1.0 to 6.0. As can be seen from this figure, instantaneous shut in pressures, when defined as above, correspond quite well to the values of the applied minimum stress. The deviation of the instantaneous shut in pressures from the applied stresses is maximum for low applied stresses and for stress ratios greater than 2.0. This is most likely the result of experimental uncertainty caused by the pre-existing fracture not being closed at low stress levels or at high deviatoric stress levels.

These values were determined by re-opening a previously created fracture which was subjected to a variety of stress states. The fracture arrest holes were open to the atmosphere and very large pressure gradients existed along the fracture. These tests show that even under extreme conditions the instantaneous shut in pressure, as defined by the inflection point in the pressure-time record following shut in, can give reasonable estimates of the minimum stress.

Estimation of the Minimum In Situ Stress from Instantaneous Shut In Pressures.

In many cases pressure-time records obtained from stress determinations performed at depth in relatively high modulus rock types such as granite, limestone and sandstone show a decrease in the magnitude of the instantaneous shut in pressure with number of pressurization cycles. In these cases it is observed that the instantaneous shut in pressures measurably decrease after each pressurization cycle until after several cycles they approach a constant value. When this phenomenon is observed, which value of the instantaneous shut in pressure should be taken as being the best estimate of the minimum in situ stress?
Results reported here suggest that the minimum instantaneous shut in pressure provides the best estimate of the minimum in situ stress. Judgement is required since the possibility of bypassing a packer or accessing an adjacent lower stress zone exist.

Data was obtained from experiments performed in a mine in North Eastern Minnesota at a depth of approximately 550 meters below surface. This location was selected because it provided access to rock in a relatively high in situ stress field. The U.S. Bureau of Mines had recently performed an in situ stress determination by overcoring which was used for comparison.

Tests were performed in three NX diamond core holes drilled from a mine drift. Hole orientations were vertical, horizontal, and inclined by 45° from horizontal. Based on integrated density and the U.S.B.M. overcoring results, the overburden stress was 15.9 MPa. The minimum horizontal stress was 22.8 MPa. Inflatable straddle packers were used to seal off the intervals. Impression packers were used to determine that axial fractures (parallel to the wellbore axis) were formed at the wellbore. Based on this, the minimum stress normal to each wellbore was calculated from the overcoring results and this value was used as the appropriate stress to compare with that inferred from hydraulic fracturing.

Each of the intervals in these holes were repressurized several times. The instantaneous shut in pressures decreased after each cycle until after several cycles they stabilized. The instantaneous shut in pressures from the first pressurization cycles overestimated the appropriate stresses determined by overcoring, while those determined from latter cycles agreed quite favorably with the appropriate overcoring values. Percent differences between the appropriate overcoring stresses and hydraulic fracturing are given in Table 1.

The same phenomena of decreasing instantaneous shut in pressure with sequential pressurization cycles has been observed in deep wells. Figure 4 is a plot of the instantaneous shut in pressure vs. cycle number from a deep gas well in Northeastern British Columbia. Pressures were monitored using two surface pressure transducers and two precise bottomhole pressure transducers. Hydrostatic pressure was added to the surface values to obtain these results. There is considerable variation in the instantaneous shut in pressure with number of cycles, after four cycles they stabilize. The results from cycle 4 were taken as being the best estimate of the minimum stress.
TABLE 1
Comparison between minimum stresses determined by hydraulic fracturing and overcoring

<table>
<thead>
<tr>
<th>HOLE ORIENTATION</th>
<th>SECTION/m</th>
<th>PERCENT DIFFERENCE USING:</th>
<th>First Shut in Press.</th>
<th>Min Shut in Press.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal</td>
<td>6.4</td>
<td></td>
<td>66.8</td>
<td>3.2</td>
</tr>
<tr>
<td>Horizontal</td>
<td>7.6</td>
<td></td>
<td>38.6</td>
<td>7.7</td>
</tr>
<tr>
<td>45°</td>
<td>7.6</td>
<td></td>
<td>42.3</td>
<td>0</td>
</tr>
<tr>
<td>45°</td>
<td>11.6</td>
<td></td>
<td>50.5</td>
<td>19.1</td>
</tr>
<tr>
<td>Vertical</td>
<td>11.6</td>
<td></td>
<td>23.8</td>
<td>2.6</td>
</tr>
</tbody>
</table>

At a shallower depth in the same deep gas well in Northeastern British Columbia the opportunity arose to compare the results of an in situ stress determination by low volume, low rate (<50 l min⁻¹) hydraulic fracturing with the stresses inferred from the pressure decline following an acid stimulation procedure. The well was cased and perforated over an 8 m interval with a single line of 56 perforations.

An in situ stress determination by hydraulic fracturing was performed. The interval was pressurized seven times and the results were analyzed using the previously described methods. The results of the seven cycles are listed in Table 2. The minimum in situ stress was inferred to be 34.5 MPa.

Figure 5 shows the pressure-time records from the first two low volume cycles. Pressures are bottomhole values, surface and downhole pressure transducers were used to monitor the stress determination. As can be seen from this figure, there is a decrease of 1.2 MPa in the instantaneous shut in pressure between the two cycles.

Subsequent to the seven cycle stress determination, a 20 m³ acid treatment was given to the zone. In this operation 20 m³ of a completion fluid are pumped at a rate of 0.5 - 1 m³min⁻¹ down tubing which is isolated from the wellbore casing by a bottomhole packer. The acidic fluid is held under pressure for one or two hours and then allowed to flow back. During the time the fluid is held under pressure, pressure in the tubing can only be reduced by leak off of the fluid to the porous zone.
Figure 6 is the pressure record of the tubing and the casing at the end of the treatment and after shut in. Pressure is maintained on the casing to minimize stresses on the tubing and the downhole packer. The decrease in surface pressure of 7.8 MPa is the elimination of the friction pressure when the fluid stops moving. This pressure drop in the tubing resulted in a slight decrease in the casing pressure due to tubing contraction. Note that the instantaneous shut in pressure determined after this large treatment is 4.1 MPa higher than the stress determined previously, at 250 s the pressure was removed from the casing and corresponding pressure perturbation on the tubing pressure can be seen. After this time the only pressure changes are due to leak off of fluid in the tubing-fracture system to the porous rock. A distinct change in slope is evident at 900 s. This is the anticipated behaviour upon closure of an open fracture. Prior to closure, leak off can occur over the entire fracture face. Subsequent to closure, fluid leak off from the tubing can only occur in the wellbore vicinity with a corresponding decrease in leak off rate. The decrease in leak off rate is manifest as a decrease closed at 900 s and that the minimum principal in situ stress is 34.7 MPa, compared to 34.5 MPa determined by low rate, low volume cyclic hydraulic fracturing.

TABLE 2
Results of cyclic pressurization at a flow rate of 50 l min⁻¹ instantaneous shut in pressure; V, volume injected to interval from wellbore during cycle

<table>
<thead>
<tr>
<th>CYCLE</th>
<th>P MPa</th>
<th>V m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>35.9</td>
<td>0.052</td>
</tr>
<tr>
<td>2</td>
<td>34.7</td>
<td>0.072</td>
</tr>
<tr>
<td>3</td>
<td>34.3</td>
<td>0.063</td>
</tr>
<tr>
<td>4</td>
<td>35.0</td>
<td>0.074</td>
</tr>
<tr>
<td>5</td>
<td>34.9</td>
<td>0.166</td>
</tr>
<tr>
<td>6</td>
<td>35.2</td>
<td>0.119</td>
</tr>
<tr>
<td>7</td>
<td>35.4</td>
<td>0.110</td>
</tr>
</tbody>
</table>
Application to Hydraulic Fracture Containment.

Another series of low rate, low volume, cyclic hydraulic fracturing stress determinations were made in this well. The purpose of these tests was to investigate the relationship between hydraulic fracture containment and in situ stresses. Instantaneous shut in pressures were determined using the previously discussed method and the minimum in situ stress was determined using the method discussed in the preceding section when multiple instantaneous shut in pressures were observed. The results of these stress determinations are shown in Figure 7. As can be seen from this figure, the minimum stresses acting both above and below the hydrocarbon zone (2095 m) are greater than the stress in the pay zone. The containment potential for this zone is excellent.

A stimulation treatment was performed on the zone. The proppant was tagged with radioactive sand so that the propped fracture height could be determined at a later time. Bottom hole treating pressures were monitored during the treatment and are displayed in Figure 8. Analysis of these indicate that the fracture began accepting fluid at a bottomhole pressure of 34.7 MPa, which is slightly above the best estimate of the minimum stress in the pay zone. The shut in pressure drop at 40 min. of 6 MPa reflects perforation friction losses, subtracting this from the peak treating pressure yields a minimum pressure of about 4 MPa over the minimum stress in the zone.

By assuming a step increase in the minimum in situ stress from 34.5 MPa in the hydrocarbon zone to 40.7 MPa in the potential barrier above the zone, Equation (1), (Simonson et. al, 1978) was used to calculate the expected vertical extent of the fracture.

\[
P - \sigma_{H \min} = \frac{K_{IC}}{\sqrt{\pi L (1 + \varepsilon)}} + \frac{2(\sigma_b - \sigma_a)}{\pi \cos^{-1}\left(\frac{1}{1+\varepsilon}\right)}
\]

where:
- \(L\) = Formation thickness
- \(K_{IC}\) = Critical Stress Intensity Factor
- \(\sigma_b - \sigma_a\) = Stress Difference
- \(\varepsilon\) = Fractional distance fracture has propagated into the high stress region.
- \(P-\sigma_{H \min}\) = Excess pressure in fracture.

Based on the following parameters, Equation (1) was iteratively solved to determine what fracture height would result from the over pressurization \((P-\sigma_{H \min})\) of 4 MPa.
It was found that the vertical fracture extension based on these conditions should be approximately 10 m to 2080 m.

Figure 9 shows the pre and post treatment gamma logs from this zone. The radioactive sand shows up as increased radioactivity on the log. As seen on this figure, there is an increased amount of radioactivity in the perforated interval and for approximately 12 meters above the zone. It is difficult to determine the exact top of the propped fracture; however, it appears to be close to that predicted by equation 1.

CONCLUSIONS:

The instantaneous shut in pressure can be defined as the pressure after shut in at the inflection point in the pressure-time record. Operationally this point may often be determined by taking the pressure at the first decrease in magnitude of the slope of the pressure-time record subsequent to shut in.

Instantaneous shut in pressures are not necessarily reproduced by repeated pressurizations. The first shut in pressure in a series of pressurizations is often significantly higher than that obtained on subsequent cycles.

Controlled laboratory and field experiments demonstrate that the minimum in situ stress acting perpendicular to the wellbore axis can be determined by low volume, low rate, cyclic hydraulic fracturing. Judgement is required in evaluating observed variations in instantaneous shut in pressures and inferring the minimum principal stress. Often the best estimate of that stress is the lowest shut in pressure observed over a number of cycles.

For application to the prediction of hydraulic fracture containment for effective hydrocarbon recovery, stresses in adjacent intervals must be measured to an accuracy of the order of 1 MPa. Apparent containment of an induced hydraulic fracture was achieved with a stress contrast of 6 MPa over a 22 m interval.

ACKNOWLEDGEMENTS

The laboratory and mine experiments were performed by J. M. Gronseth in the Department of Civil & Mineral Engineering, Institute of Technology, University of Minnesota. This research was supported by the U.S. Department of the Interior, Geological Survey, under contract number 14-08-0001-16768. The support of the U.S.G.S. and the University of Minnesota is gratefully acknowledged.
The authors wish to thank the Well Services and Well Specialists groups of Esso Resources Canada Limited for their assistance in the fracture containment work. We also appreciate the permission granted by Esso Resources Canada Limited to present this work.

REFERENCES:


FIG 1 Specimen Geometry

- Fracture arrest hole
- Hole diameters: 25mm
- Dimension: 0.38 m
FIG 2 Pressure Time Records from Laboratory Tests
Showing Determinations of the Instantaneous
Shut In Pressure Using Tangent Lines.
Figure 2a, \( \sigma_2 = 8.6 \) MPa, Figure 2b, \( \sigma_2 = 10.3 \) MPa.
FIG 3 Results of Laboratory Tests
FIG 4 Variations in the Instantaneous Shut In Pressure with Cycling.

- **SURFACE PRESSURE**
- **TRANSDUCERS + HYDROSTATIC PRESSURE**
- **BOTTOMHOLE PRESSURE TRANSUDCERS**
FIG 5  Pressure-Time Records from Deep Gas Well Stress Determination
FIG 6 Pressure-Time Record from Acid Treatment

\( \sigma_{\text{min}} = 34.5 \, \text{MPa} \)

- FRICTION PRESSURE 17.2 MPa
- 38.8 MPa
- 34.7 MPa

SURFACE PRESSURE / MPa

TIME / ks
FRACTURING PRESSURE RECORD

TIME / min

BOTTOMHOLE TREATING PRESSURE / MPa
FIG 9 Comparison of Pre Frac and Post Frac Gamma Logs
HYDRAULIC FRACTURING TO ESTIMATE
MINIMUM STRESS AND ROCKMASS STABILITY
AT A PUMPED HYDRO PROJECT

by

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Workshop on Hydraulic Fracturing Stress Measurements
December 3-5, 1981
Monterey, California
Hydraulic fracturing stress measurements were performed in interbedded shales, siltstones and sandstones to determine minimum stress levels. The tests were performed beneath a steep hillside in four boreholes at depths ranging from 227 to 727 feet, along the approximate location of a planned pressure tunnel for a pumped hydro project, which will operate under 700 feet head of water. The minimum stress levels determine whether the rock mass will resist hydraulic splitting, in case of leakage from the concrete liner. Additional steel reinforcement would be needed if stress levels were lower than the pressure head. The values of $K_0$ (ratio of minimum horizontal to vertical total stresses) were lower than expected, averaging 0.60 for shale, 0.83 for siltstone, and 1.41 for sandstone. The proximity of the measurements to the surface and to the hillside suggest the influence of unloading in the results. The contrasts in unloading moduli between shale, siltstone and sandstone at relatively low stress explain the contrasting $K_0$ values, and are also consistent with the reversed behavior at depth, where shale barriers frequently exhibit higher minimum horizontal stress than the sandstone reservoir rocks.

Size effects induced by the step from measurement boreholes of 3 inches diameter to the tunnel of 35 feet diameter are addressed. The presence of numerous pre-existing joints at tunnel scale, which were largely avoided in the borehole test locations may induce hydraulic opening of these joints in tension, and also in shear. Permeability increases caused by dilatent shear can be dramatic. It is important that the minimum total normal stress acting across each joint set exceeds potential levels of water pressure.
INTRODUCTION

Terra Tek performed 25 hydraulic fracture in situ stress measurements in four exploratory NX (3 inches diameter) boreholes, at the Rocky Mountain Pumped Hydro Project, Georgia. Test depths varied from 227 to 727 feet. The purpose of the tests was to investigate the ratio of minimum horizontal stress to vertical stress (termed $K_0$) in rock units 12, 14, 15 and 16, which will be intersected by the horizontal pressure tunnel. These rock units consist of thin to medium bedded sandstone, shale, siltstone and massive shale respectively. They vary in thickness from approximately 50-80 feet (Figure 1). The original test plan of 20 tests was extended to include units 8, 7, 4 and 3, consisting of carbonaceous shale, a bedded shale, a thin bedded siltstone, and a massive siltstone, respectively, which will be crossed by the vertical pressure shaft.

The reason for this extension in the program was the unexpectedly low values of $K_0$ measured along the trace of the future pressure tunnel. Low values of $K_0$ in locations with minimal overburden signify the possibility of hydraulic splitting of the rock mass, unless special tunnel lining designs are adopted. The hydraulic fracturing stress measurement technique appears to be a particularly realistic test method for investigating the risk of large-scale hydraulic splitting of a rock mass, due to the relatively large area of each hydraulic test fracture, compared to the much smaller sphere of influence of most other rock mechanics test techniques.

TEST EQUIPMENT

The surface equipment consisted of two high-pressure (70 MPa), low volume (0-4 gallons/minute) air compressor actuated pumps, a flow meter, and a 70 MPa strain-gauge type pressure transducer. Strip chart recorders were used to record the pressure-time histories for each test cycle. Water levels were measured prior to fracturing using an electric resistance probe, for estimation of pore or joint water pressures.

The hydraulic fractures were developed by pressurizing 4 feet long intervals of open, uncased borehole between straddle packers. Each packer element was 4 feet long and 2 5/8" - 2 3/4" diameter when deflated. A bar-drop sub was used to open the interval, following packer inflation to 600 psi. Packer and interval pressures were monitored by the surface pressure transducer. Pressures at depth were estimated by adding the pressure head generated by the relevant depth of water filled tubing. Examples of two typical surface pressure records are reproduced in Figure 2. Table 1 summarizes the results of the tests. Fourteen of the twenty-five tests were successful. The remaining eleven were unsuccessful due to deteriorated open holes, poor packer sealing, etc. These are not listed in Table 1.

TEST INTERPRETATION

Both minimum and maximum horizontal stress levels were estimated. The maximum horizontal stress levels were approximated by taking the mean value of Bredehoeft, et al (1976) and Gronseth and Detournay (1979) formulations (equations 1 and 2). Carefully controlled comparisons of these two methods of interpretation with the results of USBM overcoring measurements suggest that Bredehoeft's method may overestimate stress by approximately the same degree that Gronseth's method may underestimate stress. The following formulations were utilized:
Table 1

Summary of Test Data and Interpretations, Holes H5, H6, H7 and H8

<table>
<thead>
<tr>
<th>Date</th>
<th>Test No.</th>
<th>Depth</th>
<th>Hole</th>
<th>Unit</th>
<th>Rock Type</th>
<th>U_w</th>
<th>( \sigma_v )</th>
<th>( \sigma_{H(min)} )</th>
<th>( K_0 )</th>
<th>F. of S.</th>
<th>( \sigma_{H(max)} )</th>
<th>( \sigma_{H(min)} )</th>
<th>( p_{b1} )</th>
<th>( p_{b2} )</th>
<th>( \sigma_t )</th>
</tr>
</thead>
<tbody>
<tr>
<td>8/17</td>
<td>1</td>
<td>470</td>
<td>H6</td>
<td>16</td>
<td>Massive Shaly Siltstone</td>
<td>191</td>
<td>540</td>
<td>284</td>
<td>0.52</td>
<td>0.85</td>
<td>372</td>
<td>1.98</td>
<td>954-299</td>
<td>655</td>
<td></td>
</tr>
<tr>
<td>8/18</td>
<td>2</td>
<td>458</td>
<td>H6</td>
<td>16</td>
<td>&quot;   &quot;</td>
<td>186</td>
<td>525</td>
<td>338</td>
<td>0.64</td>
<td>1.03</td>
<td>455</td>
<td>1.90</td>
<td>858-388</td>
<td>470</td>
<td></td>
</tr>
<tr>
<td>8/18</td>
<td>3</td>
<td>407</td>
<td>H6</td>
<td>15</td>
<td>Thin to Med. Bed. Shaly Siltstone</td>
<td>164</td>
<td>470</td>
<td>616</td>
<td>1.30</td>
<td>2.05</td>
<td>918</td>
<td>1.76</td>
<td>1236-816</td>
<td>420</td>
<td></td>
</tr>
<tr>
<td>8/19</td>
<td>6</td>
<td>319</td>
<td>H6</td>
<td>14</td>
<td>Thin to Med. Bed. Shale (+ SST. Lenses)</td>
<td>126</td>
<td>367</td>
<td>280</td>
<td>0.76</td>
<td>1.05</td>
<td>432</td>
<td>1.99</td>
<td>1098-318</td>
<td>780</td>
<td></td>
</tr>
<tr>
<td>8/19</td>
<td>7</td>
<td>252</td>
<td>H6</td>
<td>12</td>
<td>Thin to Med. Bed. Silty-Shale + SST.-Shale</td>
<td>97</td>
<td>283</td>
<td>469</td>
<td>1.63</td>
<td>1.97</td>
<td>863</td>
<td>2.05</td>
<td>849-474</td>
<td>375</td>
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<tr>
<td>8/21</td>
<td>11</td>
<td>290</td>
<td>H5</td>
<td>15</td>
<td>Thin to Med. Bed. Sandy Shale</td>
<td>0</td>
<td>332</td>
<td>233</td>
<td>0.70</td>
<td>0.96</td>
<td>359</td>
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<td>685</td>
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<tr>
<td>8/24</td>
<td>14</td>
<td>623</td>
<td>H8</td>
<td>15</td>
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<td>247</td>
<td>714</td>
<td>535</td>
<td>0.75</td>
<td>1.55</td>
<td>718</td>
<td>1.80</td>
<td>1110-660</td>
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<tr>
<td>8/24</td>
<td>15</td>
<td>607</td>
<td>H8</td>
<td>14</td>
<td>Thin Bed. Shale + SST. Lenses</td>
<td>240</td>
<td>695</td>
<td>364</td>
<td>0.52</td>
<td>1.08</td>
<td>485</td>
<td>1.99</td>
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<td>640</td>
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<tr>
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<td>16</td>
<td>583</td>
<td>H8</td>
<td>14</td>
<td>&quot;   &quot;</td>
<td>230</td>
<td>668</td>
<td>341</td>
<td>0.51</td>
<td>1.04</td>
<td>424</td>
<td>1.92</td>
<td>1033-383</td>
<td>650</td>
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<tr>
<td>8/31</td>
<td>21</td>
<td>713</td>
<td>H7</td>
<td>12</td>
<td>Well Cem. Thin Bed. Quartz Sandstone</td>
<td>263</td>
<td>817</td>
<td>366</td>
<td>0.44</td>
<td>1.10</td>
<td>388</td>
<td>1.78</td>
<td>1249-509</td>
<td>740</td>
<td></td>
</tr>
<tr>
<td>9/1</td>
<td>22</td>
<td>575</td>
<td>H7</td>
<td>8</td>
<td>Massive Carbonaceous Shale</td>
<td>204</td>
<td>657</td>
<td>309</td>
<td>0.47</td>
<td>1.13</td>
<td>324</td>
<td>1.71</td>
<td>709-419</td>
<td>290</td>
<td></td>
</tr>
<tr>
<td>9/1</td>
<td>23</td>
<td>522</td>
<td>H7</td>
<td>7</td>
<td>Thin to Med. Bed. Shale</td>
<td>181</td>
<td>593</td>
<td>294</td>
<td>0.49</td>
<td>1.17</td>
<td>354</td>
<td>1.82</td>
<td>686-386</td>
<td>300</td>
<td></td>
</tr>
<tr>
<td>9/2</td>
<td>25</td>
<td>275</td>
<td>H7</td>
<td>3</td>
<td>Massive Siltstone</td>
<td>74</td>
<td>315</td>
<td>274</td>
<td>0.87</td>
<td>1.90</td>
<td>429</td>
<td>1.84</td>
<td>480-200</td>
<td>280</td>
<td></td>
</tr>
</tbody>
</table>

\( \dagger \) = mean of 4 ft. interval

\( * \) = All stresses in psi
Figure 1. Measurements of $K_0$ in Boreholes H5, H6, H8 and H7, and estimates of the factor of safety (F.S.) against hydraulic splitting of the rockmass.
Figure 2. Two examples of breakdown, extension and shut-in records obtained from surface gauges. Both records were obtained in thin to medium bedded shale at the depths indicated. $K_0$ values were 0.76 and 0.52 respectively.
\[
\sigma_{H}^{\text{(max)}} = 3P_{s1} - P_{bn} - P_{0} \quad \text{--------- (1)}
\]
\[
\sigma_{H}^{\text{(max)}} = 3P_{sn} - P_{bn} - P_{0} \quad \text{--------- (2)}
\]

where \(P_{s1}\) = first shut-in pressure
\(P_{bn}\) = minimum breakdown or extension pressure
\(P_{sn}\) = minimum shut-in pressure
\(P_{0}\) = pore pressure

Implicit in these two methods is the assumption that the tensile strength of the intact rock is represented by the difference between the first and subsequent breakdown pressures. Values of \(P_{b1} - P_{b2}\) given in Table 1 indicate mean values of 673, 518 and 563 psi in units b1, b2, 14, 15 and 16, quite similar to mean values obtained from Brazilian tensile tests of the same units (660, 769, 746 psi) (Southern Company Services, 1981). The larger scale of a hydraulic fracturing test will presumably produce lower values of tensile strength than obtained in laboratory-scale tests.

Values of minimum horizontal stress \(\sigma_{h}^{\text{(min)}}\) and \(K_{0}\) listed in Table 1 are based on the mean of \(P_{s1}\) and \(P_{sn}\). In general, a gradual reduction of the shut-in pressure was observed over the four or five tests performed at each location. An average range of values of \(+6\%\) was obtained for the \(K_{0}\) values listed in the table. Total vertical stresses at each location were estimated from measured rock densities and depth. Calculations using each individual rock unit's density and thickness resulted in a mean value of 165.25 lbf/ft\(^3\). A conversion factor of 1.146 was therefore used to convert mean depth of interval (feet) to total vertical stress (psi).

The ratio \(K_{0}\) in Table 1 is a ratio of total stresses. Values of \(\sigma_{H}^{\text{(max)}}\) are also expressed as total stresses, by adding pore pressures to values obtained from equations 1 and 2. Values of pore pressure were estimated from observed water levels in each borehole prior to testing. These estimates are unlikely to be uniformly reliable in a sedimentary sequence with hillside drainage.

Figure 3 shows \(\sigma_{v}^{\text{H(min)}}\) and \(\sigma_{v}^{\text{H(max)}}\) plotted in terms of total stresses. It will be noted that low values of \(\sigma_{v}^{\text{H(min)}}\) are linked to relatively low values of \(\sigma_{v}^{\text{H(max)}}\) and vice versa. This is an artificial result caused by the formulation of equations 1 and 2. Since shut-in pressures are lower than extension pressures, the ratios of \(\sigma_{v}^{\text{H(max)}}/\sigma_{v}^{\text{H(min)}}\) will be limited to \(<2.0\) when expressed as total stresses. In the present project, values of \(\sigma_{v}^{\text{H(min)}}\) were the only data of direct relevance to hydraulic splitting of the rock mass, so the above limitation is not of concern.

FRACTURE ORIENTATION

The theoretical orientation of a hydraulic fracture is parallel with the direction of maximum horizontal stress, and perpendicular to the minimum horizontal stress. The shut-in pressure represents the equilibrium pressure between outward pumping and return flow towards the interval, under the influence of \(\sigma_{h}^{\text{(min)}}\). Quite a large area of fracture is involved, giving a good picture of
Figure 3. Hydraulic fracturing stress measurements from boreholes H5, H6, H7 and H8, plotted in terms of total stresses. The scatter bars result from taking the first shut-in pressure \( P_{SI} \) and the minimum \( P_{SN} \) when estimating \( \sigma_h(\text{min}) \) and \( \sigma_H(\text{max}) \).
the full-scale stress level, and a correspondingly reliable large-scale measure-
ment of the potential for hydraulic splitting of the rock mass in case of leakage
of the pressure tunnel.

The limited time made available for mobilization and field testing made it
impossible to obtain the necessary slim-hole impression packer equipment. Conse-
quently, fracture orientation was not measured on this occasion. Care was taken
to select test locations away from jointed parts of the core, in particular,
away from steeply inclined joints which might well parallel the principle stress
direction. Since few results indicated \( K_0 \) close to 1.0, it is not suspected
that sub-horizontal bedding was involved in the breakdowns.

TEST FAILURES

In certain instances the pressure records showed evidence of fracture of the
formation caused by the packer, and other cases of probable connection to permeable
lenses. A normal breakdown pressure would be reached, but continued pumping would
not maintain any extension pressure. For example, in hole H5, it was discovered
that the original water table 120 ft. below the collar was draining 190 ft. further
down the hole to the floor level of the exploratory drainage tunnel (see Figure 1).
In the upper part of the hole, the packers could be pulled up the hole without
deflating them from the standard 600 psi pressure. The severe erosion and weath-
ered state of the rock resulted in only one unambiguous result out of five tests
in this hole. Dubious packer seating was probably the chief problem.

DISCUSSION OF RESULTS

The test depths in the present stress investigation were limited to the range
227-727 feet, and each borehole was relatively close to a steep hillside. Low
values of \( K_0 \) were obtained, an experience also reported from equivalent near-
surface tests by Alheid et al (1982). In the present tests, even the intervals
furthest away from the slope showed low values of \( K_0 \) (0.44, 0.47, 0.49) in the
shale units.

Values of \( \sigma_h (\text{min}) \) in shale consistently lower than in the interbedded sand-
stones is the inverse of the situation usually measured at much greater depth in
reservoir rocks. Typically, shale barriers will exhibit values of \( \sigma_h (\text{min}) \) several
hundred to a thousand psi larger than \( \sigma_h (\text{min}) \) in the reservoir sandstones. The
key to this different behavior has been explained in two different ways. The
simple explanation is that shale has a lower deformation modulus than sandstone
and therefore attracts less stress. At great depth (i.e. 10,000 ft.) the shale is
assumed to behave in a less than elastic manner, and being unable to support as
large a stress difference as sandstone, therefore exhibits a higher value of
\( \sigma_h (\text{min}) \).

A second and possibly more viable explanation of the inversion is the differ-
ences in the complete load-unload stress-strain cycle of a typical shale and sand-
stone. As pointed out by Abou-Sayed (1982) inelastic deformation during the
burial and erosion history may have a significant influence on today's state of
stress. Most shales exhibit a marked hysteretic stress-strain loop, which pro-
duces a high unloading modulus when stress is reduced from a high value (deep
burial). Conversely, the shale will exhibit a very low unloading modulus when
stress is reduced (by erosion) from a low value. Provided the more elastic
stress-strain loop for sandstone lies between the above extreme values for shale,
then the inversion of \( \sigma_h (\text{min}) \) for shale and sandstone is explained.
An analogy might also be drawn between shale and heavily jointed rock. Extremely high unloading moduli at high stress and extremely low unloading moduli at low stress are a marked feature of stress-strain behavior of jointed rock (Barton and Lingle, 1982). Low values of $K_0$ in near-surface jointed rock masses are therefore to be expected, in addition to the above influence of rock type.

**IMPLICATIONS FOR DESIGN**

Prior to the stress measurements described, the most critical position for a pressure tunnel lining failure was thought to be the zone located roughly between holes H5 and H6, where the minimum overburden of 360 ft. is found (Figure 1). This translates into a (total) vertical stress of about 410 psi, giving (for $K_0 > 1.0$) a theoretical factor of safety of about 1.36 when compared with the pressure head of 303 psi generated by the nominal 700 feet head of water.

Figure 1 shows the values of $K_0$ measured in the different units in each of the four boreholes. Since lower values of $K_0$ may be acceptable where overburden depths are greater, the data has also been expressed in terms of a hypothetical factor of safety (F.S.) where:

$$F.S. = \frac{\sigma_H (min)}{(700-h)\gamma_w}$$

where $h$ = height above tunnel roof

$$\gamma_w = 62.4 \text{ lbf/ft}$$

In the case of stress measurements made in H7, parallel with the planned axis of the vertical shaft, the values of F.S. are calculated relative to the normal pool elevation of 1390 feet.

The values of F.S. shown in Figure 1 represent conservative estimates, since pressure losses that would result from high velocity flow through the rock mass surrounding a leaking concrete lining have been ignored. There is no reliable basis for estimating such losses. In essence the values of F.S. represent the capacities of the given rock units to resist opening of existing vertical joints. Since stress orientations were not measured on this occasion, it is unknown which of the two observed steeply dipping sets of joints in each unit will be governed by these limited factors of safety.

It is clear from the ratios $\sigma_H (max)/\sigma_H (min)$ listed in Table 1 that in most instances, only one steeply dipping set will be liable to hydraulic splitting should significant lining failure take place. If the minimum horizontal stress orientation were consistently parallel to the tunnel axis, then the values of $\sigma_H (max)/\sigma_H (min) > 1.0$ (all cases) would have a positive effect in reducing the likelihood of hydraulic splitting parallel with the tunnel axis. While this may be the case close to the hillside, the larger scale tectonic features of the area suggest that $\sigma_H (min)$ is probably oriented perpendicular to the tunnel at greater depth under the slope.

Steel reinforcement is normally used to distribute the circumferential strain experienced by a concrete pressure tunnel lining into a large number of fine longitudinal cracks, in preference to the two major cracks occasionally observed in failed, unreinforced linings. For optimum design of this expensive reinforcement, it is important that realistic distributions of rock stress and deformation moduli are obtained for numerical analyses. A key factor is the actual circumferential stress concentrated around the immediate perimeter of the tunnel, this being a function of both the far field stress and the modulus of the disturbed zone.
A design which ensures the development of only very fine cracks in the concrete lining also ensures large water pressure gradients through the lining, and a correspondingly reduced likelihood of tensile splitting or shear displacements along pre-existing joints in the surrounding rock mass.

BREAKDOWN BY SHEARING

A potential size effect is introduced when the results of hydraulic fracturing from 3 inch diameter boreholes is used for predicting the hydraulic splitting of a jointed rock mass from a 35 foot diameter pressure tunnel. In the second case, the presence of joints is unavoidable since the spacing of both the sub-vertical sets is considerably smaller than the tunnel diameter. If we make the conservative assumption that cracking of the concrete lining is possible, then penetration of water at high pressure into at least one of the joint sets is a possibility. If the joint set with the minimum value of total normal stress acting across it is not parallel with the principal stress direction, then a shear component is possible. Shear displacement resulting from anisotropic stress and reduced effective normal stress may be a valid "breakdown" mechanism.

Figure 4 indicates the considerable reduction in shear stiffness that occurs as effective normal stress reduces and block dimensions increase for two hypothetical joints of different strength and roughness (Barton, 1981). The potential reduction in shear resistance may be sufficient to generate shear displacement on an incipient or pre-existing tight joint. The examples in Figure 5 showing increase in permeability with shear for two scales of joint permeability tests indicate that more than an order of magnitude increase in permeability can occur for a shear displacement of no larger than 1 mm, especially when the effective normal stress is reduced by the penetration of water under pressure.

From the point of view of pressure tunnel lining design, it would therefore seem important to ensure that cracking is carefully controlled by suitable reinforcement in those locations where the total normal stress component acting across a joint set is little more than the potential joint water pressure. Unless the critical joint set is oriented perpendicular to $\sigma_h(\min)$, the resulting stress component (total normal stress) would be larger than $\sigma_h(\min)$.

Acknowledgements

The author is indebted to Georgia Power Company and Southern Company Services for permission to publicize this data, and to Ahmed Abou-Sayed for interesting discussions and practical advice.
Breakdown by shearing is facilitated by penetration of water under pressure, which increases the effective block size and reduces the effective normal stress. Constitutive models of joint behavior indicate that both the above effects will reduce shear stiffness considerably. Upper curves represent rough joints (JRC=15) in a strong rock (JCS=150 MPa), lower curves represent smooth joints (JRC=5) in a weaker rock (JCS=50 MPa). After Barton, 1981.
Figure 5. Hydro-mechanical constitutive models which couple shearing, dilation and permeability indicate that joint permeability may increase dramatically with moderate shear displacements. Breakdown by shearing would increase leakage progressively as effective normal stress was reduced by the increasing flow.


DO INSTANTANEOUS SHUT-IN Pressures Accurately
Represent the Minimum Principal Stress?

By

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ABSTRACT

Comparisons between overcoring and hydraulic fracturing stress measurements have indicated that shut-in pressure is a positive indicator of the in situ stress regime. Some of the features which preclude ready interpretation of shut-in curves are outlined. An interpretative technique based on a pressure vs log (t + Δt)/Δt plot is applied to a suite of data generated by laboratory hydraulic fracturing tests, and the implications are discussed.

INTRODUCTION

Hubbert and Willis (1957) stated, "Once a fracture has been started, the fluid penetrates the parting of the rocks and pressure is applied to the walls of the fracture. This reduces the stress concentration that previously existed in the vicinity of the wellbore and the pressure, ΔP, required to hold the fracture open in the case of a nonpenetrating fluid is then equal to the component of the undistorted stress field normal to the plane of the fracture. A pressure only slightly greater than this will extend the fracture indefinitely, providing it can be transmitted to the leading edge...

"The minimum down-the-hole injection pressure required to hold open and extend a fracture is therefore slightly in excess of the original undistorted regional stress normal to the plane of the fracture."

On this basis, instantaneous shut-in pressures are regularly used as indicators of the minimum in situ principal stresses (Kehle, 1964; von Schoenfeldt, 1970; Haimson, 1972; Riegiers, 1974; Bredehoeft et al., 1976; Zoback et al., 1977; McLennan, 1980).

And in fact, Gronseth and Detournay (1979) and Haimson (1981) (refer to Tables 1 and 2) have established the general trend of reliability of shut-in pressures through comparisons of hydraulic fracturing and overcoring measurements.

Consequently, it would seem that instantaneous shut-in pressures are "positive" indicators of in situ stress, provided they can be reliably ascertained from pressure-time records.

However, Figure 1 indicates an all too common pressure response, reflecting complicated transient behavior and making the selection of the proper shut-in pressure value rather subjective. Furthermore, there is an added uncertainty as to what in situ characteristic the shut-in pressure is reflecting for certain complicated pressure-time records.

†This is by necessity only a partial list.
FACTORS COMPLICATING READY INTERPRETATION OF
INSTANTANEOUS SHUT-IN PRESSURES

The guilty mechanisms masking shut-in pressures have been outlined elsewhere (Aamodt and Potter, 1978; Zoback et al., 1978; McLennan, 1980) and are briefly summarized here.

Leakoff

In permeable formations, the pressure decay behavior (Figure 1) reflects pressure loss by permeation, either through the matrix or via pre-existing discontinuities. For example, fluid can bleed into secondary fractures intersecting the "primary" fracture system being pressurized at the wellbore.

Equipment

It is likely that on any fracturing operation there will be some leakage in the mechanical pressurization system. The likelihood of this increases with depth as operating pressures become higher.

Packer Bypass

There are two possibilities. The first is an imperfect seating resulting in some leakage past the packers. The second is leakage via a fracture growing past one or both of the packers.

Fracture Inclination

If an inclined fracture is pressurized either due to inflation of an existing inclined discontinuity or the generation of an inclined fracture due to stress or structural control, the secondary principal stress calculated from the shut-in pressure may not be an indication of the correct principal stress (Mizuta and Kobayashi, 1980; McLennan, 1980).

Change in Fracture Orientation

As has been emphasized in the past, an initiated fracture is directed according to local conditions (either geological, stress related or mechanical, i.e., perforations). The propagating fracture will tend to adjust to an optimum orientation. Consequently, nonplanar features may be developed. The two circumstances where this may be especially critical are as follows.

1. Where significant geologic control is developed (extreme foliation, well-developed bedding, weak interbeds, discontinuities). In some cases, geological control may be more dominant than stress control.

2. In situations where $\sigma_v < \sigma_{\text{HMIN}}$ and rubber packers are used, there are strong indications (Roegiers and McLennan, 1981; Rummel, 1981) that the initial created fracture would be along the wellbore axis (depending to some extent on hole angle and stress conditions) and would reorient itself to a more favorable stress orientation with
adequate propagation. Consequently, shut-in behavior is going to be "anomalous."

In Situ Stress Heterogeneities

During fracture propagation, especially with moderate pumping volumes, there is a strong possibility for the fracture to extend into adjacent zones of different stress levels. These stress levels must be reflected in the shut-in response of the fracture both through the compliance and the stress levels in the regions intersected by the fracture.

Incomplete Fracture Closure

If shut-in pressure is viewed as a pressure level at which the fracture closure rate dramatically decreases, one must evaluate the consequences of fracture closure being prevented. Potential mechanisms for the prevention of complete fracture closure might be:

1. Inelastic deformation,
2. Slippage and asperity override in the plane of the fracture,
3. Slippage at the fracture boundaries (i.e., along an interface boundary), or
4. If any proppant is used in a fracture, complete closure can be prevented and stresses dependent on compliances and fracture width can be "locked into" the formation.

Stress Level

If the shut-in pressure is regarded as a pressure level reflecting a minimum pressure required to extend the fracture, the measured pressure will to some degree be in excess of the in situ principal stress. Rosepiler (1979) cited a figure of 200 psi as being representative of the pressure in excess of the minimum principal stress. The validity of this figure has not been established. Regardless, the thought-process is illustrated by looking at the simplistic Perkins and Kern pressure representation.

\[ P_{ext} = \sigma_{MIN} + \sqrt{\frac{\mu \gamma E}{2(1 - \nu^2)L}} \]

\[ P_{ext} = \text{Pressure to Extend Fracture} \]
\[ E = \text{Young's Modulus} \]
\[ \gamma = \text{Surface Energy} \]
\[ \nu = \text{Poisson's Ratio} \]

\[ ^* \text{It must be strongly indicated that this formula for a penny-shaped fracture of radius L is cited for illustrative purposes only.} \]
This relationship suggests that an excess equilibrium pressure will exist on shut-in but does emphasize that the excess decreases as the fracture is extended a relatively small distance. The implication is that the fracture may continue to extend a small distance on shut-in until an equilibrium pressure is reached at which additional extension is not possible. This pressure, as predicted from formulas of this nature, is slightly larger than the in situ stress acting across the fracture. This may in fact be one of the contributing factors to superior shut-in pressure readings with multiple pressure cycles.

Along these lines of stress concentration effects, one must ask whether the shut-in pressure represents the average stress level across the fracture and whether any near wellbore stress concentrations are still acting on the fracture.

Pore Pressure

Pore pressure may in fact be a consideration. For example, in deep, over-pressured wells in South Texas, the minimum principal stress has been consistently found to decrease with production of the wells and consequent drawdown causing local alteration of the stress field.

IMPLICATIONS

The implications are as follows.

A. Shut-in curves showing classic pressure drop followed by no decay should still be evaluated carefully to be certain of the phenomena being measured. For example, for measurements in an area of high lateral stress, do the final shut-in pressure cycles (Zoback and Pollard, 1978) reflect a representative value of the vertical stress component? If not, why? (influence of discontinuities, stress concentrations due to proximity to underground cavities, inclined fractures, combinations of vertical, horizontal and/or inclined fractures).

B. Anomalous shut-in curves, masking pressure effects either as a function of leakoff or fracture morphology effects, should ideally be used to one's advantage in identifying in situ conditions, rather than being discarded. This may involve more comprehensive curve analysis than has been often done in the past. This may include, for example, utilization of reservoir engineering concepts to a greater degree. An important step in this direction will be to define a characteristic of a typical curve which does in fact reflect the desired stress level. The techniques of multiple shut-in cycles seem to be one practical field procedure for defining a representative shut-in pressure (Zoback and Pollard, 1978; Gronseth and Detournay, 1979) and may be useful in defining the complete stress tensor under awkward stress conditions (i.e., shallow depths). The general procedure of repeated fracturing until stabilization probably ensures that the major influence of wellbore proximity is overcome. Other field techniques which may prove useful might be the use of more viscous fluids designed to reduce leakoff to a negligible quantity.
EXPERIMENTAL MEASUREMENTS

A series of laboratory hydraulic fracturing experiments is described below. These tests served to highlight the potentially complicated nature of the pressure decay behavior on shut-in along with reasonable mechanistic explanations for some of the encountered anomalies. The pressure curves were evaluated in several manners in order to ascertain if there were consistent characteristics of the decay curves which reflected one or more of the applied polyaxial stresses (refer to Figures 2 and 3).

The experimental program consisted of loading 0.2-m cubed samples in a polyaxial cell and fracturing (at a servo-controlled flow rate) 0.006-m wellbores (using a miniature straddle packer configuration). Three types of samples were tested:

A. Plexiglas (allowing visual mapping of induced fractures),
B. Medium-grained granite, and
C. Mortar blocks (purposely air-cured in order to develop an approximately orthogonal fracture system).

An initial interpretative phase involved examination of pressure-decay records plotted as pressure vs time. Some of the tests showed a leakoff behavior due to ultimate fracturing past the packers and leakoff through other fracture systems or to the edges of the blocks. Hence, it was felt that there were good candidates for evaluating potential techniques to disassociate fracture closing effects from those of permeation (primary or secondary).

ANALYSIS OF SHUT-IN CURVES

The first data reduction procedure was an analysis of the pressure vs time behavior on shut-in. The curved nature of these plots made it extremely difficult to readily correlate between any characteristic feature on the curve and the known stress applied across the fracture.

As a next step, it was recognized that for numerous years reservoir engineering used particular data manipulations to determine behavior patterns. In an attempt to apply similar manipulations to shut-in analysis, both log-log and semi-log plots of pressure, time, square root of time and ratio of \((t + \Delta t)/\Delta t\) (where \(t\) is the time of pressurization and \(\Delta t\) is the time since shut-in) were prepared. Such analysis was earlier suggested by Sun and Mongan (1974). Similar analysis is discussed by Aamodt (1981) and Doe et al. (1981).

A successful predictive method appeared to involve plotting pressure \(P\) vs log \((t + \Delta t)/\Delta t\). \(P\) vs log \(\Delta t\) curves provided the next most satisfactory results.

Table 3 indicates the results of such a predictive method. It must be stressed that there was little or no obvious indication on pressure-time plots of a distinctly representative pressure level. Figures 4 to 9 are
typical situations. As is shown, there is a pressure level (range) at which there is a slope change. This inflection point appears to correspond well with the in situ stress.

In many stress measurement programs, a higher stress value has been selected. For example, where the initial rapid pressure drop on shut-in decreases in rate, this point may correlate with a rapid (instantaneous) fracture closure which is followed by complete closure at a much lower rate. Medlin and Masse (1981) reported results supporting this philosophy stating:

"Fracture closure measurements after shut-in show that there are two well-defined periods of different behavior. Early closure is controlled by leakoff and fluid efficiency is a dominant factor. Later closure is controlled by residual strain and creep properties of the rock are dominant. A transition between these two modes is marked by a slope discontinuity in the P_0 decline curve. This discontinuity can be identified with the ISIP in field treatments or with an event near it in time. Our experimental results show that the ISIP is likely to be higher than the true earth closure stress with possible errors of 20% or more."

The possible implication is that some shut-in pressures picked to date may be too high. The stress measured may relate to closure stress discussed by Nolte (1979) and Nolte and Smith (1979), where the closure pressure is differentiated from the in situ stress. Also, as mentioned, Rosepiler (1979) asserted that the measured pressure is higher (= 200 psi is claimed) than the in situ stress acting across the fracture. The consequence is that determination of a pressure level from a P vs log (t + Δt)/Δt plot may be more indicative of the stress regime as opposed to being a function of the fracture characteristics.

Another feature very strongly suggested by this study was the influence of fracture orientation. Especially in cases where a large stress difference exists, even a slightly inclined fracture can record an anomalously high additional stress component. Figures 9 to 11 clearly show the manifestation of this in the laboratory experiments performed. As a result, as mentioned elsewhere (Aamodt and Potter, 1978), interpretative caution is essential.

A further observed phenomenon was that for relatively small stress levels acting across the fracture, fracture pressure decay:

A. Was not as rapid as for large stresses, and

B. Did not completely decay to the expected levels even after long periods of time (Figure 12).

This suggests nonrecoverable behavior on fracturing.

†On P vs Δt plots.
A final feature of interest, observed under certain circumstances, was a stress plateau in the pressure decay curve. Figures 13 to 16 indicate that this plateau may be strongly indicative of in situ stress conditions. The implication is that no irregularity of the pressure decay curve can be considered as being insignificant.

In complicated systems, such as where fracturing is at shallow depths or pre-existing fractures are reopened, shut-in curves may be indicative of unexpected situations. For example, particularly in a material with relatively high tensile strength, it may be more energetically feasible to re-open an existing fracture (if it intersects the wellbore) than to create a new, more favorably oriented fracture (refer to Figure 17). Under these circumstances, shut-in curve analysis reflects fairly well the stresses acting across the fracture rather than a current principal stress (Mizuta and Kobayashi, 1980). As a result, unconventional interpretation may be misleading. However, on the bright side, if horizontal and vertical fractures exist or are created, it may be possible to discern additional information from multiple points on the shut-in curve (refer to Figures 18 to 20).

**SUMMARY**

- It has been found that selection of a pressure level as an inflection point on a P vs log (t + Δt)/Δt plot provides representative predictions of the in situ stress level.

- Care must be taken to differentiate between the pressure level at which the fracture closure rate drastically declines and the stress acting across the fracture.

- Fracture inclination must be evaluated and considered wherever possible.

- In low stress regimes, stress levels determined from shut-in curves may be somewhat high due to nonrecoverable effects.

- It may be possible to determine more than one in situ stress from multiple points on the semi-log, shut-in curve.

- All features and irregularities of the pressure decay curves are significant.

**REFERENCES**


### TABLE 1

**COMPARISON BETWEEN THE MINIMUM STRESS DETERMINED BY OVERCORING AND BY HYDRAULIC FRACTURING**

<table>
<thead>
<tr>
<th>Hole</th>
<th>Section (ft)</th>
<th>First Instantaneous Shut-In Pressure</th>
<th>Minimum Instantaneous Shut-In Pressure</th>
<th>Percent Difference Using</th>
</tr>
</thead>
<tbody>
<tr>
<td>10105</td>
<td>21</td>
<td>+ 66.8</td>
<td>+ 32</td>
<td>+ 66.8</td>
</tr>
<tr>
<td>10105</td>
<td>25</td>
<td>+ 38.6</td>
<td>+ 7.7</td>
<td>+ 38.6</td>
</tr>
<tr>
<td>10106</td>
<td>25</td>
<td>+ 42.3</td>
<td>0</td>
<td>+ 42.3</td>
</tr>
<tr>
<td>10106</td>
<td>38</td>
<td>+ 50.5</td>
<td>+ 19.1</td>
<td>+ 50.5</td>
</tr>
<tr>
<td>10107</td>
<td>38</td>
<td>+ 23.8</td>
<td>+ 2.6</td>
<td>+ 23.8</td>
</tr>
</tbody>
</table>

†From Gronseth and Detournay (1979).

### TABLE 2

**COMPARISON BETWEEN THE MINIMUM HORIZONTAL PRINCIPAL STRESS MEASURED BY HYDRAULIC FRACTURING AND BY OVERCORING**

<table>
<thead>
<tr>
<th>Location</th>
<th>Depth (m)</th>
<th>$\sigma_{HMIN}$ (MPa)</th>
<th>Hydraulic Fracturing</th>
<th>Overcoring</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nevada Test Site</td>
<td>380</td>
<td>3.5 (N55°W)</td>
<td></td>
<td>2.5 (N45°W/91°)</td>
</tr>
<tr>
<td>Helms Pumped Storage Project</td>
<td>300</td>
<td>5.5 (N65°W)</td>
<td></td>
<td>7 (N74°W/104°)</td>
</tr>
<tr>
<td>Bad Creek Pumped Storage Project</td>
<td>230</td>
<td>15.5 (N30°W)</td>
<td></td>
<td>17.5 (N32°W/112°)</td>
</tr>
<tr>
<td>Hanford</td>
<td>50</td>
<td>1.5 (N15°E)</td>
<td></td>
<td>2 (N8°E/95°)</td>
</tr>
<tr>
<td>Darlington, Ontario</td>
<td>70</td>
<td>8 (N20°W)</td>
<td></td>
<td>8 (N20°W)</td>
</tr>
<tr>
<td>Strippa, Sweden</td>
<td>320</td>
<td>11.5 (N14°E)</td>
<td></td>
<td>10.5 (N54°E)</td>
</tr>
</tbody>
</table>

†From Haimson (1981).
<table>
<thead>
<tr>
<th>Test</th>
<th>Cycle</th>
<th>Material</th>
<th>Applied Principal Stress (psi)</th>
<th>Measured Pressure (psi)</th>
<th>Average Fracture Inclination (°)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td></td>
<td>Mortar</td>
<td>( \sigma_v = 500 ) ( \sigma_{HMAX} = 0 ) ( \sigma_{HMIN} = 0 )</td>
<td>530, 0</td>
<td>4°</td>
<td>Average slope is uniform and pressure decays to zero.</td>
</tr>
<tr>
<td>6 2</td>
<td></td>
<td>Mortar</td>
<td>0</td>
<td>320</td>
<td>335</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td></td>
<td>Mortar</td>
<td>400</td>
<td>428</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>4</td>
<td>Mortar</td>
<td>( \sigma_v = 750 ) ( \sigma_{HMAX} = 400 ) ( \sigma_{HMIN} = 0 )</td>
<td>700, 420, 295</td>
<td></td>
<td>Multiple points on shut-in curve.</td>
</tr>
<tr>
<td>9</td>
<td>2</td>
<td>Mortar</td>
<td>( \sigma_v = 750 ) ( \sigma_{HMAX} = 400 ) ( \sigma_{HMIN} = 0 )</td>
<td>720, 430</td>
<td></td>
<td>Multiple points on shut-in curve.</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>Granite</td>
<td>( \sigma_v = 3,000 ) ( \sigma_{HMAX} = 0 ) ( \sigma_{HMIN} = 0 )</td>
<td>210, 0</td>
<td>4°</td>
<td>Slow decay due to low horizontal stress level.</td>
</tr>
<tr>
<td>12</td>
<td></td>
<td>Granite</td>
<td>( \sigma_v = 1,500 ) ( \sigma_{HMAX} = 400 ) ( \sigma_{HMIN} = 0 )</td>
<td>585, 350</td>
<td>5°</td>
<td>Fracture was curved, no distinct slope change (( \sigma_{HMAX} ) acts across a pre-existing fracture).</td>
</tr>
<tr>
<td>13</td>
<td>10</td>
<td>Granite</td>
<td>( \sigma_v = 3,000 ) ( \sigma_{HMAX} = 660 ) ( \sigma_{HMIN} = 0 )</td>
<td>800 and plateau at 1,100</td>
<td>3°</td>
<td>816</td>
</tr>
<tr>
<td>14</td>
<td></td>
<td>Granite</td>
<td>( \sigma_v = 3,000 ) ( \sigma_{HMAX} = 400 ) ( \sigma_{HMIN} = 60 )</td>
<td>Plateau at 480 psi</td>
<td>2°</td>
<td>560</td>
</tr>
<tr>
<td>15</td>
<td>2</td>
<td>Granite</td>
<td>670</td>
<td>680</td>
<td>10°</td>
<td>660 Slope change after plateau.</td>
</tr>
<tr>
<td></td>
<td>reload</td>
<td>Granite</td>
<td>670</td>
<td>630</td>
<td>10°</td>
<td>660</td>
</tr>
<tr>
<td>16</td>
<td></td>
<td>Granite</td>
<td>1,000</td>
<td>Plateau at 1,050 to 1,100</td>
<td></td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>3</td>
<td>Granite</td>
<td>( \sigma_v = 0 ) ( \sigma_{HMAX} = 480 ) ( \sigma_{HMIN} = 0 )</td>
<td>Slight plateau at 500</td>
<td>10°</td>
<td>There is an aberration at 530 psi (( \sigma_{HMAX} ) acts across a pre-existing fracture).</td>
</tr>
<tr>
<td>19</td>
<td></td>
<td>Granite</td>
<td>660</td>
<td>690</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td></td>
<td>Granite</td>
<td>2,215</td>
<td>290</td>
<td>2°</td>
<td></td>
</tr>
<tr>
<td>21</td>
<td></td>
<td>Granite</td>
<td>( \sigma_v = 3,000 ) ( \sigma_{HMAX} = 400 ) ( \sigma_{HMIN} = 0 )</td>
<td>400</td>
<td></td>
<td></td>
</tr>
<tr>
<td>22</td>
<td>11</td>
<td>Plexiglas</td>
<td>1,500</td>
<td>1,480</td>
<td>1,830</td>
<td></td>
</tr>
<tr>
<td></td>
<td>12</td>
<td></td>
<td>1,750</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>24</td>
<td>8</td>
<td>Plexiglas</td>
<td>1,500</td>
<td>1,495</td>
<td>1,030</td>
<td></td>
</tr>
<tr>
<td></td>
<td>12</td>
<td></td>
<td>1,000</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1^As deduced from P vs log (t + \( \Delta t \))/\( \Delta t \).

2^This is in fact the stress acting across the fracture.
Figure 1. A Common Pressure vs Time Situation.

Figure 2. Polyaxial Loading Cell.
Figure 3. Loading System.

Figure 4. Indication of Point of Inflection.
Figure 5. Indication of Point of Inflection.
Figure 6. Indication of Point of Inflection.
Figure 7. Indication of Point of Inflection.

Figure 8. No Slope Change Below the Minimum Principal Stress.
Figure 9. Indication of Point of Inflection: slightly inclined fracture leads to measured pressure in excess of the minimum stress.
Figure 10. The point of inflection reflects an additional stress component from the vertical load (refer to Table 3).
Figure 11. Away from the wellbore, the fracture was inclined at approximately 5°, possibly causing the upper slope change.
Figure 12. The stress level remained anomalously high even after a relatively long period of time.
Figure 13. Plateau on Pressure Decay Curve.

Figure 14. Plateau on Pressure Decay Curve.
Figure 15. Plateaus on Pressure Decay Curve.
Figure 16. Unexplained Pressure Plateau. This may reflect an inclined feature.
Figure 17. Reopening of an Existing Fracture.

Figure 18. Possible Presence of Multiple Inflection Points (mortar sample with quasi-orthogonal fracture system).
Figure 19. Possible Presence of Multiple Inflection Points (mortar sample with quasi-orthogonal fracture system).
Figure 20. Possible Presence of Multiple Inflection Points (mortar sample with quasi-orthogonal fracture system).
STRESS MEASUREMENTS IN SALT IN A DEEP BOREHOLE, SOUTHEASTERN UTAH

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ABSTRACT

Five successful hydraulic fracture tests were conducted in a 25- to 30-cm (10- to 12-inch) diameter borehole in bedded salt strata 954 to 1,477 meters (3,130 to 4,846 feet) below ground surface in southeastern Utah. Breakdown pressures were observed in two of the three shallower tests, indicating an induced fracture. Impression packer results confirmed this result. The pressure time histories of the two deeper tests showed no breakdown pressure indicating either fluid flow into a pre-existing fracture or inelastic deformation of the borehole. The magnitude of the maximum horizontal stress is interpreted to be 1.5 times the minimum horizontal stress, and the magnitude of the minimum horizontal stress corresponds to a reasonable lithostatic gradient of 26 kpa/m (1.15 psi/ft). The maximum horizontal stress is oriented approximately ENE - WSW.

INTRODUCTION

An important consideration in the siting of an underground nuclear waste repository in salt is the stability of deep openings. In situ state of stress and the material properties of salt ultimately dictate the feasibility of mining and reinforcing an underground structure. In an attempt to determine the in situ state of stress in salt, several hydraulic fracture tests were conducted in borehole GD-1 in the Gibson Dome area of the Paradox Basin in southeastern Utah (Figure 1). The objectives of the hydraulic fracture tests were two-fold: (1) to determine whether the Paradox Basin salt, a material with moderate deformability, could be hydraulically fractured; and (2) to evaluate the in situ state of stress.

SITE DESCRIPTION

Borehole GD-1 was drilled in the Gibson Dome area to a maximum depth of approximately 1,946 meters (6,384 feet). The diameter of the borehole ranged from 25 to 30 cm (10 to 12 inches) in the intervals tested. The stratigraphy encountered in the borehole included bedded sedimentary deposits of sandstone, siltstone, limestone and dolomite of Permian and Pennsylvanian age from the surface to a maximum depth of 798 m (2,618 feet). The sedimentary deposits overlie the Paradox Formation of Pennsylvanian age and consist of distinct
beds of salt separated by interbed sequences of anhydrite, dolomite and fine clastic rock. The Paradox Formation was deposited in 29 depositional cycles with a total thickness of 881 m (2,890 feet) in the GD-1 borehole. The base of the formation is at 1,679 m (5,507 feet).

The salt found in the Paradox Formation at GD-1 is 95% pure and forms beds from 2 to 105 m (7 to 346 feet) thick. Anhydrite bands within the salt beds comprise 2 to 5 percent of the rock mass in each depositional cycle. The anhydrite is found in two forms: (1) laminar anhydrite, which forms bands approximately 1 to 2 mm (0.04 to 0.08 inch) thick and spaced 2 to 10 cm (1 to 4 inches) apart; and (2) diffuse 2- to 3-cm (1-inch) thick bands of anhydrite sand in a salt matrix.

TECHNIQUE

The hydraulic fracture technique was used for in situ stress measurements because it enables measurements in deep boreholes. The technique was first described by Hubbert and Willis (1957) and was further refined by Haimson and Fairhurst (1967).

The technique involves raising the fluid pressure in a sealed segment of a borehole until a tensile fracture is induced. Continued pumping opens the fracture and extends it away from the borehole. When pumping ceases, the pressure in the borehole comes to an equilibrium level as the horizontal stress closes the fracture. Subsequent analyses of the pressure-time histories of the tests yield the magnitudes of the principal stresses.

The assumptions implicit in the technique are: (1) one of the principal stress axes is vertical; (2) the fracture propagates parallel to the maximum horizontal stress and perpendicular to the minimum horizontal stress; (3) the rock strata are homogenous and isotropic; and (4) the rock strata deform elastically.

Haimson and Fairhurst (1968) demonstrated that a fracture would form when the breakdown pressure, \( P_B \), is:

\[
P_B = 3S_h - S_H - P_p + T
\]

where \( S_h \) is the minimum horizontal stress perpendicular to the fracture, \( S_H \) is the maximum horizontal stress parallel to the fracture, \( P_p \) is the pore pressure, and \( T \) is the tensile strength.

Bredehoeft and others (1976) showed that the fracture opening pressure, \( P_F \), can also be used to calculate \( S_H \) in cases where the tensile strength is not known. Zoback and others (1980) successfully utilized this technique in several tests conducted in California. As shown by Bredehoeft and others (1976), the fracture will propagate when the fracture opening pressure, \( P_F \), is:

\[
P_F = 3S_h - S_H - P_p
\]
The azimuth of the maximum horizontal stress corresponds to the direction of fracture propagation and can be determined from the orientation of the hydraulic fracture at the borehole wall. Impression packers and/or seismic televiewers are commonly used for this determination.

An idealized pressure-time history is shown on Figure 2. Breakdown and fracture opening pressures are shown. The instantaneous shut in pressure, ISIP, is defined as the minimum horizontal stress, \( S_h \). The ISIP and the fracture opening pressure, \( P_F \), are determined where the pressure versus time curve begins to depart from linearity after shut in of the first pressurization cycle and pump-in during the second pressurization cycle. \( P_B \), \( P_F \), ISIP, and \( S_h \) are shown on Figure 2.

**METHODOLOGY**

Test zones were isolated with inflatable rubber packers. Care was taken to choose test zones that were: (1) totally within a single cycle of salt and, (2) unfractured. To choose an unfractured interval within the salt cycle the core was carefully examined for pre-existing fractures. None were observed within the test zones chosen. Figure 3 is a schematic of a test zone and the equipment used. Five of the test zones were 4.1 m (13.5 feet) long; one of the test zones was 30.5 m (100 feet) long. In all tests borehole drilling fluid completely (a brine, density equal to 1.4 g/cc) filled the borehole and the test zone below a closed shut in valve. The packers were inflated with a downhole pump for the 4.1-m (13.5-foot) test zones and with surface nitrogen pressure for the 30.5-m (100-foot) test zone. Table 1 lists the test intervals, depths, and data gathered.

The test sequence consisted of lowering a straddle packer test tool ("drill-stem test tool" in oilfield terminology) to the desired elevation and inflating the packers (the downhole pump was powered by rotating the tubing string). The shut in valve was opened and pressure inside the test zone was increased at a slow constant volumetric injection rate of between 1.9 to 19 liters per minute (0.5 to 5 gallons per minute) until a fracture was induced or until a maximum pressure plateau was achieved. Surface wellhead pressure and the quantity of fluid injected at the wellhead were measured. Immediately after the test zone was fractured, the system was shut in and pressure was monitored for a period of time. Fluid was then let out or added to the system in a series of bleed, shut in, pump-in, bleed cycles. The system was pressurized again at a higher flow rate until a maximum pressure plateau was encountered. The system was shut in again and a final bleed, shut in, pump-in, bleed cycle was conducted. The test was terminated by reducing surface pressure to atmospheric pressure.

The tests utilized a high-pressure/low-flow hydraulic "triplex" piston pump capable of delivering between 2 to 38 liters per minute (0.5 to 10 gallons per minute) at pressures to 69 Mpa (10,000 psi). The low injection volume of this pump arrangement was utilized to obtain an accurate record of peak breakdown pressure. Fluid volume was measured by observing the fluid level in a 167-liter (44-gallon) fluid supply tank equipped with a graduated sight glass.
Surface injection pressure was monitored by a bourdon-type pressure gauge and a sensitive digital quartz-crystal transducer (QCT). Downhole pressure was measured by lowering a pressure transducer on a wireline to a depth of 30 m (100 feet) above the test zone. Pressure was not monitored below the bottom packer. Backup pressure measurements were made within the test zone and annulus above it with a digital memory recorder and a mechanical scratch-type recorder.

An impression packer consisting of a conventional inflatable packer wrapped with soft rubber was used to record the orientation of the fracture.

The accuracy of the downhole wireline sensors indicated downhole pressure within ± 0.5 percent of the sum of the surface injection pressure and hydrostatic pressure of the fluid filling the tubing. The accuracy of the surface pressure measurements was approximately ± 14 kpa (± 2 psi). The volume measurements were accurate to 0.76 liters (± 0.2 gallons), or within ± 0.5 percent of a typical injection test volume of 140 liters (37 gallons).

Compliance measurements were made to test the stiffness of the overall system. The measurements consisted of 2 components: (1) tool compliance within the test zone below the shut in valve; (2) the compliance of the tubing, wellhead piping, and hydraulic pressure connections above the shut in valve. These measurements were used to correct volumetric strain measurements made in the salt strata as part of the overall geotechnical program.

DATA ANALYSIS

Six hydraulic fracture tests were attempted. Five of the tests, GDST-6, -6A, -7, -8, -9 were conducted in 4.1-m (13.5-foot) intervals. GDST-6 was not successful. GDST-4A was conducted in a 30.5-m (100-foot) interval. The pressure time histories of GDST-4A, -6A, -7, -8, -9 are shown on Figures 4 and 5. Two of the shallowest tests (GDST-8 and -9) exhibited a breakdown pressure, as shown on Figures 4a and 4c. The two deepest tests (GDST-6A and -7), shown on Figures 5a and 5b, did not show a breakdown pressure. GDST-4A, shown on Figure 4b, did not exhibit the classic breakdown pressure but did exhibit a distinct maximum pressure during the first pressurization cycle.

At 956 m (3,137 feet), test GDST-9, a breakdown pressure was observed at 30.4 MPa (4,400 psi) indicating that a fracture had been initiated (Figure 5b). An impression packer inflated in the test zone confirmed this observation. The magnitude of the instantaneous shut in pressure, ISIP, measured during the first, second and third pressurization cycles was approximately the same. The magnitudes of the fracture opening pressures of the second and third pressurization cycles are reproducible, providing confidence in the values chosen.

GDST=Geotechnical Drill Stem Test
At 1,106 m (3,629 feet), test GDST-4A, an attempt was made to fracture a 30.5-m (100-foot) interval. The pressure time history of this test is shown on Figure 4b. Although a breakdown pressure followed by decreasing pressure as pumping continued was not observed, the maximum pressure of this first pressurization cycle is greater than subsequent pressure maximums of the second and third pressurization cycles. The low volumetric strain rate (only about 14 percent of the rate for shorter zone tests) might create a condition favorable for slow crack propagation. The shape of the pressure time curve, particularly the first and second pressurization cycles, favors this interpretation. The breakdown pressure and ISIP were 35.2 Mpa and 33.8 Mpa (5,100 and 4,900 psi), respectively.

At 1,273 m (4,176 feet), test GDST-8, breakdown was observed at 39.3 MPa (5,700 psi). The ISIP is 35.9 MPa (5,200 psi). As in GDST-9, the fracture opening pressure of the second and third pressurization cycles are of nearly equal magnitude and are reproducible.

At 1,395 m (4,577 feet), test GDST-7, a breakdown pressure was not observed. The magnitude of the maximum pressure of the first pressurization cycle (Figure 5a) was nearly equal to the ISIP. The pressure maximum of the second and third pressurization cycles were slightly greater than the ISIP. The typical explanation of this type of pressure time history is that a pre-existing fracture was encountered. However, a careful examination of the core showed no fractures. An alternative explanation is that the hole is plastically deforming.

A breakdown pressure was also not observed at 1,477 m (4,846 feet), during test GDST-6A. Little difference was observed in the pressure maximums and ISIP of the first, second, and third pressurization cycles. As in the previous test, it is assumed that this type of pressure time history is caused by plastic deformation.

The fracture-opening pressure, taken from the second or third pressurization cycles, and the instantaneous shut in pressures, as defined on Figure 2, were used to calculate the maximum and minimum horizontal stresses. To determine the fracture opening pressure, a straight line was drawn through the data points of the second and third pressurization cycle. The point at which the data deviated from the line was defined as the fracture-opening pressure. A similar technique was used to determine the ISIP during the first pressurization cycle. The pressure-time history of GDST-8 is enlarged on Figure 6 to illustrate the determination of breakdown pressure, fracture opening pressure, and ISIP.

Table 1 is a summary of the in situ stresses and test pressures interpreted from the pressure-time curves. The magnitudes of $P_B$, and ISIP are plotted versus depth on Figure 7a. The magnitudes of the inferred in situ stresses, $S_H$ and $S_h$, are plotted versus depth on Figure 7b. The lithostatic stress gradient is assumed to be 26 kPa/m (1.15 psi/ft). The lithostatic gradient is represented by a solid line on Figures 7a and 7b. The pore pressure is assumed to be drilling fluid pressure; the gradient is 13.6 kPa/m (0.6 psi/ft).
The magnitudes of the maximum and minimum horizontal stresses, $S_H$ and $S_h$, increase with depth. Assuming a pore pressure gradient of 13.6 kPa/m (0.6 psi/ft), the values of the maximum horizontal stress, $S_H$, range from 40.8 MPa (5,920 psi) at 956 m (3,137 feet) to 56.3 MPa (8,170 psi) at 1,477 m (4,846 feet). The impression packer results indicate that the maximum horizontal stress is oriented ENE - WSW at a depth of 956 m (3,137 feet).

Volumetric strain measurements of the salt strata were made during the unloading pressure sequences as part of the overall geotechnical program for GD-1 (Nelson and Kochershans, 1981). These measurements provide additional data on the material properties of the salt strata and were used in the interpretation of the hydraulic fracture data. Volumetric strain versus depth is plotted on Figure 8. At depths corresponding to the shallowest tests volumetric strain is small. As test depth increases the volumetric strain increases. This observation is consistent with the hydraulic fracture results which suggests that the salt strata becomes more plastic as a function of depth.

DISCUSSION

Salt strata were fractured to depths of 1,219 m (4,000 feet) using standard oilfield drilling equipment. Several factors contributed to this success. Careful inspection of the core for pre-existing fractures facilitated choosing a fracture-free interval, and utilization of a high-pressure/low-flow hydraulic pump facilitated easy observation of breakdown pressure. Fluid flow was monitored with relative ease by monitoring reservoir volume at the surface without the use of flow meters.

The maximum horizontal stress in borehole GD-1 is generally about 1.5 times the minimum horizontal stress. These values seem unreasonably high considering the plastic nature of salt. There are several sources of error in the determination of the maximum horizontal stress: (1) the uncertainty in the determination of pore pressure; (2) the uncertainty of the magnitude of the tensile strength of salt; and (3) the possibility that the assumption of elastic response may not be strictly valid for the salt strata encountered in GD-1.

In borehole GD-1 the assumptions used for the pore pressure determinations may be the source of error. If pore pressure in salt is assumed to be equal to the lithostatic stress (vertical stress), the values for the maximum horizontal stress are nearly equal to the vertical and minimum horizontal stress and thus indicate hydrostatic stress conditions. An additional determination of $S_H$ was calculated assuming that the pore pressure gradient is equal to the lithostatic pressure gradient. These calculations are included in Table 2. In this case $S_H$ ranges from 27.9 MPa (4,043 psi) at 956 m (3,137 feet) to 38.1 MPa (5,522 psi) at 1,106 m (3,629 feet). The shear and tensile strength calculations also change slightly.

Impression packer results were disappointing. Only one successful impression was achieved out of four attempts. The rubber was commonly scraped off and/or damaged traveling in and out of the hole. A borehole televiewer may improve
The lack of a breakdown pressure in the deeper tests is indicative of either a pre-existing fracture or a change in material properties. Since a careful examination of the core for GD-1 showed no pre-existing fractures the latter explanation is more plausible. In addition, unloading volumetric strain measurements showed substantial strain below 1,219 m (4,000 feet) as shown in Figure 8.

To assess the potential influence of the non-elastic properties of salt, the hydraulic fracture test results of GD-1 were compared with the results of tests in other rock types. An unexpected but encouraging result was the fact that the pressure time histories of the shallower tests (depths less than 1,220 m) showed a breakdown pressure characteristic of hydraulic fractures induced in brittle elastic rocks.

Other in situ stress measurements have been made within the Colorado Plateau in the Piceance Basin, Colorado (Bredehoeft and others, 1976) and Rangely, Colorado (Raleigh and others, 1972). The results of hydraulic fracture tests in the Piceance Basin indicate near hydrostatic stress conditions at 0.5 km depth with the maximum horizontal stress oriented approximately N70°W to N80°W. At Rangely, Colorado stress measurements made at depths of approximately 1,800 m indicate that the magnitude of the maximum horizontal stress is approximately twice the vertical stress and one and one-half times the minimum horizontal stress. The maximum horizontal stress axis is oriented N70°E. The unusually high deviatoric stress level is thought to be influenced by the oil field activities and not due to the regional stress field (Raleigh and others, 1972). The direction of the maximum horizontal stress axis measured at GD-1 is consistent with the results at Rangely, Colorado.

Pure salt normally displays plastic behavior when subjected to small deviatoric stresses. Results of the tests in GD-1 tentatively suggest that salt behaves in a relatively brittle manner at depths less than 1,219 m (4,000 feet) and in a more plastic manner at depths greater than 1,219 m (4,000 feet). Volumetric strain measurements support this observation. Triaxial laboratory tests of the salt core taken from the borehole at confining pressures reproducing the pressure conditions encountered in GD-1 are being conducted to test this observation.

Hydraulic fracture data were compared with stress orientations inferred from earthquake focal mechanism data in the Paradox Basin (Wong and Simon, 1980). Microearthquake activity has been observed in the proximity of the confluence of the Green and Colorado Rivers, Southeastern Utah. The earthquake activity is located in the precambrian basement (at depths greater than 2 km) below the salt in the Paradox Basin. The focal mechanisms indicate strike-slip faulting with predominantly east-west compression. The intermediate stress axis is near vertical. The earthquake activity suggests a stress state other than hydrostatic. The non hydrostatic stress state and orientation of the stress axes are consistent with the hydraulic fracture measurements.
CONCLUSIONS

The results of hydraulic fracture measurements in the Paradox salt strata indicate that successful fracture tests can be conducted. The results of the tests indicate a non-hydrostatic stress state at depths shallower than 1,219 m. The maximum horizontal stress axis is oriented ENE - WSW consistent with other hydraulic fracture measurements in the Colorado Plateau and with earthquake focal mechanism data (Wong and Simon, 1980).

The test results yield reasonable values of the magnitudes of the minimum horizontal stress. The magnitude of the maximum horizontal stress seems unreasonably high considering the material properties of salt. The limitations imposed by the initial elastic assumptions and the pore pressure estimates may be responsible.

At depths above 1,219 m salt appears to behave in a relatively brittle elastic manner as evidenced by the shape of the pressure time curve. At depths below 1,219 m salt appears to behave in a more plastic manner.
REFERENCES


<table>
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<tr>
<th>Test</th>
<th>Depth (m)</th>
<th>Length (m)</th>
<th>Temp (°C)</th>
<th>$P_B$ (MPa)</th>
<th>$P_F$ (MPa)</th>
<th>$P_o$ (MPa)</th>
<th>$S_h$ (MPa)</th>
<th>ISIP = $S_h$ (MPa)</th>
<th>$S_v$ (MPa)</th>
<th>$T_o$ (MPa)</th>
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$S_h$ values in parentheses are calculated using breakdown pressure instead of fracture opening pressures and provide a lower bound estimate of $S_h$.

$P_B$ = Breakdown Pressure

$P_F$ = Fracture-Opening Pressure

$P_o$ = Pore Pressure

$S_h$ = Minimum Horizontal Stress

$S_v$ = Vertical Stress

$T_o$ = Tensile Strength

$S_M$ = Maximum Shear Stress

ISIP = Instantaneous Shut In Pressure
Table 2

Table of Recalculated Values of $S_h$, $S_h$, & $S_v$
Assuming Pore Pressure Equal to Lithostatic Gradient

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$P_o$ = Pore Pressure
$P_f$ = Fracture-Opening Pressure
Figure 1: LOCATION MAP OF PARADOX BASIN, UTAH
LEGEND

$P_B$ = Breakdown Pressure
$P_F$ = Fracture Opening Pressure
ISIP = Instantaneous Shut-in Pressure
$S_h$ = Minimum Horizontal Stress

Figure 2: IDEALIZED PRESSURE–TIME HISTORY
Figure 3: SCHEMATIC OF TEST ZONE SET-UP FOR HYDRAULIC FRACTURE TESTS
Figure 4: PRESSURE TIME HISTORIES OF GDST-9, -4A, -8
Figure 5: PRESSURE TIME HISTORIES OF GDST-6A AND GDST-7
Figure 6: ENLARGEMENT OF GDST-8
Figure 7: SUMMARY PLOTS OF HYDRAULIC FRACTURE DATA
Figure 8: VOLUMETRIC STRAIN VERSUS DEPTH
IN SITU STRESS DETERMINATION IN DEEP WELLS AS AN AID TO STIMULATION DESIGN

A. Abou-Sayed, Terra Tek

Introduction

At the present time the only technique which has been demonstrated to be viable for in situ stress determinations at depths greater than a few hundred meters is a small scale hydraulic fracturing operation. The technique is well understood and frequently demonstrated for open hole operations (von Schoenfeldt, 1970; Haimson, 1978; Bredhoeft et al., 1976; Zoback and Pollard, 1978). The hydraulic fracturing technique measures the minimum in situ stress directly by measuring the instantaneous shut in pressure following the pressurization and breakdown of a packed off section of a borehole. During the ten year period in which hydraulic fracturing has been routinely utilized in deep boreholes for stress determination, interpretive techniques have been refined to the point whereby the maximum secondary principal stress in deep holes can be computed to the same relative degree of precision as stress relaxation techniques yield in shallow boreholes. The minimum principal stress is not computed—it is measured directly and is independent of the material properties of the rock mass. Questions as to the propagation of fractures in a direction normal to the maximum stress directions have been resolved in laboratory experiments which illustrate the rate dependence of such phenomena. The hydraulic fracturing technique averages stresses over relatively large areas and therefore is unaffected by inhomogeneties in the rock mass. The criticisms which have been presented against using hydraulic fracturing as a stress measuring technique in deep holes have one important fact in common; they are not criticisms of applicability of the technique as much as they are criticisms of the methods utilized to interpret the raw data.

Hydraulic fracturing equipment is proven, rugged, and durable; many of the components are borrowed directly from the petroleum production industry and have a history of successful development and a proven work record. The few "delicate" components of the tools are protected within a strong housing and are never exposed to adverse wellbore fluids. Should a tool become stuck, highly developed techniques are available to free it from the hole. The robust design of the equipment insures that when it is recovered, after being subjected to tension, torque and hydraulic jarring, damage will be minimal and the tool is almost ready for reuse.

The hydraulic fracturing technique possesses a major operating advantage which, realistically, cannot be claimed for either the relaxation or ultrasonic techniques; it will function in a cased borehole. Once again, the interpretive techniques are still in the debatable stages, but large scale laboratory work will remedy any discrepancies in interpretive techniques.

Other techniques, utilizing for example strain relief or velocity birefringence, are proposed from time to time, but to date none have been successfully demonstrated in a deep hole. As the name suggests, strain relief or relaxation methods are techniques which induce a stress change in the body and, by the detection of the subsequent material displacement, allow the calculation of the stress state. The most serious objections to the relaxation techniques lie in the fact that stress is not measured but calculated from measured strain, the direct implication of this being a requirement that material properties of the rock mass be known accurately. When temperature effects, such as those which would be encountered in a deep borehole, are considered, the
The problem of sensing displacement and calculating stress becomes more complicated. The displacement must be sensed with mechanical devices, conventional or friction bonded strain gauges, all of which are prone to the adverse effects of temperature such as expansion and creep. Additionally, the material properties of the rock mass are also highly dependent upon temperature and thus the calculated stresses are subject to at least two levels of introduced error. It must also be considered that in all probability, core will not be available from deep drilled holes. Material properties, including the effects of temperature and the presence of fractures, must be estimated before stress can be calculated.

The only strain relief technique which completely reduces the stress state to zero is overcoring; other techniques such as sidewall undercoring or cutting a slot to relax the media do not achieve total stress relief and thus the calculated stresses must be "adjusted" to account for this. Overcoring has not been successfully performed at depths greater than about 300 meters; the lack of complete strain relief for the other relaxation techniques coupled with temperature effects on both the material properties of the rock mass and behavior of the strain sensing device itself makes strain relief an illogical choice for a stress determination tool at depth. It should be noted, however, that for stress determination near the surface, at moderate temperatures, where overcoring can be performed and the cores recovered for biaxial modulus testing, the strain relief technique is perhaps the most reliable stress determination technique available today.

The nature of strain relief techniques in a deep borehole would require an arrangement of expanding electric or hydraulic jacks and motors to perform the undercoring or slot cutting activities. This unprotected equipment would be very prone to jamming by silt sized particles on the rams or pieces of rock from the borehole wall falling off during the drilling or slotting activities. There is no conceivable way in which relaxation techniques could be utilized in cased boreholes. The stiffness of the steel casing would require an extreme sensitivity in the strain sensing device. In any event, the lack of knowledge of the bond between the casing and rock mass, plus the thickness of the intervening cement region, make the analytic solution, which is required for the interpretation of relaxation measurement, intractable.

Ultrasonic techniques have been shown to be reliable indicators of stress states in isotropic materials under carefully controlled laboratory conditions; there have been recent laboratory demonstrations that the technique is applicable to some rock types (Aggson, 1978). The technique is somewhat analogous to photoelasticity, but interference techniques are not sufficient to determine stress differences. Velocity changes, which vary by only a few percent must be measured directly. As with relaxation techniques, material properties must be known accurately to calculate the stress state. The unknown variation of material properties again affects the results of the calculations which in turn are relatively complex, requiring a computer for Fourier correlations. The technique requires the generation of a polarized shear wave which is a non-trivial problem. The presence of micro-cracks has a pronounced effect on the results of the test, causing velocity variations an order of magnitude higher than the velocity levels which must be measured to calculate the stress field.

Geophysical logging tools are routinely available that generate shear waves in deep boreholes. However, the intent of these investigations is to take advantage of the attenuation of shear waves in the presence of fractures. These tools make no attempt to measure velocities but rather look for changes in amplitude. The present tools require a fluid filled borehole and do not work unless the shear wave velocity in the rock is greater than the compressional wave velocity in the field. The implication here
is that any device proposed for ultrasonic detection of stress must use pads directly contacting the borehole wall. Logging companies have been understandably unsuccessful in developing tools with this capability since the requirement of exposing delicate equipment to an adverse borehole environment is an invitation to tool malfunction and loss. The ultrasonic technique is unproven as a reliable stress indicator in rock, and is untried in a real borehole environment. Additionally, the technique stands little chance of success in working in a cased borehole.

**Stress Determination at Depth**

The state-of-the-art then, today, is that a technique exists for stress determination at depth, but it is best understood for open hole conditions. Very little data has been published (Daneshy, 1973) with regard to stress determination in cased wells; however, because of the costs incurred if a tool should become jammed in an open portion of the hole, stress determinations must be typically carried out in cased wells.

A series of laboratory tests have been performed to determine the feasibility of using the hydraulic fracturing technique as a means of in situ stress determination in cased wellbores (Voegele, et al., 1981). The objective of these tests was to determine whether the stresses, in particular the minimum horizontal stress, applied to the specimens could be estimated by analysis of the pressure-time records obtained during the hydraulic fracturing of the specimens.

The majority of these tests were performed in 30 cm x 30 cm x 45 cm specimens of hydrostone Super-X loaded in triaxial compression. API standard 7 in. O.D. - 23 lb/ft casing was simulated using 28.6 mm x 26 mm steel tubing which was centered in the model before casting. The samples and casing were perforated using a small right angle drive drill which was positioned inside the casing with a rod. These perforations were approximately 3 mm in diameter and extended approximately 8 mm into the hydrostone.

Two perforation configurations were used in these tests. These consisted of either: two perforations drilled 180° apart at the mid-point of the sample; or perforations drilled in a helical arrangement. The helical perforations were confined to the middle one-third of the specimen and were located 1.0 cm apart along the axis of the casing and 30° apart tangentially around the casing.

Samples with the 180° perforations were tested with the perforations oriented at 0° and 90° to the maximum horizontal stress. The purpose of these tests was to determine whether the fracture direction would be controlled by perforation orientation or, as suggested by Daneshy (1973), the fracture would be properly oriented parallel to the maximum stress, independent of perforation orientation.

It was found that the fractures always initiated through the perforations even when the perforations were oriented at 90° to the maximum stress. It was also found that these fractures re-oriented themselves as they extended. Even though these fractures became re-oriented, it was not possible to reliably estimate the applied stresses from the pressure-time records. This is not surprising in that the fracture paths were somewhat tortuous and that the pressures were measured in the wellbore and hence would be a measure of some average stress acting perpendicular to the fracture. It is also likely that the method used to create the perforations led to a more realistic simulation than that used by Daneshy. The post shut-in portion of the pressure-time records could, perhaps, yield information as to
the state of stress acting perpendicularly to the fracture at various locations, however, such an analysis is beyond the present state-of-the-art and would require more work in numerical modeling and experimental verification before such an analysis could be considered valid.

By perforating the wellbore with a helical pattern, there is a very good chance that one perforation will be oriented in or very near to the direction of the maximum horizontal stress and as such the fracture would initiate in the proper orientation. Results of the tests performed with the helically perforated specimens show that this is indeed the case.

In general it was found that the induced fracture initiated from one perforation, in the proper orientation. There was some indication of several perforations being intersected by the induced fractures; however, this may be due to the rather close spacing of the perforations along the casing. For the most part the fractures were planar and oriented in the proper direction by the time they had propagated to several wellbore diameters into the rock mass.

For those cases where the preferred fracture orientation was vertical (the vertical stress was the intermediate or maximum principal stress) it was found that the instantaneous shut-in pressure was a reliable estimate of the minimum applied horizontal stress when analysis techniques based upon minimum shut-in pressures were utilized. The estimation of the maximum horizontal stress were not so straightforward; it must be borne in mind that the actual presence of the casing in the wellbore coupled with the cementing pressure history to which the casing has been subjected present a marked deviation from the standard open hole case upon which the interpretive theory is based. Nonetheless, it was found that for vertical fractures a reasonable estimate of the maximum horizontal stress could be estimated from the minimum reopening and shut-in pressures. This is not too surprising in light of the minimum level of interaction to be expected between the medium and the relative flexible inclusion represented by the casing.

For those cases where the preferred fracture orientation was horizontal (the vertical stress was the minimum principal stress) the instantaneous shut-in pressure could not always be related directly to the minimum principal stress. For tests in larger blocks the relationship between instantaneous shut-in pressure and minimum principal stress was observed although the fracture has to be extended a significant distance from the wellbore before this was so. As expected the other two principal stresses were not calculable from the data since standard interpretive techniques are based upon stress concentrations around wellbores and fractures propagating parallel to the wellbore. A strictly correct interpretive technique for horizontal fractures would probably require an analysis based upon fracture growth in two directions.

Application of In-Situ Stress Determination to Stimulation Design

The design of fracturing treatments are generally based upon the assumption that the vertical height of the fracture is known, and that this height remains a constant from the wellbore to the point of deepest lateral penetration. This fracture geometry may be quite accurate in the presence of strong barriers to vertical fracture growth. In fact, MHP treatments with results consistent with design predictions, appear to be in reservoirs where the adjacent rock layers form effective barriers to vertical fracture growth (Murphy and Carney, 1977). One must expect, however, to encounter many situations in which natural barriers to vertical fracture migration do not exist.
The application of the fundamental principles of fracture mechanics has led to rapid development of both quantitative and qualitative predictions of hydraulic fracture growth and geometry based on knowledge of the in situ material properties and in situ stress. Recent studies of hydraulic fracturing (Simonson, et al., 1978) have delineated those factors that affect fracture geometry and fracture containment within the pay zone. These factors include (i) the contrast in material properties, (ii) the contrast in in situ stress, (iii) the contrast in in situ stress gradients and frac fluid density.

Daneshy (1978), Cleary (1978) and Advani, et al., (1978) further discussed the effect of the contrast in material properties (including the interface) on the created fracture geometry. The material barrier concept (contrast in mechanical properties) appears to provide the basis of the success encountered in the Wattenberg experiment (Fast, et al., 1975). In this field, the pay zone had lower elastic moduli than the bounding shale layer. Figure 1 summarizes the mechanical properties of the pay zone and bounding formation in various gas fields where MHF treatments have been performed (Voegele, et al., 1981). As is apparent from data in the figure, the moduli contrast does not seem to prevail in many gas fields. While such data may be discouraging it does not necessarily follow that the lack of "material barriers" precludes the application of MHF treatments. An alternative, the stress barrier, shows significant promise in countering the lack of material barriers to contain an MHF within a pay zone. In fact, MHF appears very successful in the stimulation of the Cotton Valley Limestone, a formation which has no material-property barriers adjacent to the pay zone (Kozik and Holditch, 1981). In a case where there are no material barriers, the contrast in in situ stress and stress gradient is essential to the treatment design.

The stress state within a given region is typically consistent to the extent that at least its orientation remains fairly constant, as illustrated in Figure 2. This figure is a compilation based upon geologic evidence as well as in situ stress measurements and illustrates the stress state in terms of stress regimes, each with a different potential for faulting. As can be seen in those areas where several data points exist, the orientation is relatively constant; also, the boundaries between areas of relatively constant orientation are fairly well delineated. When magnitude as well is considered however, a different picture emerges. Figure 3 is a compilation of data from publishable in situ stress measurements; it is separated according to rock type. The stress difference axis is primarily an indication of preferred fracture plane; a positive stress difference indicates a preferential vertical fracture. Furthermore, a high differential stress is indicative of a stronger material. It is also a convenient axis to compare points from the same region to examine relative containment potential of the different horizons. For example, examine the cluster of data points at $\sigma_V = 90$, and $20 < (\sigma_V - \sigma_{MIN}) < 30$; these six points are from two wells within 1 km of each other. Two interesting points can be observed, namely; (i) the stress differences between the shales and sandstones; and (ii) the scatter in the data. There is also an interesting trend in the figure which supports a qualitative correlation between stress difference and rock type (Figure 4) as suggested by Abou-Sayed, et al., (1981). There is a strong theoretical basis for this correlation. General laboratory response of granites, sedimentary rocks and salt to applied loads implies that soft, high ductility material (higher principal strain ratio during inelastic flow) as well as materials not capable of sustaining large deviatoric stresses (low principal stress ratio in uniaxial strain tests) possess higher minimum horizontal stresses. In the elastic regime this response has widely been attributed to
Poisson's ratio effect. Examination of Figures 5(a) and (b) illustrates that there is strong field evidence to support this qualitative observation. Close examination of Figure 5(b), however, illustrates an important point; although the general trend is seen to exist, it must be concluded that, owing to the overlap of the boundaries, containment design cannot be made on the basis of lithology alone. This boundary overlap reinforces the requirement of in situ stress measurement on a case by case basis.
Figure 1. Graphic illustration of moduli contrast for tight gas reservoirs. (Voegele et al, 1981)
Figure 2. Stress regimes in the Contiguous United States, illustrating the consistency of orientation within a given regime. (Zoback and Zoback, 1980)

Figure 3. Data base of in situ stress measurements plotted according to rock type. (Lindner and Halpern, 1978; Swolfs, 1981; Voegele et al, 1981)
Figure 4. Qualitative correlation between stress difference and overburden stress (depth) for various rock types. (Abou-Sayed et al, 1981)

Figure 5. Stress difference as a function of overburden stress according to rock type. Prepared from data in Figure; compare to theoretical relationship expressed in Figure 4. (Voegele et al, 1981)
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HYDRAULIC FRACTURING IN ALBERTA TAR SAND FORMATIONS - A UNIQUE MATERIAL FOR IN SITU STRESS MEASUREMENTS

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ABSTRACT

Extensive tar sand deposits are located in northern and northeastern Alberta, Canada. The deposits consist of very viscous bitumen impregnating dense unconsolidated sand. A small portion of the deposit can be extracted by surface mining techniques, but the majority will require in situ processing.

The in situ stress state may have profound effects on many proposed in situ process technologies. However, the complexity of the tar sand material results in numerous anomalies in interpretation of classical hydraulic fracturing stress tests. As a further complication, existing data suggests that certain extraction processes may significantly alter the in situ stress regime during the life of the extraction project.
1. INTRODUCTION

Lower Cretaceous Heavy Oil Sands occur in a discontinuous trend extending from the Peace River deposit in northwestern Alberta through Wabasca, Athabasca and southwards to Cold Lake, (Figure 1). There is a southerly extension of the trend into the Lloydminster area straddling the Alberta - Saskatchewan border, where there are numerous Lower Cretaceous pools containing 16° - 24° API oil. These latter pools produce hydrocarbons by conventional methods, allied with some enhanced recovery processes.

In contrast, the heavy oil deposits contain bitumen ranging from 8° - 15° API which must be extracted either by surface mining or more exotic thermal stimulation techniques.

Table 1 gives recent in-place reserve estimates for the cretaceous oil sand deposits. Table II gives the general stratigraphic column for the area.

A correlation chart of the heavy oil areas is shown in Figure 2. Heavy oil sands are confined to the Mannville Group, which is divisible into a Lower and an Upper Unit. Lower Mannville sediments are mainly non-marine sands and form the oil reservoirs at Peace River and Athabasca. Upper Mannville sediments contain non-marine clastics, together with inter-tongues of marine sands and shales and constitute the oil-bearing sands at Wabasca and Cold Lake and the upper portion of the oil zone at Peace River. Correlations into the Cold Lake area are rather difficult, but the relationships shown represent at least a close approximation. Sands at Cold Lake have not been named but are referred to as the 'D' Unit, which is equivalent to the Gething Sands at Peace River and the McMurray Sands at Athabasca, while the 'C', 'B' and 'A' Units are of Upper Mannville age and correlate in total with the Bluesky - Spirit River of the Peace River area, and the Wabasca - Clearwater - Grand Rapids of the Athabasca Area.14

The oil sand deposits are underlain unconformably by Paleozoic sedimentary strata, (Figure 2). Surficial deposits are the debris left by the melting of an ice sheet which covered the whole of northern Canada to a depth of several thousand feet during the Pleistocene epoch. After the ice melted the original deposits were subjected to wind erosion. The finest-grained material was blown away and redeposited elsewhere as loess, while the sand-sized particles were blown into large migrating dune fields and the coarser-grained particles were left behind as lag deposits. As the climate ameliorated the dunes were stabilized by vegetation and the depressions on the poorly drained surface formed peat bogs and muskeg.

The thickness and stratigraphy of the surficial deposits of the area are not well known except where open pit mining is in progress. At this locality at least six stratigraphic units have been recognized in the glacial drift.3
The tar sand deposits, of which the Athabasca deposit represents by far the largest, are in general extremely heterogeneous. This fact alone makes any attempt to define average engineering properties very difficult. Even between adjacent wells stratigraphic detail and measured properties may show extreme variation. Sampling and testing problems unique to this material, as discussed later in this paper, compound the problem. Table III gives a range of material properties gleaned from the literature.\textsuperscript{3,5,6,7,8,9,14,16,18,19.}

Resource recovery is restricted to the following:

i) surface mining which is economic for only a small percent of the resources where it outcrops near Fort McMurray,

ii) In situ processing generally using thermal stimulation from surface wells, and

iii) hybrid techniques such as mine assisted in situ processing (MAISP).\textsuperscript{18}

The majority of the resources must be recovered using one or both of the later two techniques. At present such technology is in the experimental pilot program stage.\textsuperscript{15}

2. IN SITU PROCESSES

The term 'in situ process' is used to include both normal in situ and hybrid techniques (ii and iii above). The total in place tar sand reserves are estimated to be 950 billion barrels of which, 74 billion barrels or 8% are suitable for surface mining. The amount of in place bitumen which has more than 500 feet of overburden and must be recovered by in situ methods is 741 billion barrels, or 78% of the deposit. The remaining 135 billion barrels is overlain by 150 - 500 feet of overburden and may be recovered by some form of mining or by novel in situ recovery techniques.\textsuperscript{16}

The most critical factor in the recovery process is the extremely high viscosity of the bitumen in situ. For successful application of any in situ technique it is necessary to reduce the viscosity or otherwise mobilize it so that it will flow to the producing wellbore.

Plots of viscosity versus temperature for bitumen from each of the four major deposits are shown in Figure 3, with the "dot" on each line indicating the reservoir temperature and corresponding oil viscosity.\textsuperscript{16}

A large number of in situ pilot studies have been completed and/or are in progress in the various tar sand deposits (Table IV). The processes studied include, but are not restricted to, the following:
i) steam stimulation
   - cyclic steam (huff and puff)
   - steam flood

ii) in situ combustion
   - forward and reverse combustion
   - combustion plus waterflood

iii) electrical heating

iv) emulsification

v) hybrid (i.e. MAISP)

For in situ recovery impermeable overlying strata are required to prevent the loss of injected fluids, such as steam and air, to overlying formation. Such strata should be adequately thick to prevent loss of fluids by this means.\textsuperscript{18}

In situ processing schemes can be regarded as a permutation of unit operations as shown in Table V. Step two, Table V, is very critical since failure here leaves little chance of further progress.

Although there is sometimes natural injectivity, in many cases communication must be artificially induced. Many operators have attempted to obtain this injectivity enhancement by fracturing the formation. Furthermore, it is generally desirable to obtain horizontal communication paths, since this is the geometry required for maximum volumetric sweep efficiency. However, effective matrix stresses and natural anisotropy in the underground formation, rather than fracture design, probably control fracture direction and at the present time it appears that in general no guarantee can be placed on the direction of a hydraulic fracture in oil sands. Therefore, alternative methods of communication initiation, such as the use of horizontal wells and electrolinking, are worthy of pursuit.\textsuperscript{18}

3. PROPOSED STRESS REGIMES

Published data on the state of stress in Alberta tar sands is very limited. Brooker stated that horizontal stresses exceed the vertical stress by a factor of about three at the relatively shallow depths of open pit mining operations.\textsuperscript{2}

Dusseault presents theoretical stress distributions assuming (a) monotonic burial, (b) burial to 1000m, then excavation and (c) burial with 2000m of ice acting 50% effectively followed by removal and suggests that Brooker's ration of $\frac{\sigma_H}{\sigma_V} = 3.0$ is not unrealistic. The author further suggests that the depth at which the minor principal stress changes from vertical to horizontal is probably between 350 and 450 meters.\textsuperscript{5}
Tests performed in the McMurray formation southeast of Gregoire Lake over a perforated interval of 308 to 317m included an initial cold water fracture for stress measurement purposes. The results indicated that the cold water hydraulic fracture produced was vertical.12

Settari and Raisbeck present data from short term hydraulic fracture experiments and interpret that fractures in the Cold Lake deposit at depths of ≈ 420m are probably vertical while those in the Wabasca deposit at depths of ≈ 240m may be horizontal.19

4. INFLUENCE OF IN SITU STRESS REGIME ON IN SITU PROCESSES

Conventional hydraulic fracture operations are usually performed in rocks of high tensile strength and at considerable depth in a single well or in several widely spaced injection points. Injection volumes are generally small compared to the reservoir volume.7 The two major geometric characteristics of hydraulic fractures are the fracture plane orientation and the fracture propagation direction when a new increment of fluid is injected.

Theoretical and field studies suggest that the in situ stress regime forms the first order control on hydraulic fracture orientation and propagation direction.1,4,8,15,19 Given three orthogonal principal stresses a fracture will open against the direction of the least of these stresses and will propagate parallel to the largest stress.

Generally the success of in situ recovery methods in tar sands is dependent in large measure on the initial step of creating sufficient flow paths. In many cases the permeability enhancement is achieved by fracturing the formation.19 Naturally knowledge of fracture geometry, and hence the in situ stress, is critical to successful process planning and control. Furthermore for many hybrid processes, (i.e. MAISP), in situ stress measurements are also critical parameters for facility design.

For most proposed in situ production schemes in Alberta's oil sands, fractures will be generated by injection of fluid volumes of a significant percentage of the reservoir volume, from closely spaced injection points including many pressurization cycles and including large inputs of thermal energy. Oil sands are cohesionless materials and the depths at which injection will take place are relatively shallow (200-600m). There is evidence that the reservoir behavior is not adequately explained by conventional approaches as a result of gross system changes that may result from the injection process.7 The remainder of this paper deals with problems with interpretation of small volume stress measurement hydraulic fractures and in relating such measurements to large volume, high temperature process production fractures.

5. HYDRAULIC FRACTURE IN SITU STRESS MEASUREMENTS IN ALBERTA TAR SAND
5.1 IN SITU STRESS DETERMINATIONS BY HYDRAULIC FRACTURING

Unlike other methods of in situ stress determination, hydraulic fracturing does not explicitly require knowledge of mechanical properties of the "rockmass" under consideration. The parameters necessary for the computation of the stresses acting perpendicular to the borehole axis are measured directly. It has been shown numerous times, in both laboratory and field measurements of this technique that hydraulic fractures extend in the direction of the maximum in situ stress.\textsuperscript{11,15,20}

In situ stress determinations by hydraulic fracturing are similar to production-related hydraulic fracturing procedures with the exception that they are best performed using low rates, without the use of proppants in the fracturing fluid.

In its simplest form, hydraulic fracturing consists of sealing off a section of a wellbore and injecting fluid into this section until a fracture is created at the wellbore. This fracture is extended by continued injection of the fracturing fluid.

An idealized pressure-time record of fluid pressure in the sealed off section of a wellbore during hydraulic fracturing is shown in Figure 4. The breakdown pressure, $P_b$, is the pressure at which the fracturing occurs. The fracture extension pressure $P_f$, is the pressure occurring while fluid is injected into the fracture, causing it to extend. Finally, when fluid injection is discontinued, the pressure rapidly drops to a 'stable' value known as the instantaneous shut-in pressure, $P_s$. The instantaneous shut-in pressure corresponds to a state of quasi-static equilibrium between the in situ stress acting to close the fracture and the pressure in the fracture to hold it open.

Open hole hydraulic fracture stress measurements are conducted using a straddle packer configuration consisting of two rubber seal elements mounted a set distance apart on a steel mandrel. These elements "straddle" a zone to be fractured (Figure 5). The zone is isolated from the rest of the hole by inflating these sealing elements, forcing them against the borehole wall. This sealed-off zone can then be pressurized until hydraulically induced fractures occur and/or pre-existing discontinuities open up.

Fracture orientation at the wellbore can be determined using impression packers, Figure 6. Impression packers are inflatable thick-walled rubber elements, wrapped with soft, semi-cured rubber. Upon deflation the rubber wraps retain a negative image of the created fracture trace which can be oriented to obtain azimuth and dip of the fracture.

Hydraulic fracture stress measurements can also be performed through perforated casing, Figure 7. In such cases fracture orientation must be determined remotely.
5.2 EXISTING HYDRAULIC FRACTURE STRESS DATA

Hydraulic fracturing for in situ stress determination was performed in the Athabasca deposit during February, 1981, by tti GEOTECHnical resources ltd., as part of phase I of the Gulf/AOSTRA, MAISP project. Although the results are proprietary, they indicate a similar trend to those which can be estimated from the pressure-time records of Holzhausen,11 (Figure 8, Table 6). The time scale for the pressure time record for the above work, Figure 9, makes it difficult to estimate the shut-in pressure which is representative of the total minimum principal stress.

Before discussing the measured stress results further some key geotechnical characteristics of the tar sand deposits should be noted: 5,9

i) Tar sands are almost entirely cohesionless. The bitumen acts as as a very viscous Newtonian fluid and no grain-to-grain cementitious material is present.

ii) Resulting from over-consolidation and mild diagenesis the oil sands are denser than "normal" sands. They have an intimately interlocked (pressure solutioned) fabric, are extremely incompressible at high confining stress, show strain-weakening behavior and dilate considerably during shear at moderate stress levels.

iii) The viscous bitumen impedes fluid movement through the strata, but the bitumen viscosity is very temperature dependent. The absolute permeability of the sand is very high however due to its high porosity and lack of cement.

iv) As the result of the exsolution of pressure-dissolved gases, the fabric of the oil sand is grossly disturbed if confining stresses are removed rapidly. This phenomenon is observed even in cores from shallow depth and physical properties derived from laboratory analysis are thus very suspect.

Production fractures in the McMurray formation are performed in very bitumen rich "pay" zones. Such zones, without thermal stimulation, may generally be considered completely impermeable. Under such conditions the maximum horizontal stress is generally calculated using:

\[ \sigma_{HMAX} - 3\sigma_{HMIN} = \frac{Pb + To - Po}{3} \]

Where

- \( \sigma_{HMAX} \) - Maximum horizontal stress
- \( \sigma_{HMIN} \) - Minimum horizontal stress = \( Ps \)
- \( Ps \) - Instantaneous shut-in pressure
- \( Pb \) - Breakdown pressure
- \( To \) - Hydraulic fracture tensile strength
- \( Po \) - Pore pressure
Several interpretation problems, unique to the tar sands, are inherent in the analysis. One problem involves the value of pore pressure selected. The first results in Table 6 assume a hydrostatic pore pressure condition. However, Hackbarth, (Figure 10) predicted subhydrostatic pore pressures. Whether these pore pressures are real or only reflect DST's which are too short in duration is uncertain. However, if lower values of pore pressure are used significantly different stress conditions are indicated (Table 6, Figure 11). Maximum stress differences of this magnitude could have an influence on shaft and tunnel design in the tar sands.

However, fracturing through perforations and the relatively higher flow rates may suggest higher values for the maximum horizontal stress ($\sigma_v$ is estimated as 6.86 MPa on an average depth of 312 m and $\gamma_t \approx 2.2$ g/cm$^3$).

Holzhausen's data indicates a significant pressure drop after breakdown, (Figure 9), suggesting a marked hydraulic fracturing strength. Similar phenomena have been observed more recently by GEOTECH. However, as noted earlier, the tar sands are a completely cohesionless material ($T_o = 0$). Dusseault states that for carefully controlled hydraulic fracture stress measurements in cohesionless tar sand there should be very little difference between the breakdown pressure, fracture propagation pressure and the instantaneous shut-in pressure. Dusseault's statement presupposes that the in situ tar sand cannot sustain the stress concentration due to the wellbore itself. Existing data suggests, however, that either:

i) at least in some zones tar sand can withstand moderately high stress concentrations, or

ii) that some zones do possess an inherent tensile strength.

It is the author's opinion that:

i) is the more probable explanation.

As noted earlier bitumen rich tar sand zones are essentially impermeable without thermal stimulation. However it is possible for the bitumen to transmit pore pressure without allowing any significant leakoff. This factor could again affect the estimate of the maximum horizontal stress.

The in situ material properties of the tar sands are as yet poorly understood. Severe sample disturbance problems have not been adequately overcome and result in laboratory derived parameters being highly suspect. Equally uncertain is the degree and significance of wellbore damage, due to the same processes of gas exsolution and fabric disturbance, on downhole in situ measurements, including hydraulic fracture stress measurements. Hence the applicability of theories based on elasticity and fracture mechanics to this complex material remain uncertain.
Even with the above limitations the derived stresses correlate reasonably well with proposed theoretical stress regimes. The correlation with the situation of monotonic burial described by Dusseault, Figure 12, is poor. However Dusseault further stated "The preceding theoretical development of a stress field (sic monotonic burial) is highly unlikely in nature, even in the non-tectonic terrain within which the McMurray Formation is found." In this vein, Dusseault hypothesized two alternate loading situations involving stratigraphic burial-removal, and glacial loading. The predicted stresses from these models show somewhat better correlation to those measured (Figures 13 and 14). Recent measurements by GEOTECH bracket the fore-mentioned models even more closely. Comparisons of this nature are tenuous because it is not possible to ascertain a relationship between the present pore pressure and the "historical" pore pressure. For comparative purposes it is assumed that the "historical" pore pressure was equal to the "present" pore pressure plus a hydrostatic supplement associated with previous geological history.

The previous data indicates that $\sigma_H' = \sigma_V'$, suggesting that this zone is close to the transition where fractures change from vertical to horizontal.

The models proposed by Dusseault for sediment removal and glacial loading would seem to involve a loading situation as shown in Figure 15.

Figure 16 shows an idealized deposit of dry cohesionless sand with a horizontal ground surface. The sand extends infinitely in all directions. At point A in the interior of the deposit the vertical pressure on a horizontal plane is:

$$P_v = \gamma z$$

where

- $\gamma$ = unit weight of the deposit
- $z$ = depth

The horizontal pressure on vertical planes at Point A is considered to be:

$$P_h = K_0 P_v = K_0 \cdot \gamma \cdot z$$

where $K_0$ is known as the coefficient of earth pressure at rest (Figure 16). The value of $K_0$ depends on the relative density and method of deposition of the sand.

It is suggested that a more representative situation might be as shown in Figure 17.

The inset in Figure 17 is based on experimental work by Hendron. While it is appreciated that there are significant property differences the trend of some unloading of $\sigma_H$ is regarded as being important, specifically the variation of $K_0$ with the over-consolidation ratio (OCR).
For the depth where these stress measurements were made, the over-consolidation ratio is relatively small suggesting that the current $K_0$ value reflects only a modest increase from the initial $K_0$ value.

Tables 7 and 8 indicate potential stress regimes for:

i) 1000 m of stratigraphic burial followed by erosion considering $\sigma_H'/\sigma_V' = f(OCR)$

ii) 2000 m of glacial loading, considering $\sigma_H'/\sigma_V' = f(OCR)$

In each case the initial $K_0$ value was assumed to be equal to $(1 - \sin \phi)$. While the validity of this assumption cannot explicitly be established, the $K_0$ values investigated are within the feasible range of $\frac{\sigma_H}{\sigma_V} = \frac{\sigma}{1-\nu}$, representative of the horizontal component due to gravitational loading of a flatlying horizontal configuration.  

The preceding tables and figures indicate that due to the relatively small over-consolidation values at the depths under consideration, the stress elevation due to unloading is not going to produce a dramatic increase in the horizontal stress regime at the depths under consideration. General expectations of high stress values are often clouded by near surface measurements where the OCR is significantly higher. For example, as reported $K_0$ might approach 3 at shallower depths.  

The features which make predictions of the stress regime in this manner difficult, are:

i) assessing the initial value of $K_0$

ii) assessing the variation of $K_0$ with OCR

6. POTENTIAL INFLUENCE OF IN SITU PROCESSING ON THE IN SITU STRESS FIELD

Production fracture orientation, (vertical versus horizontal), is critical to in situ process design. Much of the tar sand deposits lie in a transitional zone where evidence suggests that the minor principal stress changes from vertical to horizontal.

It is evident that careful measurement of the in situ stress regime may have critical implications to successful process technology development. At present however there are a number of uncertainties in relating the stress field, measured using small volume cold water fractures, to what may take place during long term, high temperature, process steam fracturing.

Continued large volume injection of a liquid into a cemented reservoir by continuous fracture results in growth of fracture area as permitted by injection rate, liquid "bleed-off" rate, pressure
distribution of the fluid within the fracture, reservoir and fluid properties, and other factors until an equilibrium state is approached at which time little further fracture growth occurs. It has been stated that such equilibrium is doubtful in continuous fracture of the shallow oil sands because the processes of injection and reservoir heating may induce large stress changes and also perhaps cause changes in formation properties through heating, erosion and remolding of oil sands. The fracture fluid properties may also be altered as eroded sand and bitumen become entrained during injection. Figure 18 shows schematically an ideal pressure time curve for a fracture in brittle rock versus long term fracture behavior in tar sand.

Reported injection pressures for a long term injection experiment in a single well at 312 m depth showed an initial injection pressure of 5600 kPa (initial $K_0$ of 0.85). Gradual rises of injection pressure to values about 1.15 to 1.22 times overburden, followed by sudden drops to about overburden, occurred repeatedly over a period of several weeks, each build-up taking from one to three days at approximately constant injection rates (Figure 19).

These events are interpreted as vertical fracture development, stress build-up, and breakthrough in a horizontal or inclined fracture. Breakthrough releases the pressure on the vertical fracture, permitting closure and reinjection in the vertical fracture, followed by another cycle of gradual stress accretion and breakthrough.12

Based on published literature and on numerical studies, Dusseault concluded that for the shallow reservoirs of the oil sands where there are no dramatic anisotropic rock properties, vertical fractures would be generated only for limited fracture operations. Where massive injection volumes occur over long time periods at a large number of closely spaced injection wells, he concludes that horizontal fractures must be ultimately generated.6

Figure 20 shows an example of steam injection history for a Cold Lake well. One interpretation is that the wellhead pressure shows a proportional variation with injection rates. This would not be observed if most injection occurred through the fracture. Rather with increasing time the well response resembles that of an unfractured well due to thermal input which mobilizes a progressively larger area around the initial fracture path.19

An alternate interpretation of the data suggests that although there is some obvious correspondence between injection rate and well head pressure, careful examination suggest that rates and pressures are not totally coupled, and that the effect could be related to local wellbore stresses. A logical explanation of the increasing pressure is that increase of the minor principal stress is taking place because of injection of fluid in planes normal to that stress, and because of the input of thermal energy which causes one-dimensional thermal
expansion in the same direction as the minor principal stress.  

There is another uncertainty with respect to hydraulic fracture orientation in oil sands. If either one or two of the other principal stresses are very similar in magnitude to the value of the minor principal stress, then lithologic variability, bedding structures, or small variations in stress direction will have a significant effect on fracture orientation and fracture propagation direction. It is quite probable that if a vertical fracture were initially generated and propagated upwards, it could become horizontal or at least begin curving over in that zone where minor principal stress direction changes over to vertical. This latter hypothesis may have important consequences as vertical fractures would not be expected to break directly upwards to the surface because of high lateral stresses near the surface.

Finally it is extremely important to recognize the difference between Darcy flow and fracture flow during stimulation, injection, and production cycles. A well production situation can take advantage only of Darcy flow towards the wellbore if no propped fracture exists, although gradients can be significant. Fissure flow, which takes place during hydraulic fracture operations (stimulation or injection), is not necessarily reversible when wells are drawn down because fractures reheat (and clog with bitumen) and only Darcy flow can occur. Once the driving force (injection) which has created the fracture ceases, fractures close and flow conditions may become dramatically different.

7. SUMMARY

The Alberta tar sands are a unique, four phase, material. Recovery of the majority of the deposit requires the development of successful in situ processing techniques. A thorough understanding of the existing stress state in situ and its interaction with process technology is critical to efficient bitumen recovery. Interpretation of in situ stress based on classical low volume cold water hydraulic fracture tests is plagued with numerous problems resulting from the unique nature of the material. The most critical problems result from:

i) poor understanding of pore pressure effects due to the presence of highly viscous bitumen,

ii) observation of apparent hydraulic fracture strength in a cohesionless (To = 0) material,

iii) non-reliable material properties due to (at present) unavoidable sample disturbance,

iv) unknown amount and effect of wellbore damage, and

v) uncertainty of applicability of elasticity and fracture mechanics theories to tar sand materials.
Compounding the above problems in interpretation of the existing stress field, the interaction of the stress field with large volume, high temperature process production fracturing is also a matter of debate. Published data suggests that large volume process steam stimulation may sufficiently influence the in situ stress state to alter fracture orientation. Certain in situ processes require either communicating horizontal fractures or non-communicating vertical fractures for success. Continued research into the above areas is imperative to the ultimate development of this extensive resource.
ACKNOWLEDGEMENTS

The author wishes to thank Gulf Canada Resources Ltd. for permission to publish this paper, based in part on experience gained in Phase I of the Gulf/AOSTRA, MAISP experiment. The author expresses special appreciation to Dr. D. Devenney of Gulf Canada Resources for his guidance and helpful comments throughout the project and in review of this paper. The author further wishes to acknowledge Dr. J. McLennan of Dowell Inc., (formerly of tti GEOTEchnical resources ltd.), who analyzed the field data.
REFERENCES


### Updated Proved In Place Reserves in The Cretaceous Oil Sand Deposits

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"The Oil Sands of Canada — Venezuela" — 1977

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| BULK DENSITY | 2.05 - 2.18 g/cm³ | Saturated bulk density for well-sorted, oil-rich sand. |
| | 2.15 - 2.27 g/cm³ | Saturated bulk density for variable oil content poorly sorted and silty sands. |
| | 2.24 - 2.40 g/cm³ | Saturated bulk density for oil-poor and oil-free silty sands, silts and silty clays. |

| POROSITY | 28 - 36% | Oil-rich, well-sorted sands. |
| | 23 - 30% | Variable oil content, poorly sorted and silty sands. |
| | 15 - 25% | Oil-poor and oil-free silty sands, silts and silty clays. |

| VOID RATIO | 0.39 - 0.56 | Oil-rich, well-sorted sands. |
| | 0.30 - 0.43 | Variable oil content, poorly sorted and silty sands. |
| | 0.18 - 0.33 | Oil-poor and oil-free silty sands, silts and silty clays. |

| SATURATION | 0.94 - 1.0 | Oil-rich, well-sorted sands. |
| | 0.93 - 1.0 | Variable oil content, poorly sorted and silty sands. |
| | 0.89 - 1.0 | Oil-poor and oil-free silty sands, silts and silty clays. |

| BITUMEN CONTENT | 12% - 16% | Oil-rich, well-sorted sands. |
| | 0% - 10% | Variable oil content, poorly sorted and silty sands. |
| | 0% - 4% | Oil-poor and oil-free silty sands, silts and silty clays. |

| COMPRESSIBILITY (300 m depth) | $10^{-6} - 10^{-7}$ kPa⁻¹ |

| YOUNG'S MODULUS (300 m depth) | $0.2 - 3.0$ GPa |
| For $o' = 1$ MPa | $E = 0.3$ GPa |
| For $o' = 4$ MPa | $E = 1.1$ GPa |

| SHEAR STRENGTH (TRIAXIAL) | For $o'_n = 1$ MPa | $T = 1.2 - 2$ MPa |
| For $o'_n = 4$ MPa | $T = 6$ MPa |

| (DIRECT SHEAR) | For $o'_n = 1$ MPa | $T = 0.8 - 1.4$ MPa |
| For $o'_n = 4$ MPa | $T = 2.4 - 3.8$ MPa |

| PERMEABILITY | Highly variable up to 50 md. |

| BITUMEN PROPERTIES |
| a) Specific Gravity | 1.002 - 1.027 |
| b) Viscosity | $0.6 - 60$ kPa·s |
| c) Specific Heat | $0.35$ cal/gm°C |
| d) Calorific Value | $17,900$ BTU/lb. |
| e) Pressure Solubility | $0.067$ kPa⁻¹ (OIL), $0.0005$ kPa⁻¹ (H₂O) |
| f) Temperature Solubility | $0.007$ (OIL), $0.014$°C⁻¹ (H₂O) |

| THERMAL PROPERTIES |
| a) Specific Heat of Mineral Matter | $0.18$ cal/gm°C |
| b) Specific Heat of Formation | $0.22$ cal/gm°C |
| c) Thermal Conductivity | $k = 1.27 - 2.25 + 0.39 S_s + 0.74 W/m°C$ |

256
<table>
<thead>
<tr>
<th>Athabasca In-Situ Pilots — Active and Announced</th>
</tr>
</thead>
<tbody>
<tr>
<td>Name:</td>
</tr>
<tr>
<td>Process:</td>
</tr>
<tr>
<td>Reservoir:</td>
</tr>
<tr>
<td>Overburden/Pay (ft)</td>
</tr>
<tr>
<td>Pattern:</td>
</tr>
<tr>
<td>Size (acres/well)</td>
</tr>
<tr>
<td>Cost: ($)</td>
</tr>
</tbody>
</table>

(1) Plus Pacific Petroleum
(2) With Canada-Cities Service and Imperial Oil

<table>
<thead>
<tr>
<th>Athabasca In-Situ Pilots Completed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date</td>
</tr>
<tr>
<td>Combustion Pilots</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
</tbody>
</table>

*Overburden/Pay (ft)
**TABLE IV (cont'd)**

<table>
<thead>
<tr>
<th>Cold Lake In-Situ Pilots — Active</th>
</tr>
</thead>
<tbody>
<tr>
<td>Name: Ethel Pilot</td>
</tr>
<tr>
<td>Imperial Oil</td>
</tr>
<tr>
<td>Murphy Oil Co.</td>
</tr>
<tr>
<td>Norcen Energy:</td>
</tr>
<tr>
<td>Union Texas</td>
</tr>
<tr>
<td>Name: Imperial Oil</td>
</tr>
<tr>
<td>Start Date:</td>
</tr>
<tr>
<td>Process: Huff &amp; Puff, Steam</td>
</tr>
<tr>
<td>Stimulation</td>
</tr>
<tr>
<td>Reservoir: Formation Depth/Pay ft.</td>
</tr>
<tr>
<td>Pattern: Type</td>
</tr>
<tr>
<td>Size (acres/well)</td>
</tr>
<tr>
<td>Cost ($)</td>
</tr>
<tr>
<td>(1) With Japan Oil Sands Company</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Cold Lake In-Situ Pilots — Announced and Active with Expansion Plans</th>
</tr>
</thead>
<tbody>
<tr>
<td>Name: Marguerite Lake</td>
</tr>
<tr>
<td>Status: Start spring 1978</td>
</tr>
<tr>
<td>Process: Steam Stimulation and Combustion</td>
</tr>
<tr>
<td>Reservoir: Formation Depth/Pay (ft)</td>
</tr>
<tr>
<td>Pattern: Type</td>
</tr>
<tr>
<td>Size (acres/well)</td>
</tr>
<tr>
<td>Cost ($)</td>
</tr>
<tr>
<td>(1) Plus Hudson's Bay Oil and Gas and PanCanadian</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Cold Lake In-Situ Pilots — Completed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date</td>
</tr>
<tr>
<td>-----------------------</td>
</tr>
<tr>
<td>Imperial Oil</td>
</tr>
<tr>
<td>Ethel Pilot</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>BP Exploration</td>
</tr>
<tr>
<td>Triad Pilots Phase 1, 2</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Supertest - Grand Centre</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Great Plains</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Hudson’s Bay Oil &amp; Gas</td>
</tr>
<tr>
<td>Frenchman’s Butte</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Chevron Canada</td>
</tr>
<tr>
<td>Beaver Crossing</td>
</tr>
</tbody>
</table>

*Overburden Pay-ft.*
### TABLE IV (cont’d)

**Peace River In-Situ Pilots — Announced and Completed**

<table>
<thead>
<tr>
<th>Name:</th>
<th>Process:</th>
<th>Reservoir: Formation</th>
<th>Depth/Pay ft.</th>
<th>Pattern: Type</th>
<th>Size (acres/well)</th>
<th>Cost ($)</th>
<th>Status:</th>
<th>Company:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peace River In-Situ Pilot</td>
<td>Steam Pressure Depressurization in underlying water zone</td>
<td>Upper Bullhead</td>
<td>1800-90</td>
<td>7 - 7 spots</td>
<td>2.33</td>
<td>58 MM Phase I</td>
<td>Planning</td>
<td>Shell Canada Ltd.</td>
</tr>
<tr>
<td>Shell Canada Ltd.</td>
<td>— Steam Injection Drive — Combustion</td>
<td>Upper Bullhead</td>
<td>1800-90</td>
<td>2 wells</td>
<td>2 wells</td>
<td>1 well</td>
<td>2 wells</td>
<td></td>
</tr>
</tbody>
</table>

**Wabasca and Grosmont In-Situ Pilots**

<table>
<thead>
<tr>
<th>Name:</th>
<th>Process:</th>
<th>Reservoir: Formation</th>
<th>Depth/Pay ft.</th>
<th>Pattern: Type</th>
<th>Size (acres/well)</th>
<th>Cost ($)</th>
<th>Status:</th>
<th>Company:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wabasca Pilot</td>
<td>Cyclic Steam Stimulation, Steam Flooding, Solvents, Combustion</td>
<td>Grand Rapids A</td>
<td>800-40</td>
<td>3 - five spots</td>
<td>0.2 - 1.25</td>
<td>8 MM</td>
<td>Present</td>
<td>Gulf Oil Canada</td>
</tr>
<tr>
<td>Chipewyan River</td>
<td>Cyclic Steam Stimulation</td>
<td>Grosmont Carbonate</td>
<td>700-1200</td>
<td>3 wells</td>
<td>3 wells</td>
<td>2.1MM</td>
<td>1975-1977</td>
<td>Union Oil Co. of Canada</td>
</tr>
<tr>
<td>Buffalo Creek</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1977</td>
<td></td>
</tr>
</tbody>
</table>

**Summary — Current, Constructing and Planned In-Situ Pilots**

<table>
<thead>
<tr>
<th>Athabasca</th>
<th>Cold Lake</th>
<th>Peace River</th>
<th>Wabasca</th>
<th>Grosmont</th>
</tr>
</thead>
<tbody>
<tr>
<td>AOSTRA/Amoco</td>
<td>AOSTRA BP Exploration</td>
<td>AOSTRA Shell Can.</td>
<td>Gulf Oil</td>
<td>Union Oil</td>
</tr>
<tr>
<td>AOSTRA/In-Situ R&amp;E</td>
<td>Chevron Canada</td>
<td>AOSTRA/Numac</td>
<td>Gulf Oil</td>
<td></td>
</tr>
<tr>
<td>AOSTRA/Numac</td>
<td>Imperial Oil</td>
<td>Petro-Canada Expl.</td>
<td>Murphy Oil</td>
<td></td>
</tr>
<tr>
<td>Petro-Canada Expl.</td>
<td>Norcen Energy</td>
<td>Texaco Exploration</td>
<td>Union Texas</td>
<td></td>
</tr>
<tr>
<td>WECO Development</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Investigations</th>
<th>Test Wells</th>
<th>Depth Range (feet)</th>
<th>Total Number Investigations</th>
<th>Total Number Wells</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>55+</td>
<td>200-1600</td>
<td>16</td>
<td>266+</td>
</tr>
<tr>
<td>1</td>
<td>31</td>
<td>1120-1200</td>
<td>1800</td>
<td>1365</td>
</tr>
<tr>
<td>1</td>
<td>11</td>
<td>1350</td>
<td>700-1200</td>
<td>700-1200</td>
</tr>
</tbody>
</table>

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TABLE V
CLASSIFICATION OF UNIT STEPS
FOR IN SITU RECOVERY METHODS

1. ACCESS
A. VERTICAL/SLANT WELLS
B. HORIZONTAL WELLS

2. COMMUNICATION INITIATION
A. NATURAL PERMEABILITY
   (E.G., BASAL WATER SAND)
B. FRACTURING WITH STEAM
C. FRACTURING WITH AIR
D. FRACTURING WITH COLD WATER
E. ELECTROLINKING
F. SONICS

3. COMMUNICATION DEVELOPMENT
A. ELECTROLINKING
B. COLD EMULSIFICATION
C. COLD SOLVENT
D. STEAM AND SOLVENT
E. STEAM AND GAS
F. STEAM AND EMULSIFIER
G. AIR–COMBUSTION

4. VISCOSITY REDUCTION
A. STEAM AND/OR HOT WATER
B. STEAM PLUS ADDITIVE
C. SOLVENT
D. AIR–COMBUSTION

5. DISPLACEMENT
A. STEAM
B. AIR–COMBUSTION

NO. OF POSSIBLE COMBINATION PROCESSES:
\[ (-2 \times 6 \times 7 \times 4 \times 2) = 672 \]
(NOT ALL COMBINATIONS ARE PRACTICAL)
(After Raisbeck & Card, 1978)

TABLE 6
STRESS CALCULATIONS FOR HOLZHAUSEN et al., 1980

<table>
<thead>
<tr>
<th>PARAMETER</th>
<th>Value (MPa)</th>
<th>Based on Hydrostatic Pore Pressure</th>
<th>Based on Measured Pressure Heads</th>
</tr>
</thead>
<tbody>
<tr>
<td>( P_0 )</td>
<td>3.12</td>
<td></td>
<td>2.0</td>
</tr>
<tr>
<td>( P_{b1} ) (D.H.) *1</td>
<td>9.6</td>
<td></td>
<td>9.6</td>
</tr>
<tr>
<td>( P_s ) (D.H.)</td>
<td>5.2</td>
<td></td>
<td>5.2</td>
</tr>
<tr>
<td>( P_{prop} ) (D.H.)</td>
<td>-5.5</td>
<td></td>
<td>-5.5</td>
</tr>
<tr>
<td>( \sigma_{HMIN} )</td>
<td>5.2</td>
<td></td>
<td>5.2</td>
</tr>
<tr>
<td>( \sigma_{HMAX} )</td>
<td>-7.0</td>
<td></td>
<td>-6.0</td>
</tr>
<tr>
<td>( \sigma_v )</td>
<td>-6.86</td>
<td></td>
<td>-6.86</td>
</tr>
<tr>
<td>( \sigma_{HMIN}/\sigma_v )</td>
<td>.76</td>
<td></td>
<td>.76</td>
</tr>
<tr>
<td>( \sigma_{HMAX}/\sigma_v )</td>
<td>1.02</td>
<td></td>
<td>1.17</td>
</tr>
</tbody>
</table>

*1 D.H. = Down Hole
*2 Pore Pressure estimated from Hachbarth (1970)
### Table 7

**STRATIGRAPHIC BURIAL - REMOVAL (1000 m)**

<table>
<thead>
<tr>
<th>DEPTH (m)</th>
<th>$\phi$</th>
<th>$\sigma_v$ MPa</th>
<th>$\sigma_v'$ MPa</th>
<th>$K_O$ (loading)*1</th>
<th>OCR</th>
<th>$K_O$ (unloading)*2</th>
<th>$\sigma_H'$ MPa</th>
<th>$\sigma_H$ MPa</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>312</td>
<td>30</td>
<td>6.86</td>
<td>3.74</td>
<td>0.5</td>
<td></td>
<td></td>
<td>4.21</td>
<td>1.0</td>
<td>3.74 6.86</td>
</tr>
<tr>
<td>1312</td>
<td>30</td>
<td>28.86</td>
<td>15.74</td>
<td>0.5</td>
<td></td>
<td></td>
<td>4.21</td>
<td>1.0</td>
<td>3.74 6.86</td>
</tr>
<tr>
<td>312</td>
<td>30</td>
<td>6.86</td>
<td>3.74</td>
<td>0.36</td>
<td>4.21</td>
<td>0.72</td>
<td>2.69</td>
<td>5.81</td>
<td>312 m burial</td>
</tr>
<tr>
<td>1312</td>
<td>40</td>
<td>28.86</td>
<td>15.74</td>
<td>0.36</td>
<td>4.21</td>
<td>0.72</td>
<td>2.69</td>
<td>5.81</td>
<td>312 m burial</td>
</tr>
<tr>
<td>312</td>
<td>40</td>
<td>6.86</td>
<td>3.74</td>
<td>0.23</td>
<td>4.21</td>
<td>0.46</td>
<td>1.72</td>
<td>4.84</td>
<td>312 m burial</td>
</tr>
<tr>
<td>1312</td>
<td>50</td>
<td>28.86</td>
<td>15.74</td>
<td>0.23</td>
<td>4.21</td>
<td>0.46</td>
<td>1.72</td>
<td>4.84</td>
<td>312 m burial</td>
</tr>
<tr>
<td>312</td>
<td>50</td>
<td>6.86</td>
<td>3.74</td>
<td>0.23</td>
<td>4.21</td>
<td>0.46</td>
<td>1.72</td>
<td>4.84</td>
<td>312 m burial</td>
</tr>
</tbody>
</table>

*1 - Based on $1 - \sin\phi$

*2 - Estimated increase in $K_O$ on unloading for an OCR of 4 - 5

(All analysis assume Hydrostatic pore pressures)
<table>
<thead>
<tr>
<th>Depth</th>
<th>$\phi$ (°)</th>
<th>$\sigma'_v$ (MPa)</th>
<th>$\sigma'_v$ (MPa)</th>
<th>$K_o$ (loading)$^1$</th>
<th>OCR</th>
<th>$K_o$ (unloading)$^2$</th>
<th>$\sigma'_h$ (MPa)</th>
<th>$\sigma_h$ (MPa)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>312</td>
<td>30</td>
<td>6.86</td>
<td>3.74</td>
<td>.50</td>
<td></td>
<td>3.5</td>
<td>0.90</td>
<td>3.37</td>
<td>6.49</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>6.86</td>
<td>3.74</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>312 m burial After 2000 m glaciation</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>6.86</td>
<td>3.74</td>
<td>.36</td>
<td></td>
<td>3.5</td>
<td>0.65</td>
<td>2.43</td>
<td>5.55</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>6.86</td>
<td>3.74</td>
<td></td>
<td></td>
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<td></td>
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<td>312 m burial After 2000 m glaciation</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>6.86</td>
<td>3.74</td>
<td>.23</td>
<td></td>
<td>3.5</td>
<td>0.41</td>
<td>1.53</td>
<td>4.65</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>6.86</td>
<td>3.74</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>312 m burial After 2000 m glaciation</td>
</tr>
</tbody>
</table>

*1 - Based on $1 - \sin \phi$

*2 - Based on an estimated 80% increase in $K_o$ on unloading for these OCR values

(All analysis assume hydrostatic pore pressures)
Figure 3. Viscosity-Temperature Relationship of Bitumen.
(After Nicholls et al., 1977)
FIGURE 4
IDEALIZED PRESSURE-TIME CURVE

FIGURE 5
INFLATABLE STRADDLE PACKER CONFIGURATION

CROSSOVER SUB TO DRILL PIPE

UPPER PACKER

FLUID PORTS

LOWER PACKER

INSTRUMENTATION "BOMB" WAS ATTACHED HERE
Figure 6: Schematic representation of the impression packer configuration.
Figure 7: Schematic of Typical Downhole Test and Instrument Arrangement N.T.S.

- Tubing String to surface
- Schlumberger conducting Wireline
- Compression Packer
- Schlumberger HMS-A Downhole Pressure - Temperature Recorder
- Perforations (two at 90° Phasing)
- Casing
- Bridge Plug
- Amerata Pressure Gauge

T.D.
FIGURE 8: STRESS REGIME BASED ON THE ASSUMPTION OF A HYDROSTATIC PORE PRESSURE DISTRIBUTION

Pressure head versus depth - Athabasca Oil Sands Area

Figure 10: (After Hackbarth, 1970)

Predicted stress conditions

Figure 11: Predicted stress conditions
FIGURE 13: STRESS REGIME FOR 1000 m BURIAL-REMOVAL (MODIFIED AFTER DUSSEAULT, 1977)

- D - Maximum historical vertical stress
- H - Present effective horizontal stress
- G - Present effective vertical stress
- D - Maximum historical vertical stress
- A - Maximum historical vertical stress for 1000 m burial-removal

V - Vertical stress for 1000 m burial-removal (approximation)
E - Effective horizontal stress
F - Effective horizontal stress
FIGURE 14  STRESS REGIME FOR GLACIAL LOADING (MODIFIED AFTER DUSSEAULT, 1977)

FIGURE 15

UNLOADING

LOADING
Figure 16 Coefficient of Earth Pressure at rest (Peck et al, 1974)

Lateral stress during one-dimensional compression. Minnesota sand: \( k_0 = 0.62, D = 0.34 \). (From Henderson, 1963.)
Ideal P-Time Curve for Brittle Rock

P-T Curves in Oil Sand

Pressure–Time Behavior of Oil Sands During Fracture

Figure 18
Figure 19: Cyclic Pressure Drops During Steam Injection (only 4 of 8 events represented)
FIGURE 20
PRESSURE - INJECTION HISTORY DURING INITIAL CYCLE OF STEAM INJECTION
A COMPARATIVE STUDY OF DEEP HYDROFRACTURING AND
OVERCORING STRESS MEASUREMENTS AT SIX
LOCATIONS WITH PARTICULAR INTEREST TO THE
NEVADA TEST SITE

Bezalel C. Haimson
University of Wisconsin
1509 University Avenue
Madison, Wisconsin 53706
TOPIC II

COMPARISON OF STRESS MEASUREMENT
TECHNIQUES AND RELATED TOPICS

MODERATOR - W. F. BRACE
ABSTRACT

Six case histories are described in which deep hole hydrofracturing stress measurements were compared with independently conducted overcoring tests. All the comparisons show good to excellent agreement with respect to both stress magnitudes and directions.

Of particular interest is the case of the Nevada Test Site. There, two tunnel complexes were used for a total of four sets of in situ stress measurements, two by overcoring and two by hydrofracturing. The results of the independently conducted tests show very close agreement between the measured stresses. In addition, focal mechanism solutions confirm the results with respect to both stress directions and relative magnitudes.

The comparisons of all the reported tests clearly demonstrate the ability of the hydrofracturing technique to obtain a reliable estimate of the in situ state of stress.
INTRODUCTION

Hydrofracturing as a method of estimating the stress regime in rock has been treated theoretically [e.g. 1, 2, 3, 4] and tested in the laboratory [e.g., 5, 6]. Moreover, in most published cases of field hydrofracturing measurements the results appear plausible [7, 8, 9, 10, 11]. The problem is, however, that there is no direct way to verify field results. The farfield boundary conditions are typically unknown, and no theory or model in existence to date can predict the state of stress at a point in rock. We can obtain some indication of the reliability of results from the consistency of different measurements in the same hole, or the agreement between tests in adjacent holes; in addition, nearby geologic structures such as faults, dikes or folds can provide a qualitative check to measured stresses. These and other devices cannot, however, unequivocally determine whether the measured values represent the existing stresses. One way of confirming the validity of results is to compare the stresses derived by one method with those independently determined by another. If the agreement is close and not coincidental it can be used as evidence that both method appear to measure the real stress condition at the site.

The hydrofracturing technique, which has rapidly developed in this country as the primary method for measuring stresses as great depths, has been directly compared with other methods in only a few cases. These comparisons, we believe, serve to reinforce the general reliability of hydrofracturing, while pointing out that it is not a precision instrument. The present paper is a report on those sites where two or more sets of in situ stress measurements were conducted and includes a discussion on the quality of the comparisons. The hydrofracturing procedure used in all cases has been described elsewhere [7, 8, 12].

Particular emphasis is placed on the Nevada Test Site experience since no fewer than four sets of measurements, each by an independent group, have been conducted there over the years (two sets of hydrofracturing and two of overcoring). In addition, indirect verification of results with respect to stress directions and relative magnitudes has been provided by focal mechanism solutions of local earthquakes.

THE NEVADA TEST SITE

The stress measurements reported here were conducted in the Rainier Mesa of the Nevada Test Site. The Mesa is a flat-topped mountain, approximately rectangular in shape with dimensions of 4.5 km length and 2 km width, and bounded by steep cliffs on all four sides. The approximate coordinates are 37° 11'N, 116° 13'W, and the elevation is about 2170 m, some 300-400 m above the background. The Rainier Mesa consists of some 450 m thick series of tuff and tuffaceous sandstone overlying massive Paleozoic rocks. A complex of tunnels has been excavated in the tuff for the purpose of underground nuclear weapons testing.
Chronologically, the Nevada Test Site (NTS) provided the first direct comparison between the hydrofracturing and the overcoring stress measuring methods [13]. The NTS has also been the location of, perhaps, the most comprehensive series of stress measurements anywhere in the world. Between 1970 and 1980 four series of tests were conducted. The first two series were carried out in tunnel complex U12n. Hooker et al [14] conducted overcoring tests using the Bureau of Mines borehole deformation gage, and Haimson et al. [13] conducted hydrofracturing stress measurements. The second group of tests was carried out in tunnel complex U12g, some 3 km south of U12n. Overcoring tests in U12g were run by Ellis and Ege [15]. Hydrofracturing tests in U12g were carried out intermittently between 1974 and 1980, constituting probably the most complete series of such tests ever undertaken, and have been recently reported by Smith et al [16]. In addition to the four sets of in situ measurements there has been a number of focal mechanism solutions worked out for seismic events in the general area of Nevada Test Site. Most of these solutions have been reported by Smith and Lindh [17].

Tunnel Complex U12n

I. Hydrofracturing

A total of 12 hydrofracturing tests were conducted in the tuff of the Rainier Mesa in the vicinity of tunnel complex U12n [13], one test in each of the two horizontal and three vertical holes drilled from the tunnel to a depth of some 25m, and seven additional tests in a 250m vertical hole drilled from the slope of the Mesa in the same general area.

Figure 1 summarizes the hydrofracturing stress results. Impression packer tests in the tunnel area yielded traces of vertical fractures oriented at N35°E. Stress magnitudes increase gradually with depth (taken from the top of the Mesa). The two horizontal principal stresses were directly determined by hydrofracturing. The vertical stress was calculated based on the density of the tuff. The measurements in the vicinity of the tunnel which were conducted in different boreholes (depths of 380 and 410m in Figure 1) yielded similar stress values, which also coincided with those determined in the deep vertical hole. This consistency of results from tests run in separate holes provided strong confidence in the values obtained. Based on linear regression analysis the variation of the measured stresses with depth within the range of 230-410 m is given by:

\[
\begin{align*}
\sigma_1 & \equiv \sigma_{Hmax} = 1 + 0.021 D \quad \text{at N35°E} \\
\sigma_2 & \equiv \sigma_V = 0.018 D \\
\sigma_3 & \equiv \sigma_{Hmin} = -1 + 0.012 D \quad \text{at N55°W}
\end{align*}
\]

where stresses are in megapascals $\sigma_1$, $\sigma_2$, $\sigma_3$ are the principal compressive stresses in order of decreasing magnitudes, $\sigma_V$, $\sigma_{Hmin}$, $\sigma_{Hmax}$ are the vertical and horizontal secondary principal stresses, and $D$ is depth in meters.
FIGURE 1. Variation with depth of stress magnitudes and $\sigma_{H_{\text{max}}}$ direction at the Nevada Test Site, and overcoring results at 380m depth (D is depth in meters).
II. Overcoring

Several years prior to the hydrofracturing tests the Bureau of Mines had conducted a series of overcoring borehole-deformation-gage measurements in the same tunnel (U12n) using short holes drilled in different directions [14]. They obtained a complete stress tensor for the vicinity of the tunnel at a depth of 380m below the mesa cap:

\[
\begin{align*}
\sigma_1 &\approx \sigma_{H_{\text{max}}} = 8.5 \text{ MPa at N}47^\circ\text{E/110}^\circ \\
\sigma_2 &\approx \sigma_{V} = 5.7 \text{ MPa at N}42^\circ\text{E/20}^\circ \\
\sigma_3 &\approx \sigma_{H_{\text{min}}} = 3.5 \text{ MPa at N}44^\circ\text{W/90}^\circ
\end{align*}
\]  

(2)

where the first angle represents the bearing, and the second angle is the inclination measured from the downward vertical.

III. Comparison of Results

Table 1 and Figure 1 provide a direct comparison between the hydrofracturing and overcoring results. Overcoring suggests that the principal stresses act in planes that are somewhat inclined to the horizontal and vertical (up to 20°). Hydrofracturing appears insensitive to such inclination. However, the overcoring horizontal and vertical secondary principal stresses presented in Table 1 and Figure 1 match surprisingly well, both in magnitude and direction, the hydrofracturing results. Table 1 shows that the difference in stress magnitudes measured by the two methods at the tunnel level is only 1 MPa. This close agreement is excellent in field tests in general, and in comparing results of two entirely different methods in particular. Moreover, the maximum horizontal stress directions are practically identical (10° difference).

Tunnel Complex U12g

I. Hydrofracturing

Subsequent to the tests in tunnel U12n, Sandia National Laboratories undertook a program of detailed hydrofracturing measurements in tunnel complex U12g, including "mineback" detection of fractures away from the testhole. More than 100 hydrofractures were conducted. Summarizing the hydrofracturing stress magnitudes and directions obtained in Tunnel U12g at a location under the mesa cap (426m below the surface) where overcoring tests were also conducted, Smith et al [16] report:

\[
\begin{align*}
\sigma_1 &\approx \sigma_{H_{\text{max}}} = 7.5 \text{ MPa at N}40^\circ\text{E/90}^\circ \\
\sigma_2 &\approx \sigma_{V} = 7.3 \text{ MPa at N}50^\circ\text{W/0-7}^\circ \\
\sigma_3 &\approx \sigma_{H_{\text{min}}} = 3.0 \text{ MPa at N}50^\circ\text{W/83-90}^\circ
\end{align*}
\]  

(3)
Table 1. Nevada Test Site, Tunnel 12n
Stress Comparison at Tunnel Level (380 m depth)*

<table>
<thead>
<tr>
<th>Stress</th>
<th>Hydrofrac(HF)</th>
<th>Overcoring(OC)</th>
<th>HF-OC</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MPa/Direction</td>
<td>MPa/Direction</td>
<td>MPa/Direction</td>
</tr>
<tr>
<td>$\sigma_{Hmax}$</td>
<td>9 / N35°E</td>
<td>8 / N45°E</td>
<td>1 / 10°</td>
</tr>
<tr>
<td>$\sigma_{Hmin}$</td>
<td>3.5 / N55°W</td>
<td>2.5 / N45°W</td>
<td>1 / 10°</td>
</tr>
<tr>
<td>$\sigma_V$</td>
<td>7</td>
<td>6</td>
<td>1</td>
</tr>
</tbody>
</table>

*Given in terms of secondary principal stresses in the horizontal and vertical planes.

Table 2. Nevada Test Site, Tunnel U12g
Stress Comparison at Tunnel Level (426 m depth)*

<table>
<thead>
<tr>
<th>Stress</th>
<th>Hydrofrac(HF)</th>
<th>Overcoring(OC)</th>
<th>HF-OC</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MPa/Direction</td>
<td>MPa/Direction</td>
<td>MPa/Direction</td>
</tr>
<tr>
<td>$\sigma_{Hmax}$</td>
<td>7.5 / N40°E</td>
<td>8.5 / N22°E</td>
<td>1.0 / 18°</td>
</tr>
<tr>
<td>$\sigma_{Hmin}$</td>
<td>3.0 / N50°W</td>
<td>2.6 / N68°W</td>
<td>0.4 / 18°</td>
</tr>
<tr>
<td>$\sigma_V$</td>
<td>7.3</td>
<td>6.8</td>
<td>0.5</td>
</tr>
</tbody>
</table>

*Given in terms of secondary principal stresses in the horizontal and vertical planes.

Table 3. Nevada Test Site - Focal Mechanism Solutions

<table>
<thead>
<tr>
<th>Location</th>
<th>$\sigma_{Hmin}$ Direction</th>
<th>$\sigma_{Hmax}$ Direction</th>
<th>Inferred Relative Stress Magnitudes</th>
</tr>
</thead>
<tbody>
<tr>
<td>37.2°, 116.5°</td>
<td>N45°W</td>
<td>N45°E</td>
<td>$\sigma_{Hmax} \geq \sigma_V &gt; \sigma_{Hmin}$</td>
</tr>
<tr>
<td>37.2°, 116.5°</td>
<td>N45°W</td>
<td>N45°E</td>
<td>$\sigma_{Hmax} \approx \sigma_V &gt; \sigma_{Hmin}$</td>
</tr>
</tbody>
</table>
The principal stresses act in the vertical and horizontal planes approximately, with the intermediate component subvertical and the largest compression horizontal at N40°E.

II. Overcoring

An overcoring technique was used at a point in Tunnel U12g in order to estimate the stress tensor by an independent method [15]. The results are:

\[
\begin{align*}
\sigma_1 &\approx \sigma_{\text{Hmax}} = 8.5 \text{ MPa at N22°E/90°} \\
\sigma_2 &\approx \sigma_{\text{V}} = 6.8 \text{ MPa at N83°W/7°} \\
\sigma_3 &\approx \sigma_{\text{Hmin}} = 2.6 \text{ MPa at N68°W/83°}
\end{align*}
\]

III. Comparison of Results

The overcoring results in U12g as presented above suggest that two of the principal stresses deviate by 7° from the vertical and the horizontal planes. Hydrofracturing yielded very similar results, showing that when a sufficient number of tests are run and sophisticated means of detecting fracture directions are used (such as "mineback") hydrofracturing can be as sensitive as any instrumental method (e.g. overcoring) to minor deviations of stress inclination.

Table 2 compares the overcoring and hydrofracturing results in Tunnel U12g in terms of the secondary horizontal and vertical principal stresses. The table highlights the excellent agreement between the two methods both with respect to directions (within 18°), and with respect to magnitudes (within 1.0 MPa).

Focal Mechanism Solutions at Nevada Test Site

A large number of focal mechanism solutions have been worked out for the area of Nevada around NTS. Most of these are reported by Smith and Lindh [17] and appear to be rather consistent both with respect to inferred horizontal stress directions (\(\sigma_{\text{Hmax}}\) at NE, \(\sigma_{\text{Hmin}}\) at NW) and with respect to the relative magnitudes (\(\sigma_{\text{V}} \approx \sigma_{\text{Hmax}} > \sigma_{\text{Hmin}}\)).

Two focal mechanisms for the actual area of the Nevada Test Site were reported earlier by Hamilton and Healy [18] and are presented in Table 3. They agree both with many of the solutions for southern Nevada, and with the in situ measurements at NTS.

Discussion

With respect to stress determinations Nevada Test Site is probably the most thoroughly investigated site anywhere in the world. Four sets of in situ measurements (two by hydrofracturing and two by overcoring) conducted by four different groups over a period of ten years in the same general area should provide an acceptable comparison of the methods involved. The focal mechanism
<table>
<thead>
<tr>
<th>Stress</th>
<th>Site</th>
<th>Hydrofracture (MPa/Direction)</th>
<th>Overcore (MPa/Direction)</th>
<th>Focal Mechanism (Direction)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_{H_{\text{max}}}$</td>
<td>U12n</td>
<td>9.0 / N35°E</td>
<td>8.0 / N45°E</td>
<td>N45°E</td>
</tr>
<tr>
<td></td>
<td>U12g</td>
<td>7.5 / N40°E</td>
<td>8.5 / N22°E</td>
<td></td>
</tr>
<tr>
<td>$\sigma_{H_{\text{min}}}$</td>
<td>U12n</td>
<td>3.5 / N55°W</td>
<td>2.5 / N45°W</td>
<td>N45°W</td>
</tr>
<tr>
<td></td>
<td>U12g</td>
<td>3.0 / N50°W</td>
<td>2.6 / N68°W</td>
<td></td>
</tr>
<tr>
<td>$\sigma_{V}$</td>
<td>U12n</td>
<td>7.0</td>
<td>6.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>U12g</td>
<td>7.3</td>
<td>6.8</td>
<td></td>
</tr>
<tr>
<td>Relative Magnitudes</td>
<td>U12n</td>
<td>$\sigma_{H_{\text{max}}} &gt; \sigma_{V} &gt; \sigma_{H_{\text{min}}}$</td>
<td>$\sigma_{H_{\text{max}}} &gt; \sigma_{V} &gt; \sigma_{H_{\text{min}}}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>U12g</td>
<td>$\sigma_{H_{\text{max}}} &gt; \sigma_{V} &gt; \sigma_{H_{\text{min}}}$</td>
<td>$\sigma_{H_{\text{max}}} &gt; \sigma_{V} &gt; \sigma_{H_{\text{min}}}$</td>
<td>$\sigma_{H_{\text{max}}} &gt; \sigma_{V} &gt; \sigma_{H_{\text{min}}}$</td>
</tr>
</tbody>
</table>
solutions serve as an additional powerful instrument for indicating the reliability of the in situ methods.

Table 4 provides a comparison of all the methods and measurements at the tunnel level. The uniformity of results both with respect to magnitudes and directions for the Rainier Mesa is remarkable. The mean value of $\sigma_{Hmax}$ at the tunnel level (approximately 400m depth) from the four sets of in situ measurements based on Table 4 is (giving each set of measurements equal weight):

$$\sigma_{Hmax} = 8.25 \pm 0.65 \text{ MPa} \text{ at } N35^\circ E \pm 10^\circ$$

The values of $\sigma_{Hmin}$ and $\sigma_V$ are similarly very close from one set of measurements to the next and their respective mean values are:

$$\sigma_{Hmin} = 2.9 \pm 0.45 \text{ MPa} \text{ at } N55^\circ W \pm 10^\circ$$

$$\sigma_V = 6.8 \pm 0.56 \text{ MPa}$$

The relative magnitudes of the principal stresses are uniformly

$$\sigma_{Hmax} > \sigma_V > \sigma_{Hmin}$$

with $\sigma_{Hmax}$ only slightly larger than $\sigma_V$.

It is noteworthy that the focal mechanism solutions at the Nevada Test Site confirm both the horizontal principal stress directions and the relative stress magnitudes of the in situ measurements (Table 4). The directions are within $10^\circ$ and the strike-slip fault conditions indicated by the in situ stress measurements were observed in both focal mechanisms. The accompanying normal fault movement inferred in one of the solutions alludes to the possibility that $\sigma_{Hmax}$ and $\sigma_V$ are not substantially different in magnitude, just as determined by the in situ tests.

The close agreement at the Nevada Test Site between the three methods of estimating stress could be a result of unusual uniformity of local geologic conditions, and of a particularly good rock for performing hydrofracturing and overcoring tests and for obtaining the seismic data required for unambiguous focal mechanism solutions. However, the comparison also demonstrates that at the Nevada Test Site the hydrofracturing technique yielded reliable, repeatable and accurate results which underwent rigorous verifications by use of other methods and independently run hydrofracturing.
THE HELMS PUMPED STORAGE PROJECT

The Helms Project of the Pacific Gas and Electric Company, located in the central Sierra Nevada Mountains, is a 1050 megawatt hydroelectric pumped storage facility. As part of the pre-excavation site investigation and design nine successful hydrofracturing stress measurements were conducted in granite in two NX drillholes, seven in a vertical hole between the depths of 119 and 326m, and two in an inclined hole at depths 239, and 271m [19].

The inclined hole was 30° off the vertical in a N27°E direction. The latter parallels the general direction of the hydrofractures in the vertical hole (N25°E). The strategy behind using this particular inclined hole was that if axial vertical fractures were obtained, they would confirm the stress directions determined in the vertical hole. The fractures induced were indeed nearly axial, with subvertical dips (80°SE). The tests in the inclined hole increased our confidence in the results obtained in the vertical hole and reinforced the assertion that the principal stresses were approximately vertical and horizontal, with the maximum horizontal stress (σHmax) oriented at N25°E.

Figure 3 gives the measurements as a function of depth. The steady increase with depth of all the stresses can be approximated by linear regression:

\[
\begin{align*}
\sigma_1 &\approx \sigma_{Hmax} = -0.6 + 0.035 \, D \quad \text{at } N25°E/80°-90° \\
\sigma_2 &\approx \sigma_V = 0.027 \, D \quad /0°-10° \\
\sigma_3 &\approx \sigma_{Hmin} = 3.5 + 0.006 \, D \quad \text{at } N65°W/90-100°
\end{align*}
\]

The vertical stress was calculated from the measured rock density.

The uniqueness of the Helms hydrofracturing stress measurements was that for the first time a verification of results was provided through the use of the inclined hole. As shown in Figure 3 the inclined hole results cannot be distinguished from those obtained in the vertical hole, with respect to both magnitudes and directions.

The results of these pre-excavation stress measurements were checked some years later against a series of overcoring borehole-deformation-gage tests conducted from a drift just off the site of the hydrofracturing test holes at the future powerhouse level [20]. Table 5 and Figure 2 juxtapose the overcoring horizontal and vertical secondary principal stresses versus the hydrofracturing results at 300m depth.

The overcoring principal stresses are inclined up to 30° from the vertical. This is plausible in view of the mountainous terrain, and considering that the hydrofractures were also inclined up to about 10°. The overcoring secondary principal stresses (Table 5) are reasonably close to the hydrofracturing results with the largest discrepancy in the σHmax value (5 MPa). The directions of the horizontal stresses agree within 10°.
FIGURE 2. Variation with depth of stress magnitudes and $\sigma_{H_{\text{max}}}$ direction at Helms, and overcoring results at 300m depth (D is depth in meters).
Table 5. Helms Project-Stress Comparison at Powerhouse Level (300m depth)

<table>
<thead>
<tr>
<th>Stress</th>
<th>Hydrofrac(HF)</th>
<th>Overcoring*(OC)</th>
<th>HF-OC</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MPa</td>
<td>MPa</td>
<td>MPa/Direction</td>
</tr>
<tr>
<td>$\sigma_{H_{\text{max}}}$</td>
<td>10 at N25°E</td>
<td>15 at N17°E</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(15.5 at N01°W/65°)</td>
<td>-5/8°</td>
</tr>
<tr>
<td>$\sigma_{H_{\text{min}}}$</td>
<td>5.5 at N65°W</td>
<td>7 at N73°W</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(6.7 at N74°W/104°)</td>
<td>-1.5/8°</td>
</tr>
<tr>
<td>$\sigma_V$</td>
<td>8</td>
<td>7.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(7.5 at N43°E/29°)</td>
<td>0.3</td>
</tr>
</tbody>
</table>

*Given in terms of secondary principal stresses in the horizontal and vertical planes, with principal stress values shown in parenthesis together with their bearing and inclination from the downward vertical.

Table 6. Bad Creek Project - Stress Comparison at Powerhouse Level (230m depth)

<table>
<thead>
<tr>
<th>Stress</th>
<th>Hydrofrac(HF)</th>
<th>Overcoring*(OC)</th>
<th>HF-OC</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MPa</td>
<td>MPa</td>
<td>MPa/Direction</td>
</tr>
<tr>
<td>$\sigma_{H_{\text{max}}}$</td>
<td>24 at N60°E</td>
<td>28.5 at N56°E</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(29 at N57°E/110°)</td>
<td>-4.5/4°</td>
</tr>
<tr>
<td>$\sigma_{H_{\text{min}}}$</td>
<td>15.5 at N30°W</td>
<td>17.5 at N34°W</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(18.5 at N32°W/112°)</td>
<td>-2.5/4°</td>
</tr>
<tr>
<td>$\sigma_V$</td>
<td>6</td>
<td>11.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(10 at N S72°E/15°)</td>
<td>-5.5</td>
</tr>
</tbody>
</table>

*Given in terms of secondary principal stresses in the horizontal and vertical planes with principal stress values shown in parenthesis together with their bearing and inclination from the downward vertical.
The 1000 megawatt Bad Creek Pumped Storage Project of Duke Power Company is located along the south-eastern edge of the Blue Ridge Escarpment in the north-west corner of South Carolina. A 275m vertical NQ hole was drilled in the Toxaway gneiss from the surface to the level of the future underground powerhouse. As part of the pre-excavation site investigation seven successful hydrofracturing stress measurements were conducted at different depths in the hole between 120m and 270m [19]. The results indicated a state of rather high horizontal stresses, and a steady increase in stress with depth with consistent least horizontal principal stresses, and a rather wide scatter in the magnitudes of the largest horizontal principal stress (Figure 3). The direction of \( \sigma_{\text{Hmax}} \), based on the vertical fracture impressions we have obtained averages N15°E in the top 150m but appears to readjust at N60°E in the 180-275m range (Figure 3). The state of stress in this range is given by:

\[
\begin{align*}
\sigma_1 & \equiv \sigma_{\text{Hmax}} = -8 + 0.14 \ D & \text{at N60°E} \\
\sigma_2 & \equiv \sigma_{\text{Hmin}} = -3 + 0.08 \ D & \text{at N30°W} \\
\sigma_3 & \equiv \sigma_v = 0.026 \ D
\end{align*}
\]

The vertical stress in equation (9) is based both on rock density and on hydrofracturing results which yielded both vertical and horizontal fractures and respective shut-in pressures [19].

Unlike the previous two case histories, the least horizontal compressive stress is not the smallest overall principal stress but rather the intermediate component. The negative (tensile) values that the two horizontal stresses appear to attain at shallow depths may or may not be real. In general, extrapolations beyond the range of depths within which tests have been carried out is not recommended.

The stress results were used as the basis for laying out a pilot tunnel into the powerhouse area. After the pilot tunnel was completed, the in situ stresses were determined again in the general area of the planned powerhouse using the borehole-deformation-gage method (H. G. McKay and R. E. Steffens, personal communication, 1978). Again, the results are in good agreement with the hydrofracturing horizontal stresses (Table 6, Figure 3). The least principal stress is the subvertical component, with the other principal stresses acting at about 15° from the horizontal and in directions practically identical to those determined by hydrofracturing. The magnitudes of both horizontal secondary principal stresses are within 2.5 and 4.5 MPa respectively from the hydrofrac results, but because of the high stress magnitudes those discrepancies are relatively small. Only the vertical stress is substantially different; however, we note that this is nearly the minimum principal stress which is the least accurately determined in overcoring.
FIGURE 3. Variation with depth of stress magnitudes and \( \sigma_{H \max} \) direction at Bad Creek, and overcoring results at 230m depth (D is depth in meters).
Two sets of in situ stress measurements were conducted in conjunction with the excavation of the Near-Surface Test Facility (NSTF) in the basalt of Gable Mountain, Hanford Reservation, State of Washington. Hydrofracturing was carried out in a 120m, NQ size vertical hole near the western end of Gable Mountain. The six tests conducted in this hole were part of the pre-excavation design stage of the underground test facility [21]. The borehole-deformation-gage technique was used after the completion of the NSTF [22]. Both methods of stress measurements were adversely affected by the fractured nature of the basalt (averaging 15 fractures per meter).

The hydrofracturing results and the juxtaposed overcoring measurements at the NSTF level (approximately 50m below the collar of the test hole) are shown in Figure 4. While the \( \sigma_{\text{Hmin}} \) values appear to increase steadily with depth, \( \sigma_{\text{Hmax}} \) is constant in the top 55m but increases substantially below that. Hence no attempt was made to fit a curve to its variation with depth. A sharp change occurs at 55m in the \( \sigma_{\text{Hmax}} \) direction, from an average of N75°W in the 0-55m range to N45°W in the 55-70m segment. The state of stress as obtained by hydrofracturing is:

\[
\begin{align*}
\sigma_1 &= \sigma_{\text{Hmax}} = 14 \text{ at N75°W (0-55m)} \\
&= 22 \text{ at N45°W (55-70m)} \\
\sigma_2 &\approx \sigma_3 = \sigma_{\text{Hmin}} = 0.6 + 0.015 \text{ D} \\
\sigma_2 &\approx \sigma_3 = \sigma_V = 0.028 \text{ D}
\end{align*}
\]

The vertical stress was based on the basalt density. It should be noted that because of the large magnitudes of \( \sigma_{\text{Hmax}} \) it could not be calculated by using the field value of the tensile strength. The latter can be estimated from the pressure-time records [12] only if \( \sigma_{\text{Hmax}} < 3 \sigma_{\text{Hmin}} \) (in absence of pore pressure, which was the case at the NSTF). At Gable Mountain \( \sigma_{\text{Hmax}} \) is considerably greater than \( \sigma_{\text{Hmin}} \) so only the laboratory-determined hydrofracturing tensile strength of extracted core could be used (equal to 17 MPa in the 0-55m interval, and 25.5 MPa in the 55-70m range). Typically, field obtained tensile strengths are considerably lower than laboratory ones by a factor which may vary from site to site and from rock to rock. Basalt is particularly susceptible to size effect. Recent hydrofracturing tests in Iceland basalt yielded laboratory tensile strengths of 16 MPa for specimen and borehole sizes similar to those in the NSTF case. Field determined tensile strengths were however about 3 MPa [23]. Hence, at NSTF we can only assert with some certainty that the hydrofracturing-determined \( \sigma_{\text{Hmax}} \) is an upper limit of the value. A comparison between the results of the two stress methods is given in Table 7.
FIGURE 4. Variation with depth of stress magnitudes and $\sigma_{\text{Hmax}}$ direction at Gable Mountain, and overcoring results at 50m depth (D is depth in meters).
Table 7. Gable Mountain, Hanford - Stress Comparison at NSTF-Level (50m depth)

<table>
<thead>
<tr>
<th>Stress</th>
<th>Hydrofrac (HF)</th>
<th>Overcoring *(OC)</th>
<th>HF-OC</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MPa</td>
<td>MPa</td>
<td>MPa/Direction</td>
</tr>
<tr>
<td>σ_Hmax</td>
<td>14 at N75°W</td>
<td>7 at N81°W/101°</td>
<td>-7/6°</td>
</tr>
<tr>
<td>σ_Hmin</td>
<td>1.5 at N15°E</td>
<td>2 at N8°E/95°</td>
<td>-0.5/7°</td>
</tr>
<tr>
<td>σ_V</td>
<td>1.5</td>
<td>2 at N65°E/167°</td>
<td>-0.5</td>
</tr>
</tbody>
</table>

*Given in terms of the principal stresses which because of their small inclination are approximately equal to the secondary values in the horizontal and vertical planes. The inclination is given with respect to the downward vertical direction.

Table 8. Darlington, Ontario - Stress Comparison at 70m Depth

<table>
<thead>
<tr>
<th>Stress</th>
<th>Hydrofrac (HF)</th>
<th>Overcoring (OC)</th>
<th>HF-OC</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MPa</td>
<td>MPa</td>
<td>MPa</td>
</tr>
<tr>
<td>σ_Hmax</td>
<td>12 at N70°E</td>
<td>12 at N70°E</td>
<td>0</td>
</tr>
<tr>
<td>σ_Hmin</td>
<td>8 at N20°W</td>
<td>8 at N20°W</td>
<td>0</td>
</tr>
</tbody>
</table>
Figure 4 and Table 7 show good agreement between the two methods with respect to $\sigma_{Hmin}$ magnitude, and a major discrepancy (2:1 ratio) for values of $\sigma_{Hmax}$. In view of the discussion above regarding the tensile strength such a discrepancy could be expected. The overcoring $\sigma_{Hmax}$ result is probably more reasonable, but additional tests may be needed to settle the difference. The agreement as far as the directions of the horizontal stresses is again remarkable. We note that $\sigma_{Hmax}$ direction (N75°W–N80°W) at 50m depth is also roughly parallel to the Gable Mountain axis, which should be expected in view of the limited width of the mountain and the proximity of the test hole (250m) to the very steep south-southwestern slope. Another important agreement between the measurements is with respect to the relative stress magnitudes. Both sets of results show the vertical and the least horizontal stresses to be approximately equal and small as compared to $\sigma_{Hmax}$.

DARLINGTON, ONTARIO

The existence of high horizontal stresses in many of the Silurian, Ordovician and Precambrian rocks of Ontario has been well documented and recognized in recent years [24,25]. It was with this background information in mind that Ontario Hydro decided to incorporate in situ stress measurements into the conceptual design and evaluation of underground nuclear power stations. A 303m deep NQ size test hole was drilled for generic study purposes on the construction site of the Darlington Generating Station near Bowmanville, Ontario, 65 km east of Toronto, on the north shore of Lake Ontario. The test hole penetrated through 26m of overburden, 193m of Ordovician limestone and siltstone, and 84m of Precambrian granitic gneiss. An unprecedented follow-up to the hydrofracturing tests consisted of their verification by overcoring measurements in the depth range of 26-88m, and by borehole TV camera scanning of the hydraulically induced fractures.

As shown in Figure 5 a total of ten hydrofracturing tests were conducted, six of them in the limestone between 46m and 208m, and four in the gneiss (228-300m). The top five tests yielded both vertical and horizontal hydrofractures as well as two respective shut-in pressures. This enabled, like in the case of the Bad Creek tests, the estimation of the vertical stress directly from the hydrofracturing results [26,27]. The results revealed two apparently different or decoupled stress fields, one in the limestone and another in the gneiss.

The hydrofracturing-determined stress field in the Ordovician limestone (46-200m):

$$
\begin{align*}
\sigma_1 & \equiv \sigma_{Hmax} = 12 + 0.004 D \text{ at N70°E} \\
\sigma_2 & \equiv \sigma_{Hmin} = 8 + 0.004 D \text{ at N20°W} \\
\sigma_3 & \equiv \sigma_{Hf} = 2 + 0.025 D \\
\sigma_3 & \equiv \sigma_V = 0.026 D
\end{align*}
$$
FIGURE 5. Variation with depth of stress magnitudes and $\sigma_{Hmax}$ direction at Darlington, based on hydrofracturing and overcorning measurements.
where $\sigma_v^{Hf}$ is the vertical stress as determined by hydrofracturing. The value of $\sigma_v$ was obtained from rock density.

The hydrofracturing $\sigma_v$ values are consistently larger than the density-based $\sigma_v$ by about 2 MPa in the depth range considered. The variation with depth of both horizontal stresses is very small in this Paleozoic zone (0.6 MPa) and for all practical purposes can be considered negligible. The direction of $\sigma_{Hmax}$ is quite consistent at N70°E (Figure 5).

In the Precambrian granitic gneiss (228-300 m) the hydrofracturing state of stress is:

$$
\begin{align*}
\sigma_1 &= \sigma_{Hmax} = 16 + 0.009 D \text{ at N23°E} \\
\sigma_2 &= \sigma_{Hmin} = 9 + 0.008 D \text{ at N67°W} \\
\sigma_3 &= \sigma_v = 0.026 D \quad (12)
\end{align*}
$$

A sharp increase in horizontal stress magnitudes is observed accompanied by a similar change in stress directions. The transition in direction is evidenced by the lowest test in the limestone (207 m depth) which yields a $\sigma_{Hmax}$ direction of N32°E. The decoupling of the stress regime along the Precambrian-Paleozoic contact is probably due to the existence of residual stresses in the Precambrian rocks prior to the deposition and formation of the Ordovician sediments. The vertical stress in this depth range could not be determined by hydrofracturing, and is based on rock density.

The stress magnitudes in the Precambrian granite have not been checked by an independent method, but the orientation of the hydrofractures were verified by a borehole TV camera which confirmed their inclinations (vertical) and strikes within an average of 5° from the ones determined by the hydrofracturing impression packer-magnetic orienting tool.

Using a borehole-deformation-gage Ontario Hydro also independently conducted a series of overcoring stress measurements at shallow depths at a site about 1.6 km from the hydrofracturing test hole. The original tests and results which were published by Haimson and Lee [26,27] were considered by Ontario Hydro as not sufficiently reliable, and a new set of overcoring tests in two separate vertical holes were conducted in 1980 at depths from 26 to 88 m (C. Lee, personal communication, 1981). The results are plotted in Figure 5. These tests could only yield the horizontal principal stresses. It is evident from the top three overcoring tests, at depths of 26-30 m, that the stresses just under the overburden are relieved, not unlike the behavior observed for example at Waterloo, Wisconsin [12]. On the other hand, in the 58-88 m zone the four overcoring tests yield results that cannot be distinguished from the hydrofracturing stresses. This holds true for both magnitudes and directions as shown in Table 8. We note again that the thrust faulting type condition ($\sigma_{Hmax} > \sigma_{Hmin} > \sigma_v$) is confirmed by both stress methods. A similar relative stress situation was encountered in the Bad Creek case, and was verified by both hydrofracturing and overcoring.
Table 9. Stripa, Sweden - Stress Comparison at 320m Depth

<table>
<thead>
<tr>
<th>Stress</th>
<th>Hydrofrac (HF) MPa</th>
<th>Overcoring*(OC) MPa</th>
<th>HF-OC MPa/Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(a) 16.5 at N80°W</td>
<td>23.5 at N67°W</td>
<td>-7/13°</td>
</tr>
<tr>
<td>σ_Hmax</td>
<td>(b) 18 at N76°W</td>
<td>18.5 at N36°W</td>
<td>-0.5/40°</td>
</tr>
<tr>
<td></td>
<td>(a) 10.5 at N10°W</td>
<td>12.5 at N23°E</td>
<td>-2/13°</td>
</tr>
<tr>
<td>σ_Hmin</td>
<td>(b) 11.5 at N14°E</td>
<td>10.5 at N54°E</td>
<td>+1/40°</td>
</tr>
<tr>
<td></td>
<td>(a) 8.5</td>
<td>13</td>
<td>-4.5</td>
</tr>
<tr>
<td>σ_V</td>
<td>(b) 8.5</td>
<td>10</td>
<td>-1.5</td>
</tr>
</tbody>
</table>

*Overcoring results are given in terms of secondary principal stresses in the horizontal and vertical planes, with the principal stresses, and their inclination shown in parenthesis.

(a) based on linear approximation of stress-depth variation.

(b) based on actual test results at 326m for overcoring and at 318, 326 and 329m for hydrofracturing.
FIGURE 6. Variation with depth of stress magnitudes and $\sigma_{Hmax}$ direction at Stripa, based on hydrofracturing and overcoring measurements ($D$ is depth in meters).
A 380m vertical NX testhole was drilled from the surface into granite just outside the perimeter of the underground Stripa Mine, Sweden which has been used for generic nuclear waste disposal research. This is probably the first case in which the same test hole has been used for exhaustive stress testing both by hydrofracturing and by overcoring (in this case the Leeman triaxial cell technique modified by the Swedish State Power Board for deep measurements [28]). Details of the tests and results are given by Doe et al [29]. The averages of 16 hydrofracturing and 17 overcoring tests have been clustered in Figure 6 around five depths. The hydrofracturing results show a consistent increase with depth:

\[
\begin{align*}
\sigma_1 & = \sigma_{Hmax} = 3.5 + 0.04 \ D \quad \text{at N80°W} \\
\sigma_2 & = \sigma_{Hmin} = 2 + 0.027 \ D \quad \text{at N10°E} \\
\sigma_3 & = \sigma_V = 0.026 \ D
\end{align*}
\] (13)

The overcoring stresses which generally increase with depth, are less consistent and display a larger scatter than the respective hydrofracturing stresses. A quick comparison between the two sets of results (Figure 6) show that the overcoring vertical and maximum horizontal stresses are considerably higher than the density based \(\sigma_V\) and the hydrofracturing \(\sigma_{Hmax}\). There is, however, good agreement with respect to \(\sigma_{Hmin}\) magnitude, as well as the directions of the horizontal stresses.

Table 9 is a comparison of results at 320m depth which is the depth of interest in the mine. Owing to the availability both of direct measurements in the immediate vicinity of this depth and of the general trend of stress vs. depth as shown in Figure 6, two sets of comparisons have been made (Table 9) based on data from Doe et al [29]. The agreement between interpolated stresses (case a in Table 9) is good for \(\sigma_{Hmin}\), fair for \(\sigma_{Hmax}\), poor for \(\sigma_V\) and excellent for \(\sigma_{Hmax}\) direction. However, the agreement between direct measurements at about 320m (case b) is considerably better, with each of the horizontal stresses differing by only 1 MPa, but with a somewhat larger discrepancy in stress directions (within 40°).

DISCUSSION AND CONCLUSIONS

It has been said that hydrofracturing is well developed but poorly understood. It is perhaps difficult to challenge this statement except for pointing out that the same may be said about almost any other rock field test now in use. This paper certainly does not add much to the understanding of the method; rather it is an attempt to verify whether it works. The inescapable answer, based on the six comparisons with overcoring is an unequivocal yes, provided we keep in mind that hydrofracturing is not a precision measurement.
Table 10. Summary of Stress Comparisons

<table>
<thead>
<tr>
<th>Site</th>
<th>HF-OC</th>
<th>(HF-OC)/HFx100%</th>
<th>(HF-OC) Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\sigma_{Hmin}$</td>
<td>$\sigma_{Hmax}$</td>
<td></td>
</tr>
<tr>
<td>NTS-U12n</td>
<td>1 MPa</td>
<td>1 MPa</td>
<td>29</td>
</tr>
<tr>
<td>NTS-U12g</td>
<td>0.4 MPa</td>
<td>1 MPa</td>
<td>13</td>
</tr>
<tr>
<td>Helms</td>
<td>-1.5 MPa</td>
<td>-5 MPa</td>
<td>-27</td>
</tr>
<tr>
<td>Bad Creek</td>
<td>-2 MPa</td>
<td>-4.5 MPa</td>
<td>-13</td>
</tr>
<tr>
<td>Gable Mt.</td>
<td>-0.5 MPa</td>
<td>7 MPa</td>
<td>-33</td>
</tr>
<tr>
<td>Darlington</td>
<td>0 MPa</td>
<td>0 MPa</td>
<td>0</td>
</tr>
<tr>
<td>Stripa (a)</td>
<td>-2 MPa</td>
<td>-7 MPa</td>
<td>-19</td>
</tr>
<tr>
<td>Stripa (b)</td>
<td>1 MPa</td>
<td>-0.5 MPa</td>
<td>10</td>
</tr>
</tbody>
</table>

HF-OC = Hydrofracturing stress-overcoring stress
(a) Based on linear approximation of stress-depth variation
(b) Based on actual test results at 325 m ±10m.
device but a method of estimating the magnitudes and directions of the horizontal principal stresses in deep vertical drill holes.

Analyzing the comparisons in Tables 1-9 we first notice that the stresses determined by hydrofracturing and overcoring are basically in the same "ball park". This approximate coincidence of results supports the assertion that both methods, using radically different approaches, estimate the same field parameter, namely the stress regime. Two major conclusions can be drawn directly from these tables:

1. Both methods yield the same relative stress magnitudes ($\sigma_Y > \sigma_{Hmax} > \sigma_{Hmin}$, or $\sigma_{Hmax} > \sigma_Y > \sigma_{Hmin}$, or $\sigma_{Hmax} > \sigma_{Hmin} > \sigma_Y$).

2. The inclinations of the overcoring principal axes are usually within 30° from the hydrofracturing principal axes (vertical and horizontal).

Additional conclusions can be drawn either from Tables 1-9 or from the summary given in Table 10:

3. The directions of the horizontal principal stresses as determined by the two methods are typically within 10°.

4. The magnitudes of $\sigma_{Hmin}$ as determined by the two methods are within 2 MPa. This is equivalent to a discrepancy of up to 30% of the hydrofracturing $\sigma_{Hmin}$ value.

5. The magnitudes of $\sigma_{Hmax}$ as determined by the two methods are typically within 5 MPa. A Gable Mountain and Stripa (case a) the differences are slightly higher, but they are not typical as explained above. In terms of percentage of the hydrofracturing $\sigma_{Hmax}$ value the discrepancy can reach 50%.

The above conclusions, we believe, provide a strong support for the reliability of hydrofracturing. The almost identical results with respect to stress directions and the magnitude of $\sigma_{Hmin}$ are remarkable. The larger discrepancy in the estimation of $\sigma_{Hmax}$ magnitude is expected owing to the rather illusive 'tensile strength' parameter which enters the calculation in the hydrofracturing method. One must also be aware that the discrepancy may in part be due to a tendency to exaggerate stress magnitudes in overcoring by using a rock modulus based on core testing which is often higher than the equivalent field value. Still the discrepancy is tolerable in most cases and more importantly, in most uses.

Hydrofracturing has been used so far in four major areas: in pre-excavation site investigation and design of large underground openings, in design of in situ mining projects (such as oil, and hot-dry-rock geothermal energy extraction), in waste disposal studies, and in tectonophysics and earthquake research. In all these endeavors the information provided by deep-hole hydrofracturing stress measurements is, despite possible inaccuracies in
principal axes inclinations and \(\sigma_{H\text{max}}\) magnitude, extremely valuable and sufficiently accurate for most utilizations. Recent advances in statistical fracture mechanics [30] appear to provide the necessary tools for a rational estimation of the field tensile strength of rocks based on laboratory tests. Thus, even the uncertainty surrounding the accuracy of hydrofracturing \(\sigma_{H\text{max}}\) may soon be a thing of the past.

ACKNOWLEDGEMENTS

Many thanks to the different private companies and national organizations who sponsored the hydrofracturing results reported herein. I also thank the many authors whose papers on overcoring and hydrofracturing I have used. Special thanks to Tom Doe who enabled me to participate in the Stripa tests and allowed the use of the results prior to their formal publication. A major part of this paper was presented at the 22nd U.S. Rock Mechanics Symposium, MIT, June 1981.

REFERENCES


DETERMINATION OF THE STATE OF STRESS AT THE STRIPA MINE, SWEDEN

Thomas W. Doe
Earth Sciences Division
Lawrence Berkeley Laboratory

ABSTRACT

The state of stress at the Stripa test mine in Sweden has been studied through a program of hydraulic fracturing and overcoring stress measurements performed both in a 381 meter deep vertical borehole drilled from the surface, and from shorter boreholes drilled around the heater experiment drifts.

Far-field measurements were obtained in the deep vertical hole using the Swedish State Power Board's deep hole Leeman triaxial cell and by hydraulic fracturing. The two methods are in good agreement on the orientation of the maximum horizontal stress and in the interpolated stress values for the depth of the test facility. Based on a regression analysis of the stress data versus depth the following conclusions are made: (1) that a determination of stress at a particular depth should be made by interpolation of data from well above and below the depth of interest, (2) extrapolation of values beyond the depth range of the data cannot be done with confidence, and (3) stress determinations should be based on more than just a few measurements.

The hydraulic fracturing experiments are all interpreted using the first breakdown pressure and a tensile strength term. The tensile strength term is based on analysis of laboratory tensile strength data, and is compensated for size effect through methods of statistical fracture mechanics.

Near-field measurements were made from a series of two horizontal boreholes and one vertical borehole in the heater test area. The vertical hole was used for Swedish State Power Board Leeman cell measurements and hydraulic fracturing. One horizontal hole was used for hydraulic fracturing, and the other was used for overcoring using the University of Luleå (LuH) triaxial cell, the USBM borehole deformation gage, and the CSIRO hollow inclusion triaxial cell. All of the data are in excellent agreement that the maximum stress in the area of the heater drift trends parallel to the drift and is horizontal. The methods all agree in the magnitudes of the stresses, however there are some discrepancies in the orientations of the two lesser stresses when comparing data from the horizontal holes and the vertical hole. Specifically, the hydraulic fractures from both the vertical and the horizontal hole showed two shut-in pressures, yet the fractures in either one hole or the other should have been normal to the least stress. Similarly, the
overcoring measurements by the triaxial cell methods do not agree as to the orientations of the least and intermediate stresses in the area directly under the floor of the heater experiment drift.

An acoustic experiment to monitor the propagation of the hydraulic fractures was partially successful for only one test. No acoustic activity was recorded for the first breakdown. Considerable activity was registered during reinjection of the fracture, the locus being about two meters from the well in a direction about forty five degrees from the azimuth of the fracture impression on the borehole wall. About two minutes passed between the beginning of the pumping and the peak of the acoustic emissions.

The maximum stress direction rotates from being northwest in the far-field to being northeast in the near-field. This rotation appears to be due to the influence of the mine as a whole rather than being a local adjustment near the heater experiment drifts.

INTRODUCTION

For the past several years the Stripa Mine in central Sweden has been the site of hydrologic and rock mechanics field testing to evaluate the technology of storing radioactive wastes in granitic rocks. Data on the state of stress has been recognized as a necessary parameter for the analysis of the data at the site, and over the past two summers we have been carrying out a program of in situ stress measurements by hydrofracturing and a variety of overcoring techniques.

In addition to the primary purpose of the stress measurement experiments, which was to determine the state of stress for the purpose of analyzing the heater test data, the work at Stripa has provided an opportunity to compare the results of several stress measurement techniques at a common site, and to measure the effect of a large mine on the state of stress.

It was hoped that this work might help to resolve some of the controversy surrounding stress measurement techniques. Hydraulic fracturing, which has become very popular as a deep measurement method, has been questioned over such issues as the non-colinearity of the hole with principal stress directions, the role of rock tensile strength, and the interpretation of shut-in pressure records. Overcoring measurements have been notorious for a large degree of scatter in the data as well as questions about the influence of small-scale, local heterogeneities on the results of strain cell measurements. Hydraulic fracturing and overcoring have not been carried out in a common borehole nor have many measurements by the two methods been made in the immediate vicinity of one another underground.

The Stripa stress measurement program has been carried out in two stages. The goal of the first stage was to determine the state of stress at a location where the influence of the mine openings would be small. We are calling this the far-field stress, and the program
for obtaining its values involved drilling a 381 meter borehole, SBH-4, at a location about 300 meters north of the mine (Figure 1). During the drilling the Swedish State Power Board performed seventeen stress measurements using their unique, deep-hole triaxial cell. The measurements were performed in groups of about four at four depth levels: 100, 200, 300, and 380 meters. After the hole was completed, sixteen hydrofracturing tests were performed between 25 meters and 369 meters depth, with a larger proportion of the measurements made around the depth of the underground test facility at about 320 meters.

The goal of the second stage of the program was to determine the state of stress in the immediate vicinity of the full scale heater tests underground (Figure 1). This near-field state of stress was determined using hydrofracturing, the Swedish State Power Board deep hole triaxial cell, and a variety of more familiar overcoring techniques including the USBM borehole deformation gage, the University of Luleå (LuH) triaxial cell, and the CSIRO triaxial cell.

FAR FIELD MEASUREMENTS

Stress Measurement Data

The measurement of the stresses in deep hole SBH-4 has been described in Doe (et al., 1981). The procedures used in the overcoring are summarized in Figure 2. The Swedish State Board triaxial cell has been adapted from the Leeman triaxial cell for use in deep holes by wireline emplacement. The cell measures the complete state of stress though the response to overcoring of three strain gage rosettes, each having three components. The rosettes are cemented to the wall of a 36 mm pilot hole which is then overcored with a conventional 76mm (NX) double tube core barrel. The principal stress data from the overcoring are summarized in Figure 3. The data exhibit a large degree of scatter in the magnitudes, however there is consistency in the orientations of the principal stresses. The greatest principal stress is oriented horizontally, however, the other principal stresses are generally skewed with respect to the vertical and horizontal. Hence the usual assumption in hydrofracture data analysis that the borehole is oriented in the direction of one of the principal stresses is not met.

Methods for analyzing the hydraulic fracturing data are described in the Appendix. Briefly, the methods used the first breakdown pressure and a tensile strength term determined in laboratory testing. This method is considered more reliable than second breakdown techniques (Bredehoeft, et al., 1976; Zoback, et al., 1980) for sites where the horizontal stress ratio exceeds two (Doe, et al., 1981), as for such ratios the theoretical second breakdown pressure is less than the shut in pressure. The tensile strength term has been derived using methods of statistical fracture mechanics.
Figure 1. Location of stress measurement boreholes at Stripa Mine. SBH-4 is a 381 meter hole drilled from surface; BSP-1, 2, and 3 are drilled in the vicinity of the heater test drifts underground. Inserts show location of mine in Sweden as well as the orientations and magnitudes of the horizontal stresses determined in the surface hole and underground.

Figure 2. Procedures used in Swedish State Power Board stress determinations. (1) drilling of 76mm borehole, (2) drilling 36mm pilot hole, (3) inspecting the core, (4) running the strain gage carrier into the hole by wireline, (5) setting the strain gages and taking first set of gage readings, (6) removing strain gage carrier from hole, (7) overcoring, and (8) removing strain gaged core from hole and taking final strain gage readings.
(Ratigan, 1981) which take into account the differences of size effect and sample geometry between laboratory tests and field fracturing tests.

The orientations of the fractures were obtained using a wireline impression packer which contained a borehole survey compass for packer orientation. The wireline operation saved considerable time over emplacement methods using rigid tubing.

Comparison of the Far-Field Hydrofracturing and Overcoring Results

Two bases for comparing the results of the overcoring and hydraulic fracturing are used in this paper, the orientation of the maximum horizontal stress, and the magnitudes of the maximum and minimum horizontal stresses at a depth of 320 meters in the hole, which is the depth of the test facility. The horizontal stresses are used for comparison because the hydrofracture test is generally thought to measure mainly the stress components normal to the borehole. The stress magnitude at the test facility depth is determined by interpolation of a linear regression of stress versus depth.

The data for the orientation of the maximum horizontal stress, versus depth are shown in Figure 4. The mean orientations of the maximum horizontal stress directions agree within a one degree of N 83° W for the two techniques. The 95% confidence levels for the means are determined using the methods of Mardia (1972) and are both about ± 20 degrees, thus one can conclude that the correspondence between the overcoring and hydraulic fracturing is quite good. The confidence intervals could have been improved to about ± 15 degrees had over twenty measurements been made. Further improvement in the statistics with larger numbers of measurements is probably not practical from the standpoint of cost and from the lack of suitable test zones.

The magnitudes of the secondary principal stresses for the overcoring and the hydrofracturing are shown as a function of depth in Figures 5 and 6. The data have been fitted to regression lines whose coefficients are given in Table 1. The horizontal stress magnitude for the two methods also agree closely. The hydraulic fracturing has somewhat better confidence intervals than the overcoring, particularly for the horizontal minimum stress, but both methods provide estimates for the mean stress values at the depth of the test facility within ±20% or better. At the depth of the Stripa test facility the regression values are:

\[
\begin{array}{ccc}
\sigma_{\text{Hmax}} & \sigma_{\text{Hmin}} \\
\text{Hydrofracturing} & 22.1 \pm 2.1 & 11.1 \pm 0.8 \\
& \text{(First Breakdown Method)} & \\
\text{Hydrofracting} & 16.3 \pm 2.2 & 11.1 \pm 0.8 \\
& \text{(Second Breakdown Method)} & \\
\text{Overcoring} & 25.4 \pm 2.9 & 12.1 \pm 2.4 \\
\end{array}
\]

* maximum and minimum horizontal stresses, MPa.
Figure 3. Lower hemisphere stereographic projection of results of overcoring stress measurements in SBH-4. Figures given at left denote depth ranges for each row of plots. Stress magnitudes are given in MPa.

Figure 4. Orientations of maximum horizontal stress versus depth as determined by hydraulic fracturing and overcoring in SBH-4.
Figure 5. Magnitudes of horizontal stresses determined by overcoring in SBH-4. Triangles are maximum horizontal stress, circles are minimum horizontal stress. Curved lines are the 90% confidence intervals for the ordinate to the regression line. Large, open data points are the values of stress at the depth of the test facility as predicted by the regression. Error bars on either side of the open points are equal to the standard error of estimate.
Figure 6. Magnitudes of horizontal stresses determined by hydraulic fracturing in SBH-4. See Figure 5 for explanation of symbols.
The standard errors of estimate for the all measurements and the confidence intervals for the regression slopes are as high as ± 50% for the magnitude data. This large amount of uncertainty suggests that a stress measurement program consisting of only a few measurements at a site may be insufficient to adequately determine the stress magnitudes.

The large standard errors of estimate and the large confidence intervals for the slopes of the regression lines show that reliable predictions of the in situ stresses at depth cannot be made either on the basis of a few measurements or by extrapolating the results of a set of measurements taken at shallow depth.

NEAR-FIELD MEASUREMENTS

The second phase of the Stripa stress measurement program was to measure the in situ stress in the immediate vicinity of the the full scale heater experiment (Figures 1 and 7). Three holes were drilled for the purposes of stress measurement. Hole BSP-1 (BSP stands for bergspännung, or rock stress in Swedish) was drilled vertically downward from the center line of the full scale drift to a depth of 25 meters. This hole was 76mm in diameter and was used for hydraulic fracturing and for overcoring by the Swedish State Power Board method. Two holes were drilled from the extensometer drift, an opening excavated parallel to the full scale drift at a lower level to allow the installation of horizontal extensometers in the original heater experiment. Hole BSP-2 was drilled with a diameter of 76mm to a length of 20 meters and was used exclusively for hydrofracturing. The hole was drilled at an angle three degrees downward from the horizontal to assure that the hole would remain full of water during the hydrofracturing tests. Hole BSP-3 had a diameter of 150 mm and was drilled to length of 12 meters for use in USBM, CSIRO, and LuH triaxial cell measurements. The hole was drilled at a small angle upward from the horizontal to assure that water would drain from the hole and not affect the bonding of the triaxial strain cells.

An acoustic emission experiment was set up by Ernest Majer of Lawrence Berkeley Laboratory to detect the propagation of the hydraulic fracture and, hopefully, to map its location. The layout and results of the acoustic experiment are discussed elsewhere in this volume (Majer and Mc Evilly, 1982).

In addition to the simple comparison of the stress values from the various overcoring techniques, the underground experiment had several other objectives including:

- investigating the effect of the hole orientation on the hydrofracture results,
- measuring the influence of the extensometer drift and full scale drifts on the in situ stress orientations and magnitudes,
o investigating the correspondence of the acoustically mapped
hydrofracture plane with the plane normal to the least principal
stress determined by the overcoring.

Predicted State of Stress in Full Scale Drift Area

Chan, et al. (1981) performed a series of two-dimensional
boundary element calculations of the stress field in the area of the
full scale drift based on the far field measurements. The results,
shown in Figure 8, allowed some prediction of what should be expected
from the field measurements. Along BSP-1, the vertical hole drilled
from the centerline of the full scale drift downward, the principal
stress orientations and magnitudes do not vary much from the farfield
values. Along the horizontal holes, BSP-2 and BSP-3, there is
considerable change due to the influence of the extensometer drift.
The maximum stress is vertical near the drift, and it rotates towards
the horizontal as the holes approach the full scale drift.

RESULTS OF OVERCORING MEASUREMENTS

The results of the overcoring measurements (excluding the CSIRO)
are presented in Table 2.

Swedish State Power Board Leeman Cell Measurements (BSP-1)

A total of six measurements were made with the Swedish State
Power Board Leeman cell. meters below the floor of the full scale
drift. The principal stress data are given in Table 2 and the
orientation data are shown in Figure 9. The direction of the maximum
principal stress is very consistent among the measurements and is
oriented parallel to the axes of the two drifts. The intermediate
principal stresses are oriented off the vertical an average of about
60 degrees. The minimum principal stresses are within about 30
degrees of the horizontal. There is little discernable trend to the
changes in orientation of the minor principal stresses with depth.

LuH Triaxial Cell Measurements (BSP-3)

Eight University of Luleå LuH triaxial cell measurements were
made at hole lengths between 2.5 and 11.2 meters. The magnitudes of
the principal stresses are given in Table 2 and the orientations are
shown in Figure 10. The magnitudes of the principal stresses vary
along the length of the hole, but not to the extent which was
predicted by the modelling. The maximum principal stress is
consistently parallel to the axis of the drifts and coincides closely
with the direction measured by the Power Board. The intermediate and
minor principal stresses are nearly 45 degrees off the vertical and
horizontal directions near the collar of the hole. As the hole
proceeds towards the full scale drift, the intermediate stress
Figure 7. Diagram showing relative positions of the full scale drift, the extensometer drift, and the stress measurement boreholes. Orientations of typical hydraulic fractures shown for the 76mm holes, BSP-1 and 2. Swedish State Power Board overcores were taken in BSP-1; USBM, CSIRO, and LuH overcores were taken in BSP-3. Approximate values for the magnitude and orientation of stress ellipsoid from the overcoring are shown in the lower right hand part of the diagram.

Figure 8. Stress distributions around full scale and extensometer drift as predicted by boundary element calculation based on the far field stress results (Chan and Saari, 1981).
rotates towards the horizontal and the least stress rotates towards
the vertical. This rotation is consistent with the predictions of
the boundary element model predictions shown in Figure 8.

The mean orientations of the principal stresses agree well with
those measured by the Power Board in BSP-1; however, one would expect
that the Lulea measurements closest to the end of the hole, which is
near the centerline of the full scale drift, would be the ones most
closely coinciding with the Power Board results. Such is not the
case as the measurements at the end of BSP-3 show the greatest
divergence with the Power Board results.

**USBM Borehole Deformation Gage Measurements (BSP-3)**

The USBM borehole deformation gage was used in the same hole as
the LuH cell and CSIRO cell measurements. Unlike the triaxial strain
cells, the USBM gage measures only the stress components normal to
the hole axis. This disadvantage is balanced against the greater
rapidity and reliability of the USBM gage. Strain cell measurements
and deformation gage measurements complement one another when used in
the same hole. The strain cells provide the three dimensional
information, and the deformation gage provides the larger number of
measurements necessary for confidence in the stress determination for
a site.

Nine USBM measurements were made at hole depths ranging from 1.1
to 9.7 meters. The results of the USBM measurements are plotted
along with the secondary stress data for the LuH cell measurements in
Figure 11. The agreement is excellent for both magnitude and
orientation. The mean secondary stresses with 90% confidence levels
for the means are

\[
\begin{align*}
\sigma_{\text{Max}}^* & \quad \sigma_{\text{Min}}^* \\
(\text{MPa}) & \\
\text{LuH} & 20.0 \pm 3.3 & 4.5 \pm 0.8 \\
\text{USBM Gage} & 17.5 \pm 2.3 & 4.3 \pm 1.4
\end{align*}
\]

The maximum secondary stress, which is very close to being the
maximum principal stress, is horizontal for both techniques.

**CSIRO Triaxial Cell**

The CSIRO triaxial cell (Worotnicki and Walton, 1976) is a
hollow cylinder which is grouted into a 38 mm pilot hole and then
overcored. The cell is similar to the Leeman triaxial cell in that
it contains three strain gages rosettes with three components each.
The data reduction methods are the same as those for the Leeman cell
except for modifications to allow for the effect of the cylinder.
The CSIRO cell has several practical advantages over the Leeman cell
including the protection of the electronic circuitry from the

* maximum and minimum stresses normal to borehole
Figure 9. Lower hemisphere stereographic projection of the hydrofracture planes and the principal stress directions determined by overcoring in the vertical borehole, BSP-1. Identification numbers are given for each test (see Tables 2 and 3 for depths).
Figure 10. Lower hemisphere stereographic projection of the hydrofracture planes and the principal stress directions determined by the LuH cell (solid symbols) and CSIRO (open symbols) overcoring in the sub-horizontal holes, BSP-2 and 3. Identification numbers given for each test (see Tables 2 and 3 for depths).
drilling fluids and the capability for monitoring the strain gage outputs during the overcoring. It has a disadvantage in that the cements require seventeen hours or more to cure to an acceptable hardness.

Five CSIRO measurements were made in BSP-3. Despite using curing times in excess of seventeen hours, the first two measurements did not appear to adequately bonded to the pilot borehole walls. Even after switching to a faster curing cement for the final three measurements, the gage values showed an average drift rate of about five microstrains per minute before and after the overcoring. As there are only three measurements, confidence levels for the mean magnitudes are not presented. The orientation and magnitude data are calculated using strain data from which the linear drift has been subtracted. The data, shown in Figure 10, are consistent with the LuH results both in orientation and in magnitude.

NEAR FIELD HYDRAULIC FRACTURING MEASUREMENTS

Location, Equipment, and Procedures

Hydraulic fracturing stress measurement were carried out in both the vertical borehole, BSP-1, and the horizontal borehole, BSP-2. Nine measurements were carried out in BSP-1 over 0.6 meter test intervals ranging in depth from 2.3 meters to 20.2 meters. Eight measurements were performed in BSP-2 using the same test interval length and range of distance from the extensometer drift walls of 3.8 meters to 16.7 meters.

The equipment and procedures used for conducting the tests and evaluating the results were essentially the same as those used for the far field stress measurement work in SBH-4. The results, given in Table 3, are calculated using the first breakdown pressures and the tensile strength values determined by Ratigan (1981). It was assumed that the underground test area was drained of water, thus the pore pressure term was taken as zero. The only major difference in the procedures from those described for the SBH-4 work was the addition of a fast pumping cycle in the pressure time record. The fast pumping cycle was added after the first of the secondary repressurization cycles with the purpose of extending the fracture as far as possible for the sake of the acoustic monitoring. The pumping rate for the fast pumping cycle was 4.5 liters per minute, which was limited by the air-driven, positive-displacement pump. In contrast to this pumping rate, our first breakdown and second breakdowns were performed at a rate of about 1 liter per minute. The slow pumping cycle for determining the fracture reopening pressure was run at about 0.25 liters per minute.
Figure 11. Maximum and minimum stresses normal to the direction of BSP-3 measured by LuH triaxial cell and USBM deformation gage overcoring. Right hand scale gives the values for the plot of the angle between the maximum stress direction and the vertical.
Orientation of Hydraulic Fractures

The orientation of the hydraulic fractures was determined by an impression packer which was lowered into the hole on scribed tubing. Figure 9 shows the orientation of the hydrofracture planes at the borehole wall for the vertical hole, BSP-1, and Figure 10 shows the fracture orientations for the horizontal hole, BSP-2.

The fracture orientations in BSP-1 are strongly aligned parallel to the axis of the full scale and extensometer drifts, and thus agree closely in orientation to the maximum principal stress direction determined by the overcoring measurements.

The fracture orientations in the horizontal hole strike parallel to the drift axis and are shallowly dipping except for the deepest measurements. Several of the measurements are nearly perpendicular to the minimum principal stress determined by the LuH cell overcoring in BSP-3.

Interpretation of Secondary Breakdown Records

The shut-in pressures determined both from the breakdown records and from the slow pumping cycle decreased in value with additional pressurization cycles. This drop in the shut-in pressure value was noted in tests from both the vertical and the horizontal holes, and the drop was especially marked after the fast pumping cycle. The initial shut pressures and the final shut-in pressure values have the same average values for the two holes as shown in Table 3.

Previous investigators have interpreted the reduction in shut-in pressure as an indication that the fracture is changing its orientation from a plane coaxial with the borehole to a plane normal to the minimum principal stress (Zoback and Pollard, 1978; Haimson, 1978). It is not clear from the data presented here whether or not that hypothesis is valid.

If we consider the LuH cell measurements from the horizontal hole, BSP-3, to be accurate, then the hydrofractures in the horizontal hole BSP-2, shown schematically in Figure 7, would be normal to the minimum principal stress. Under those conditions one would expect that the first shut-in pressure values in BSP-2 would equal the minimum stress and there would no reduction in shut-in value with additional pumping cycles. Unfortunately, such reductions are observed.

If we consider the Power Board Leeman cell results in the vertical hole, BSP-1, to be accurate, then neither of the hydrofracture boreholes is normal to the least principal stress, and it is coincidental that the first and second shut-in pressure values have the same values in the two holes despite the fact that the holes are orthogonal to one another.

It is possible that the shut-in pressure is influenced by other factors than the minimum principal stress value, such as fracture length, interconnection with other fractures, and the fracture normal
stiffness (Narasimhan and Palen, 1981). While the data on secondary shut-in pressures are not conclusive, they do not readily support interpretations based on changes in fracture orientation.

**Acoustic Mapping of the Hydrofracture Propagation**

A complete description of the acoustic mapping experiment is contained in Majer and McEvilly (1982, this volume) and will not be repeated in detail here. The major points were that the acoustic activity was only observed during the fast pumping, and that the location of the activity was located within two meters of the well. The acoustic activity did not begin until two minutes into the fast pumping cycle, which suggests that the time required to propagate the hydraulic pressure to the crack tip might be appreciable. Unfortunately, the primary data recorder was damaged in shipment to Sweden, and a substitute which had not been optimized for the signal collection was obtained at the last minute. Of the twelve stations placed on the walls of the drifts and in instrument holes, only three successfully measured activity. The small number of recording stations was insufficient to accurately locate the position of the plane of the hydrofracture. The acoustic activity was generally located directly to the north of the hole rather than to the northeast along the axis of the full scale drift, which would have been the direction predicted by the impression packer data from the wall of the borehole. Such data would suggest that the fracture was changing orientation away from the borehole, which would be consistent with the vertical minimum stress data from the horizontal boreholes.

**Comparison of Near Field Stress Measurement Results**

The agreement between the results of the overcoring and the hydraulic fracturing for the near-field measurements is best in the magnitude and orientation of the maximum principal stress. All the techniques are in agreement that that the direction of the maximum stress is horizontal and parallel to the axes of the full scale and extensometer drifts. The magnitudes for the stresses cover a range within about ± 10% of 22 MPa.

The values for the magnitudes of the intermediate and least stresses are in general agreement; however, a number of inconsistencies exist in the orientation results. These have been discussed above, and can be summarized as (1) the inconsistency in the secondary shut in pressures between the tests run in the two orthogonal holes, and (2) the divergence in orientation between the two Leeman cell methods for the measurements made underneath the full scale drift. As the results of both the hydraulic fracturing and the overcoring are ambiguous as to the orientations of the minor stresses, it is not possible to use the results from one measurement method to answer the questions posed by the other.
INFLUENCE OF MINE ON STATE OF STRESS AT STRIPA

One of the striking aspects of the comparison of the near field and far field stress data is the change in orientation of the maximum principal stress from northwest in SBH-4 to northeast in the full scale drift area. The rotation appears to be caused by the mine as a whole rather than being a local effect from the full scale and extensometer drifts. This conclusion can be based on Carlsson’s (1978) Leeman cell measurements about 40 meters away. Carlsson’s results are similar to the overcoring results obtained in this study both in magnitude and in orientation. Another set of measurements was performed recently in a deep borehole drilled from the 360 meter level of the mine. In this hole the Swedish State Power Board performed two sets of four measurements each at hole depths of 150 and 300 meters. Their results (Strindell and Andersson, 1981) also recorded a northeast trend to the maximum horizontal stress.

DATA REQUIREMENTS FOR STRESS MEASUREMENT PROGRAMS

If the goal of a stress measurement program is to obtain stress magnitude data for a particular depth, then the measurement program can be designed to obtain the estimate by (1) performing a number of measurements at the depth of interest, or (2) interpolating the value from a linear regression on measurements taken over a range of depths. The requirements of sample size for obtaining a given confidence interval can be obtained in any good statistical reference book such as Crow, et al. (1960).

Assume one used the first approach, and the data had a mean of 22 Mpa and standard deviation of about 5 MPa, which is the value for the hydrofracture maximum stress between 300 meters depth and the end of the hole. One would need to perform about 13 measurements to have data with a 90% confidence interval of ± 10% for the means. Since there may not be that many suitable test zones in the depth range of interest, one may need to use the linear regression approach. It is difficult to specify a number of tests required to obtain a particular confidence interval, because the quality of the estimate will depend on how the data are distributed with respect to depth. A program where the stresses are measured from the surface to a depth twice as great as the horizon of interest will provide data with the highest degree of confidence for designing the underground facility. If testing to such great depths is not practical, tests should be made as deep as the target depth and preferably somewhat deeper. For a site that had similar variances in the stress data as Stripa, one would expect that at least ten to fifteen measurements would be necessary for the ± 10% confidence intervals.

If a site yields stress orientation data similar in dispersion to that measured at Stripa, attempts should be made to obtain about 20 readings for a confidence of ± 15 degrees. More than 20 measurements may not be practical from the standpoint of both cost...
and the availability of suitable test zones.

CONCLUSIONS

Overcoring and hydraulic fracturing stress measurements provided comparable results both in a deep vertical hole drilled from the surface and in shorter holes drilled from underground openings. The results of the LuH, USBM, and CSIRO methods gave consistent results when run in the same hole, BSP-3.

The hydrofracturing data were interpreted using both first breakdown methods, which require a tensile strength term, and second breakdown methods. The first breakdown methods used a tensile strength term obtained through a statistical fracture mechanics analysis (Ratigan, 1981). The first breakdown results agreed more closely with the overcoring data than the second breakdown results.

In the underground tests, the orientations and magnitudes of the intermediate and least stresses were consistent when comparing hydrofracturing and overcoring results from holes of the same orientation. However, the vertical hole measurements (BSP-1) suggested that the least stress was close to horizontal, while the horizontal hole measurements indicated that the least stress was vertical.

As shown in Figure 7, the hydrofracture orientations were influenced by the orientation of the boreholes, as the hydrofractures tended to align themselves in the plane defined by the borehole axis and the maximum principal stress directions.

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Chan, T. and K. Saari, 1981, Preliminary modeling of the in situ stress state in the area of the full scale heater experiments. Earth Sciences Division, Lawrence Berkeley Laboratory, unpublished technical memorandum


APPENDIX - ANALYSIS OF HYDRAULIC FRACTURING DATA

Calculation of Maximum Horizontal Stress Using Tensile Strength Term

The maximum horizontal stress results were determined using both the first breakdown technique, which requires the use of a tensile strength term, and Bredehoeft et al.'s (1976) second breakdown method. The first breakdown method is given by the familiar equation

\[ P_{bl} = 3 \sigma_{Hmin} - \sigma_{Hmax} + T - P_h \]

where \( P_{bl} \) is the first breakdown pressure, \( P_{si} \) is the shut in pressure, \( P_h \) is the pore pressure, and \( \sigma_{Hmax} \) and \( \sigma_{Hmin} \) are the maximum and minimum horizontal stresses. The pore pressure term was calculated from the hydrostatic gradient from the ground surface for the deep hole, SBH-4, data. The region around the full scale drift was considered to be drained thus no pore pressure term was used. Figure A1 shows the location of the various pressure terms on a typical pressure-time record from the Stripa work. The methods for obtaining the tensile strength and the shut-in pressures are described below. The second breakdown method used substitutes the difference between the first and second breakdown pressures for the tensile strength term. The value of the maximum horizontal stress interpolated to the depth of the test facility was 22.1 MPa for the first breakdown analysis versus 16.8 MPa for Bredehoeft's (et al., 1976) second breakdown method. The maximum stress value calculated using the first breakdown method was in better agreement with the
25.3 MPa value interpolated from the overcoring.

The first breakdown method has been little used in recent years because the laboratory tensile strength values for cores with small diameter boreholes were larger than one would expect for the borehole diameters used in the field. Ratigan (1981) has shown that the size effect can be handled using statistical fracture mechanics methods. His analysis of the hydrofracture tensile strength of the Stripa granite has been described elsewhere and does not bear repeating here, except that he concluded that there was an apparent tensile strength below which failure should not occur; this strength is equal to the square root of a critical strain energy release rate, which was 10.4 MPa for the Stripa Granite. The surface area of the test sections used in the hydraulic fracturing field tests was sufficiently large that the apparent tensile strength of the boreholes should have been the equal to 10.4 MPa. This tensile strength value has been used in analyzing the field hydrofracture measurements.

The tensile strength approach to analyzing the hydrofracturing data has several advantages over second breakdown methods. First, the second breakdown method as originally proposed by Bredehoeft, et al. (1976) would give theoretical second breakdown pressures less than the shut-in pressure value for stress state where the horizontal stress ratio is greater than two (Doe, et al., 1981). Zoback, et al. (1980) have proposed a modified secondary breakdown interpretation technique which considers the second breakdown to be a break in the slope of the pressure build-up curve. This method, while very promising, has yet to be independently confirmed through laboratory tests or through a detailed analytical or numerical analysis. Also, it is not clear how critical to the analysis is the assumption that the previously induce hydrofracture has negligible permeability. Using a first breakdown analysis with a reliable and realistic tensile strength term may be more effective for situations where core is available and for test systems where constant rate cannot be easily maintained, such as with air driven, positive displacement pumps.

**Determination of Shut-in Pressure**

A typical pressure-time record is shown in Figure A1. Three to six secondary pressurization cycles were used for measurement, and the shut-in pressures did not vary with the repeated pressurization. Shut-in pressures were determined using the fracture reopening pressures from a single slow flow rate pressurization cycle. An alternative method of obtaining the shut-in pressure involved using a semi-logarithmic plot of the pressure versus time for the period immediately following the first breakdown. An example of a semi-log plot is given in Figure A2. The logarithmic curves typically had a break in slope corresponding closely with the fracture reopening pressures from the slow pumping. The rationale for using the
A semi-logarithmic plot is the analogy between the post breakdown pressure decay and a pulse permeability test on a single fracture where the permeability decreases when the pressure in the fracture falls below the minimum in situ stress. It should be noted that this idea has not yet been developed theoretically and its main justification is the correspondence with the fracture reopening pressures from the slow pumping cycles.

Figure A1. Typical pressure time record for hydraulic fracturing test in SBH-4 (Test 17, 304 meters)

Figure A2. Semi-logarithmic plot of pressure after first breakdown versus time for test 11, SBH-4, 329 meters depth.
Table 1. Regression Statistics for SBH-4 Stress Measurement Results as a Function of Depth.

<table>
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<th>Slope* (MPa/m)</th>
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<th>Correlation Coefficient</th>
<th>Standard Error of Estimate (MPa)</th>
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* given with 90% confidence intervals
** correlation coefficient versus depth does not pass significance test
Table 2. Overcoring Data for BSP-1 and BSP-3 (Orientations given in Figures 10 and 11)

BSP-1 Swedish State Power Board Leeman Cell

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BSP-3 University of Luleå Leeman (LuH) Cell

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*given with 90% confidence intervals for means
Table 3
Hydraulic Fracturing Data for BSP-1 and BSP-2 **
(Orientations given in Figures 9 and 10)

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* given with 90% confidence levels for means

** numbers on shut in pressures refer to initial and final values; for BSP-2 data the HMax and HMin terms refer to stresses normal to the borehole and not necessarily horizontal stresses; the subscript min refers to the interpreted minimum stress from the final shut in pressure value.
Experiments of the In-Situ Stress Measurements Using the Stress Relieving and Hydraulic Fracturing Techniques

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Abstract

This paper presents the results of the in-situ stress measurements by stress relieving and hydraulic fracturing techniques in the same borehole drilled into the quartz diorite at depth between 0 and 90 meters in the village Shelongcheng, south of the Yi Xian ( county ), Hebei Province, as follows:

\[
\begin{align*}
\sigma_{H_{\text{max}}} &= 34 + 0.30H \text{ kg/cm}^2 \\
\sigma_{H_{\text{min}}} &= 25 + 0.31H \text{ kg/cm}^2 \\
\sigma_{V} &= 11 + 0.23H \text{ kg/cm}^2
\end{align*}
\]

Where \( H \) is depth in meters. The direction of the maximum mean principal horizontal stress was N 64° + 28W, and

\[
\begin{align*}
\sigma_{H_{\text{max}}} &= 49 + 0.28H \text{ kg/cm}^2 \\
\sigma_{H_{\text{min}}} &= 26 + 0.21H \text{ kg/cm}^2 \\
\sigma_{V} &= 11 + 0.23H \text{ kg/cm}^2
\end{align*}
\]

the maximum principal horizontal stress is orientated N 80° + 36E.

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Either of the two methods mentioned could give reliable results, but the hydrofracture technique as a stress measuring method is simpler and easier to handle; it is free from the restrictions of the borehole depth and the stress state in rock, it is a promising method for stress measurements. The paper further suggested that the in-situ stress measurement method is significant to the research related to the action features of the tectonic stress field as well as the development and occurrence of earthquake.

Introduction

For the purpose of studying the action features of tectonic stress field, since 1973 we have made in-situ measurements of absolute stresses using the stress-relieve method in five regions: North China, East China, Southwest China, Northwest China and Center-South China. It has been proved that the method is effective in the research of the regional tectonic stress field, but it requires measuring instruments with high sensitivity. These sophisticated instruments were hard to operate and they were easily effected by the elastic parameters of rock and the contact conditions with the borehole wall etc. Therefore the stress relieve method is used only in shallow boreholes and places where the differential stresses are low. Furthermore, the results obtained with this method should be checked by other methods. The hydraulic fracturing method is
a technique developed in recent years to measure stress directly. It can remedy the defects in stress relieving. With an eye to clarify which one is the more reliable and easier to perform we have conducted several tests using both the stress relieving and hydrofracturing techniques in a borehole at about the same identical depth.

Results of Stress Relieving

Table 1 gives four sets of data obtained by the piezomagnetic stressmeter in the borehole. From Tab. 1 we can see that the four sets of data are in good agreement with each other. The variations of the maximum and minimum principal horizontal stresses with depth are as follows:

\[ \sigma_{\text{Hmax}} = 34 + 0.30H \text{ kg/cm}^2 \]
\[ \sigma_{\text{Hmin}} = 25 + 0.31H \text{ kg/cm}^2 \]

where \( H \) is depth in meters. The average direction of the maximum principal horizontal stress was approximately N 64° + 20°W. From the above relationships it is evident that the gradient in variations for both the maximum and the minimum principal horizontal stress with depth are close in value.

Results of Hydraulic Fracturing

1. Experimental Equipment

Fig. 1 shows the measuring system of the hydraulic fracturing. Two inflatable rubber packers were located at the depth to be investigated to seal off the testing borehole segment. The fracture
testing segment is about 2.3m. long. Fracture fluid was injected into the sealed off segment with specified flow rate, pressurizing until fracture occurred at the wall of borehole. Both the pressure and flow rate changes versus time were simultaneously plotted on a x-y recorder through a pressure transformer and a flow rate gauge. The inclination and direction of the hydraulic fracture were determined by means of an impression packer (see Fig. 2) and a well T.V. (see Fig.4).

2 Measurement Technique and Experimental Results

Before the test was started the intact borehole segment without primary joints were selected as the hydraulic fracture segments by means of macroscopic and well T.V. Observation. The test borehole were divided into four fracture segments at depth of 18.05 - 20.36m, 38.49 - 40.81m, 70.98 - 73.29m and 82.00 - 84.45m. In view of rock intact and lower permeability. Water was adopted as fracturing fluid. Fig. 5.1 shows pressure changes versus time interval of 18.05 - 20.36m: the pressure to set up the packer was approximately 150 kg/cm², the pressure to open the injection valve was approximately 100 kg/cm². The pressure increased gradually with the injection of water in the sealed-off segment. Once the fracture of borehole wall occurred the pressure were dropped immediately. At this moment the critical fracture initiation pressure \( P_c \) on the record curved was approximately 88 kg/cm². When the injection pump was shut down and the test pipelines were sealed, it was observed that the instantaneous shut-in pressure \( P_s \) was approximately 29 kg/cm².
and then pressure reduction rate dropped off. The observed long-time shut-in pressure $P_{ASIP}$ was approximately 17 kg/cm$^2$ when pressure reduction rate change slowed down. Later the injection pump was started a second time, and the pressure in the sealed-off interval increased again, the fracture reopening pressure $P^1_b$ was measured to be 31 kg/cm$^2$ when pressure increasing rate is slowed.

The experiment was repeated several times until the obtained results of $P_s$ and $P^1_b$ were much the same as that of $P_s$ and $P^1_b$. The following three in-situ principal stresses were obtain from measurements:

$$\sigma_{H\text{min}} = P_s = 29 \text{ kg/cm}^2$$
$$\sigma_{H\text{max}} = 3\sigma_{H\text{min}} - P^1_b - P_o + P_f$$
$$= 54 \text{ kg/cm}^2$$
$$\sigma_V = P_{ASIP} = 17 \text{ kg/cm}^2$$

Where the pore pressure $P_o = 2 \text{ kg/cm}^2$, was approximately equal to the head pressure of the test segment. The frictional resistance of fluid $P_f$ was rather small, we have neglected it in this experiment. The direction of the new vertical fracture was found to be approximately N $71^\circ$ W (see Fig. 3) from the impression packer, from N $85^\circ$ W to N $89^\circ$E (see Fig. 4) by the well-T.V.. Using the same method, the other three sets of result were obtained. Pressure versus time curves are shown in Fig. 5.2, 5.3, and 5.4.

The values of the pressure measurements and the calculated stress are listed in Tab. 2 and Tab. 3 respectively.
As seen from the calculated stress results, the magnitudes of the principal stresses varied linearly with depth (see Fig 6). The relations between them are as follows:

\[
\begin{align*}
\sigma_{H_{\text{max}}} &= 49 + 0.28H \text{ kg/cm}^2 \\
\sigma_{H_{\text{min}}} &= 26 + 0.21H \text{ kg/cm}^2 \\
\sigma_V &= 11 + 0.23H \text{ kg/cm}^2
\end{align*}
\]

It should be pointed out that the values of measured principal stresses deviated comparatively far from the stress-depth curves for the fractured interval of 70.98 - 73.29m.

Discussion

The values of the principal stresses and their directions, obtained either from the stress-relieving or from the hydrofracturing, well agreed with each other. As shown in figure 6, the gradient in variation of the principal stresses versus depths were in accordance for both the methods. The consistency between the directions of the principal stresses obtained from the stress-relieving is better than those from the hydrofracturing which was bably caused by the inaccuracy of the orientation devices and the local defects of the borehole wall. The directions of the maximum principal horizontal stresses determined by both the methods were mainly from NWW to near EW (see Tab. 1, 3. and Fig. 7). The maximum principal stress values measured by hydrofracturing was slightly greater
than those from stress-relieving. The reason for the difference was that the pressure at fracture initiation could not be accurately determined during the hydrofracturing and thus the calculated maximum principal stress could not be accurately obtained on the other hand, the stress-relieving measurements were influenced by the wall conditions, for instance the transducer was not in full contact with the borehole wall.

The differences between the minimum principal stress values obtained from the two methods were relatively small, this could be due to the fact that the instantaneous shut-in pressure could be accurately determined during hydraulic fracturing. In hydraulic fracturing the values of the breakdown pressure are not the same at different rates of pressurization. The rock tensile strength obtained by hydraulic fracturing and by Brazilian indirect tension tests (in the cores of the borehole) were close in magnitude. The mean rock tensile strength determined by the Brazilian indirect tension tests was approximately 54 kg/cm² as shown in Tab.4, and the value obtained by hydraulic fracture in the fields was approximately 49.3 kg/cm² (see Tab. 3).

It is natural that the in-situ tensile strength was slightly lower than that of the intact pieces of rock measured in laboratory because the rock in well was acted upon by the pore pressure and is more saturated than the core, besides there is the size effect to be considered.
Newly-produced fractures in four measurements of hydraulic fracturing were all vertical or nearly vertical (with an inclination of $85^\circ$); horizontal fractures have not been discovered. In accordance with the measurements, $\sigma_V$ was the minimum principal stress. As pointed out by the theory and experiments of Haimson, Hubbert et. al., the fractures should be vertical to the rubber packer, even with $\sigma_V$ not being the medium principal stress. However, away from the wall of the hole the fracture could become horizontal, this phenomenon could be verified from the appearance of the gradual shut-in pressure on the pressure-time curve. The fact that in our experiments the gradual shut-in pressure varied linearly with depth and was paralleled with the theoretical curve seems also related to this phenomenon.

Summarizing, we can see that both methods were effective and reliable in the in-situ stress measurements for the research of tectonic stress fields. The hydraulic fracture method was simple and effective, it could not be effected by the elastic parameters and was suitable for different depths, including areas with large differential stresses. Thus the hydrofracturing method is widely used in deep stress measurements.

It has been found that the results from overcoring were effected by the accuracy of the measuring devices and their contact conditions with the walls as well as local rock conditions. The H. F. Method, though not having the above
mentioned drawbacks, its accuracy could be effected by the local rock defects (microfracture or joints) in the walls of the hole etc. Therefore, either methods would require many measurements. However, with the help of statistics, the results obtained could be comparatively good.

The In-Situ Stress Measurement and Tectonic Stress Fields

The in-situ stress measurements are now mainly used in the research of stress state of the crust of the earth, to establish the relationship between tectonic stress fields and earthquakes. Tab. 5 and Fig. 8 show the in-situ stress measurement results of recent years in North China.

As seen from the date even though the measurement of the direction of the in-situ maximum principal stress were not very consistant, but from them the general direction was found to be NWW to near EW. This result was in good agreement with the solution of Tangshan earthquake (7.8 Magnitude) mechanism of N75°E for the direction of principal compressive stress of release and also the results obtained from shallow earthquakes in Northeast China and North China. This also proved that the North China belonged to a same stress field. Due to the differences of tectonic positions, stress concentrations and also measurement errors, the in-situ principal stresses and directions obtained from different measurement locality differed. On the other hand, after the Tangshan earthquake of 7.8 magnitude in 1976, the in-situ maximum principal horizontal stress was measured to be approximately 25 kg/cm² at the seismic centre. This
was much smaller than the maximum principal horizontal stress of 60 kg/cm² measured in LuanXian, 55 km away from the seismic centre. Similar results were obtained from the south Longling earthquake in Yunnan province in the same year. It is believed that this phenomenon reflected the characteristics of the instantaneous stress variation during the stress release re-adjustment process of an earthquake.

To sum up, we might conclude that even though the in-situ principal stresses and their directions were controlled mainly by the regional tectonic stress fields it still could be effected by the local tectonic conditions, stress concentrations, energy releases and adjustments. Therefore, it is important to consider local effects of the tectonic conditions in the in-situ stress measurements. In a seismically dangerous area a rational arrangement of monitoring posts and stress measurement localities are of significance. In addition, while trying to obtain regional tectonic stress fields conditions through measurements done in mining tunnels, the geologic structural condition of the mine and the local excavating effects should be considered, since these effects sometimes may be misleading and over shadowing the real features of the regional tectonic stress field.
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### Table 1 Results of Stress Relieving Measurements

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<tr>
<th>No</th>
<th>Depth of relieving m (m)</th>
<th>( \sigma_{H_{\text{max}}} ) ( \text{kg/cm}^2 )</th>
<th>( \sigma_{H_{\text{min}}} ) ( \text{kg/cm}^2 )</th>
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### Table 2 Results of pressure measurement by hydraulic fracture

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<th>( P_{b_{\text{c}}} ) ( \text{kg/cm}^2 )</th>
<th>( P_{s_{\text{c}}} ) ( \text{kg/cm}^2 )</th>
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Table 3 Results of the in-situ principal stresses by calculation

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<th>$\sigma_{\text{Hmin}}$ (kg/cm$^2$)</th>
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Table 4 Brazilian indirect tension test results

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<th>No</th>
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Table 5. Results of the in-situ stress measurements in North China

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Fig. 1 Experimental equipment of the stress measurement by hydraulic fracture

Fig. 2 Photograph of impression packer
Fig. 3. Photograph of impression packer and the sketch from the hydraulic fracture at 18.05-20.36m.

Fig. 4 Borehole televiewer record for the hydraulic fracture at 19.97m.
Fig. 5 Surface pressure versus time during hydrofracturing
Fig. 6 Variation of in-situ stress with depth

- $\sigma_{\text{Hmax}}$, by hydraulic fracturing.
- $\sigma_{\text{Hmin}}$, by hydraulic fracturing.
- $\sigma_v$, by hydraulic fracturing.
- $\sigma_{\text{Hmax}}$, by stress relief.
- $\sigma_{\text{Hmin}}$, by stress relief.
- $\sigma_v$, by theoretical computation.
Fig. 7 Variation of directions of $\sigma_{H_{\text{max}}}$ with depth
1. Direction of the in-situ maximumPrincipal horizontal stress.
   ( * by hydraulic fracturing )

2. Direction of the solution of earthquake mechanism.

Fig. 8 The in-situ stress measurement results in North China.
Strain relief Stress Measurements along the San Andreas Fault in Southern California

by

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ABSTRACT

Near surface, strain relief stress measurements were made in the Mojave desert southeast of Palmdale, California at two sites during the summer of 1980, using the U.S. Bureau of Mines technique to depths of about 30m. Our field data and finite element modeling studies demonstrate that thermally induced stress dominates the results obtained in the upper 6m. At depths greater than 6m the average orientation for the horizontal maximum compressive stress at these sites is N22°W ± 9° at 2km from the San Andreas fault and N13°W ± 2° at 20km from the fault. These compare favorably with an average of N21°W determined by using nearby hydrofracture stress measurements [Zoback et al., 1980]. Savage et al., [1981] also found a NNW orientation for the maximum shortening from a geodetic network with a 15km aperture in the Palmdale area. The principal stress magnitudes at the near surface sites is also consistent with those obtained by Zoback et al., to depths of 849m. The fact that the same orientation is recovered by three different techniques which sample to different depths and over different areal extents and consistent magnitudes are measured by two of these argues strongly that tectonic stress is being measured.

These observations can best be interpreted with the aid of finite element models of the San Andreas fault in southern California. A stress field similar to that observed regionally is developed away from the fault, when displacements corresponding to relative motion between lithospheric plates are applied on the boundaries of the models. Near the fault, however, the model principal stresses are rotated counterclockwise by about 20° to 35° to azimuths which are very close to those measured near Palmdale. This stress rotation appears to be dependent on the rheology near the fault and fault orientation.
INTRODUCTION

A knowledge of the state of stress in the vicinity of active faults is one necessary ingredient in the understanding of the constitutive properties of fault zones and the mechanism of earthquake generation. Theoretical (Rogers and Chinnery, 1973) and laboratory (Barber and Sowers, 1974) research on strike-slip faults indicate that changes in the orientation and magnitude of the stress field should be expected in the vicinity of these faults, when they are loaded. These changes in stress are most likely a result of some combination of lowered rigidity near the fault, the strain accumulation/release history of the fault, fault geometry and asperities along the fault, which serve as locking points.

Our objective in initiating this research was to study the rheological properties and the nature of the strain accumulation and release cycle along fault zones. The work of Castle et al. [1976] in identifying the southern California uplift heightened the interest of the scientific community in the section of the San Andreas fault that last ruptured in 1857. Simple estimates of slip using global plate velocity data suggested repeat times of 100 to 200 years for an earthquake the size of the 1857 shock. Later more precise estimates of recurrence by Sieh [1978] are in general agreement with these numbers. Thus, the area of the San Andreas fault near Palmdale appeared to have a high strain accumulation and seemed a logical place to test if the stress field near faults was indeed altered as in the model studies, and if that alteration could be used to constrain the physical properties of the fault and the degree of strain accumulation.

Since the area in the vicinity of this section of the San Andreas fault is relatively aseismic, there are insufficient fault plane solutions of earth-
quakes to define the principal stress orientations. Thus an effort was made during the summer of 1977 to measure in situ stress using a strain relief technique at shallow depths [Sbar et al. 1979; Tullis, 1981]. At that time it was questionable whether in situ stress measurements could indeed detect tectonic stress. To demonstrate this became one of our objectives in these early studies. Within the next year hydraulic fracturing stress measurements were initiated in a profile from the San Andreas fault northward into the Mojave block [Zoback and Roller, 1979]. The strain relief measurements are characterized by consistent orientations at adjacent sites, but significant variation between groups of sites, while the hydraulic fracturing technique produced little data on stress orientation. Zoback and Roller did, however, detect a decrease in stress magnitude as the fault is approached from the north.

Subsequent to the above research, further strain relief stress measurements made in the winter and summer indicated that measurements less than 6m from the surface may be seriously affected by thermally induced stress. [Flaccus and Richardson 1981; Sbar and Richardson, 1981]. The latest strain relief stress measurements were made to depths of about 30m at two sites south of Palmdale during the summer of 1980. A third site was attempted in the winter of 1980 - 1981, but was aborted due to technical problems. This most recent data is reported in this paper and compared with nearby deep hydrofracture stress measurements obtained by Zoback et al. [1980] and geodetic strain data in southern California from Savage et al. [1981]. Through this comparison, a case is made that shallow strain relief stress measurements can detect tectonic stress. These results are then interpreted in terms of the regional tectonics of southern California, and fault rheology and geometry with the aid of finite-element models of southern California.
SITE DESCRIPTION

The surface geology of the western Mojave Desert consists of vast expanses of late-Tertiary alluvium which fill wide shallow basins between scattered buttes of Mesozoic granites and quartz monzonites (Figure 1). These buttes are often highly fractured and weathered. To the southwest the foothills of the San Gabriel Mountains contain sandstone conglomerates (late Miocene-early Pliocene Punchbowl Formation and the Paleocene-Eocene San Francisquito Formation), quartz monzonites, granites, and metamorphic rocks.

Sites LRS and TKY (Figure 1) are on a flat outcrop surrounded by alluvium among the low-lying ridges of the Punchbowl formation between the San Andreas and Punchbowl faults. The two sites are within 10m of each other. These sites are on the north limb of a large syncline whose axis strikes N70°W and plunges west. The attitude of the bedding is N40°-50°W, 50°SW. The Punchbowl formation consists of massive, light buff, cross-bedded, coarse terrestrial sandstone with large lenses of pebble and cobble conglomerate [Noble, 1954; Dibblee, 1967]. We made an effort to avoid the conglomerate whenever possible in the selection of intervals for stress measurement. The formation is remarkably unfractured, although there are bedding plane fractures spaced at 1 to 3m intervals and some vertical joints with locally varying attitudes, spaced at 3 to 7m intervals. In the cores obtained only few fractures were encountered.

Site IMS (Figure 1) was drilled in the Mesozoic quartz monzonites which form the basement of much of the western Mojave Desert. Gravity data [Mabey, 1960] indicate that in the western Mojave, there are two northeast trending basement highs separating three alluvium-filled basins. This site is along the southeasternmost of these highs on the southern lobe of Piute Butte about
20 km northwest of the San Andreas fault. Near the top of the core several steeply dipping fractures striking N40°E were found. Between about 6m and 14m depth a highly fractured zone of altered rock was encountered in which no measurements could be made. No systematic fracture orientation was present at these depths. Near the bottom of the hole steeply dipping fractures striking N85°E and gently dipping fractures striking N30°W were found.

TECHNIQUE

The U.S. Bureau of Mines borehole deformation (USBM) technique was used at all sites reported in this manuscript (Figure 2). A detailed description of the USBM technique is found in Hooker and Bickel [1974]. The gauge consists of twelve individual foil-resistance strain gauges mounted two apiece on six cantilevers, which are housed within a cylindrical steel borehole tool in a manner which will record horizontal displacements of a 3.8 cm (EX) borehole. The gauge is azimuthally oriented and set at a specified depth inside the 3.8 cm borehole and overcored in our operation by a 15.9 cm outer diameter - 14.3 cm inner diameter coring bit (Figure 2). The length of core cut in a single measurement is usually about 30 cm.

During the overcoring process the 3.8 cm borehole deforms in response to relaxation of the core upon stress relief. The USBM gauge records this deformation as a change in borehole diameter from which in situ stress may be calculated using the elastic moduli of the rock. The deformation observed is affected by the presence of fractures and strong rock anisotropy within the core. The method used to determine rock properties for the calculation of stress partially compensates for these effects.
Field Procedure  Figure 3 shows a typical borehole deformation curve recorded with the USBM gauge. Three horizontal components of diametrical borehole displacement are recorded from which a displacement ellipse and its azimuth are computed. The introduction of drilling fluid by a pump during overcoring sometimes results in a slight compression of the strain gauges and/or deformation of the 3.8 cm borehole, resulting in a displacement offset. To avoid this offset interpretations of strain relaxation data are restricted to the "pump on" period. A slight compressional bulge is observed just before the strain relaxation indicated by the large expansion. This may be due to the stress concentration around the end of the annulus as the core is being freed from the surrounding rock. The same effect is observed by Hooker et al. [1979] in a two-dimensional numerical simulation of the overcoring process. Drill down pressure was not included in the numerical work, so we assume the bulge is not due to that. The displacement for each component was then found by fitting horizontal lines to the data before the bulge and after the strain relaxation and noting their difference.

Once a displacement ellipse is determined from the borehole deformation measured during overcoring, an estimate of the elastic moduli of the rock is necessary to compute the corresponding stress. To obtain static determinations of these moduli, the USBM gauge is reoriented inside the 14.3 cm cores at the same position it occupied during the initial overcoring. A biaxial compression chamber applies a known radial load to a 20 cm length of core containing the gauge. Elastic parameters are obtained by the secant method using the recorded displacements in each of the three gauge-component directions as functions of applied pressure (Figure 4). Because crack closing and rock anisotropy cause the moduli to be stress dependent the moduli used for stress calculations are determined at the approximate displacement
magnitudes observed during the initial overcoring of each core. The unloading rather than loading part of the curve is used, since strain relief is an unloading process. A small non-recoverable strain is observed only during the first loading and unloading cycle. When the maximum applied pressure is constant, the second and later cycles return to the same value of displacement at zero applied pressure.

Data Reduction. Since we do not measure a sufficient number of elastic moduli to exactly correct for the anisotropy of the rock, we use an approximate technique developed by Tullis [1981] to account for anisotropy in calculating the stress. Equation (1) which yields Young's modulus for the isotropic case, is applied separately along each of the three axes of the strain cell.

\[ E_i = \frac{D^2}{D^2 - d^2} \cdot \frac{2P_o}{U_i} \]  

where \( E_i \) is Young's modulus, \( D \) is the outer diameter of the core, \( d \) is the inner diameter of the core, \( P_o \) is the applied radial pressure and \( U \) is the increase in diameter of the inner hole upon release of the pressure. The three values of \( E_i \) obtained are averaged to form \( \bar{E} \). Equation (2) is then applied to calculate a modified \( U_i \) along each axis.

\[ U_i^m = U_i \cdot \frac{E_i}{\bar{E}} \]  

Following this procedure the equations cited by Merrill and Peterson [1961] for the deformation of a borehole in an infinite plate with stress applied at infinity are applied to calculate the stress using the average Young's modulus \( \bar{E} \) and the modified displacements \( U_i^m \). Calculations were made for both the
plane strain and plane stress cases. A clear explanation of this procedure is found in Tullis [1981] and will not be repeated here.

DATA

The data reported in this paper were taken at sites IMS and LRS in the summer of 1980. TKY was sampled in the summer of 1979 and is included for comparison with LRS. The measurements at LRS and IMS were made to depths of about 30m specifically to avoid the effects of thermally induced stress. The previous year TKY was drilled to a depth of 10m. Observations by Hooker and Duvall [1971] and our previous measurements in the Mojave Desert [Flaccus and Richardson, 1981; Sbar and Richardson, 1981] convinced us that thermally induced stress is a significant source of noise in the upper 6m. The data below the thermal stress level appear to primarily reflect tectonic stress based on a comparison with geodetic strain and other stress data in the area.

The Punchbowl formation in which LRS and TKY were drilled is essentially unfractured. This permitted us to make a relatively large number of measurements in each of these holes. Both sets of data are plotted in Figure 5 to show the consistency in data from year to year (Tables 1 and 2); however, only the LRS data are used in the following analysis for reasons that will be explained below.

Twenty-one sets of elastic moduli were measured out of 40 possible at LRS because of breakage of the core on removal from the hole or core barrel. These moduli would be equivalent to the Young's moduli for isotropic rocks. The moduli were averaged to the values listed in Table 2 at a depth of 2.95m for example, and applied to all of the displacement data for which no moduli were determined. The average values are essentially isotropic, since the anisotropy at this site appears to be random (Figure 6; Table 2). This may
be a result of the scattered conglomeratic sections observed in the core. Also note that the degree of anisotropy is relatively low for most individual measurements. Poisson's ratio, also needed in the calculation of stress, could not be measured with the biaxial chamber used to determine the rock stiffness. A value of 0.4 was selected because of the relatively soft nature of these rocks [Jaak Daemon, personal communication, 1982]. The moduli determined on the cores from TKY were measured under a loading rather than unloading situation and were therefore unusable for calculating stress from strain relief data. Thus, the average moduli determined from the LRS data were used for computing stress at TKY.

In the calculation of stress from displacement an assumption of plane stress or plane strain must be used. With the exception of the near-surface points neither assumption is exact. We chose the plane strain equations for these data as a better approximation. Calculations for both cases were made and are discussed below. A comparison of the azimuths of the maximum displacement and the maximum horizontal stress in Table 2 demonstrates that the effect of the first-order anisotropic correction to the moduli is small. The azimuthal change is a function of both the percent anisotropy of the particular sample and the ratio of the maximum to minimum horizontal stress. The less the anisotropy and the greater the ratio the less the azimuth is influenced.

The other effect of the rock stiffness is on the magnitude of the stress. Only one measurement at LRS seems to be strongly modified compared to the others. The sample at 9.53m has a stiffness approximately twice that of the other data (Figure 6) and as a result has an unusually high calculated stress. The displacement of this sample is also relatively high. There is no obvious reason that can be seen in the core to explain the high
modulus observed. One other measurement stands out at 27.58m in the stress plot (Figure 5). This sample has the largest displacement of any measurement below 6m, but unfortunately uses the average moduli to compute the stress. If the true moduli of the rock were significantly lower than the average, these values might be more in line with the other data. In general the stress and displacement plots are quite similar in pattern indicating that the rock moduli are similar among the various samples.

An obvious feature of the stress values for LRS and TKY is the unusually high stress observed near the surface, which decreases exponentially with depth. Both the maximum and minimum horizontal stresses are affected. This can be attributed to the seasonal component of thermally induced stress. The magnitude of this stress is calculated by Richardson and Flaccus [in preparation] for realistic parameters at this site. The calculated curve in Figure 5 incorporates a 1.6 MPa, isotropic, tectonic stress component. The Young's modulus was varied until the curve approximately fit the data, at a value of 18 GPa, about four times the average for this site. This is the only parameter that is not close to the measured or expected values for this rock. If the plane stress assumptions were used for the near surface points, their magnitudes would be reduced about 15%, allowing a modulus of about 14 GPa to be used to fit the data. Another contributor to the discrepancy in stiffness could be in the method we used to determine the moduli for the stress calculation. The modulus determined using the biaxial chamber is not simply a Young's modulus, but is a linear combination of two or more elements of the stiffness matrix, depending on the nature of the rock anisotropy. Although these moduli are suitable for the calculation of stress, they may not be the appropriate ones for use in the expression for the thermal stress. Considering the simplicity of the Richardson and Flaccus model at
this time, a value of 14 to 18 GPa seems acceptable. The solid line of Figure 5 shows the resulting thermally induced stress using a one dimensional model. The fit of this curve to the data is reasonably good and strongly argues for thermal stress as the dominant factor in the measurements of the upper 6m.

The parameters we are most interested in are the mean azimuth and magnitude of the principal horizontal stresses and any possible variation of these with depth. We also seek to demonstrate that the observed stress is tectonic in origin. For the following analysis the upper 6m of the data are removed to eliminate the influence of thermal stress.

Fisher statistics [Mardia, 1972] are used to find the mean and standard deviation of the mean for the azimuthal data. In this method each azimuth is treated as a unit vector. The assumption is made that the azimuthal variation of the maximum horizontal compressive stress is random about some mean value. The azimuthal data are further edited by eliminating those samples which have poor resolution is azimuth. A measure of this is the ratio of the maximum to the minimum horizontal stresses (e.g. see Table 2). If the two stresses are approximately equal, the stress ellipse is essentially a circle and the azimuthal resolution is poor. Table 3 shows the effect of eliminating successively higher ratio data. For LRS the mean converges to N22° W with a standard error of the mean of ±9° for ratios > 1.4. The TKY data with a population of 6 indicate a mean of N44° W with a standard error of ±9°. The LRS data between 6m and 10.6m have a mean of N33°W with a standard error of ±5°. There may be a physical reason for the trend of the maximum horizontal stress to be more northwesterly at this depth range. Also there are several measurements between 19 and 21m that trend east-westerly. The reasons for these systematic variations in azimuth are not obvious in the data. It is
possible that local variations in the stress field can be caused by the heterogeneous nature of this rock. It is clearly valuable in this case to have a large number of samples to average these variations. Although the TKY data appear to be consistent with the LRS data both in magnitude and azimuth, we have chosen not to use them since they would bias the statistics of a reasonably uniformly sampled data set. The LRS mean azimuth is plotted in Figure 1 and referred to in later discussion.

A similar average azimuth was determined for the sites XTLR, MOJ1 and MOJ2 for the five fair or better quality azimuths of Zoback et al. [1980]. These three sites were lumped to include as many data as possible in the average. The average for all three sites is N21°W ±7° (Table 3). A separate average is plotted in Figure 1 for the two measurements at MOJ1, and the three at MOJ2 and XTLR since they are at different locations. The overall average is essentially the same as that for LRS.

The average magnitude for the maximum horizontal stress for depths below 6m is 1.62MPa. The average for the minimum horizontal stress is 1.07MPa, while that for the shear stress is 0.28MPa. A gentle increase with depth can be seen for each of these parameters. A linear regression of stress versus depth was computed for a variety of cases some of which are listed in Table 4. Basically all data were included below cutoff levels of 6, 7, and 9m for LRS. Data were not excluded by ratio, since this characteristic only applies to azimuthal reliability. The difference in either the slope or intercept among the different tests is not significant considering the standard errors of the data. The values for depth > 6m are used in the remainder of this paper. The errors are relatively large because of the small depth range over which the data were taken. It should be noted that both the maximum and minimum horizontal stress values are greater than the vertical stress due to
lithostatic loading in this depth range, which has an intercept of zero and a slope of 0.0235 MPa/m.

The magnitude data can easily be compared with the hydrofracture stress measurements determined by Zoback and Roller (1979) and Zoback et al. (1980) at the sites MOJ1, MOJ2 and XTLR shown in Figure 1. Magnitudes were measured at XTLR to a depth of 849m. A linear regression on the stresses obtained at XTLR alone, with MOJ2, and then MOJ1 and MOJ2 is listed in Table 4. The slopes found are quite well determined for the hydrofracture data because of the depth range covered, and compare favorably with ours at LRS. The intercepts, are poorly constrained, but are not statistically different from ours. A plot of the regression line for LRS with depth > 6m is superimposed on the data for XTLR, MOJ1 and MOJ2 for comparison in Figure 8. The purpose of this exercise is not to claim that we can predict the stress at 800m depth from data in the upper 30m, but to demonstrate the reliability of stress measurements at shallow depths.

Only sixteen measurements were made at IMS compared with the forty at LRS because the rock was more fractured at the former site. There was no evidence here for thermally induced stress as seen at LRS. We suspect this may be because the fractures at IMS close as the rock expands partially relieving the stress. A Poisson's ratio of 0.2 was selected for this site based on typical values for granitic type rocks (Haas, 1981). The displacements at IMS are lower than those at LRS, but the stiffness is four to five times greater yielding higher stresses (Table 5). The displacements shown in Figure 9 have more scatter than the stresses in Figure 10. At this site the effect of the moduli is to reduce the variation in magnitude. Although only seven sets of moduli were determined, it can be seen that the anisotropy in this rock, a quartz-monzonite, is quite uniform. The magnitude
of the moduli is lower for the shallower measurements (Figure 11). This may be because the shallower rock is more weathered. The amount of anisotropy is still not that great at this site and does not significantly change the azimuth in the calculation of stress (Table 5). The change in stress magnitude using the plane stress equations instead of plane strain is less than 5% for this site, which is not significant.

The mean azimuth of the data > 6m is N13°W ±2°. Since there are no measurements between 5.26m and 14.63m, only data > 14m are sampled. The rock at this site is highly fractured between these depths, and unsuitable for measuring stress. The azimuthal data have little scatter, thus the mean is tightly constrained. Also note that the ratio is uniformly higher for the IMS data than the LRS data, implying that the azimuths are better constrained. The mean at IMS is not statistically different from those at LRS or XTLR, MOJ1, and MOJ2.

The stress magnitudes do not vary in any systematic way with depth at this site, so there is no point doing a regression on the data. The average values are, however, higher than those at LRS. The maximum horizontal stress is 2.08MPa the minimum horizontal stress is 0.76MPa and the shear stress is 0.66MPa. This is consistent with the observations of Zoback et al. in which they noted higher shear stresses at sites farther away from the San Andreas fault than MOJ1 and MOJ2. Their normal stresses were also higher. The higher magnitudes could simply result from the greater stiffness for the rock at IMS compared with LRS.

TECTONIC INTERPRETATION

It is difficult to demonstrate unequivocally that the stresses measured in the Palmdale are indeed tectonic in origin. However several arguments can be posited that support this assumption. The stress orientation measured at
sites IMS and LRS, which are 22 km apart, is essentially the same. This orientation is the same as that obtained by Zoback et al. [1980] near site LRS using the hydrofracture technique (Figure 1). In addition the azimuth of maximum shortening averaged over 9.6 years by Savage et al. (1981) for their Palmdale network with an aperture of 15 km is of nearly identical orientation (Figure 12). The axis of maximum shortening and the axis of maximum compression are equivalent in an isotropic medium. Also the results at LRS when extrapolated to the depth of the hydrofracture measurements produce similar magnitudes. Both the hydrofracture data and the geodetic results measure deformation to much greater depth than the 30m we have obtained. Either the agreement in orientation is fortuitous, or we are all measuring a regional scale phenomenon, which implies that it is tectonic in origin.

Other influences on our stress measurements should also be considered. Excessive relief in topography can produce significant stresses [Harrison 1976; Jaeger and Cook, 1969]. At both sites IMS and LRS the relief is minimal and calculations of topographic stress produce no significant influence on our results. Thermal effects are clearly of concern for shallow stress measurements. Our measurements, however, extend to depths clearly beneath the zone of thermal influence. Residual stress remaining from previous tectonic events or due to some characteristic of the rock is another problem which must be considered. A double overcore was made at site LRS which produced very little strain-relief compared with the original overcore [Sbar et al. 1979]. This suggests that very little residual stress was stored in that rock. Double overcores were not made at site IMS, but the similar orientation for the principal stresses at both sites in very different kinds of rock of different ages makes the possibility of residual stress less likely.
We argue that the stress obtained at sites IMS and LRS is tectonic in origin and can be interpreted with other data from southern California in terms of the regional framework. Figure 13 shows that for the entire San Andreas region including areas as far east as the California-Nevada border the maximum compressional stress inferred from fault plane solutions of earthquakes is N14°E ± 9° [Sbar, 1982]. Stress and strain data from Palmdale and surrounding areas near the fault, however, indicate a maximum compressive stress of about N20°W. This counterclockwise rotation of about 35° can be interpreted using the finite element models developed by Richardson and Bergman [1979] and Sbar and Richardson [1981]. In those models a counterclockwise rotation of the principal stresses is observed in the vicinity of the San Andreas fault (Figure 14). This is a result of the change in the orientation of the San Andreas fault system in the Big Bend region. Different fault orientations with respect to the regional stress field will produce different amounts and directions of rotation. In this model the elements comprising the fault are thinner than the remainder of the elements by a factor of ten. They are also more compliant than the elements away from the fault. These combined effects cause the stresses to be greater along the fault.

The strain data of Savage et al. [1981] strongly support this model. All of their networks that span the locked portion of the San Andreas fault show a counterclockwise rotation of the maximum shortening to NNW, while those networks in other parts of southern California show the regional trend for the maximum shortening. We suspect that the amount of strain accumulation also influences the amount of rotation, but elastic models in Figure 14 are unable to demonstrate this.
SUMMARY

In the course of this research program we believe that we have learned how to make reliable measurements of tectonic stress near the surface. Initially we made measurements to only 3m depth. Analysis of our field data and finite element modeling of thermal stresses indicates that we would have to sample below 6m to obtain reliable observations of tectonic stress. The results at two sites presented in this paper agree favorably in orientation and magnitude with those obtained in the same area by Zoback et al. [1980] at depths to 849m with the hydrofracture technique, and with those of Savage et al. [1981] from geodetic observations. We measured azimuths of N22° W ± 9° and N13° W ± 2° compared with N21° W ± 7° for Zoback et al. and NNW for Savage et al. These data plus other data along the locked section of the fault from Savage et al. (Figure 12) all show a counterclockwise rotation away from the azimuth of the regional horizontal maximum compressive stress (N14°E). This rotation is also observed in the numerical models of the San Andreas fault in southern California [Richardson and Bergman, 1979] and can be explained in terms of fault rheology, orientation and effective thickness.

REFERENCES


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TABLE 2 - Stress Data for Site LRS
Min
Horiz
Stress
MPa

Azim
of Max
Stress
E of N

Ratio

m

Max
Horiz
Stress
MPa

0.39
1.93
2.51
2.95
3.30
4.04
4.45
4.85
6.55
6.96
7.37
8.10
8.53
9.09
9.53
10.06
10.54
11.43
12.12
12.37
13.26
14.91
15.32
15.72
16.81
17.25
17.78
18.57
18.97
19.38
21.03
21.51
22.40
22.91
23.37
24.16
24.56
25.88
26.34
27.58

5.17
3.63
2.87
3.93
3.14
2.63
2.43
2.42
1.40
1.45
1.36
1.56
1.29
1.21
2.69
1.58
1.26
1.42
1.87
1.34
1.98
1.01
1.84
1.21
1.89
1.77
1.63
1.65
1.32
2.08
1.41
0.73
1.30
1.97
1.96
1.53
1.62
2.45
1.83
4.19

4.90
2.79
2.25
3.60
2.68
2.32
1.78
1.56
0.73
1.17
0.48
0.99
0.92
0.69
0.72
1.36
0.92
1.27
1.27
0.84
1.86
0.85
1.42
0.81
1.23
1.57
1.41
1.44
1.04
1.20
0.63
0.20
1.02
1.48
1.63
0.36
0.32
1.83
U15
3.10

-86
-57
-62
27
86
60
-46
-51
-39
-48
-34
-23
-28
-27
-23
-73
-60
46
-2
-85
-2
-78
38
-28
-1
-50
77
-52
-65
56
-77
85
-40
-27
-28
-38
-13
-10
-29
29

1.1
1.3
1.3
1.1
1.2
1.1
1.4
1.6
1.9
1.2
2.9
1.6
1.4
1.7
3.7
1.2
1.4
1.1
1.5
1.6
1.1
1.2
1.3
1.5
1.5
1.1
1.2
1.1
1.3
1.7
2.2
3.7
1.3
1.3
1.2
4.3
5.1
1.3
1.6
1.4

Depth

Shear
Stress
MPa
.14
.42
.31
.16
.23
.15
.32
.43
.34
.14
.44
.29
.19
.26
.98
.11
.17
.08
.30
.25
.06
.08
.21
.20
.33
.10
.11
.10
.14
.44
.39
.27
.14
.24
.17
.59
.65
.31
.34
.55

Max
Horiz
Displ
um
100.7
42.1
83.1
63.4
88.4
42.5
42.0
43.4
26.6
24.2
31.3
32.3
19.6
18.7
31.1
17.6
20.9
22.5
31.0
24.0
31.8
21.4
29.2
30.1
34.7
28.4
26.5
26.7
28.4
46.3
27.3
15.1
32.6
34.1
36.7
32.4
35.4
42.9
31.9
73.4

Min
Horiz
Displ
inn

Ajzim
of Max
Displ
E of N

90.5
28.1
45.7
52.1
65.8
33.5
22.5
17.5
6.1
15.8
-0.0
12.4
9.0
5.7
-3.7
11.8
13.3
13.7
17.5
9.3
27.2
15.3
18.3
12.8
13.3
22.8
20.1
20.7
20.9
35.7
3.8
-1.1
17.7
18.8
19.9
-3.4
-5.4
22.9
11.0
38.6

-81
-71
-71
23
64
57
-44
-50
-38
-46
-37
-21
-27
-29
-15
-69
-63
51
-12
-85
0
-74
47
-27
-1
-46
75
-48
88
29
-77
85
-38
-26
-15
-37
-12
-9
-36
28

374

Max
Stiff

Min
Stiff

GPa

GPa

3.40
6.55
2.85
4.25
2.70
4.25
4.25
4.25
4.25
4.25
5.26
3.80
5.20
5.11
10.29
6.71
4.41
5.42
4.67
4.25
4.25
3.33
4.75
3.04
4.25
4.25
4.25
4.25
3.60
3.69
4.25
4.25
3.20
4.25
4.71
4.25
4.25
4.25
4.92
4.25

3.35
5.13
2.29
4.10
2.14
4.10
4.10
4.10
4.10
4.10
3.26
3.50
4.80
4.83
6.42
6.13
3.72
4.23
3.55
4.10
4.10
3.12
4,02
2.98
4.10
4.10
4.10
4.10
2.52
1.36
4.10
4.10
2.78
4.10
3.57
4.10
4.10
4.10
4.07
4.10

Azim
of Max
Stiff
E of N
53
-31
-5
-84
-64
-E4
-84
-84
-84
-84
52
-55
69
7
-69
34
-52
-36
18
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71
9
-80
-84
-84
-84
-84
-45
60
-84
-84
60
-84
88
-84
-84
-84
23
-84

Stiff
Anisot
%
2
22
20
3
21
3
3
3
3
3
38
8
8
5
38
9
16
22
24
3
3
6
15
2
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3
3
3
30
63
3
3
13
3
24
3
3
3
17
3


### TABLE 3 - Mean Azimuth of Maximum Horizontal Stress

<table>
<thead>
<tr>
<th>Site</th>
<th>Mean E of N</th>
<th>Standard Error of Mean</th>
<th>Number of Observations</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>IMS</td>
<td>-13°</td>
<td>±2°</td>
<td>9</td>
<td>D* &gt; 14m</td>
</tr>
<tr>
<td>LRS</td>
<td>-25°</td>
<td>±8°</td>
<td>28</td>
<td>D &gt; 6m, R &gt; 1.2</td>
</tr>
<tr>
<td>LRS</td>
<td>-22°</td>
<td>±8°</td>
<td>23</td>
<td>D &gt; 6m, R &gt; 1.3</td>
</tr>
<tr>
<td>LRS</td>
<td>-22°</td>
<td>±9°</td>
<td>18</td>
<td>D &gt; 6m, R &gt; 1.4</td>
</tr>
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<td>±5°</td>
<td>7</td>
<td>6m &gt; D &gt; 10.5m</td>
</tr>
<tr>
<td>TKY</td>
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<td>±9°</td>
<td>6</td>
<td>D &gt; 6m, R &gt; 1.7</td>
</tr>
<tr>
<td>XTLR, MOJ1 &amp; MOJ2</td>
<td>-21°</td>
<td>±7°</td>
<td>5</td>
<td>80m &gt; 787m</td>
</tr>
<tr>
<td>Palmdale Trilateration</td>
<td>-15°</td>
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<td></td>
<td>~80km x 25km</td>
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* D is the depth
## TABLE 4 - Regression analysis of Stress Magnitude

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<th>Site</th>
<th>Number of Observations</th>
<th>Dependent Variable</th>
<th>Zero Depth Intercept</th>
<th>Slope MPa/m</th>
<th>Correlation Coefficient</th>
<th>Remarks</th>
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<tbody>
<tr>
<td>LRS</td>
<td>32</td>
<td>( \sigma^a ) 1H</td>
<td>1.09 ±0.57</td>
<td>0.0379±0.158</td>
<td>0.402</td>
<td>D &gt; 6m</td>
</tr>
<tr>
<td>LRS</td>
<td>30</td>
<td>( \sigma ) 1H</td>
<td>1.05 ±0.58</td>
<td>0.0396±0.177</td>
<td>0.389</td>
<td>D &gt; 7m</td>
</tr>
<tr>
<td>LRS</td>
<td>27</td>
<td>( \sigma ) 1H</td>
<td>1.03 ±0.62</td>
<td>0.0408±0.215</td>
<td>0.355</td>
<td>D &gt; 9m</td>
</tr>
<tr>
<td>XTLR, MOJ1 &amp; MOJ2</td>
<td>15</td>
<td>( \sigma ) 1H</td>
<td>0.778 ±2.15</td>
<td>0.0422±0.021</td>
<td>0.985</td>
<td>80m &gt; D &gt; 49m</td>
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<tr>
<td>XTLR &amp; MOJ2</td>
<td>10</td>
<td>( \sigma ) 1H</td>
<td>2.50 ±2.42</td>
<td>0.0397±0.029</td>
<td>0.979</td>
<td>149m &gt; D &gt; 849</td>
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<tr>
<td>XTLR</td>
<td>7</td>
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<td>0.251 ±2.72</td>
<td>0.0428±0.049</td>
<td>0.969</td>
<td>266m &gt; D &gt; 849</td>
</tr>
<tr>
<td>LRS</td>
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<td>( \sigma ) 2H</td>
<td>0.702 ±0.540</td>
<td>0.0267±0.151</td>
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<tr>
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<td>0.0279±0.168</td>
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</tr>
<tr>
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<td>0.0238±0.202</td>
<td>0.229</td>
<td>D &gt; 9m</td>
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<tr>
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<td>( \sigma ) 2H</td>
<td>1.51 ±1.14</td>
<td>0.0215±0.019</td>
<td>0.984</td>
<td>80m &gt; D &gt; 849m</td>
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<tr>
<td>XTLR, &amp; MOJ2</td>
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<td>( \sigma ) 2H</td>
<td>2.08 ±1.26</td>
<td>0.0205±0.015</td>
<td>0.979</td>
<td>149m &gt; D &gt; 849m</td>
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<tr>
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<td>0.966</td>
<td>266m &gt; D &gt; 849m</td>
</tr>
<tr>
<td>IMS</td>
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<td>( \sigma ) v</td>
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<td>0.0262</td>
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</tr>
<tr>
<td>LRS</td>
<td></td>
<td>( \sigma ) v</td>
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<td>0.0235</td>
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</tr>
</tbody>
</table>

\( a \) - \( \sigma_{1H} \), Maximum horizontal stress; \( \sigma_{2H} \), minimum horizontal stress; \( \sigma \) \( v \), vertical stress

\( b \) - D, Depth
<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Azim of Max</th>
<th>Stiff</th>
<th>% Anisot</th>
<th>Max</th>
<th>Ratio</th>
<th>Min</th>
<th>Stiff</th>
<th>MPa</th>
<th>E of N</th>
<th>um</th>
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FIGURE CAPTIONS

Figure 1. Geologic map of the study area. The heavy arrows denote our strain relief sites. Light arrows are hydrofracture sites of Zoback and Roller [1979] and Zoback et al. [1980].

Figure 2. Cross sectional view of U.S. Bureau of Mines overcoring technique before and after overcoring.

Figure 3. Example of a strain relaxation history during overcoring of the U.S.B.M. gauge. Values along the abscissa indicate distance of overcoring. The location of the strain gauge (GD) is 120 mm below the depth where overcoring begins. Values along the ordinate are displacement recorded by the gauge, which is transformed to stress. Symbols: triangle-North, square-Southeast, star-Southwest.

Figure 4. Applied pressure vs. diametral contraction for samples from sites IMS and LRS. Secant moduli are determined from these curves at approximately the deformation observed in the original overcore. Notation as in Figure 3.

Figure 5. Stress values for LRS and TKY. The solid symbols are the maximum horizontal stress. The open symbols are minimum horizontal stress. Circles are LRS data. Squares are TKY data. Azimuth is indicated by the bar on the solid symbols. Up is north. Azimuths are only plotted for data with a ratio >1.4.

Figure 6. Moduli for LRS. Symbols as in Figure 5.

Figure 7. Displacement data for LRS. Symbols as in Figure 5.

Figure 8. Hydrofracture stress measurements from Zoback and Roller [1979] and Zoback et al. [1980] for sites XTLR, MOJ1 and MOJ2 within 4km of the San Andreas fault in southern California. The values with large error bars indicate maximum horizontal stress, and the ones with small error bars are least horizontal stress. The solid lines are extensions of the regression lines for the data from site LRS for depths > 6m (Table 4). The maximum horizontal stress line is to the right.

Figure 9. Displacement data for IMS. Symbols as in Figure 5.

Figure 10. Stress values for IMS. Symbols as in Figure 5.

Figure 11. Moduli for IMS. Symbols as in Figure 5.

Figure 12. Map of southern California showing the locations of the seven trilateration networks and the average principal strain rates measured at each. Each network is identified by name, period covered by the surveys, and the principal strain rates (extension reckoned as positive) in μstrain/yr. The directions of the principal strains are indicated by the diagram beside each label. The heavy sinuous lines represent the major faults. From Savage et al. [1981].
Figure 13. Lower hemisphere equal area projection of P (solid circle) and T (open circle) axes from fault plane solutions of earthquakes along the San Andreas fault system. The arrows pointing inward denote the inferred maximum horizontal compressive stress and those pointing outward mark the least horizontal compressive stress. From Sbar [1982].

Figure 14. Principal stresses for model A4, where stress has been concentrated along the fault by lowering the effective thickness across which elastic stress can be supported to 10 km. The stresses are rotated counterclockwise along the fault, with an orientation of about N15° for the maximum compressive stress. From Sbar and Richardson [1981].
OVERCORING A BUREAU OF MINES GAUGE
Figure 3

AZIMUTH UI = -86°

LRS #30

DIAMETRAL DEFORMATION (mm)

OVERCORE DEPTH (MM)

GD
Figure (M M)
Figure 10

IMS

HORIZONTAL STRESS (MPa)

0 1 2 3 4 5

DEPTH (M)

10 15 20 25 30

389
TEHACHAPI 1973-1979.9
\( \dot{e}_1 = 0.10 \pm 0.03 \)
\( \dot{e}_2 = 0.15 \pm 0.03 \)

GARLOCK 1973-1979.1
\( \dot{e}_1 = 0.02 \pm 0.03 \)
\( \dot{e}_2 = 0.14 \pm 0.03 \)

PALMDALE 1971-1980.6
\( \dot{e}_1 = 0.17 \pm 0.02 \)
\( \dot{e}_2 = 0.18 \pm 0.02 \)

CANYON 1974-1980.2
\( \dot{e}_1 = 0.08 \pm 0.02 \)
\( \dot{e}_2 = 0.13 \pm 0.03 \)

LOS PADRES 1973-1980.4
\( \dot{e}_1 = 0.06 \pm 0.14 \pm 0.03 \)
\( \dot{e}_2 = 0.13 \pm 0.03 \)

ANZA 1974-1980.4
\( \dot{e}_1 = 0.06 \pm 0.02 \)
\( \dot{e}_2 = 0.14 \pm 0.02 \)

SALTON 1972-1979.9
\( \dot{e}_1 = 0.14 \pm 0.02 \)
\( \dot{e}_2 = 0.21 \pm 0.02 \)
San Andreas Fault
Figure 11

STIFFNESS (GPa)

DEPTH (M)

---

390
MODEL A4: PRINCIPAL STRESSES

N

50 KM

100 BARS
A hydraulic fracture may be created by pumping water into a cavity in a rock mass at sufficiently high pressure. Until fracture extension begins, the pressure history in the cavity is determined by the flow rate, the cavity shape, and the strength of the rock. If the rock is permeable and a pre-existing fluid pressure, $P_0$, exists in the pores of the rock, the concept of a surface pressure may be replaced by a body force on the rock, equal to the negative gradient of the fluid pressure. The integral of this body force in a direction normal to the wall is equal to the cavity pressure, $P$, minus the pore pressure, $P_0$, and if the gradient is concentrated very near to the wall, this pressure may be treated as the effective pressure acting on the wall of the cavity to create fracture. This effective pressure, combined with the matrix stresses in the rock, must create a tensile stress greater than the rock strength in order for fracture to occur. All else being equal, this will occur first in a plane normal to the least principal matrix stress, $\sigma_3$, (considered positive in compression).

In any case, as the largest dimension of the fracture becomes several times that of the largest dimension of the cavity, the cavity shape no longer influences the pressure. It depends only on the flow rate, the matrix stress normal to the fracture face, and the strength of the rock. In a brittle material, like most rocks, the fracture opening is small compared to the fracture length, and if the stress field in the earth does not vary greatly about the fracture, the fracture will grow in two dimensions. In this case, as the fracture becomes larger, the strength of the rock no longer influences the cavity pressure, which depends only on the flow rate and earth stress. If the flow is now stopped, the fracture pressure equalizes, and, ideally, there is a time during which the pressure is constant, until the fluid permeates into the rock and the fracture closes. This pressure is defined as the instantaneous shut-in pressure, or ISIP, and, if it is observable, is a measure of the least principal matrix stress in the rock. Because it is independent of cavity shape and rock strength, both of which are unknown in practice, it is usually considered the most reliable measure of earth stress at depth.
Shut-in Measurements in Crystalline Rock

The Los Alamos National Laboratory has drilled four holes into crystalline basement rock as part of the Hot Dry Rock (HDR) Geothermal Energy Development Program. The experimental site is on the western flanks of the Valle Caldera, a volcano which has been active in the last million years. The site known as Fenton Hill, is located about 50 km west of Los Alamos, New Mexico. The last hole, designated EE-3, was drilled to a depth of 4 km and cased to 3.08 km. The open hole section is inclined at 28 to 36 degrees from vertical. In order to determine the earth stress in the open hole section, the well was pressurized at several different flow rates varying from 0.57 \( \ell/s \) to 2.52 \( \ell/s \).

The initial pressurization, at 1.07 \( \ell/s \), and the first repressurization, are shown in Figure 1. A vent and a seven hour waiting period took place between these pressurizations. After a temperature log, the repump was terminated with a 54 minute shut-in, shown in Figure 2.

The initial pressurization curve deviated from a straight line near 6 MPa, indicating some pre-existing permeability in the well. The shut-in curve had no region of constant pressure from which the ISIP could be inferred. Such behavior has characterized all of the fractures made at the Fenton Hill site. Other methods of finding the fracture pressure had to be used, and it is the purpose of this paper to define one such method which has been found useful, not only for determining the ISIP during fracture creation, but also for determining the pressure inside the fracture, near the exit and entrance wellbores, when a circulation of fluid through a fracture is taking place.

The wellbore pressure, itself, is not a satisfactory measure of the pressure inside a fracture in these rocks for several reasons. There is a pressure drop concentrated near the wellbore because the flow rate is highest there. If the flow rate is decreased, permeation limits the pressure which can be attained within a reasonable time to a value less than the earth stress. It has been determined experimentally that the pressure inside the fracture can be found by a method of analysis which was originally suggested by Muskat for analyzing pressure buildup in oil wells. Such an analysis has become known here as the "Muskat method", and is described below:

Muskat Method

The basic assumption of the Muskat method is that, after a short transient period, the shut-in pressure approaches an asymptotic value, \( P_a \), in an exponential fashion, i.e., if \( P_a \) is subtracted from \( P \) at each time, \( t \), and the result is plotted, \( \ln(P-P_a) \) vs. \( t \) will be a straight line. Various values of \( P_a \) are tried until the best straight line fit is found. The transient period is usually only a few minutes, while the straight line may extend over several hours.

The straight line is extrapolated back to the time of shut-in, giving a pressure \( P_e \). Then \( P_e + P_a + \) hydrostatic pressure is taken as the pressure inside the fracture during pumping, a value very near the total earth stress if the fracture is large. Figure 3 shows a Muskat analysis of the shut-in Figure 2. The value of \( P_a + P_e \) is 8.73 MPa.
Because of the observed permeation at pressures below the peak pressure in Figure 1, it was decided to pump the fracture at several different rates and ascertain that the earth stress had actually been reached in the first experiment. Figure 4 shows the pressure history of the second experiment and Figure 5 shows the Muskat analysis of the shut-in pressure/time curve after the highest flow rate. $P_a + P_e$ was 9.01 MPa, about 3% higher than before, which is considered to be good agreement.

As a check on the Muskat method, the peak pressures reached at each flow rate were plotted against flow. Adding in a point at the origin and drawing a smooth curve through the points, we have Figure 6. The knee in the curve was taken as 9.1 ± 0.2 MPa, and provides a second measure of the fracture opening pressure, in good agreement with the Muskat values.

The fracture depth was found from temperature logs to be 3.09 km below the surface. Hydrostatic pressure at this depth was calculated to be 28 MPa, so the total earth stress was 37 MPa. This value fits smoothly with the data of previous experiments, given in Table I. The variation of stress with depth may be anomalous at Fenton Hill, since it is only 2.7 km from the ring fault of the Valles Caldera.

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Figure 1. Initial Pressurization and Repump of EE-3.
Figure 2. Pumpup and Shut-in at 1.07 l/s.
Figure 3. Muskat Analysis of Shut-in Curve of Figure 2.
UNITED STATES DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY

PROCEEDINGS OF
WORKSHOP XVII

WORKSHOP ON HYDRAULIC FRACTURING STRESS MEASUREMENTS
VOLUME II

Convened under Auspices of
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Compiled by
Muriel Jacobson

This report is preliminary and has not been reviewed for conformity
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nomenclature. Parts of it were prepared under contract to the USGS
and the opinions and conclusions expressed herein do not necessarily
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MENLO PARK, CALIFORNIA
1982
Figure 4. Second Experiment in EE-3. Shut-in Curves at Higher Flow Rates.
FLOW RATE = 2.52 \text{ l/s}

P_e + P_a = 9.01 \text{ MPa}

P_a = 5.89 \text{ MPa}

TIME (min)

Figure 5. Muskat Analysis of Last Shut-in Curve Shown in Figure 4.
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HYDRAULIC FRACTURING

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Figure 6. Alternative Method for Deriving Fracture Opening Pressure.
NOTE ON EFFECTS OF INFILTRATION ON THE CRITERION FOR BREAKDOWN PRESSURE IN HYDRAULIC FRACTURING STRESS MEASUREMENTS

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ABSTRACT:

A review of the literature on hydrofracturing stress measurement has shown a need for clarification of the effects of fluid infiltration on the criteria for crack initiation and breakdown.

The theoretical criterion of crack initiation, based on infiltration effects, has been found to be supported by published laboratory tests under conditions of normal infiltration at low stresses, as well as of negligible infiltration at high stresses. Further confirmatory tests are needed, but it seems likely that the simple criterion currently in use, derived for the condition of no infiltration, may become superseded by a criterion requiring knowledge of a crack porosity parameter which is stress dependent and stress anisotropic, for which little data is presently available.

The rate of infiltration into the initiated fracture determines the time lag to the breakdown pressure. Formulae are given for the time lag and show that it is highly sensitive to some parameter values (modulus, tensile strength, stress), nearly proportional to viscosity, and less influenced by infiltration into the freshly exposed faces of the propagating crack. The time lag becomes indefinitely large as tensile strength tends towards zero. The predictions have not been checked against the published data from rate-controlled tests.
1. INTRODUCTION

The simple formula in use today for the breakdown pressure in hydrofacturing stress measurements is generally regarded as enabling an estimate only of the major principal stress, $\sigma_1$ to be determined, to an accuracy, say, of 25%,

$$P_b = 3\sigma_2 - \sigma_1 + T - P_o$$

where

- $P_b$ = breakdown pressure
- $\sigma_1$ = major horizontal principal stress
- $\sigma_2$ = minor horizontal principal stress
- $T$ = tensile strength
- $P_o$ = virgin pore pressure

The first two terms on the right are due to Hubbert and Willis (1957), and the last two, to Scheidegger (1962). The formula does not include the effects of infiltration on the breakdown. The object of this note is to discuss the infiltration effects with a view to future improvements in the accuracy of measurement of $\sigma_1$. The infiltration effects comprise,

(i) infiltration into the borehole wall surface
   (a) the effective stress tensile failure criterion
   (b) the compressive poro-elastic stress

(ii) infiltration into the propagating crack
   (a) viscous flow into the crack
   (b) absorption into the crack surfaces

2. INFILTRATION INTO THE BOREHOLE WALL SURFACE

(a) Effective Stress Tensile Failure Criterion

Scheidegger's introduction of the term $P_o$ in (1) was a nominal but incorrect step toward the use of an effective stress tensile failure criterion. He regarded the rock as impervious during the time of the test, but with tensile strength reduced by the pre-existing pore pressure. The effective stress criterion was introduced by means of a fracture mechanics analysis by Abou-Sayed et al., (1978),

$$P_b = 3\sigma_2 - \sigma_1 + T - P_b$$

(2)
They regarded the fracture as being initiated at a microcrack in the borehole periphery, where the pore pressure attains the applied fluid pressure, no matter how small the macroscopic depth of the fluid infiltration may be, even in nominally impervious rocks. They commented on the difference between (1) and (2), but had no explanation as to why the equation (1), with its lack of rigor, should be approximately correct for general application to hydrofracture, whereas the more rigorously derived equation (2) was clearly not generally applicable.

The error in equation (2) was due to overlooking the state of compressive stress developed in the peripheral skin of the borehole wall by the infiltrated fluid under pressure.

(b) Compressive Poro-Elastic Stress

Geertsma (1966) had already given the corrected hydrofracture criterion allowing for the wall skin reinforcement due to poro-elastic compressive stress, and assuming failure by effective stress at the wall surface,

\[ P_b = 3\sigma_2 - \sigma_1 + T - P_b + \beta(P_b - P_o) \]

where, \( \beta = \alpha(1-2\nu)/(1-\nu) \)

\( \alpha = \) crack porosity (coefficient of Biot).

The criterion was written by Haimson and Fairhurst (1967) in the form:

\[ P_b = \frac{T}{C} \left[ 3\sigma_2 - \sigma_1 + T \right] - \frac{1}{C}P_o \]

where

\( C = 2-\beta \quad \frac{1}{C} = \beta/(2-\beta) \)

3. LABORATORY TESTS CONFIRMING SURFACE INFILTRATION EFFECTS

Conformation of the equation (3) for the effects of infiltration into the rock pores, came from tests on a range of rocks, by Edl (1973). The rock property test determinations of the crack porosity \( \alpha \) varied with the stress level. At high stresses, for which \( \alpha \to 0 \), the breakdown pressure agreed with equation (3), with \( \frac{1}{C} = 0.5 \), and demonstrated the invalidity of the old theory, (1). This confirmed the applicability of the effective stress term, even when the rock was effectively impervious - the condition analysed by Abou-Sayed et al., (equation (2)). At low stresses, where \( \alpha \) tends towards unity, Edl (1973) found that \( \frac{1}{C} \approx 1 \). Haimson (1976) reported that for low to moderate stress levels, which represented most field situations, the results did not distinguish between the new and old theories. Paradoxically, the new theory, for pervious rock, could not be distinguished from the old theory, for
impervious rock. Results which distinguish between the theories, as applied to pervious rock at low stresses, are given in the work of Kim and Gray (1978). Kim and Gray's objective was the study of fracture propagation, using rapid pressurization, and they did not analyse their results with respect to the hydrofracture criterion (3), although all test data to enable this analysis to be made were given. The writer computed the following values of the coefficient \( \frac{1}{C} \) from the test data given for Berea sandstone:

- From the hydrofracture breakdown pressures at stresses covering the range 0 to 34.5 MPa, \( \frac{1}{C} = 0.72 \).
- From the rock properties tests (bulk modulus and Poisson's ratio at stresses over the range 0 to 34.5 MPa), \( \frac{1}{C} = 0.73 \).

Such close agreement in \( C \) values - sufficient to validate equation (3) in preference to (1) - was however, not obtained from data given for tests on a limestone, possibly owing to the rapid pressurizing.

4. FRACTURE INFILTRATION

(a) Distinction Between Crack Initiation Pressure and Breakdown Pressure

Increased injection rates were frequently observed to give increased breakdown pressure.

Zoback et al., (1977) suggested that this was explained by fluid pressure loss in the propagating crack due to viscous drag, the crack being only partly filled, leaving a vacuum tip. Crack initiation was observed to be distinct from breakdown under some test conditions (Zoback et al., 1977). Crack initiation was observed by acoustic emission recording, and found to be consistently related to rock stress (in test blocks) and tensile strength, independently of fluid viscosity. Breakdown was however sometimes delayed and at higher pressure than for crack initiation, depending on pressurizing rate and fluid viscosity.

We therefore revise (3) to:

\[
P_i = \frac{1}{C} (3\sigma_2 - \sigma_1 + T) - \frac{1}{C} P_0
\]

\[
P_b = P_i + \Delta
\]

where \( P_i \) = crack initiation pressure, \( \Delta \) = borehole pressure rise in the time delay until the injection rate into the fracture attains the pumping rate.

(b) Calculation of Time Lag of Fracture Infiltration

The fluid injection rate into a hydrofracture has been studied for established fractures propagating with constant fluid injection rates (Geertsma and Klerk, 1969; Abbé et al., 1976) but a theory for breakdown time lag requires the condition of constant excess pressure in the borehole. Zoback et al., (1978) obtained an iterative coupled stress-flow solution for the propagation of the initiating crack, with constant borehole pressure, and removed the assumption used by the above-mentioned
workers (Geertsma and Klerk, 1969; Abe et al., 1976) that the fluid pressure be taken as constant along the crack, but did not evaluate the initiation-breakdown time lag. The theory of Khristianovic and Zheltov (1955) applies to the required condition of fluid injection into the crack at constant injection pressure. It gives a solution to the present problem if assumptions are made on the initial length of micro-crack, and on the affect of the presence of the borehole. Khristianovic and Zheltov found that the volume of the propagating hydrofracture increased exponentially with time, i.e.

\[ V = V_0 e^{t/c} \]

with the time constant,

\[ c = \frac{100\eta E^2}{P^3 f(P/\sigma)} \]

(\( \eta \), fluid viscosity; \( E \), rock modulus; \( P \), borehole pressure at breakdown; \( \sigma \), stress field perpendicular to the crack) where \( f(P/\sigma) \) has the values

<table>
<thead>
<tr>
<th>( P/\sigma )</th>
<th>1.15</th>
<th>1.35</th>
<th>1.7</th>
<th>2.1</th>
<th>3.5</th>
<th>4.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f(P/\sigma) )</td>
<td>290</td>
<td>43</td>
<td>5.5</td>
<td>1.02</td>
<td>0.22</td>
<td>0.1</td>
</tr>
</tbody>
</table>

For example, with water (\( \eta = 10^{-3} \) Pa s), rock modulus \( E = 70 \) GPa taking \( \sigma = 20 \) MPa, \( P = 42 \) MPa, we obtain \( c = .007 \) sec. For the initial volume we consider a pre-existing crack of length \( l_0 \), whose width \( 2w_0 \) arises from pressurization to breakdown pressure,

\[ V_0 = 2w_0 l_0 = 8l_0^2 \frac{(P - \sigma)}{E}, \text{ per unit length of borehole.} \]

Taking arbitrarily \( l_0 = 0.1 \) mm with the above data, \( V_0 = 10^{-10} \) m\(^3\) per metre borehole length. Ignoring at this stage the presence of the borehole, the crack propagates with length \( l = l_0 e^{ht/c} \), and width \( w = w_0 e^{ht/c} = 4l(P - \sigma)/E \). The presence of the crack in the periphery of the borehole has the effect of widening the borehole, and increasing the borehole volume by the order \( \Delta V_B = 2aw \) per metre length of borehole. Hence the total volume of the borehole and crack increases by,

\[ V_T = V_B + V \]

\[ = 8l_0 \frac{P - \sigma}{E} e^{ht/c} \left(1 + \frac{l_0}{a} e^{ht/c}\right) \]

and the rate of increase of fluid volume is,

\[ \frac{dV_T}{dt} = 8a c \frac{P - \sigma}{E} e^{ht/c} \frac{l_0}{2c} \left(1 + 2 \frac{l_0}{a} e^{ht/c}\right) \]

\[ = V_0 \frac{a}{l_0} e^{ht/c} \frac{l_0}{2c} \left(1 + 2 \frac{l_0}{a} e^{ht/c}\right) \]  

(8)
The order of magnitude of the time delay to breakdown pressure is found by equating the rate of increase of fluid volume to the pumping rate, \( R \).

Taking for example numerical values of \( R \) from 1 to 5 litres per minute, with the values of \( \ell, P, \sigma \) given above, and a borehole of 100 mm diameter \((a = 0.05 \text{ m})\) the equation (8) becomes

\[
R = \frac{dV}{dt}
\]

i.e.

\[
16.7 \text{ to } 83 = 10^{-10} \frac{50}{0.1} e^{\frac{bt}{c}} \frac{1}{0.014} (1 + 2 \frac{0.1}{50} e^{\frac{bt}{c}})
\]

\((x \times 10^{-6} \text{ m}^3 \text{s}^{-1})\)

The first term on the right is dominant, and we obtain

\[
e^{\frac{bt}{c}} = 5 \text{ to } 23,
\]

\[
e^{\frac{bt}{c}} = 1.6 \text{ to } 3.2,
\]

\[
\ell = 0.5 \text{ to } 2.3 \text{ mm},
\]

\[
t = 22 \text{ to } 44 \text{ millisec.}
\]

(c) Infiltration into the Crack Surfaces

A correction to the flow-rate equation (9) is required to allow for absorption of fluid into the crack walls.

The absorption of fluid in the crack surfaces has been studied for established fractures propagating with constant fluid injection rates, but not for the initiation of hydrofracture. Available solutions for fluid absorption in general use are based on diffusion (Howard and Fast, 1957; Le Tiran and Dupuy, 1967; Geerstsma and Klerk, 1969; Shuck and Advani, 1975) and do not allow for poro-elastic behaviour of the rock mass. Poro-elastic behaviour is allowed for by Jaeger and Cook (1969, in the consolidation solution of one-dimensional movement of pore-water) but under the restriction that the fluid was assumed to be incompressible. The solution for compressible fluid is obtained by substituting in Jaeger and Cook's (1969) solution, the consolidation coefficient obtained for compressible pore fluid by Alexander (1976). The following expression is found for the quantity of fluid (compressible) infiltrating into unit rock surface area in the crack walls, exposed to the fluid for time \( t \):

\[
q = \left(P_1 - P_0\right) 2t^{1.5} \frac{\alpha k}{\pi n K} \left(1 + \frac{n K}{\alpha K_w}\right) , \text{ m}^3/\text{m}^2
\]

\[(P_1, P_0 \text{ fluid pressure in crack and initial pore pressure in the rock mass; } k = \text{permeability; } \eta, \text{ viscosity of fluid; } n, \text{ volume porosity; } K, K_w, \text{ bulk elastic moduli of rock and water}). \] Note that omission of the unity gives Howard and Fast's formula. e.g. For a rock with permeability to viscosity ratio, say \( k/\eta = 10 \text{ millidarcies} - 0.01 \text{m}^2 \text{s}^{-1} (\text{GPa})^{-1} \); crack porosity \( a = 1 \); volume porosity \( n = 5\% \); and \( K, K_w = 35, 35 \text{ GPa} \) respectively; \( P_0 = 2 \text{MPa} \) (corresponding to a test depth of 200 m) and the numerical data used previously, we find \( A = 1.04 \times 10^{-3} \text{ m sec}^{-1} \).
Consider a crack of initial length $2l_0$, and length $2l$ at time $t$, and $2l$ at time $t_2$, and ignore the presence of the borehole. The time of exposure of a crack surface element of length $dx$ is

$$t = t_2 - t_x = 2c \log \frac{l}{x}$$

and the total volume of water absorbed into the crack walls (comprising 4 half-lengths $l$) is,

$$Q = \int_{l_0}^{l} \frac{q}{dx} = \int_{l_0}^{l} 4A t^b dx = \int_{l_0}^{l} 4A \sqrt{2c} \sqrt{\log \frac{l}{x} - \log x} \cdot dx$$

The rate of total absorption into the crack surfaces is

$$\frac{dQ}{dt} = 4A \sqrt{2c} \sqrt{\log \frac{l}{l_0}} \cdot \frac{dl}{dt} = 4A l \sqrt{\frac{\log \frac{l}{l_0}}{2c}} N \quad (11)$$

For the crack propagation data obtained from (9) and (10) we obtain $dQ/dt = 22$ to $187 \times 10^{-6}$ m$^3$s$^{-1}$ per metre length of borehole.

The surface absorption rate therefore, in this example, is not small compared with the flow-rate into the crack and borehole, (9). It is therefore necessary to include the surface absorption rate in equation (9). The revised flow rate equation is,

$$R = \frac{dV_T}{dt} + \frac{dQ}{dt}$$

$$= 8a \frac{p-a}{E} \frac{l}{2c} (1 + \frac{2l}{a}) + 4A l \sqrt{\frac{\log \frac{l}{l_0}}{2c}}$$

(12)

Inserting the numerical data, but generalizing the viscosity gives,

$$R = \frac{3.5}{N} \times 10^{-6} \frac{l}{l_0} \{1 + \frac{l/l_0}{250} + \sqrt{\log \frac{l}{l_0}}\}$$

where $N = \eta/\eta$ water. The respective terms are due to borehole widening, increase of crack volume, and absorption in the crack faces.

The previously obtained crack lengths for flow-rate balance (following (g)) are reduced to $l = 0.25$ to $1.0$ mm, attained at the reduced 'delay times', $t = 13$ to $32$ millisec.

The effect of fluid viscosity on the delay is illustrated by the following estimates (Table I, for viscosities 1, 10 and 100 times that of water, with the numerical data as used previously.)

410
TABLE 1

<table>
<thead>
<tr>
<th>n/n&lt;sub&gt;water&lt;/sub&gt;</th>
<th>c, sec</th>
<th>ℓ&lt;sub&gt;mm&lt;/sub&gt;</th>
<th>Delay t sec</th>
<th>Borehole widening</th>
<th>Crack fluid</th>
<th>Absorbed fluid</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.007</td>
<td>0.25</td>
<td>0.013</td>
<td>8.4</td>
<td>0.1</td>
<td>8.2</td>
<td>16.7</td>
</tr>
<tr>
<td>10</td>
<td>0.07</td>
<td>1.8</td>
<td>0.4</td>
<td>6.0</td>
<td>0.4</td>
<td>10.3</td>
<td>16.7</td>
</tr>
<tr>
<td>100</td>
<td>0.7</td>
<td>13</td>
<td>6.8</td>
<td>4.5</td>
<td>2.2</td>
<td>10.0</td>
<td>16.7</td>
</tr>
</tbody>
</table>

Hence, whilst the time constant is proportional to viscosity, so also are, very roughly, the crack length and delay time at flow balance. The rate of change of crack volume is dominated by the rate of change of borehole volume and volume of absorbed fluid, at all viscosities in this example.

Estimation of the delay to attainment of maximum pressure in practice is outside the scope of the present note. It is assumed that the difference Δ between initiation and breakdown pressures will be of the order of the rate of borehole pressure increase before cracking multiplied by the 'delay time' as above calculated.

The time constant tends to infinity as tensile strength T → 0. Hence for a repeated breakdown test after a first hydrofracturing operation the possibility of high time lag and associated overpressurising error exists. This may explain the anomalously high breakdown pressure obtained by Zoback et al., (1977) in tests on pre-fractured laboratory samples using high viscosity fluid.

5. CONCLUSIONS

Laboratory tests confirming the crack initiation criterion based on infiltration into crack pores have been reported for values of the crack porosity parameter toward the lower and upper ends of the range 0.1 of possible values. The breakdown pressure ranged from 50 to 72% respectively of that predicted by the commonly used criterion, which takes no account of infiltration. Further tests at low stresses are needed, carried out on the lines of those reported by Kim and Gray (but with lower pressurizing rates). The stress dependence of the crack porosity will include development of crack anisotropy with increased stress, which has not been allowed for in the theoretical analysis.

The dependence of the breakdown time lag on values of the various parameters associated with viscous infiltration into the crack, viz modulus, stress, rock tensile strength, and absorption into the crack surfaces, has been analysed. The time lag is not a source of significant error in practice with low viscosity fluid and moderate rates of pressurization, but may explain some apparently anomalous effects. For example, the high breakdown pressure reported by Zoback et al., (1977) for a pre-cracked specimen using high viscosity fluid, may possibly be due to the predicted indefinitely large time lag as tensile strength approaches zero.
6. REFERENCES


ANALYSIS OF INJECTION TESTS FOR IN-SITU STRESS DETERMINATION

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ABSTRACT

The classical interpretation of hydraulic fracturing tests for in-situ stress determination relies on a few a-priori hypothesis among which the most constraining is that the fracture extends perpendicularly to the least principal stress. However this does not hold for rock masses with strong anisotropic tensile strength; this anisotropy may be a rock matrix property; most of the time it is associated with preexisting weakness planes of the rock mass such as recemented fractures.

A new interpretation technique is proposed based on shut-in pressure measurements for fractures developed in various directions. In general, six different orientations are required. If one of the principal stress is vertical and equals the weight of overburden then only three different fracture orientations are needed. The validity of this new technique is discussed in the context of two in-situ stress determinations, the first one in a granitic rock mass, the other one in salty deposits.
I. INTRODUCTION

Hydraulic fracturing is now widely used for in-situ stress measurements (e.g. Haimson 1978, Zoback and Zoback 1980, Rummel 1980). Yet little effort has been devoted to develop means for ascertaining the accuracy of such stress measurements.

The purpose of this paper is first to discuss the hypothesis assumed with the now classical hydraulic fracturing technique for stress measurements. Then two alternative injections tests, which may help in some instances refine the stress determination, will be presented ; namely the "reopening pressure" test and the "shut-in" pressure test. Both these tests take advantage of the fact that a hydraulic fracture does not lay necessarily in a plane perpendicular to the direction of the minimum principal stress.

Field results are finally presented as illustration of these concepts.

2. THE CLASSICAL HYDRAULIC FRACTURING TECHNIQUE

Many authors (e.g. Hubbert and Willis (1957), Scheidegger (1962), Kehle (1964), Fairhurst (1964), Haimson and Fairhurst (1969)) have suggested that hydraulic fracturing can be used as an in-situ stress measurement technique for both impervious and permeable rocks.

With this technique a portion of a borehole is sealed off with a straddle packer. When the straddle packer isolates a portion of the borehole in which no preexisting fracture exists, a fluid injection test yields a typical pressure-time record, as indicated on figure 1 (pressure is supposed to be measured in the interval between the two packers).

The pressure first rises to a maximum value called the breakdown pressure \( P_b \), then, as injection proceeds at a constant flow rate, the pressure drops and stabilizes at the so-called propagation pressure \( P_p \). When injection stops, a shut-in pressure \( P_s \) can be observed if the wellbore remains sealed off.

If the borehole is parallel to one of the principal stress directions and if the rock is linearly elastic and isotropic with respect to both its elastic behaviour and its "strength", the breakdown pressure and the shut-in pressure can help determine some of the components of the local stress tensor. In cases considered by these authors, the borehole is vertical as well as one of the principal stress components (which is simply taken as the overburden pressure).
Because most straddle packers are inflattable, only vertical fractures can be developed from vertical boreholes (Haimson and Fairhurst 1969) even in regions very close from ground surface. Accordingly, since it is assumed that the rock is linearly elastic and isotropic with respect to both, its elastic properties and its strength characteristic, the well known following relationships are used to determine the horizontal principal stresses magnitude (for impervious rocks):

\[ -\sigma_1 + 3\sigma_2 + \sigma_T = P_b \tag{1} \]
\[ \sigma_2 = \frac{P_s}{\sigma} \tag{2} \]

where \(\sigma_1\) and \(\sigma_2\) are the principal stress components in the plane perpendicular to the borehole axis (\(\sigma_1 > \sigma_2\), compressions are positive) and \(\sigma_T\) is the so-called "tensile strength" of the rock. From Griffith theory of fracture (Griffith 1921) it can be shown that, for such a geometry, the fracture extends in the direction perpendicular to that of \(\sigma_2\).

With this theory, a first difficulty is encountered with the definition of an accurate method for determining the rock "tensile strength". A second one arises from the fact that the break-down pressure \(P_b\) may not coincide with fracture inception: It merely corresponds to the moment when the rate of injection flow equals that of the flow into the fracture. This moment, for high injection flow rates and viscous fluids may not coincide with fracture inception (Zoback et al. 1977).

In addition the validity of the basic hypothesis underlying equations (1) and (2), that is that the fracture extends in a plane normal to the \(\sigma_2\) direction (\(\sigma_2\) is the minimum principal stress in the plane perpendicular to the borehole axis) is very difficult to establish with certainty.

Indeed, according to Griffith's criterion of fracture, a hydraulic fracture occurs when:

\[ \Delta w(ds) - \Delta u(ds) > \Delta \Omega(ds) \tag{3} \]

where \(\Delta w(ds)\) is the work of external forces associated to crack growth \(ds\);
\(\Delta u(ds)\) is the strain energy variation of the rock mass;
\(\Delta \Omega(ds)\) is the surface energy absorbed by crack growth \(ds\).

When there is strict equality between both terms of equation (3), the fracturing process is quasistatic whilst if the left term is larger than the right one some kinetic energy is generated.
The formation of a new surface can be considered as an irreversible dissipative process so that for quasistatic fracture propagation, the theorem of minimum potential energy leads to:

\[
\left[ \Delta u(x) + AD(x) \right] - \left[ \Delta u(x_0) + AD(x_0) \right] \leq 0
\]

where \( \Delta u(x) \) is the strain energy variation associated with any virtual crack configuration \( (x) \), the configuration \( (x_0) \) corresponding to the actual extended crack, \( AD(x) \) represents the quantity of surface energy dissipated by fracture extension configuration \( x \).

Now, if the rock is isotropic with respect to its strength (\( AD(x) = AD(x_0) = \gamma da \), where \( \gamma \) is Griffith's free surface energy per unit area), equation (4) becomes:

\[
\left[ \Delta u(x) - \Delta u(x_0) \right] \leq 0
\]

from which it can be shown that the hydraulic fracture extends perpendicularly to the \( \sigma_2 \) direction. But if the rock is anisotropic with respect to its strength (\( AD(ds) = \gamma(n)da \)), orientation of the fracture may not coincide with that of \( \sigma_1 \). Here the concept of anisotropy must be taken in its broadest sense; it may refer either to a rock matrix property or to a more or less recemented preexisting joint. In the later case, even though the rock matrix may be isotropic with respect to its strength, the rock mass is not.

This suggests that the orientation of a hydraulic fracture cannot be assumed, a priori, to lay in a plane perpendicular to the \( \sigma_2 \) direction so that equations (1) and (2) may not be valid.

Attention has been devoted to the definition of additional tests which would ascertain the validity of the many hypothesis implicit with the classical interpretation of hydraulic fracturing data.

3. THE REOPENING PRESSURE TEST

Means have been thought to develop hydraulic fractures parallel to the borehole axis but in any desired azimuth at the wellbore wall. Indeed the reopening pressure \(^*\) for such fractures could provide a verification of the stress determination established from the classical hydraulic

\(^*\) the reopening pressure is defined as the pressure required to open a fracture at the wellbore wall.
The tangential stress at any point of the wellbore wall in impervious rocks is:

\[
\sigma_{\theta \theta} = (\sigma_1 + \sigma_2) - 2(\sigma_1 - \sigma_2) \cos 2\theta - P_r
\]  

Equation (7) is redundant with respect to equations (1) and (2) so that an information on accuracy of the determination of the \(\sigma_1\) orientation can be obtained. This suppose that the oriented fracture is developed in a zone not too distant from that where the classical hydraulic fracture used for the stress determination has been created, in order to avoid difficulties with local stress variations.

Since a hydraulic fracture initiates at points where the tangential stress at the wellbore wall is minimum, such fractures can be generated, theoretically, in any desired orientation thanks to the use of hydraulic curved jacks exerting normal pressures onto two symmetrical sections of the wellbore (see figure 2). For such loading conditions the tangential stress is (see e.g. Jaeger and Cook 1969):

- for the borehole section where the jacks exert their pressure:

\[
\sigma_{\theta \theta} = (\sigma_1 + \sigma_2) - 2(\sigma_1 - \sigma_2) \cos 2\theta + P_j - \frac{4a}{\pi} P_j - P_w
\]  

- for the borehole section where no jack is applied:

\[
\sigma_{\theta \theta} = (\sigma_1 + \sigma_2) - 2(\sigma_1 - \sigma_2) \cos 2\theta - \frac{4a}{\pi} P_j - P_w
\]

where \(2\alpha\) is the angle along which one jack is acting , \(P_j\) is the jack pressure and \(P_w\) is the hydrostatic pressure exerted by the fluid in the wellbore.

If \(\sigma_1\) and \(\sigma_2\) are known from a previous test, \(P_j\) can be determined so that \(\sigma_{\theta \theta}\) reaches its minimum value at one of the corners of each jack (if the jack orientation is parallel to the \(\sigma_1\) direction, \(\sigma_{\theta \theta}\) reaches a minimum at both corners of each jack). Accordingly a hydraulic fracture will initiate at the orientation imposed by the jacks.

A tool providing such loading facilities has been built (76 mm diameter, 1 cm spacing between both packers, 90° angle for each jack, 400 b maximum jack pressure). Tests conducted at the bottom of a limestone quarry (borehole depth ranging from 3 to 6 meters) have shown that for
these site conditions ($\sigma_1 \approx 33$ bars, $\sigma_2 \approx 6$ bars) fractures could be developed at 45° angles from the $\sigma_1$ direction (Charlez 1981), and that the fractures extend in their own directions for at least 20 cm.

Laboratory work has been conducted to investigate experimentally the path that fractures, initiated in a plane normal to a uniaxial stress field, would follow as they extend away from the drill hole (Cornet 1976). Results (see figure 3) indicate that for low flow rates, the fracture extends in its own plane; the higher the uniaxial stress field, the lower the flow rate necessary to obtain this propagation scheme. As the flow rate increases, a pressure gradient develops inside the fracture and this one tends to get oriented parallel to the uniaxial stress field direction. For even higher flow rates, no percolation occurs inside the preexisting fracture and a new one is developed in the uniaxial stress direction. In addition, the lower the flow rate, the lower the pressure necessary to propagate the fracture (see figure 3 and 4), for a uniaxial stress $\sigma$ equal to 5.5 MPa and a flow rate $\dot{v}$ equal to 28.4 cm$^3$/sec, the breakdown pressure is equal to 17.1 MPa, whilst for $\sigma = 7.6$ MPa and $\dot{v} = 56.0$ cm$^3$/sec, the breakdown pressure is 15.6 MPa. Had the breakdown pressure been independent of flow rate, it should have been lower for the second loading conditions than for the first one since, at the wellbore, before pressurization occurs, the minimum tangential stress is - 5.5 in the first case and - 7.6 in the second one.

A numerical investigation has been conducted to determine whether the results observed experimentally could be generalized to any fracture orientation. For the laboratory experiments, the fracture lay in a plane perpendicular to the uniaxial stress field so that no shear component exists in this plane. But for the configuration obtained with our curved jack instrument the fracture initiates at a 45° angle with respect to the $\sigma_1$ direction, that is in a direction where shear is maximum.

The numerical model used for crack propagation analysis (Cornet 1979) is based on Griffith's energy criterion of fracture as stated by equation (5) (only isotropic materials have been considered). Equation (5) is solved by applying the displacement-discontinuity technique (Crouch 1976) to various crack increment configurations and then by choosing that which yields the largest strain energy release rate. Numerical values were such that $\sigma_1 = 10$ MPa, Young's modulus = 70,000 MPa, Poisson's ratio = 0.25, original crack length = 10 cm.
Result are shown on table 1 for a crack inclined at 30° with respect to $\sigma_1$ direction and for a crack inclined at 45°. Crack increment length is 0.125 cm.

**TABLE 1 : Propagation of a hydraulic fracture inclined by an angle $\theta$ with respect to a uniaxial stress field**

<table>
<thead>
<tr>
<th>$\theta$</th>
<th>Pressure in the fracture in MPa</th>
<th>Change of orientation with crack growth ($\beta$)</th>
<th>Strain energy release rate (MPa·cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>30°</td>
<td>8.9</td>
<td>-45°</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>18.8</td>
<td>-22.5°</td>
<td>0.11</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>-7.5°</td>
<td>2.01</td>
</tr>
<tr>
<td></td>
<td>300</td>
<td>0</td>
<td>18.7</td>
</tr>
<tr>
<td>45°</td>
<td>16</td>
<td>-30°</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>-22.5°</td>
<td>0.03</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>-15°</td>
<td>0.14</td>
</tr>
<tr>
<td></td>
<td>80</td>
<td>-7.5°</td>
<td>0.62</td>
</tr>
</tbody>
</table>

\[ \sigma_1 \]

\[ \theta > 0 \]
For a granite, the surface energy is of the order of 50 Joules/m² (i.e. 0.005 MPa.cm) which corresponds to a strain energy release rate of the order of 0.01 MPa.cm. This shows that if the fluid pressure within the fracture is just large enough to insure quasistatic propagation, the fracture will get oriented progressively with the \( \sigma_1 \) direction. This suggests that fractures generated at 45° with respect to the \( \sigma_1 \) direction, thanks to the jacking system, will eventually get oriented parallel to the major principal stress direction as they propagate outside the zone of influence of the borehole (i.e. about three times the borehole radius). This distance is long enough to insure satisfactory reopening pressure reading.

The next question to be answered is whether indeed reopening pressure can be easily detected.

The reopening pressure is defined as the pressure required to cancel the tangential stress as the borehole wall where the fracture initiates. It is not necessarily the maximum pressure reached during an injection test performed after the fracture has been developed. This later value, as pointed out by Zoback and Pollard (1979), merely reflects the equilibrium between injected flow rate and absorbed flow rate (which in this case corresponds to the fluid into the fracture since the rock is supposed to be impervious or the liquid very viscous).

As can be seen on figure 4 (e.g. curve obtained for \( \sigma = 7.6 \) MPa, \( V = 1.6 \times 10^{-3} \) cm/sec), for very low flow rates, even though the fracture is closed, some fluid percolates into the fracture quite before the tangential stress at the wellbore wall is cancelled. As a consequence first of all, expression for the tangential stress at the wellbore wall is no more given by equation (7) and, secondly, the time when the fracture does open at the wellbore becomes unnoticeable on the pressure-time record.

Making use of the displacement-discontinuity technique, the influence of this percolation effect on the borehole wall tangential stress has been analysed for a fracture, with a length equal to 0.7 times the radius of the borehole, with an orientation perpendicular to the uniaxial stress field (\( \sigma_1 = 10 \) MPa). When no fluid percolates, the classical value is found (30 MPa = 3 \( \sigma_1 \)). If the pressure is assumed to be uniform to the half crack length and then drops to zero in the remaining part of the crack, the reopening pressure is found to be 15.4 MPa. If the pressure is supposed to be uniformly applied up to the fracture tip then the reopening pressure is 11.8 MPa.
Quite clearly for longer fractures, different reopening pressures would have been obtained. More precisely as the fracture gets longer and longer the influence of the borehole stress concentration on the fracture tip stress field becomes negligible so that the maximum pressure reached with very slow flow rates is representative only of the normal stress field near the fracture tip (recall we are concerned only with impervious rocks).

Accordingly, the measurement of reopening pressures for very slow flow rates yields values similar to those of the shut-in pressure whilst for larger rates, when no percolation occurs into the fracture, the reopening pressure provides a direct measurement of the tangential stress at the wellbore wall where the fracture initiates.

The interpretation of reopening pressure in terms of borehole wall tangential stress requires two additional conditions:

- The far field stress state must be such that the fracture closes back when the borehole pressure returns to its original value.
- The fracture must not extend in the zone where the packers are in contact with the rock. Indeed, in this zone the tangential stress \( \sigma_{\theta\theta} \) at the wellbore wall depends on the pressure inflating the packers.

We will conclude for now that it is possible to develop hydraulic fractures in any desired orientation at the wellbore but that these fractures tend to change orientation as they extend in zones unaffected by the borehole. Measurements of the reopening pressures require flow rates high enough to prevent percolation into the fracture if the tangential stress at the wellbore wall, where the fracture initiates, is to be determined. This reopening pressure may not coincide with the peak of the pressure-time record. Very slow flow rates provide means to determine the normal stress near the fracture tip and should give values similar to those of the shut-in pressure.

4. **THE SHUT-IN PRESSURE METHOD**

It will first be assumed, in this discussion, that hydraulic fractures are planar. Interpretation of reopening pressures in terms of tangential stress at the wellbore wall requires that the fracture be parallel to the axis of the borehole. However interpretation of shut-in pressures does not imply such a requirement: the shut-in pressure is simply a direct measurement of the normal stress exerted on the fracture plane. If \( \sigma \) is the local stress tensor and \( n \) the unit normal to the fracture plane, the shut-in pressure is :
\[ \mathbf{n} \cdot \mathbf{n} = P_s \] (10)

Consequently if a hydraulic fracturing test is conducted in a borehole domain where a preexisting weakness plane exists, it may be possible to propagate the frac along this weakness plane so as to obtain shut-in pressure measurements for orientations different from that of a classical hydraulic fracture. Alternatively if the plane is fairly permeable so that shut-in pressures reading are rendered difficult, injection at various flow rates may provide the information we are looking for; namely the value of the normal stress exerted onto that plane. If this test is conducted in a region not too distant from where a classical hydraulic fracture has been developed, results of this shut in pressure measurement are redundant with those given by equation 1 and 2 so that some appreciation of the accuracy of the stress determination can be gained.

Rock masses are never homogeneous but always exhibit some natural fracture pattern. These fractures may be still opened and permeable. Very often they are recemented by calcite or quartz, yet they represent directions of "strength" anisotropy which can be taken to advantage for developing hydraulic fractures in directions different from that of the major principal stress. As suggested by the above mentioned laboratory experiments, development of fractures along these weakness planes are easier to obtain when slow injection rates are used.

Theses planes of weakness are encountered at various depth in the wellbore so that generally shut-in pressure measurements cannot be correlated together at once. When measurements are conducted from various boreholes drilled, for example, from a tunnel or a mine shaft, it is possible to conduct tests on many planes with different orientation within the same depth range. For such test conditions, if six different natural fracture orientations are available, the complete stress tensor can be determined from shut-in pressure measurements only, that is, without any of the restrictive hypothesis which must be met with the classical interpretation method. This stress determination technique is even free from the linear elasticity hypothesis inherent to most other stress measuring techniques.

A fairly common situation is that of roughly homogeneous rock masses for which the stress variation with depth can be assumed to be linear at least in the depth range of the stress determination location. Then the stress tensor can be approximated by the following:
where $\sigma_1$ and $\sigma_2$ are supposed to be independent of depth for the depth range under consideration (they may be called tectonic stresses);

- $z$ is the depth of the test;
- $\rho g$ is vertical stress gradient;
- $\alpha_1$ and $\alpha_2$ are two unknown coefficients; for small depth intervals the linear variation of stress with depth may be assumed to be caused only by gravity so that $\alpha_1 = \alpha_2$.

Accordingly if $\rho g$ is known, equation 10 depends on four unknowns namely $\sigma_1, \sigma_2, \alpha_1$ and $\eta$ the orientation of the major principal stress.

If $\beta$ is the direction of the horizontal projection of the normal to the fracture plane, equation (10) becomes:

$$
\left(\sigma_1 + \sigma_2\right) + \left(\sigma_1 - \sigma_2\right) \cos 2(\beta - \eta) = 2 \frac{P - z \left(\alpha_1 (1 - n_3^2) - \rho \rho_3n_3^2\right)}{(1 - n_3^2)}$$

where $n_3$ is the director cosine of the normal to the fracture plane with respect to a vertical axis.

Accordingly if four different weakness planes can be used to develop hydraulic fractures in three different directions (at four different depth) then the stress can be determined. When more than four fracture planes are used, some redundancy is obtained. After linearizing equation (12) (take $X = \sigma_1 + \sigma_2$, $Y = (\sigma_1 - \sigma_2) \cos 2\eta$, $Z = (\sigma_1 - \sigma_2) \sin 2\eta$), the system can be solved by a least squares techniques.

For fracture planes which are parallel to the borehole axis, use of both reopening pressures and shut-in pressures implies that only two different fracture orientations are necessary for the complete determination of the stress tensor. Since the orientation of the major principal stress $\sigma_1$ is taken as an unknown, equation (7) becomes:

$$
\left(\sigma_1 + \sigma_2\right) + 2(\sigma_1 - \sigma_2) \cos 2(\beta - \eta) = P - z - 2 \alpha_1 z
$$

where $\beta$ has the same meaning as in equation (12).

For vertical planes ($n_3 = 0$), combining equations (12) and (13) yields the convenient result:

$$
\left(\sigma_1 + \sigma_2\right) = 4 \frac{P_s - P_r - 2 \alpha_1 z}{\alpha_1}
$$

if $\alpha_1$ is not equal to $\alpha_2$ in equation (11) then the term $2\alpha_1$ in equation (14) is replaced by $\alpha_1 + \alpha_2$. 

\[ (\sigma_1 + a_1) = 0 \] 
\[ (0 + \sigma_2 + a_2 z) = 0 \] 
\[ (0 + 0 + \rho g z) = 0 \] 

where $\sigma_1$ and $\sigma_2$ are supposed to be independent of depth for the depth range under consideration (they may be called tectonic stresses);
Equation (14) is independent of the fracture azimuth. It can be used either to determine the value of $\sigma_1$ from tests at various depth or, if $\sigma_1$ can be determined by other means, it can help verify the accuracy of the reopening pressure reading since $(\sigma_1 + \sigma_2)$ is an invariant.

Interpretation of shut-in pressures is based on the hypothesis that hydraulic fracture surfaces remain planar as they extend away from the wellbore. However, as pointed out by Daneshi (1971), fracture orientation may change as the crack extends away from the borehole. If the rock is homogeneous and isotropic this change of orientation can be caused either by a variation in relative principal stress magnitudes (Zoback and Pollard, 1979) or principal stress direction (Mizuta and Kobayashi, 1980).

If the rock mass exhibits preexisting fractures or planes of strong weakness, these fractures may induce change of orientations.

The influence of a pre-existing fissure located on the path followed by a hydraulic fracture extending in the $\sigma_1$ direction has been investigated numerically. The pre-existing discontinuity and the hydraulic fracture are assumed to be parallel to the intermediate principal stress (the vertical direction in many practical instances, that is $\sigma_3$ in our notation). It has been found that if the discontinuity is inclined at less than 45° with respect to the maximum principal stress direction, the hydraulic fracture stops when it encounters the preexisting fracture and flow occurs along the old fracture, or in the $\sigma_2$ direction. For angles larger than 45 degrees, both, the magnitude of principal stresses and the friction coefficient along the pre-existing fissure, must be known if the exact path of hydraulic fracture extension is to be determined.

On a practical basis, hydrau-frac tests conducted for stress measurements purposes must be performed in many sequences of injection and shut-in pressure measurements so as to keep track of the evolution of the normal stress near the tip of the fracture. An alternative technique is to dissociate instantaneous shut-in pressure from final shut-in pressure as proposed by Zoback and Pollard (1979).

5. FIELD RESULTS

Test in a granite rock mass

A 220 m deep, 165 mm diameter borehole drilled in granite, has been used for investigating, in-situ, initiation and propagation of hydraulic fractures. On site (at Le Mayet de Montagne, 25 km south east from Vichy in central France) the granite is covered by a less than 5 m thick soil
### Table 2: Data from hydraulic fracturing tests at Le Mayet de Montagne test site (25 km SE of Vichy, in the center of France) for borehole INAG 3-2.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>27</th>
<th>42</th>
<th>54</th>
<th>65</th>
<th>84</th>
<th>90</th>
<th>174</th>
<th>186</th>
</tr>
</thead>
<tbody>
<tr>
<td>Breakdown pressure MPa</td>
<td>22.3</td>
<td>frac.</td>
<td>frac.</td>
<td>frac.</td>
<td>10.</td>
<td>34.7</td>
<td>15.1</td>
<td>25.5</td>
</tr>
<tr>
<td></td>
<td>33.3</td>
<td>by packer</td>
<td>by packer</td>
<td>by packer</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reopening pressure MPa</td>
<td>-</td>
<td>5.4</td>
<td>5.9</td>
<td>-</td>
<td>8.5</td>
<td>5.1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Shut-in pressure MPa</td>
<td>-</td>
<td>2.1</td>
<td>4.2</td>
<td>3.2</td>
<td>4.6</td>
<td>4.4</td>
<td>5.6</td>
<td>5.4</td>
</tr>
<tr>
<td>Injected volume in m³</td>
<td>-</td>
<td>4.88</td>
<td>1.19</td>
<td>4.58</td>
<td>2.54</td>
<td>3.59</td>
<td>0.02</td>
<td>13.0</td>
</tr>
<tr>
<td>Recovered volume in m³</td>
<td>-</td>
<td>0.04</td>
<td>0.24</td>
<td>0.12</td>
<td>0.26</td>
<td>0.14</td>
<td>-</td>
<td>2.0</td>
</tr>
<tr>
<td>Pumping rate 10⁻³ m³/min</td>
<td>60</td>
<td>60</td>
<td>60</td>
<td>60</td>
<td>60</td>
<td>60</td>
<td>1</td>
<td>320</td>
</tr>
<tr>
<td>Fracturing fluid</td>
<td>water</td>
<td>water</td>
<td>gel</td>
<td>water</td>
<td>water</td>
<td>water</td>
<td>water</td>
<td>water</td>
</tr>
<tr>
<td>Orientation dip</td>
<td>multiple 82° 80° - 80° 82° 80°</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>At wellbore strike fractures N 60°E N 46°E - N160°E N50°E N57°E</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Pressure required to reopen the frac generated during the first injection (measured after the pore pressure has dropped back to its original value).
layer. Natural fractures were mapped on 12 large outcropping areas nearby the test site (within 10 km from the site) (Drogue et al. 1979). Four major fissure orientations have been identified, namely N 30° E ± 10°, N 60° E ± 10°, N 100° E ± 10°, N 155° E ± 5°.

All these fractures are subvertical (dip lies between 70 and 85°). Thermal, electrical and video loggings were used to identify fractured zones as well as quartz veins in the borehole. Their orientations, determined with a borehole TV camera, were found to be similar to those identified by the surface mapping.

Eight hydraulic fractures were generated with an inflatable straddle packer in area thought to be homogeneous or in place where a light quartz vein had been identified. Results are presented in table 2.

It can be noticed that orientation of these fractures is quite variable and that, for very shallow depth (27 and 42 m) more than one fracture occurred. For the 27 m frac, a first breakdown pressure was measured immediately after permeability tests had been conducted (pore pressure was not allowed to return to its original value). After 200 litters have been injected, the well was left opened so as to let the pore pressure return to its original value. Pumping was started again: a higher breakdown pressure than the first one was recorded. An impression packer revealed two vertical fractures. This effect is interpreted in a similar manner to that observed during the laboratory experiments described here above: for the first frac, build up of pore pressure allowed a preexisting fracture to be reopened; for the second injection test, the fast injection flow rate, as compared to the flow velocity in the preexisting fissure, induced a new fracture.

Comparison of breakdown pressures observed for the 174 m and 186 m deep frac is a good example of the influence of flow rate effect on the breakdown pressure magnitude. Both fractures are in the same direction, yet breakdown pressure is equal to 15 MPa for a 1 l/min flow rate and 29.5 MPa for a 320 l/min flow rate.

For the 90 m deep frac a 1 cm thick quartz vein intersected the borehole (direction and dip are those of the frac). So if the classical hydraulic fracturing interpretation method had been applied, results from the 90 m deep frac would have been rejected and the major principal stress direction would have been found to the roughly N 56° E. Rather the shut-in pressure method was applied to the results from the 54, 65, 90 and 174 m deep fractures. This led to the following result:
\[ \sigma_1 = 6.3 \text{ MP}_a ; \sigma_2 = 1.5 \text{ MP}_a ; \eta = 17^\circ \text{ E} ; \alpha = 0/0108 \text{ MP}_a /m . \]

Using this result to determine the shut-in pressure for the 186 m deep frac, a value equal to 5.4 MP\(_a\) is found, which compares luckily well with the 5.4 MP\(_a\) which was measured.

For the 84 m deep frac, no orientation could be determined with the borehole T.V. camera. Yet, if the fracture is assumed to be vertical, equation 14 suggests that for this fracture, \(\sigma_1 + \sigma_2 = 8 \text{ MP}_a\); this value is quite close to that which can be computed from the above mentioned stress determination. Further if indeed the fracture is vertical, its orientation can be determined from equation (12). This yields a N 58° E direction which is quite similar to that found for the fractures at depth 54, 174 and 186 m.

This is the only test for which the reopening pressure provides a satisfactory result. For the two other tests for which a reopening pressure has been measured, the value is not in agreement with the stress determination. For these two tests the fracture intersected the packers on a short distance (about 50 cm). Since, for this set of experiments, the packers remained inflated at the highest pressure reached during previous pressurisation operations, the fracture remained opened even though the fluid pressure had dropped back to its original value.

Two other boreholes have been sunk on the same site (within 30 m of the first one). One of these boreholes was intended to intersect the fracture developed in the first well at the depth of 186 m. This fracture corresponds to an injection of 13 m\(^3\) of gel; it is propped opened with sand. Intersection of the fracture was expected to occur at 175 m; the fracture was met in fact at 156 m. Identification of this intersection was established from the following observations:

- When drilling of the second borehole (which was done with the downhole percussion technique, the hole being kept empty of water during the drilling process) reached the 156 m depth, the water level in the first borehole started to be lowered.

- An injection of water with a straddle packer located at the 156 m depth in the second borehole was started. At the same time thermal logging was run continuously in the first borehole to identify places where some water would flow in. The 186 m depth was identified as the main source of flow that is the depth where the fracture has been developed.

- Injections of water, in the second borehole, at other depth than
156 m were run with continuous thermal logging of the first borehole. Negligible flows were noticed in the first borehole even though some of the injections were run on permeable natural fractures.

Chemical and thermal tracing provided means to detect the shortest distance followed by the fluid flow between the two boreholes. The value of 42 m computed for this distance is precisely that which can be computed from simple geometrical considerations (Hozanski 1980).

This demonstrates that hydraulic fractures may develop on quite long distances along directions different from that of the major principal stress once they have been initiated along weakness planes.

A few additional shut-in pressure measurements have been made. Comparison between expected and measured values is astonishingly good (see table 3). All the fracture orientations fit with one of the natural fracture direction as determined from surface mapping, none is in the exact orientation of the major principal stress.

| TABLE 3 : Comparison between expected and measured shut-in pressures : |
| --- | --- | --- | --- |
| depth in m | computed shut-in pressure in MPa | Measured shut-in pressure in MPa | Fracture orientation |
| 113 | 2.8 | 2.9 | N 20° E |
| 143 | 3.7 | 3.5 | N 32° E |
| 164 | 5.4 | 5.4 | N 155° E |

Tests in salty deposits

Stress measurements in salty sedimentary deposits, near Mulhouse (Alsace, in eastern France) were attempted from two mines, 670 m below ground level, 10 km apart. In the first mine, a 30 m deep borehole (76 mm diameter) was drilled vertically from a rectangular horizontal shaft (4 m wide, 2.2 m high). Fractures were performed at 28 m, 22 m and 17 m depth from the floor with an inflattable straddle packer (spacing of 50 cm between the packers). The three fractures were vertical and oriented in roughly the same direction (see table 4), that is N 10° E.

The interesting feature in these results is that, although the shut-in pressure is well defined for all three fractures (see figure 5, 6 and 7), it does not yield identical values for all fractures (12.2 MPa for the two upper fractures, 14.2 MPa for the lower one). This variation of stress with depth cannot be accounted for by gravity; clearly in this case, equation (12) does not apply. These variations in horizontal
stress magnitude appear to be caused by the alternance of large, soft beds of salt and shales and very stiff thin anhydrite deposits. The highest stress was measured in the vicinity of a one meter thick bed made of an alternance of anhydrite and shale.

The direction of the fractures is precisely that of a nearby, 20 km long, salt diapir which is still active. Thus it seems reasonable to assume that the fracture strikes are that of the major horizontal principal stress \( \sigma_1 \). Accordingly, the shut-in pressure readings must be close to the minimum horizontal stress magnitude. The \( \sigma_1 \) magnitude can be determined from reopening pressure readings. It is found to be equal to 21 MPa for the two upper fractures and 26 MPa for the lower one. This unusually high deviatoric stress for salty deposits is attributed to the existence of the stiff anhydrite beds.

On the second site, tests were run in an inclined borehole drilled in the direction N 107° E (that is nearly perpendicular to the direction of the vertical fractures) with an 18° dip. A fracture was initiated 18 m away from the shaft wall, that is outside the zone of influence of the shaft. A preexisting recemented fracture helped develop a fracture in the direction N 140° E with a 40° dip. Many injection tests were conducted on this fracture (see table 5). Although after each test the final shut-in pressure was fairly stable, its value kept decreasing after each new test, once about 1 litter had been injected. Each injection test was started without letting any fluid flow out of the borehole.

This continuous decrease in shut-in pressure has been interpreted as a change of orientation of the fracture during its successive extensions. The final value (12.8 MPa) compares fairly well with that observed at the same depth for the vertical fractures obtained on the first site (some 10 km away). Thus it may be concluded that the tip of this fracture must be nearly parallel to the vertical fractures described here above. Then the shut-in pressure observed for the first injection tests can be used to determine the maximum principal stress magnitude thanks to equation (10). The weight of overburden is known with some accuracy to be 15.2 MPa so that \( \sigma_1 \) is found to be 23.5 MPa. This value is in between the two values found at the first site.

Note that, for this fracture, the reopening pressure measurement cannot be used easily since the fracture is inclined with respect to the borehole axis.
<table>
<thead>
<tr>
<th>Depth from ground level</th>
<th>$P_s$</th>
<th>$P_r$</th>
<th>$P_b$</th>
<th>Vertical fracture orientation</th>
</tr>
</thead>
<tbody>
<tr>
<td>700 m</td>
<td>14.2</td>
<td>16.7</td>
<td>16.9</td>
<td>N. 10° E</td>
</tr>
<tr>
<td>694 m</td>
<td>12.2</td>
<td>15.0</td>
<td>18.9</td>
<td>N. 12° E</td>
</tr>
<tr>
<td>691 m</td>
<td>12.2</td>
<td></td>
<td>19.2</td>
<td>N. 10° E</td>
</tr>
</tbody>
</table>

Table 4: Results from a vertical borehole in salt (pressures in MPa)
<table>
<thead>
<tr>
<th>$P_s$ MPa</th>
<th>waiting time (in seconds)</th>
</tr>
</thead>
<tbody>
<tr>
<td>16.9</td>
<td>90</td>
</tr>
<tr>
<td>16.7</td>
<td>114</td>
</tr>
<tr>
<td>16.7</td>
<td>90</td>
</tr>
<tr>
<td>14.7</td>
<td>39 minutes 45 seconds</td>
</tr>
<tr>
<td>15.4</td>
<td>69</td>
</tr>
<tr>
<td>14.7</td>
<td>249</td>
</tr>
<tr>
<td>14.5</td>
<td>183</td>
</tr>
<tr>
<td>13.6</td>
<td>510</td>
</tr>
<tr>
<td>12.8</td>
<td>51</td>
</tr>
<tr>
<td>13.1</td>
<td>20</td>
</tr>
</tbody>
</table>

Table 5: variation of shut-in pressure values for an inclined fracture.
6. CONCLUSION

The accuracy of stress determinations from the interpretation of hydraulic fracturing experiments according to the classical theory rests on the validity of a few a-priori hypothesis:
- the borehole is parallel to one of the principal stress directions;
- the rock is linearly elastic and isotropic with respect to both its elastic behavior and its strength;
- the fracture remains planar as it extends away from the wellbore.

In order to circumvent this difficulty, a new technique has been proposed in which use is made of the rock mass strength anisotropy for developing fractures with various dips and stikes. The stress determination is based on shut-in pressure measurements only but may take into account reopening pressures when the fracture is parallel to the borehole axis.

When only shut-in pressures are used, theoretically six different fracture orientations are necessary for the determination of the full stress tensor (the shut-in pressure is a direct evaluation of the normal stress supported by the fracture). However only three different planes at the same depth are necessary if one of the principal stresses is assumed to be vertical and equal to the weight of overburden. For this case, if only one borehole is available, the stress determination requires fracture planes in three different azimuth but four different depth. This inverse problem remains linear if both horizontal principal stresses vary by the same amount with depth; this supposes that the depth interval remains small.

If the fractures are parallel to the borehole axis, reopening pressures can be used so that only two fractures in different directions are necessary.

This new technique has been applied to two different sites, one in granite, the other in salty deposits. For the first site, the coherence of the results is a positive test of the validity of the technique. For the second site, it was observed that stress variation with depth was not linear so that the above technique was not directly applicable. The stress field was determined according to the classical theory. Accurate measurements of shut-in pressure variations with fracture length as obtained from tests conducted on a preexisting crack provided means to ascertain the validity of the determination.
It is concluded that, even though the classical interpretation technique of hydraulic fracturing tests for stress determination purposes may be valid in many instances, it is always necessary to conduct additional injection tests on preexisting natural fracture planes to evaluate the accuracy of the determination. Observation of the same strike for all artificial hydraulic fractures is not a sound enough test of the accuracy of the stress determination.

ACKNOWLEDGEMENTS

The tests conducted at Le Mayet de Montagne and in the limestone quarry were founded by Institut National d'Astronomie et de Géophysique (A.T.P. transfert de flux de chaleur dans l'écorce terrestre) and the European Economic Community (Geothermal Research funds). The tests in the salty deposits were supported by Mines des Potasses d'Alsace.
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Figure 1: Pressure-time record for a hydraulic fracture in granite.
Figure 2: borehole loading conditions for the oriented fracture test

- $P_v$: jack pressure
- $2\alpha$: bearing angle for each jack
- $\lambda$: orientation of the jacks with respect to the $\sigma_1$ direction

$\sigma_1 \geq \sigma_2$
Figure 3: Influence of flow rate on fracture orientation

<table>
<thead>
<tr>
<th>$\sigma^*$</th>
<th>$\dot{V}$</th>
<th>$P_w$</th>
<th>Fracture Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>$MP_a$</td>
<td>$10^{-3}$ cubic cm/sec</td>
<td>$MP_a$</td>
<td></td>
</tr>
<tr>
<td>3.45</td>
<td>56.6</td>
<td>12.7</td>
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Figure 4: Pressure-injected volume curves for various flow rates and uniaxial stress conditions.
Figure 5: injection test for the fracture at 28 m.
Figure 6: injection test for the fracture at 22 m.
Figure 7: injection test for the fracture at 18 m.
A STATISTICAL FRACTURE MECHANICS DETERMINATION
OF
THE APPARENT TENSILE STRENGTH IN HYDRAULIC FRACTURE

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INTRODUCTION

The rock mechanics engineer is seldom concerned with the tensile strength of intact rock, because the inherent discontinuities are the predominant structural component that usually determines the strength of the rock mass. However, there are a limited number of important situations for which a knowledge of the apparent tensile strength of intact rock is of fundamental importance. For example, the apparent tensile strength must be known in a hydraulic fracture experiment for the determination of in situ stress if the state of stress is to be determined from the initiation of the hydraulically induced fracture. Many investigators have abandoned the use of tensile strength in hydraulic fracture test interpretation because of the difficulty in determining the appropriate magnitude and the size and stress state dependence observed in the laboratory (e.g., Haimson (1968), Scott et al (1953)).

Three observations are invariably made when intact rock samples are taken into the laboratory and tested to determine tensile strength.

(1) The apparent tensile strength depends upon the sample size (the larger the specimen, the smaller the strength).
(2) The apparent tensile strength depends upon the type of test being performed.
(3) With any given test and specimen size, a scatter (usually skewed) about the mean is obtained.

The first dilemma (commonly referred to as the size effect) is also observed with respect to compressive strength and an apparent Young's modulus, although to a lesser extent than with tensile strength (e.g., Heuze (1980)). The second observation noted above has been brushed away by using different names to refer to the strength observed in different tests. For example, the apparent tensile strength in bending is referred to as the Modulus of Rupture. The tensile strength determined by indirect tension tests is often referred to with an adjective taken from the test; for example, the Brazilian tensile strength or the split cylinder tensile strength. The third observation above is often totally neglected in the reporting of test results. Scatter about the mean is often attributed to testing methods and/or sample inhomogeneity. Thus, more often than not, the only result of the tensile testing may be the mean without the standard deviation or any of the other statistical moments. A premise of statistical fracture mechanics is that all three of the observations above are consistent with the behavior of brittle materials.
In the classical continuum concept of strength, none of the observations noted above could be attributed to the inherent behavior of the material, but rather must be attributed to the testing methods and/or the sample inhomogeneity. The probabilistic concept of the strength of brittle rock on the other hand is an integral concept rather than a differential concept which involves the geometry and volume or surface area of the specimen, the spatial stress distribution, and the distribution and strength of the failure inducing flaws within the specimen.

The purpose of this paper is to present a statistical fracture mechanics approach to determining the apparent tensile strength in hydraulic fracturing. Following development of the statistical fracture mechanics model, the apparent hydraulic fracture tensile strength of Stripa Granite (Nikiewicz, et al (1979)) will be evaluated for varying stress states and borehole sizes.

STATISTICAL FRACTURE MECHANICS

At the heart of statistical fracture mechanics is the supposition that materials inherently contain defects which eventually lead to structural collapse under increasing load. The concept of flaws or defects within materials was first popularized by Griffith (1921) in his formulation of a fracture criterion supposing flat elliptical flaws. Statistical fracture mechanics differs from a Griffith type criterion in that one hypothesizes at the onset that the flaws need not be geometrically uniform nor need they possess uniform strength. In geological materials, flaws or defects need not necessarily be physical voids, but may well be soft minerals in contact with significantly stiffer minerals (Brace (1964)). In this context, the softer minerals act much as if they were not present under the application of load. If one were able to establish the geometry of the failure inducing flaws, a general statistical fracture criterion could be established for virtually any multiaxial stress state. Although this concept is discussed by Ratigan (1981), the geometry of the failure inducing defects will be considered to be flat for the remaining portions of this paper.

The formulation of a statistical fracture criterion requires the specification of both global and local failure criteria. The terms global and local are used rather loosely; however, they will become more obvious in later portions of this paper. The global failure criterion is that which dictates the total collapse of the structure. The local failure criterion relates to the variant (in the statistical sense) which characterizes the failure of an individual material defect. Throughout this paper, material response to load shall be assumed to be brittle. In other words, release of strain energy shall be assumed to occur only through rupture or fracture of the specimen.

Three further assumptions will be made in this paper in the development of a statistical fracture criterion. Firstly, material defects shall be assumed to be present in large numbers. This is a necessary assumption in extreme value statistics (Gumbel (1958)). A quantification of large has been attempted by Jayatilaka and Trustrum (1977) who show that large may well imply 100 to 1000 defects within the specimen. The second assumption is that the material defects shall be isotropically distributed (Weibull (1939b)). This latter assumption appears to have been exclusively adopted in the rock mechanics literature involving applications of statistical
fracture mechanics models. Experimental data presented by Ratigan (1981) indicates that this assumption is inappropriate for some rocks. The third assumption is that defects do not interact with one another. This assumption allows the use of continuum concepts for determining stress states in linear elastic materials containing defects.

Global Failure Criterion

Global failure criteria for statistical fracture mechanics models essentially relate to the quantity of material defects which must fail prior to collapse of the specimen. The options available are, of course, infinite. However, one can categorize global criteria into two classes, viz;

(1) Single defect criteria
(2) Multiple defect criteria.

The single defect criteria have been termed Weakest Link Models (Weibull (1939a)), for the collapse of the specimen depends only on the failure of a single link in a series structure of links. The Weakest Link Theory was popularized by Weibull in two papers in 1939 (Weibull (1939a), (1939b))† and has received considerable attention in the literature (Finnie (1977)).

Multiple defect criteria can best be understood by again referring to the chain analogy. These criteria invariably involve links in parallel, series of links in parallel, or some other combinations of parallel and series structures (e.g., Wijk et al (1978)). Although the multiple defect criteria have received much less attention in the literature, the potential uses in progressive failure of materials by microcracking appears promising (e.g., McClintock and Mayson (1976), McClintock (1977)).

The appropriateness of a single or multiple defect criterion is difficult to assess a priori. However, microcracking (indicative of the potential need for a multiple defect criterion) may be qualitatively assessed in the laboratory by means of the microseismic or acoustic emission techniques.

Throughout this paper, the single defect or Weakest Link Model shall be assumed. Excellent treatises of the Weakest Link Theories are available in the literature (e.g., Weibull (1939a), Freundenthal (1968)) and only the essential characteristics will be repeated here.

If we define the probability of failure of link \(i\) as \(f_i\), then the survival probability \(P_s\) of a series structure of \(N\) such links is;

\[
P_s = (1-f_1)(1-f_2) \ldots (1-f_N)
\]

\[
= \prod_{i=1}^{N} (1-f_i)
\]

† Although Weibull did not use the terminology "weakest link", he nonetheless presented the mathematical form which is known as the weakest link.
If we further use the approximation \( \ln(1+f_i) = f_i \) (implying \( f_i \) small) and assume that there is a sufficiently large number of flaws so as to replace the summation with an integral, we are led to:

\[
P_s = e^{-\int_R n(\sigma) \, dR}
\]

where:

\[
R = \text{the geometric domain of the structure where defects reside}
\]

\[
n(\sigma) = \text{material function (number of flaws per unit region with strength < } \sigma)\]

\[
\sigma = \text{stress (tension positive)}
\]

\[
P_f = \text{probability of failure.}
\]

The geometric domain, \( R \), may be considered to be composed of the specimen volume as well as free surfaces of the specimen. As will be shown in the next section, this multiple domain concept also allows for multiple \( n(\sigma) \) functions; one for the volume and one for the free surfaces. If a material fails typically from a single pre-existing defect, Equation [2] is exact. However, empiricism will be introduced in the specification of \( n(\sigma) \), the local failure criterion.

Weibull defined a term \( B \), which he referred to as the risk of rupture, where:

\[
B = \int_R n(\sigma) \, dR
\]

For the remainder of this paper, the terminology cumulative failure probability, \( G \), shall be used rather than the probability of failure, \( P_f \), in order to be consistent with the literature.
Local Failure Criterion

The function \( n(\sigma) \) may be determined in the laboratory for a specific test (Evans and Jones (1978)). This methodology will not be adopted, however, due to the lack of generality. Weibull (1939b) proposed a functional form for uniaxial tension;

\[
\begin{align*}
  n(\sigma) &= \begin{cases} 
    \left( \frac{x(\sigma) - x_u}{x_o} \right)^m & \text{if } x(\sigma) > x_u \\
    0 & \text{if } x(\sigma) \leq x_u 
  \end{cases} \\
\end{align*}
\]  

where:

\( x = \) some suitable function of stress

\( x_u = \) the value of \( x \) below which rupture does not occur

\( x_o = \) scaling constant

\( m = \) Weibull modulus.

Weibull stated that the region \( R \), could well be both a volumetric region in addition to a free surface region, so that in the general case, the risk of rupture can be stated as;

\[
R = \int_S n_s(\sigma) \, dS + \int_V n_v(\sigma) \, dV
\]  

where:

\[
\begin{align*}
  n_s(\sigma) &= \begin{cases} 
    \left( \frac{x_s(\sigma) - x_{us}}{x_{os}} \right)^{m_s} & \text{if } x_s(\sigma) > x_{us} \\
    0 & \text{if } x_s(\sigma) \leq x_{us} 
  \end{cases} \\
  n_v(\sigma) &= \begin{cases} 
    \left( \frac{x_v(\sigma) - x_{uv}}{x_{ov}} \right)^{m_v} & \text{if } x_v(\sigma) > x_{uv} \\
    0 & \text{if } x_v(\sigma) \leq x_{uv} 
  \end{cases}
\end{align*}
\]
Weibull's Theory is typically mis-stated in the literature as;

\[ T_a \frac{1}{V^m} = \text{constant} \]  \[ \text{(6)} \]

where:

\( T_a \) = apparent tensile strength.

Equation [6] is a specific form of Weibull's theory which only arises following the assumptions that (1) \( x_u = 0 \) and (2) \( n_s(a) = 0 \). Both of these assumptions must be verified in the laboratory before being discarded or adopted.

Weibull selected the function \( x(\sigma) \) to be the tensile stress normal to the material defect or flaw. However, Ratigan (1981) has suggested that a more appropriate selection for the function \( x(\sigma) \) may be the strain energy release rate associated with Mode I fracture. If we assume that a material contains flat, non-interacting cracks, the critical strain energy release rate associated with Mode I fracture can be shown to be;

\[ G = k \sigma^2 \]  \[ \text{(7)} \]

where:

\( k \) = a proportionality constant

\( \sigma \) = the tensile stress normal to the crack.

Using Equation [7], we may redefine Weibull's risk of rupture term \( n(\sigma) \) as

\[ n(\sigma) = \begin{cases} \left( \frac{G - G_u}{G_0} \right)^{\alpha} & G > G_u \\ 0 & G < G_u \end{cases} \]  \[ \text{(8)} \]

where:

\[ G = \begin{cases} \sigma_n^2 & \sigma_n > 0 \\ 0 & \sigma_n < 0 \end{cases} \]

\( G_u \) = threshold strain energy release rate
\[ G_0 = \text{scaling constant.} \]

\[ \sigma_n = \sigma_1 n_1^2 + \sigma_2 n_2^2 + \sigma_3 n_3^2 \]

\[ n_1, n_2, n_3 = \text{direction cosines} \]

\[ \sigma_1, \sigma_2, \sigma_3 = \text{principal stresses.} \]

Note that when \( G_u = 0 \), \( \alpha = \frac{m}{2} \). We have introduced the terminology, \( \alpha \), so that it is not confused with the Weibull modulus, \( m \), which appears so profusely in the literature. Further, the use of the term \( \alpha \) is consistent with Freundenthal (1968) who presents a form similar to Equation [8] for \( G_u = 0 \).

The mean apparent tensile strength for the three parameter model of Equation [8] can be shown to be;

\[ T_a = \frac{\sigma_u}{2} + \int_{\frac{G_u}{2}}^{\infty} e^{-BdT} \frac{T_a}{G_u} \]

HYDRAULIC FRACTURING

We shall discuss two different types of hydraulic fracturing tests. The first type of test is that which is performed in the laboratory under controlled conditions of external loading. The second type of test which will be discussed is the field hydraulic fracture test performed in an unknown stress state and in a geometric scale significantly larger than in the laboratory.

Laboratory Hydraulic Fracture Test

The laboratory hydraulic fracture test is performed on a rock specimen usually fabricated from nominal size core. The specimen contains a concentric inner drillhole part way through the core axis (Haimson (1968)) which is pressurized until rupture occurs. External vertical and horizontal loadings may be applied to the test specimen. Assuming the internal borehole to be much smaller than the outside dimensions, the stress state (tension positive) at the borehole surface during pressurization is;

\[ \sigma_r = -p \]

\[ \sigma_\theta = p + (\sigma_{H_{\max}} + \sigma_{H_{\min}}) - 2 (\sigma_{H_{\max}} - \sigma_{H_{\min}}) \cos 2\theta \]

[10]
\[ \sigma_z = -p_{\text{axial}} \]

where:

\[ p = \text{internal borehole pressure} \]

\[ \sigma_H = \text{horizontal applied stresses} \]

\[ \theta = \text{angle from location of maximum tension} \]

\[ p_{\text{axial}} = \text{axial loading} \]

At rupture the state of stress is:

\[ \sigma_r = \{ T_\ell - 3\sigma_{H_{\text{min}}} + \sigma_{H_{\text{max}}} \} \]

\[ \sigma_\theta = T_\ell - 2 \{ \sigma_{H_{\text{max}}} - \sigma_{H_{\text{min}}} \} \{ \cos 2\theta - 1 \} \]

\[ \sigma_z = -p_{\text{axial}} \]

where:

\[ T_\ell = \text{apparent tensile strength in the laboratory hydraulic fracture test.} \]

After a test has been performed, the apparent tensile strength, \( T_\ell \), can be calculated from Equations [10] and [11]. Haimson (1968) calculated the apparent tensile strength for an extensive number of tests he performed on various rock types. Two conclusions can be drawn from Haimson's results, viz:

(1) The apparent tensile strength decreases with increasing horizontal confining pressure (\( \sigma_{H_{\text{max}}} = \sigma_{H_{\text{min}}} \)).

(2) The apparent tensile strength decreases with increasing internal borehole diameter.

In later sections of this paper, we shall show that the statistical fracture mechanics model presented can account for both of the above observations.

In order to calculate the mean apparent tensile strength for a particular laboratory test from Equation [9], we must have an expression for the risk of rupture, \( R \). Ratigan (1981) performed numerous tests on Stripa Granite and concluded that the greatest contribution to the risk of rupture came from the \( n_s \) term (see Equation [5]). Adopting this result we can
express the risk of rupture in the laboratory hydraulic fracture test as;

\[
B = \frac{A_r}{2\pi^2} \left[ \frac{T_L}{\zeta_0} \int_0^\frac{\pi}{2} \int_0^\frac{\pi}{2} F(\phi,\psi,\theta) \cos(\phi) \, d\phi \, d\psi \, d\theta \right] \tag{12}
\]

where:

- \( A_r \) = surface area of internal borehole.

\[
F(\phi,\psi,\theta) = \begin{cases} 
\frac{1}{T_L} \left[ \sigma_n^2 - \zeta_u \right] & \text{if } \sigma_n > \zeta_u^{\frac{1}{2}} \\
0 & \text{if } \sigma_n < \zeta_u^{\frac{1}{2}}
\end{cases}
\]

\[
\sigma_n = \sigma_\theta \cos^2 \phi \cos^2 \psi + \sigma_r (\sin^2 \phi - 1)
\]

The tangential and radial stresses are taken from Equation [11]. The angles \( \psi \) and \( \phi \) are used in expressing the direction cosines of Equation [8] (see Figure 1). Note that we have neglected the axial stress which results in little change in the apparent tensile strength. The contribution to the risk of rupture from the external surfaces of the test specimen has been neglected.

The expression for \( F(\phi,\psi,\theta) \) given by Ratigan (1981) mistakenly admits all positive values of \( F(\phi,\psi,\theta) \). That is to say, compressive normal stresses greater than \( \zeta_u^{\frac{1}{2}} \) are erroneously allowed.

The equation for normal stress above contains a term which is invariant with respect to location on the unit sphere in principal stress space. This term represents the internal borehole fluid pressure assumed to be acting on the defect or crack face. Without admitting this term, a decrease in apparent tensile strength with increasing confining pressure \( \sigma_{H_{\text{max}}} = \sigma_{H_{\text{min}}} \) cannot be reproduced with the present model. This term was mistakenly omitted by Ratigan (1981).

**Field Hydraulic Fracture Test**

Two quantities which are measured in the field test are breakdown pressure, \( P_c \) and shut-in pressure, \( P_S \). Employing numerous assumptions, the in situ stress state is often interpreted as;

\[
\sigma_{H_{\text{min}}} = -P_S \tag{13}
\]

\[
\sigma_{H_{\text{max}}} = T + P_c - 3P_S
\]
where T = tensile strength.

Equation [13] would be correct if tensile strength were a size invariant property. However, since it is not, Equation [13] must be replaced with:

\[ \sigma_{H_{\text{min}}} = -p_s \]

\[ \sigma_{H_{\text{max}}} = T_f(q_{ij}, C_u, C_o, \alpha, A_f) + p_c - 3p_s \]

Equation [14]

where

\[ A_f = \text{the surface area of the borehole in the field hydraulic fracture test.} \]

APPARENT TENSILE STRENGTH OF STRIPA GRANITE

Ratigan (1981) reports values of the three parameters of the statistical fracture mechanics model for Stripa granite. However, certain compressive stresses were mistakenly allowed in the function \( F(\phi, \psi, \theta) \) (see Equation [12]) in evaluating the parameters. Therefore, the parameters were reevaluated in the present work considering only the laboratory hydraulic fracture tests reported by Ratigan (1981). The parameter values were found to be:

\[ C_u = 75 \text{ (MPa)}^2 \]

\[ C_o = 0.271 \times 10^{-1} \text{ (MPa)}^2 - m^2/\alpha \]

\[ \alpha = 0.75 \]

The values of the parameters imply qualitatively that the risk of rupture is nearly proportional to the square root of the strain energy release rate above and beyond a certain threshold value. Note that a non-zero value of \( C_u \) implies that the apparent tensile strength of Stripa Granite will never be lower than 8.7 MPa, independent of the size or type of tensile test.

Using the values of the parameters for Stripa Granite, the expression for the risk of rupture of Equation [12] and the expression for the mean apparent tensile strength of Equation [9], we shall examine expected variations in the apparent tensile strength in hydraulic fracture tests. We shall define a quantity, \( T_\theta \), as the mean laboratory hydraulic fracture strength. \( T_\theta \) results from testing specimens with a 0.7 cm inside diameter and a 6.3 cm outside diameter with a pressurized borehole length of 7.6 cm.
Ratigan (1981) reported the value of $T^2$ for Stripa Granite as 16.2 MPa.

We shall further define the quantities $\gamma$ and $\delta$ as;

$$\gamma = -\frac{\sigma_{\text{H max}}}{T^2_{\ell}}$$

$$\delta = -\frac{\sigma_{\text{H min}}}{T^2_{\ell}}$$

The mean apparent tensile strength for tests with different conditions than those used in obtaining $T^2_{\ell}$ will be defined to be $T_f$.

The quantity $T_f$ over $T^2_{\ell}$ is illustrated as a function of $\gamma$ in Figure 2 for various combinations of $\gamma$ and $\delta$. Of interest in this figure is the fact that $T_f$ is less than $T^2_{\ell}$ when a uniform confining pressure is applied to the specimen; however, $T_f$ may be greater than $T^2_{\ell}$ under nonuniform horizontal stresses. Note that when $\gamma = 2\delta$, $T_f/T^2_{\ell}$ remains unchanged. The lower limit on the ratio $T_f/T^2_{\ell}$ is controlled by the magnitude of $G_u$. If we neglected the fluid pressure on the defect or crack face, the ratio $T_f/T^2_{\ell}$ would always be greater than one.

The specimen size used in estimating $T^2_{\ell}$ was not varied for the results shown in Figure 2. This is not the case when considering field hydraulic fracture tests where $A_f$ (borehole surface area for $T_f$) may be 50 to 100 times greater than $A_{\ell}$. The significance of the ratio $A_f$ over $A_{\ell}$ is illustrated in Figures 3 thru 6. Again, the parameter values are those for Stripa Granite. Doe et al. (1981) performed field hydraulic fracture tests in Stripa Granite with a value of $A_f$ over $A_{\ell}$ equal to about 80. In this case, we note that an appropriate value of apparent tensile strength for these investigators is approximately $G_u^{1/2}$ or 8.7 MPa, provided $\sigma_{\text{H min}}$ is nonzero.

**CONCLUSIONS**

A statistical fracture mechanics model has been presented based on the critical strain energy release rate associated with Mode I fracture. The model is capable of duplicating trends as well as magnitudes in the apparent tensile strength behavior in the laboratory. The model may be used in conjunction with field hydraulic fracture tests for determining in situ states of stress.

**ACKNOWLEDGEMENTS**

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Figure 1. Unit Sphere in Principal Stress Space.
Figure 2. $\frac{T_f}{T_l}$ versus $\gamma$. 

\[ \delta = 0 \]
\[ \gamma = 3\delta \]
\[ \gamma = 2\delta \]
\[ \gamma = \delta \]
Figure 3. $\frac{\bar{T}_f}{\bar{T}_i}$ versus $A_f/A_i$ for $\sigma_{H_{min}} = 0$. 

$\sigma_{H_{min}} = 0$

$\gamma = \frac{1}{4}$

$\gamma = \frac{1}{2}$
Figure 4. $\frac{T_f}{T_1}$ versus $A_f/A_1$ for $\sigma_{H_{\text{max}}} = \sigma_{H_{\text{min}}}$
Figure 5. $\frac{T_f}{T_k}$ versus $A_f/A_k$ for $\sigma_{H_{\text{max}}} = 2\sigma_{H_{\text{min}}}$.
Figure 6. $\bar{T}_f/\bar{T}_l$ versus $A_f/A_l$ for $\sigma_{H_{\text{max}}} = 3\sigma_{H_{\text{min}}}$.
HYDRAULIC FRACTURE IN ARBITRARILY
ORIENTED BOREHOLES: AN ANALYTIC APPROACH

by

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ABSTRACT

An analytic solution for stresses around a borehole arbitrarily oriented with respect to the principal stress axes is used to determine the error in assuming that model hydraulic fractures are oriented normal to the least compressive principal stress direction. Model fractures initiate parallel to the boreholes at points where the stresses in the plane tangent to borehole are least compressive. For $S_1 > S_2 > S_3$, where $S_1$, $S_2$, $S_3$ are the greatest, intermediate and least compressive principal stresses, respectively, it is found that large errors occur for boreholes inclined at angles of $0^\circ$ to $30^\circ$ from the $S_1$-$S_2$ plane when $S_3$ approaches $S_2$. For $S_3$ small with respect to $S_2$, the error is nearly equal to the inclination to the $S_1$-$S_2$ plane, except for boreholes located near the $S_1$-$S_3$ plane. For these locations the angle up from the $S_1$-$S_2$ plane at which the fracture rolls over into the $S_1$-$S_3$ plane decreases as $S_3$ approaches $S_2$. As $S_2$ approaches $S_1$, these effects are minimized. The results of this modeling indicate that errors as large as $90^\circ$ are possible in inferring that hydraulic fractures are normal to the least compressive stress direction, even when the borehole is normal or nearly normal to the least compressive stress direction.

INTRODUCTION

Hydraulic fracture in situ stress measurements offer one of the best opportunities to determine both the orientation and magnitude of stress to depths exceeding 5 km (Zoback et al., 1980; Haimson, 1978). The pressure history of pumping is related to the stress through the relationship of Hubbert and Willis (1957):

$$P_b = 3 S_{\text{min}} - S_{\text{max}} - P_o + T \quad \text{(1a)}$$

where $S_{\text{min}}$, $S_{\text{max}}$ are the minimum and maximum principal stresses perpendicular to the borehole, respectively, $P_o$ is the fluid pore pressure in the rock, $T$ is
the tensile strength of the rock, and \( P_b \), the breakdown pressure, is the pressure required to initiate the hydraulic fracture. There remain several questions about the applicability of equation 1a. A clear breakdown pressure is not always observed, and it is not always possible to accurately determine the tensile strength \( T \) of the rock. Recently, an alternative approach to equation 1a has been developed based on the reopening pressure \( P_r \), defined as the pressure required to open a closed crack on subsequent pumping cycles. Then, equation 1a may be replaced with

\[
P_r = 3 S_{\text{min}} - S_{\text{max}} - P_o
\]

after Bredehoeft et al. (1976). A second equation relating pumping history to stress is:

\[
P_s = S_{\text{min}}
\]

where \( P_s \), the shut-in pressure, is the pressure required to keep the hydraulic fracture open without further propagation (Hubbert and Willis, 1957). The fracture aligns with the borehole axis and leaves the borehole in the direction of \( S_{\text{max}} \) such that the normal to the fracture is in the direction of \( S_{\text{min}} \). Figure 1 illustrates the hydraulic fracture geometry described above. From equations 1 and 2, and the orientation of the fracture, the magnitudes and orientations of \( S_{\text{min}}, S_{\text{max}} \) can in principle be recovered.

In the derivation of equations 1 and 2 it has been assumed that one of the principal stresses aligns with the borehole. In this case, the problem reduces to the two-dimensional plane strain situation of a circular hole in an infinite plane. There is some evidence from earthquake fault plane solutions for events away from plate boundaries that the principal stresses are close to horizontal and vertical (Sykes and Sbar, 1973). Thus, for vertical boreholes, the assumption of a two-dimensional problem may be reasonable, although it is possible even for this case that the principal stresses may be inclined by as
much as 10-30° to the borehole. Inclined boreholes, such as those used for geothermal energy, or as may become available as holes of opportunity, are unlikely to be aligned with the principal stresses. It is the purpose of this paper to investigate the case where none of the principal stresses align with the borehole axis. In particular, it is important to investigate the error one makes in interpreting the orientation of the minimum principal stress from the fracture orientation.

The study of inclined boreholes continues to receive some attention in the literature. Fairhurst (1968) and Daneshy (1973) provide much of the background for the topic. Fairhurst developed a set of equations governing the stress concentrations around a cylindrical borehole in a three-dimensional stress state. Daneshy extended the equations to include the effects of a permeable formation and provided early experimental data on inclined boreholes in laboratory specimens. Recent laboratory work by Fairhurst (Mizuta et al., 1981) indicates several modes of fracture initiation related to inclination of the borehole, principal stress values, and injection rates. Laboratory experiments on boreholes confined to the S1-S3 plane, where S1, S3 are the maximum and least compressive principal stress axes, respectively, again indicates that fracture orientation depends on the geometry and values of the principal stresses (T. Dey, personal communication, 1981).

THEORY

The solution to the problem of determining the error in assuming that the fracture orientation is normal to the least compressive principal stress, S3, proceeds as follows and is similar in many respects to the derivation in Daneshy (1973). First, the principal stress state S1, S2, S3, (maximum, intermediate, and least compressive principal stress, respectively) are resolved into a six component stress tensor in a borehole coordinate system.
The borehole coordinate system is defined by two angles. One is the angle up from the S1-S2 plane toward S3 to the borehole axis and the other is the angle from S1 toward S2 to the projection of the borehole onto the S1-S2 plane. The geometry is shown in Figure 2.

The matrix equation governing the transformation of the stresses is

\[
\begin{bmatrix}
\sigma_b
\end{bmatrix} = [B]\begin{bmatrix}
S_1 \\
S_2 \\
S_3
\end{bmatrix}[B]^T ,
\]

where \([\sigma_b]\) are the stresses in the borehole coordinate system in the absence of a borehole and \([B]\) is the 3X3 matrix of direction cosines between the borehole and principal stress axes. Details of these and other expressions in this section are found in the Appendix.

Once the stresses in the borehole coordinate system are formed, the effect of the borehole itself is considered. Specifically, stress concentrations due to the borehole are considered, and three components of stress acting at the borehole wall are computed. These are a hoop stress, a stress parallel to the borehole axis, and shear stress in the plane tangent to the borehole. These three stresses, \(\sigma \omega\), \(\sigma_{xx}\), and \(\sigma_{\omega x}\), respectively, are utilized to find the maximum tensile stress acting on the borehole wall as a function of the angle \(\theta\) around the borehole axis. The angle \(\theta\) then gives the orientation of a fracture that initiates parallel to the borehole and permits the calculation of the normal to the fracture, \(n\). Finally, the angular distance \(\psi\) between \(n\) and the S3 axis is the error in assuming that the fracture orientation is normal to the least compressive stress direction. Figure 2 shows the geometry between the fracture normal \(n\), the error \(\psi\), and the S3 direction.
A suite of models have been considered to test the error as a function of the relative magnitudes of $S_1$, $S_2$, and $S_3$ and the orientation of the borehole with respect to the $S_1$, $S_2$, $S_3$ axes. The results are plotted in Figures 3-7. A stereographic projection of the first quadrant of $S_1$-$S_2$-$S_3$ space is utilized to display the results. The perimeter of the projection refers to $\psi = 0$ (see Figure 1), and corresponds to boreholes located in the $S_1$-$S_2$ plane. The two axes, labeled $S_1$ and $S_2$, respectively, refer to boreholes oriented in the $S_1$-$S_3$ and $S_2$-$S_3$ planes, respectively, with the numbers on the axes referring to the angle $\theta$, or the angle from the $S_1$-$S_2$ plane toward $S_3$ to the borehole. The error $\psi$, bounded by 0 and 90°, is contoured in the stereographic projection for values of 20, 40, 60, and 80°.

The results for the first set of models are shown in Figure 3a-f. For Figure 3a, $S_1 = -1000$, $S_2 = -100$, and $S_3 = 0$. We define a dimensionless quantity $\alpha$ by

$$\alpha = \frac{S_2 - S_3}{S_1 - S_3},$$

where $0 < \alpha < 1$. For Figure 3a, $\alpha = 0.1$. The units of stress are arbitrary, but may be taken as bars or PSI. Figure 3a shows that for $S_3$ small with respect to $S_2$, boreholes located in the $S_1$-$S_2$ plane produce a fracture that is always in the $S_1$-$S_2$ plane, or normal to $S_3$, and hence the error is 0.

Further, boreholes located in the $S_2$-$S_3$ plane indicate that the error is simply the angle $\theta$, as shown in Figure 3c, where $\psi$ is plotted versus $\theta$ for boreholes in the $S_2$-$S_3$ plane. For all other choices of $S_1$, $S_2$, $S_3$ as well, $\psi$ is equal to $\theta$ for orientations in the $S_2$-$S_3$ plane. The error in the $S_1$-$S_3$ plane depends on the relative sizes of $S_2$ and $S_3$, as shown in Figure 3d, where $\psi$ is plotted versus $\theta$ for boreholes in the $S_1$-$S_3$ plane for various values of $S_3$. In all cases for $S_2 = -100$, the error is 90° for $\theta > 45°$. As
S3 approaches S2, the angle $\phi$ at which the fracture rolls over into the borehole S3 plane decreases to about 20°. As S3 approaches S2, pathological errors develop for locations away from the S1-S3 and S2-S3 planes. Specifically, for small $\theta$ and $\phi$, the error can be as large as 90° as shown in Figure 3b for S1, S2, S3, = -1000, -100, -98. A small region, roughly bounded by $0 < \theta, \phi < 20^\circ$, becomes subject to large errors. Figure 3e shows $\psi$ for a profile with $\theta = 8^\circ, 0 < \phi < 90^\circ$ for various values of S3. This represents a profile essentially through the maximum error in Figure 3c, and shows that as S3 approaches S2, the error at $\phi = 0$ can be as large as 90°. Also, the error increases slightly for $20 < \phi < 90^\circ$, and begins to produce a non-zero error at $\phi = 0$ for an S3 of about -96. The onset of non-zero error at $\phi = 0$ is defined as the beginning of the large error mode. For S3 = 0, the error is nearly $\phi$, the inclination up from the S1-S2 plane. When the error is 90° the fracture opens up in the plane of S3 and the borehole. For $\phi = 0^\circ$, this implies that the fracture is normal to the S1-S2 plane. The error goes to 90° as $\phi$ approaches 90°, since this implies that the borehole approaches the S3 direction and all fractures must be parallel to the borehole. The error for boreholes located in the S1-S2 plane is shown in Figure 3f for various values of S3. The maximum error is centered near $\theta = 8^\circ$.

The results for models with S1 = -1000, S2 = -250, and various values for S3 are shown in Figure 4a-d. The results are similar to the previous case, except that the large error mode begins for a larger separation between S3 and S2, or equivalently, for a lower value of $\alpha$. Also, the angle $\theta$ between the S1 and S2 axes for which the error is a maximum increases from about 8° in the previous models to about 16°. The shape of the single profile changes slightly, with the major difference between Figures 3e and 4d being in the region $20 < \phi < 90^\circ$. Finally, the angle $\phi$ for boreholes in the S1-S3 plane where the error becomes 90° varies from about 60° to about 40°.
As S2 increases, there are some changes in the general pattern. The results for S2 = -500 are shown in figure 5a-d. The large error mode begins for an S3 of about -420, or $\alpha = 0.138$, as shown in Figure 5b. This is the largest relative separation between S2 and S3 for the onset of the large error mode for the models tested. The maximum error occurs for a profile of $\Theta$ about equal to 20°, as shown in Figure 5d. The main difference between this and the previous profiles occurs over the range $20 < \Theta < 90$, where the error changes more rapidly near $\Theta = 20^\circ$ as S2 approaches S1.

Results for S2 = -750 are shown in Figure 6a-d. The onset of the large error mode occurs from S3 of about -725, corresponding to $\alpha = 0.1$. Thus, the large error mode begins for a smaller relative separation between S3 and S2 for S2 = -750 than it did for S2 = -500. Again, the maximum error occurs for a $\Theta$ of about 20°.

The final set of results, for S2 = -900, are shown in Figure 7a-d. Even as S3 approaches to within .1% of S2, the error at $\Theta = 0^\circ$ does not go to 90°. Most of the variation as S3 approaches S2 occurs for intermediate values of $\Theta$. In general, the error curve has two portions. In the first, $\psi$ decays linearly from $\psi=90^\circ$ at $\Theta=90^\circ$ to a value of $\Theta$ which decreases as S3 approaches S2. Then, there is a change in the decay rate for $\psi$ into the second portion of the curve where $\psi$ decays nearly linearly to $0^\circ$ at $\Theta=0$, as shown in Figure 7d.

**DISCUSSION AND CONCLUSIONS**

One of the critical assumptions of this work is that the fracture initiates parallel to the borehole. Experimental results indicate that the fracture pattern for inclined boreholes is complex (Daneshy, 1973; Mizuta et al., 1981). In general, there is a component of the fracture along the borehole axis, although occasionally the entire fracture surface crosses the
borehole at an angle. Clearly, then, the analytic procedure outlined in this paper only applies to that portion of the fracture forming parallel to the borehole. This is the portion of the fracture, if any, that will typically be used in the field to infer the direction of $S_3$. The influence of the borehole is expected to decay rapidly with distance from the borehole, and the initial fracture should rotate upon leaving the borehole and assume an orientation perpendicular to the regional $S_3$ with several borehole radii. There is laboratory evidence for this effect (Mizuta et al., 1981). If the bulk of the fracture area is oriented normal to $S_3$, then the shut-in pressure $P_s$ should remain a good indicator of $S_3$.

It has also been assumed that there is no dependence on pumping rate. It is becoming increasingly clear that pumping rate plays an important roll in the fracture initiation process (Zoback and Pollard, 1978; Mizuta et al., 1981). The exact nature of the dependence on injection rate is still the subject of considerable research effort, but could be included in the analytic technique presented here once the dependence becomes better known.

The results presented in this paper could also be extended to the case of a permeable rock formation using the equations of Daneshy (1973). For the present, the results assume either an impermeable rock mass or injection rates high enough to minimize the effects of permeability.

The relationship between breakdown pressures for the two-dimensional and inclined borehole cases is complicated by the fact that the principal stress parallel to the borehole is not included in equation 1. Thus, no attempt has been made in this preliminary study to compare breakdown pressures for the two cases.

Within the assumptions mentioned above, the conclusions of the modeling can be summarized as follows. First, the error $\psi$ is nearly equal to $\phi$, the
inclination of the borehole to the S1-S2 plane, until S3 approaches S2. As S3 approaches S2, a large error mode develops for \( \theta < 40^\circ \) and \( 0 < \phi < 30^\circ \). This large error mode begins for the largest relative separation between S3 and S2 for an S2 of about -500. As S2 approaches S1, the error at \( \phi = 0^\circ \) is smaller as S3 approaches S2. In general, as S3 approaches S2, it is possible to get an error of 90° for boreholes located in the S1-S2 plane. These boreholes are 90° to the S3 axis, and hence this is considered a pathological error. The maximum error occurs for a \( \theta \) of about 20°. The error is independent of S1, S2, S3 for boreholes in the S2-S3 plane. Also, it is possible to get very rapid variations in the orientation of the fracture for small changes in S3 when S3 approaches S2. This may help explain apparently contradictory hydrofracture results from nearby sites. Finally, the results indicate that while the parameter \( \alpha \) describing the relative sizes of S1, S2, S3 as defined as equation 4 is useful, it alone is insufficient to describe the behavior of the error \( \psi \). Specifically, for a constant \( \alpha \) with two different values of S2 the results are not equivalent.

Laboratory testing is necessary to verify the models proposed in this paper. Orientations that are particularly sensitive to changes in the values of S1, S2, S3 occur for \( 0 \leq \theta \leq 30^\circ \) and for \( 0 \leq \phi \leq 30^\circ \), as well as the S1-S3 plane. Preliminary laboratory testing of samples with the borehole located in the S1-S3 plane with \( \phi = 30^\circ \) indicate that the fracture initiates parallel to the borehole for S1, S2, S3 = -1000, -44, 0 PSI. When S2 is increased to -110 PSI, however, the fracture is oriented in the S1-S2 plane at distances far from the borehole while near the borehole it is distorted and enters the borehole at an angle (T. Dey, personal communication, 1981). From the analytic models, the roll over should have occurred at a somewhat higher value of S2 (see Figure 3 for comparison). Thus, limited experimental work to date.
is indicative of changes in the hydraulic fracture orientation with changes of the relative magnitudes of $S_1$, $S_2$, and $S_3$ for inclined boreholes, although more laboratory and field data are needed to test and refine the model.

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APPENDIX

We first consider a principal stress coordinate system where the X1, X2, X3 axes are along the S1, S2, S3 directions, respectively. We define the borehole location by \( \mathbf{b} \), as shown below, where \( \theta \) is the angle up from the X1-X2 plane to \( \mathbf{b} \), and \( \varphi \) is the angle from X1 toward X2 to the projection \( \mathbf{b} \) onto the X1-X2 plane.

We then define a borehole coordinate system \( \mathbf{\hat{x}}_1, \mathbf{\hat{x}}_2, \mathbf{\hat{x}}_3 \) where \( \mathbf{\hat{x}}_1 = \mathbf{\hat{b}} \). We can choose \( \mathbf{\hat{x}}_2 \) to lie in the X1-X2 plane. To relate the two coordinate systems, we define a coordinate transformation matrix \([B]\) such that

\[
{\mathbf{\hat{x}}} = [B] \{\mathbf{x}\}
\]

Specifically

\[
b_{11} = \cos(\theta)\cos(\varphi)
\]

\[
b_{12} = \cos(\theta)\sin(\varphi)
\]

\[
b_{13} = \sin(\varphi)
\]

The \( b_{2i} \) terms are found as follows:

\[
b_{23} = 0,
\]

since \( \mathbf{\hat{x}}_2 \) lies in the X1-X2 plane. Then \( b_{21} \) and \( b_{22} \) are found using

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\( \hat{x}_1 \cdot \hat{x}_2 = 0 \)

and

\[ | \hat{x}_1 \times \hat{x}_2 | = 1 \]

The absolute value results in an arbitrary sign, which is chosen such that the projection of \( \hat{x}_2 \) along the \( x_2 \) axis is positive.

Finally, the \( b_{3i} \) terms are found using the cross product of \( \hat{x}_1 \) and \( \hat{x}_2 \):

\[ \hat{x}_3 = \hat{x}_1 \times \hat{x}_2 \]

Now, the three by three stress tensor \([\sigma_b]\) in the borehole coordinate system is given by

\[
[\sigma_b] = [B] \begin{bmatrix} S_1 & S_2 \\ S_3 & \end{bmatrix} [B]^T
\]

The cylindrical borehole produces a stress concentration in the medium. Along the borehole wall the three stresses of interest are \( \sigma_{\theta \theta} \), \( \sigma_{XX} \), and \( \sigma_{\theta X} \), where \( \sigma_{\theta \theta} \) is the hoop stress, \( \sigma_{XX} \) is the normal stress in the local \( \hat{x}_1 \) direction, and \( \sigma_{\theta X} \) is the shear stress. After Daneshy (1973), with changes due to a different definition of coordinate directions:

\[
\sigma_{\theta \theta} = \sigma_{22} + \sigma_{33} - 2(\sigma_{22} - \sigma_{33})\cos\theta - 4\sigma_{23}\sin\theta
\]

\[
\sigma_{XX} = \sigma_{11} - 2\nu (\sigma_{22} - \sigma_{33})\cos\theta - 4\nu \sigma_{23}\sin\theta
\]

\[
\sigma_{\theta X} = 2(\sigma_{13}\cos\theta - \sigma_{12}\sin\theta)
\]

where \( \sigma_{ij} \) refers to stresses in the borehole coordinate system. The value of Poisson's ratio \( \nu \) is taken to be 0.25.

These stresses act in the plane tangent to the borehole at the angle \( \theta \), measured toward \( \hat{x}_3 \) from \( \hat{x}_2 \). The maximum tensile stress in that plane is

\[
\sigma_p(\theta) = \frac{1}{2}[\sigma_{\theta \theta} + \sigma_{XX} + \sqrt{(\sigma_{\theta \theta} - \sigma_{XX})^2 + 4\sigma_{\theta X}^2}]
\]
which is still a function of \( \Theta \).

The maximum value of \( \sigma_p(\Theta) \) is found by iteratively sweeping through \( \Theta \) space. There are two values of \( \Theta \) between 0 and 360° where \( \sigma_p(\Theta) \) obtains its maximum value. The loci of points along the borehole axis where \( \sigma_p(\Theta) \) is a maximum forms two lines 180° apart on either side of the borehole. The fracture initiates parallel to the borehole axis along these lines.

The direction from the borehole axis to the point in the \( \hat{X}_2-\hat{X}_3 \) plane where the fracture initiates is given by

\[
\vec{f} = \hat{X}_2 \cos \Theta + \hat{X}_3 \sin \Theta
\]

The normal to the fracture \( \vec{n} \) is

\[
\vec{n} = \hat{X}_1 \times \vec{f}
\]

Now, the cosine of the angle between \( \vec{n} \) and the least compressive principal stress direction \( \hat{X}_3 \) is

\[
\cos (\psi) = \vec{n} \cdot \hat{X}_3
\]

for which the angle \( \psi \) is given by

\[
\psi = \cos^{-1} (\vec{n} \cdot \hat{X}_3)
\]

The value of \( \psi \) is chosen to lie within \( 0 \leq \psi \leq 90^\circ \), and is the error in inferring the fracture direction to be the \( X_3 \) direction.
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Fairhurst, C., Methods of determining in-situ rock stresses at great depths, TR1-68, Missouri River Div., Corps of Engineers, 1968.


Figure 1: Geometry of the borehole and induced hydraulic fracture for the case where one of the principal stresses aligns with the borehole. Equations relating breakdown and shut-in pressures to principal stresses, pore pressure and tensile strength are from Hubbert and Willis (1957).

Figure 2: Geometry of a borehole inclined with respect to the S1, S2, and S3 axes. The orientation of the borehole is specified by the angles $\phi$, $\theta$. The hydraulic fracture initiates parallel to the borehole and has a normal $n$. The angle $\psi$ is the error in degrees between the $n$ and S3 directions.

Figure 3a: Stereographic projection of the first quadrant of S1-S2-S3 space giving the orientation of the borehole and the error $\psi$. Numbers on the axes give the angle $\phi$ (see Figure 2). The angle $\theta$ varies from 0° at S1 to 90° at S2. The horizontal axis refers to borehole locations in the S1-S3 plane, while the vertical axis refers to the S2-S3 plane. The error $\psi$ is contoured for values of 20, 40, 60, and 80° for S1, S2, S3 = -1000, -100, 0.

3b: As above for S1, S2, S3 = -1000, -100, -98.

3c: Profile of $\psi$ versus $\phi$ for $\theta = 90^\circ$ (borehole located in the S2-S3 plane) for S1, S2, S3 = -1000, -100, 0. The error $\psi$ is equal to the angle $\phi$ for all choices of S1, S2, S3 for boreholes located in the S2-S3 plane.

3d: Profile of $\psi$ versus $\phi$ for $\theta = 0^\circ$ (boreholes located in the S1-S3 plane) for S1, S2 = -1000, -100. Curves labeled 1-5 correspond to S3 = 0, -30, -60, -80, -99, respectively.

3e: Profile of $\psi$ versus $\phi$ for $\theta = 8^\circ$. S1, S2 = -1000, -100, and S3 = 0, -95, -97, -98, -99 correspond to curves labeled 1-5, respectively.
Figure 3f: Profile of $\psi$ versus $\theta$ for $\theta = 0^\circ$ (boreholes located in the S1-S2 plane). S1, S2 = -1000, -100 and S3 = 0, -96, -97, -98, -99 denoted by curves 1-5, respectively.

Figure 4a: Contours of the error $\psi$ for S1, S2, S3 = -1000, -250, 0. For other details, see Figure 3a.

4b: Contours of the error $\psi$ for S1, S2, S3 = -1000, -250, -240.

4c: Contours of the error $\psi$ for S1, S2, S3 = -1000, -250, -249.

4d: Profile of $\psi$ versus $\theta$ for $\theta = 20^\circ$ for S1, S2 = -1000, -250 and S3 = 0, -215, -230, -240, -249 denoted by curves labeled 1-5, respectively.

Figure 5a: Contours of the error $\psi$ for S1, S2, S3 = -1000, -500, 0. For other details, see Figure 3a.

5b: Contours of the error $\psi$ for S1, S2, S3 = -1000, -500, -460.

5c: Contours of the error $\psi$ for S1, S2, S3 = -1000, -500, -490.

5d: Profile of $\psi$ versus $\theta$ for $\theta = 20^\circ$ with S1, S2 = -1000, -500, and S3 = 0, -400, -420, -460, -480 denoted by curves labeled 1-5, respectively.

Figure 6a: Contours of the error $\psi$ for S1, S2, S3 = -1000, -750, 0. For other details, see Figure 3a.

6b: Contours of the error $\psi$ for S1, S2, S3 = -1000, -750, -725.

6c: Contours of the error $\psi$ for S1, S2, S3 = -1000, -750, -745.

6d: Profile of $\psi$ versus $\theta$ for $\theta = 20^\circ$ with S1, S2 = -1000, -750 and S3 = 0, -700, -725, -740, -749 denoted by curves labeled 1-5, respectively.

Figure 7a: Contours of the error $\psi$ for S1, S2, S3 = -1000, -900, 0. For other details, see Figure 3a.

7b: Contours of the error $\psi$ for S1, S2, S3 = -1000, -900, -800.

7c: Contours of the error $\psi$ for S1, S2, S3 = -1000, -900, -895.
Figure 7d: Profile of $\psi$ versus $\phi$ for $\theta = 20^\circ$ with $S_1, S_2 = -1000, -900$ and $S_3 = 0, -800, -880, -895, -899$ denoted by curves labeled 1-5, respectively.
\[ P_b = 3S_{\text{min}} - S_{\text{max}} - P_0 + T \]

\[ P_s = S_{\text{min}} \]
\[ S_1, S_2, S_3 = -1000, -100, 0 \]

\[ S_1, S_2, S_3 = -1000, -100, -S \]

\[ S_1 = -1000 \\
S_2 = -100 \\
S_3 = 0 \\
\theta = 90 \]

\[ S_1 = -1000 \\
S_2 = -100 \\
\theta = 0 \]

Figure 3
Figure 3
$S_1, S_2, S_3 = -1000, -250, 0$

$S_1, S_2, S_3 = -1000, -250, -249$

$S_1 = -1000$
$S_2 = -250$
$\theta = 20$

Figure 4
\[ S1, S2, S3 = -1000, -500, 0 \]

\[ S1, S2, S3 = -1000, -580, -46 \]

\[ S1, S2, S3 = -1000, -580, -490 \]

\[ S1 = -1000 \]
\[ S2 = -500 \]
\[ \theta = 20 \]
$S_1, S_2, S_3 = -1000, -900, 0$

$S_1, S_2, S_3 = -1000, -900, -895$

$S_1 = -1000$
$S_2 = -900$
$\theta = 20$

\begin{figure}
\centering
\includegraphics[width=\textwidth]{figure7}
\caption{Figure 7}
\end{figure}
FACTORS INFLUENCING THE INITIATION ORIENTATION OF HYDRAULICALLY INDUCED FRACTURES

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ABSTRACT

This paper discusses the factors which may affect the initiation orientation of a hydraulically induced fracture in an assumed homogeneous medium. The influence of the packers upon the induced stress field in the neighborhood of the borehole is first critically reviewed. It is shown that the use of soft inflatable packers influences the fracture initiation process. Even at very shallow depths, longitudinal hydraulic fractures are generally induced. Field evidence is also presented to prove that changing the pressurization rate may create multiple fracture traces at the borehole wall. Such a technique is proposed to determine the complete stress tensor at shallow depths.

INTRODUCTION

When creating a hydraulic fracture, the stress conditions prevailing around the pressurized borehole, prior to fracture initiation, are a combination of:

A. the pre-existing in situ stress field, the borehole acting as a stress concentrator;

B. the pressure acting on the borehole wall, within the sealed-off region; and

C. the stress exerted by the packers themselves.

This last factor has been often overlooked. However, it may play an important role and dictate the initiation orientation which does not necessarily agree with that for the far-field stress component. The implication is that the trace of the fracture on the borehole wall does not always reflect the in situ, stress-tensor attitude. This will be discussed in the next section.

Moreover, once a fracture has been induced, an "opening" has been created which will "tolerate" a maximum amount of fluid flow. When this critical pumping rate is exceeded, additional fractures may be generated which are not necessarily in the same direction/orientation as the original one(s).

BACKGROUND

In interpreting hydraulic fracturing data for stress determination, a few basic assumptions are usually required. First, it is assumed that the instantaneous shut-in pressure is equal to or slightly above the in situ minimum principal stress component. The validity of this assumption is addressed in detail in another paper (McLennan and Roegiers, 1981). Second, the relationship between the breakdown pressure and the stress concentration
existing around the borehole is assumed to be known. As has been pointed out by Haimson (1968), this value is dependent on the fluid penetration and, consequently, on the fluid viscosity and the pumping rate. In addition, if one takes into consideration the observed fact that permeability is dependent on the stress field (Roegiers, 1974) it can be concluded that determination of the "exact" stress concentration is rather complex. Some researchers such as Ishijima (1980) even question the fact that peak pressure represents the initial breakdown. Ishijima's evidence is based not only on microseismic activity prior to peak load but also on microscopic observations of thin sections, suggesting significant degradation prior to "unstable breakdown." The breakdown pressure value also contains an "apparent rock strength" term which, if determined by laboratory burst tests, often leads to interpretation problems. This has been resolved by carrying out a repressurization cycle(s) (Bredheoef et al., 1976; Zoback and Pollard, 1978; Gronseth and Detournay, 1979). However, it should be pointed out that all cycles must be performed under similar conditions of pumping rate and viscosity, otherwise the stress concentrations may be different. This requires stringent flow-rate control, a condition all too often overlooked. The difficulties in selecting the representative breakdown pressure value have led to various proposals such as minimum secondary breakdown (Zoback and Pollard, 1978; Gronseth and Detournay, 1979), none of which has been fully proven to lead to better estimates. A third assumption is that the fracture trace at the borehole wall represents the direction of the maximum principal stress as well as the far-field fracture orientation. This last assumption is addressed in the next section and some pertinent field data are presented.

INFLUENCE OF PACKER TYPE AND CONFIGURATION

In 1964, Kehle examined the influence of rigid packers on the stress distribution around a borehole and, by assuming the packers were held in place by a uniform shear stress, Kehle showed that a region of longitudinal tension was induced near the end of the sealed-off interval (refer to Figure 1). Subsequently, Haimson (1968) performed a series of laboratory tests and concluded that hydraulic fractures initiated perpendicularly to the direction of the least compressive stress and asserted that this technique could therefore give a good approximation of in situ stresses as well as their orientation. Nevertheless, Haimson stated that it was virtually impossible to induce fractures normal to the wellbore even under the most favorable loading conditions when rubber packers were used. The conclusion was that rubber sealing elements -- because of their incompressibility -- exerted a radial stress against the borehole wall which was not included in Kehle's original analysis. Two years later, von Schoenfeldt (1970) suggested that the applied radial stress, exerted by the inflatable rubber sealing elements on the borehole wall, reduced the longitudinal tensile stress acting near the ends of the pressurized interval (refer to Figure 2). Roegiers et al. (1973), using finite element analyses, investigated the influence of the straddle packer stiffness on the stress distribution around a borehole during a hydraulic fracturing test. Their results showed that, for a constant pressure in the packers:

A. the location of the maximum circumferential stress shifted from under the sealing elements to the central region of the sealed-off
section when the borehole pressure exceeded one-half of the packer pressure (refer to Figure 3);

B. the use of flexible packers of small thickness enhanced the chances of inducing a "vertical" fracture (refer to Figure 4); and

C. any longitudinal packer movement (such as slippage along an inner mandrel) definitely reduced the longitudinal stress concentration and hence reduced the likelihood of initiating a "horizontal" fracture (see Figure 5).

Finally, Gronseth and Detournay (1979), using measured values for interval and packer pressures, showed that the maximum circumferential stress always occurred under the sealing elements (see Figure 6) questioning even the supposition that vertical fractures initiate within the sealed-off region.

There is also strong evidence from field experiments that "horizontal" fractures are relatively rare (Rummel, 1981). A large number of experiments in West Germany at depths as shallow as 15 m clearly revealed vertical traces at the borehole wall. However, the packers used in this case -- developed at the Ruhr-Universitaet -- were extremely soft and exhibited very large longitudinal movements as pressurization proceeded. From pressure data analyses, there is little doubt that the fracture orientation changed once the fracture grew further away from the borehole's influence (refer to Figure 6). However, the field data are not always as conclusive at slightly greater depths, especially if one considers the difficulties associated with shut-in pressure interpretations.

INFLUENCE OF PUMPING RATE AND VISCOSITY

When attempting to determine the stress field at shallow depth, the creation of a horizontal fracture has been considered as completely undesirable as it only provides information relative to the vertical stress component. As explained previously, the use of "sliding" inflatable packers may alleviate this problem although in some situations, where the rock is highly anisotropic, horizontal fractures will be induced regardless of the packer type. If this is the case, the only parameters available to overcome this structural control are pumping rate and fluid viscosity.

Assuming a horizontal fracture has been created via a typical micro-fracturing operation and that all pertinent data have been obtained (i.e., repeatability of pressure vs time curves has been assured), pumping at higher rates and with more viscous fluids will cause the pressure to increase in the sealed-off interval.† Referring to the situation shown schematically in Figure 7, one can recognize that the amount of fluid pumped between the packers must pass through a slot of limited area (crack opening x borehole circumference). As pressure increases, the width of the fracture will increase but not in direct proportion to the change in flow rate

†Even if correction has been made for tubular friction.
As a result there exists a practical limitation to the amount of fluid a horizontal fracture can accommodate. Figure 8 illustrates this concept for the flow of water through an orifice. As can be seen, a substantial pressure differential across the fracture opening can be obtained provided the discharge rate can be sufficiently varied. However, it should be pointed out that the curves plotted on this figure correspond to a fixed width. In reality, the fracture width is variable and the behavior will more likely be similar to the heavy line in this figure.

Fluid viscosity will also contribute to this pressure increase as it will enhance the pressure drop across the fracture entrance.

If the pumping rate or viscosity becomes adequately large, the pressure between the packers will increase to a level where there are two possibilities:

A. a second horizontal fracture is induced within the pressurized horizon, or

B. a new vertical fracture will be created once the pressure in the interval reaches a value sufficient to create a longitudinal fracture.

In either case, the originally induced fracture is likely to extend to some degree.

In order to avoid the first mechanism, the straddled section should be kept as small as possible such that the pressures generated in the propagating horizontal fractures add a "confining effect" in the longitudinal direction, prohibiting the formation of other horizontal features. Such an approach has recently been successful in field applications.

REFERENCES


Figure 1. Stress Distribution after Kehle (1964).
Figure 2. Stress Distribution after von Schoenfeldt (1970).
Case V

\[ \sigma_\theta = f(z) \]

Figure 3. Circumferential Stress History after Roegiers et al. (1973).
Case I & II

$\sigma_z$ – Stress Distribution
(At the distance of 0.1" from the surface)
- Steel Packer
- Rubber Packer

Figure 4. Longitudinal Stress Distribution for Rigid and Soft Packers (Roegiers et al., 1973).
<table>
<thead>
<tr>
<th>Case</th>
<th>Sealing Element Pressure, $P_p$, psi</th>
<th>Borehole Pressure $P_b$, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>3200</td>
<td>1000</td>
</tr>
<tr>
<td>B</td>
<td>3700</td>
<td>2000</td>
</tr>
<tr>
<td>C</td>
<td>4600</td>
<td>3000</td>
</tr>
</tbody>
</table>

Figure 5. Circumferential Stress History after Gronseth et al. (1979).
Figure 6. Influence of Injection Rate on Pressure Time Behavior following Shut-in (a Field Example).
Figure 7. Schematic Representation of Entrance Limitations.
Figure 8. Pressure Drop across a Rectangular Orifice.
HYDRAULIC FRACTURE INITIATION RELATED TO THE STRESS FIELD AT FAILURE

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ABSTRACT

An analysis is made of the state of stress around a borehole at failure during hydraulic fracturing treatments. The analysis utilizes the Mohr-Coulomb failure criterion combined with equations of equilibrium. Equations are derived for predicting the bottomhole fracture initiation pressure, $P_i$, for two different stress state configurations resulting in vertical fractures. Predictions of $P_i$ from these equations are compared to actual field results from 27 fracturing treatments showing very good correlations. Computation of the tangential stress at the wall of the borehole at failure yields compressive stresses for all 27 treatments. This indicates that fracture initiation may be attributed to shear failure and not tensile failure.

INTRODUCTION

When the fluid pressure in a borehole becomes sufficiently high, cracks in the surrounding formation are initiated and propagate away from the well in nearly vertical or horizontal planes. This process is known as hydraulic fracturing or hydrofracturing.

Knowledge of the magnitude of fluid pressure necessary for hydrofracture is important in operations where such fractures are intentionally induced. Such as in oil and gas well stimulation. The hydrofracture pressure is also important during drilling operations for oil and gas exploration. In this instance, unintentional hydrofracturing can cause a loss of circulation and a drop in the borehole fluid level. Therefore it is desirable to prevent such a dangerous occurrence by maintaining the borehole fluid pressure at a level which prevents fracture of the surrounding material. To do this, the pressure necessary to fracture the formation must be constantly known or predicted for the entire uncased length of the borehole. Although theories concerning hydrofracture pressure are numerous, accurate predictions of hydrofracture pressures during drilling are rare because of a lack of knowledge of the material parameters as well as a lack of recognition of the basic mechanisms involved.

Many authors have attempted to address the mechanics of hydraulic fracturing. In 1957 Hubbert and Willis (3) used elastic theory to describe the state of stress around a borehole prior to hydraulic fracturing. They then bypass this analysis to formulate a simple equation for predicting fracture propagation pressure using a non-penetrating fracture fluid. In their analysis, Hubbert and Willis assume that the formation exhibits no tensile strength.

Scheidegger (6), in 1962, continued with the elastic approach and derived equations for wellbore fracture initiation pressures for penetrating and nonpenetrating fluids. In these equations a formation tensile strength can be considered.
In 1967 Haimson and Fairhurst (2) considered hydrofracture as a two-dimensional problem and assumed the formation to be elastic, porous, isotropic and homogeneous. They combined equations of elasticity with equations of thermoelasticity to derive an expression for fracture initiation pressure for penetrating fluids.

All of these authors assumed that fracture initiation is the result of a tensile failure. However, failure at the wall of a borehole can be initiated when all three principal stresses are compressive. It is the relative magnitude of the principal stresses which can become important in predicting failure. One failure criterion which considers this is the Mohr-Coulomb failure criterion.

The purpose of the following derivation is to combine equations of equilibrium and the Mohr-Coulomb failure criterion to relate the existing stress field in the earth to hydrofracture initiation pressures.

**IMPORTANCE OF THE STRESS FIELD IN FRACTURE INITIATION AND EXTENSION**

It is the existing stress field in a formation which most affects the behavior of soil or rock during hydrofracturing processes. Assuming the formation to be homogeneous and isotropic, the orientation of fractures is dependent upon the direction and relative magnitudes of the three principal stresses in the earth's crust.

Fracture propagation is usually associated with tensile failure where fluid injected into created fractures induces stresses on the fracture walls. These stresses tend to push the walls apart creating a wedging effect for fracture propagation. Such fractures are oriented in planes perpendicular to the least principal stress. However, the initiation of failure around the borehole prior to fluid migration into the formation may be attributed to shear failure. This failure can occur while tangential stresses around the borehole are still highly compressive. The resulting failure planes or fractures are oriented as shown in Figure 1 with respect to the principal stress field. Once substantial fluid penetration into the failure planes occurs during fracture propagation, the tensile mode of failure may then prevail.
EQUILIBRIUM AND THE MOHR-COULOMB FAILURE CRITERION

Failure initially occurs if the fluid pressure in the borehole allows the stresses in the formation surrounding the hole to meet a failure criterion. For a simplified failure criterion many previous authors (2, 3, 6) have assumed that failure occurs when tensile stresses greater than the tensile strength of the formation are experienced.

Several authors have used the Mohr-Coulomb failure criterion to describe failure conditions in circular shafts. This criterion can describe a mode of failure regardless of whether the normal stresses are compressive or tensile. The Mohr-Coulomb failure criterion dictates that failure occurs when the relative magnitudes of $\sigma_{11}$ and $\sigma_{33}$ are such that the existing Mohr circle, shown in Figure 2, contacts the Mohr envelope. The Mohr envelope is generally described by the slope, $\phi$, known as the angle of internal friction, and the intercept, $c$, which is the apparent cohesion exhibited by the material. The material parameters $\phi$ and $c$ are determined from laboratory tri-axial tests.
If the pressure in the borehole is such that the failure criterion is met then a failure zone or plastic zone of radius \( b \) develops around the borehole as illustrated in Figure 3. Beyond the plastic zone is an elastic zone in which the radial and tangential stresses approach the predrilled stresses when the radius, \( r \), approaches infinity. The distance, \( b \), depends upon the magnitude of the borehole pressure.

Morgenstern (5) used the Mohr-Coulomb failure criterion to formulate equations describing hydrofracture initiation pressures. In his formulation, however, he ignores equilibrium.

Jumikis (4) used equilibrium and the Mohr-Coulomb failure criterion to describe stresses in the plastic and elastic zones around a circular shaft in rock. Westergaard (10) used the same approach to study instability around a borehole in an attempt to determine the conditions necessary for an open borehole to remain stable. Stability in open boreholes is attributed to arching of the surrounding material. Both authors assumed a spherical predrilled stress state (i.e., \( \sigma_{11} = \sigma_{22} = \sigma_{33} \)).
The same approach of combining equations of equilibrium with the Mohr-Coulomb failure criterion will be used in the following sections to derive equations for the hydrofracture initiation pressure (1).

ASSUMPTIONS IN FORMULATING THE PROBLEM

To simplify this analysis the formation is considered to be homogeneous and isotropic and the fluid in the borehole is assumed to be nonpenetrating. The analysis employs an axisymmetric approach in which the borehole is the axis of symmetry. All equations involving stress are written in terms of effective stress, $\sigma'$, as defined below (7):

$$\sigma' = \sigma - u \quad \text{...............(1)}$$

The principal effective stresses are assumed to occur in the vertical radial and tangential directions around the borehole. The vertical effective stress, $\sigma'_z$, is assumed to be constant for a given depth and computed as follows:

$$\sigma'_z = \gamma_z - u \quad \text{...............(2)}$$
Geostatic conditions are assumed which implies that the predrilled horizontal effective stresses are equal in every direction to \( K_o \sigma_z' \).

\[
K_o = \frac{\sigma_h' - u}{\sigma_z' - u}
\]

By definition of principal stresses, no shear stresses exist on planes perpendicular to the directions of \( \sigma_1' \), \( \sigma_2' \), and \( \sigma_3' \).

For the purpose of this paper, two configurations of the principal effective stresses at failure will be considered. These two configurations are shown in Table 1 and correspond to two cases in which vertical fractures are initiated.

Depending upon the stress state, different forms of the failure criterion must be considered in conjunction with equilibrium. As shown by Tschebotarioff (9), the general form of the Mohr-Coulomb failure criterion can be written in terms of effective stresses as:

\[
\sigma_{11}' - \lambda \sigma_{33}' = \beta
\]

where:

\[
\lambda = \frac{1 + \sin \phi}{1 - \sin \phi} = \tan^2 (45 + \frac{\phi}{2})
\]

\[
\beta = \frac{2c \cos \phi}{1 - \sin \phi} = 2c \tan (45 + \frac{\phi}{2})
\]

Shown in Table 1 is the failure criterion which applies to each of the two stress states considered. The resulting orientations of the failure planes are also shown in Table 1.
TABLE 1
Configuration of Principal Effective Stresses at Failure

<table>
<thead>
<tr>
<th>Case</th>
<th>( \sigma'_{11} )</th>
<th>( \sigma'_{33} )</th>
<th>( \sigma'_{22} )</th>
<th>Failure Criterion</th>
<th>Orientation of Failure Planes</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>( \sigma'_{11} )</td>
<td>( \sigma'_{33} )</td>
<td>( \sigma'_{22} )</td>
<td>( \sigma'<em>{r} - \lambda \sigma'</em>{\theta} \geq \beta )</td>
<td>( \theta )</td>
</tr>
<tr>
<td>II</td>
<td>( \sigma'_{22} )</td>
<td>( \sigma'_{33} )</td>
<td>( \sigma'_{11} )</td>
<td>( \sigma'<em>{z} - \lambda \sigma'</em>{\theta} \geq \beta )</td>
<td>( \theta )</td>
</tr>
</tbody>
</table>

DERIVATION OF EQUATIONS DESCRIBING FAILURE AROUND A BOREHOLE

The equation of static equilibrium for the two-dimensional axisymmetric case in terms of effective stresses is:

\[
\frac{\partial \sigma'_{r}}{\partial r} + \frac{\sigma'_{r} - \sigma'_{\theta}}{r} = 0 \quad \text{.................................(5)}
\]

where compression is positive. The following general solutions for radial and tangential stresses in the elastic zone satisfying Equation 5 are given by Timoshenko and Goodier (8) and are attributed to Lamé:

\[
\sigma'_{re} = B - \frac{D}{r^2} \quad \text{.................................(6)}
\]

\[
\sigma'_{\theta e} = B + \frac{D}{r^2} \quad \text{.................................(7)}
\]
where B and D are constants to be determined from boundary conditions.

The constant B is evaluated from the condition that $\sigma'_r$ and $\sigma'_\theta$ approach the predrilled horizontal effective stress, $K_o'\sigma_z^{e1}$, as the radius, $r$, approaches infinity so that Equations 6 and 7 can be written as:

$$\sigma'_r = K_o'\sigma_z^{e1} - \frac{D}{r^2} \quad \text{(8)}$$

$$\sigma'_\theta = K_o'\sigma_z^{e1} + \frac{D}{r^2} \quad \text{(9)}$$

Since the failure criterion is not involved in Equations 8 and 9 these relationships are identical for both cases I and II. However the constant D will differ for each case.

For case I the principal effective stress configuration dictates that the failure criterion be written as:

$$\sigma'_r - \lambda\sigma'_\theta = \beta \quad \text{(10)}$$

where $\sigma'_r = \sigma'_{11}$ and $\sigma'_\theta = \sigma'_{33}$

For the plastic zone, Equations 5 and 10 are combined yielding the following first order differential equation:

$$\frac{\partial \sigma'_r}{\partial r} + \frac{(\lambda - 1)\sigma'_r}{\lambda r} = -\frac{\beta}{\lambda r} \quad \text{(11)}$$

This differential equation and its solution differ for each case because the failure criterion differs for each case. The solution to Equation 11 results in the following equations for the radial and tangential stresses in the plastic zone:

$$\sigma'_r = Ar \left( \frac{1}{\lambda} - 1 \right) + \frac{\beta}{1 - \lambda} \quad \text{(12)}$$

$$\sigma'_\theta = \frac{1}{\lambda}Ar \left( \frac{1}{\lambda} - 1 \right) + \frac{\beta}{1 - \lambda} \quad \text{(13)}$$

where A is a constant to be determined from boundary conditions.
The constants $A$ and $D$ are evaluated by equating plastic and elastic stresses at the plastic-elastic boundary in the following manner:

\[ \text{at } r = b \]

\[ \sigma_{rp}^* = \sigma_{re}^* \quad \text{(14)} \]

and

\[ \sigma_{\theta p}^* = \sigma_{\theta e}^* \quad \text{(15)} \]

which results in two equations with two unknowns, $A$ and $D$. For case I the resulting expressions for $A$ and $D$ in terms of $b$ are as follows:

\[ A = 2 \left[ \frac{K \sigma_t' + \frac{\beta}{\lambda - 1}}{(1 + \frac{1}{\lambda}) b} \right] \quad \text{(16)} \]

\[ D = \left[ \frac{(1 - \frac{1}{\lambda}) K \sigma_t' - \frac{\beta}{\lambda}}{1 + \frac{1}{\lambda}} \right] b^2 \quad \text{(17)} \]

If the pressure in the borehole, $P_w$, follows the effective stress concept such that

\[ P_w = p + u \quad \text{(18)} \]

where $p$ is the portion of borehole pressure transmitted to the soil matrix, then the radius of the plastic zone, $b$, is found by letting $\sigma_{rp}^* = p$, for $r = a$, and then solving for $b$, yielding:

\[ b = a \left[ \frac{2 \left( \frac{K \sigma_t' + \frac{\beta}{\lambda - 1}}{1 + \frac{1}{\lambda}} \right)}{p + \frac{\beta}{1 - \lambda}} \right] \frac{\beta}{1 - \lambda} \quad \text{(19)} \]

The limiting effective failure pressure, $p_f$, for which failure begins at the wall of the borehole, is determined by letting $b = a$ in Equation 19 and then solving for $p$. This yields:

Case I:

\[ p = p_f = \frac{2\lambda K \sigma_t' + \beta}{1 + \lambda} \quad \text{(20)} \]
This expression also can be obtained by letting $p_f = \sigma_{te}^r$ when $\sigma_{te}^r$ and $\sigma_{0}^r$ satisfy the failure criterion for $r = a$. This same derivation is made for case II, yielding the following expression for $p_f$ (1):

**Case II:**

$$p_f = \frac{(2\lambda K - 1) \sigma_z^t + \beta}{\lambda} \quad \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots (21)$$

The limiting borehole pressure, $P_i$, for which failure or fractures are initiated, is written as:

$$P_i = p_f + u \quad \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots (22)$$

All resulting equations derived for cases I and II are tabulated in Appendix A.

**APPLICATION TO FIELD DATA**

In order to check the reliability of Equations 20 and 21 for predicting fracture initiation pressure, results of 27 stimulation treatments in the Austin Chalk and Wilcox formations in southern Texas are analyzed. These results consist of job descriptions as well as surface treating pressure and rate logs for each treatment. The initial portion of a typical surface treating pressure log is shown in Figure 4. The breakdown (fracture initiation) pressure seen at the surface, $P_b$, and the instantaneous shut-in pressure, ISIP, are labelled.

![Typical Surface Treating Pressure Log](image-url)
The actual bottomhole pressure, \( P_{\text{act}} \), observed at breakdown is calculated as follows:

\[
P_{\text{act}} = P_b + P_h - P_p - P_{pf} \tag{23}
\]

where
- \( P_h \) = hydrostatic pressure in the well
- \( P_p \) = friction pressure down the tubing
- \( P_{pf} \) = friction pressure through the perforations

Values for \( P_h \), \( P_p \), and \( P_{pf} \) also are obtained from the treatment records.

Due to a lack of triaxial test results on these two formations certain assumptions were required. The Austin Chalk is a relatively weak, naturally fractured chalk formation and the Wilcox is a relatively well consolidated sandstone. The following values for the material parameters \( \phi \) and \( c \) were assumed for these two formations:

<table>
<thead>
<tr>
<th></th>
<th>Austin Chalk</th>
<th>Wilcox</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \phi )</td>
<td>30°</td>
<td>30°</td>
</tr>
<tr>
<td>( c )</td>
<td>0</td>
<td>500 psi</td>
</tr>
</tbody>
</table>

The overburden pressure, \( \sigma_z \), was assumed to vary as 1 psi/ft. The reservoir pressure or pore pressure, \( u \), was also obtained from the treatment records.

The value of \( K \) was calculated from Equation 3 using the instantaneous shut-in pressure, ISIP, to calculate \( \sigma_h \) as follows:

\[
\sigma_h = ISIP + P_h \tag{24}
\]

The ISIP was always taken shortly after breakdown which is the only way in which the ISIP can be used in determining \( \sigma_h \) with any validity. Once a significant fracture width has been created and the adjacent formation rock has been compressed the ISIP can yield a much higher value for \( \sigma_h \) than actually exists.

All of the data for the 27 treatments in the Austin Chalk and the Wilcox are tabulated in Appendix B. This data was used along with the above equations to predict values of \( P_i \) for all 27 treatments. These results along with \( P_{\text{act}} \) are shown for the Austin Chalk data in Table 2 and for the Wilcox data in Table 3. In order to determine which case applies to each treatment the tangential stress at failure, \( \sigma_{0p} \), was calculated at \( r = a \). By comparing the relative magnitude of this value to \( P_i \) and \( \sigma_z \), the applicable case was determined. It is important to mention at this point that the tangential effective stress at the wall of the borehole at failure was calculated to be compressive for all 27 data sets.
As a means for comparison two other models from previously men­tioned authors were used to predict \( P_i \). First, Scheidegger's equation for non-penetrating fluids was used as follows:

\[
P_i = 3\sigma_{33} - \sigma_{22} - u
\]

It was assumed that \( \sigma_{22} = \sigma_{33} = \sigma_h \) for this equation. Second, the following equation proposed by Morgenstern was used:

\[
P_i = \frac{\sigma'}{2} \left[ \frac{(1 + K_o)}{2} - \frac{\sigma'}{2} \left( 1 - K_o \right) \right] - \frac{c}{2 \sin \phi} + c \cot \phi + u
\]

These results are also shown in Tables 2 and 3.

The results for the Austin Chalk show that values for \( P_i \) calculated from Equations 20 and 21 seem to match the values of \( P_{act} \) fairly well. Scheidegger's equation on the other hand yields values for \( P_i \) which are significantly higher than \( P_{act} \) for most of the treatments. In contrast, predicted values of \( P_i \) from Morgenstern's equation are much lower than \( P_{act} \) for every treatment. The results for Equations 20 and 21 in the Wilcox seem to exhibit a bit more scatter about the values of \( P_{act} \). Scheidegger's predictions are again consistently high yet Morgenstern's equation predicts much more accurate breakdown pressures in the Wilcox than in the Austin Chalk. Of course it must be recognized that if other values of \( \phi \) and \( c \) were assumed the results of such a comparison might vary.

In an effort to quantitatively compare the performance of these equations in predicting \( P_i \), the standard error of the predicted values, \( S_p^2 \), was calculated for each model in both formations as follows:

\[
S_p^2 = \sqrt{\frac{\sum (P_{act} - P_i)^2}{n-1}}
\]

These results which are also shown in Tables 2 and 3 show that Eqs. 20 and 21 yield much more accurate predictions of \( P_i \) than the other two models for the particular assumptions made for \( \phi \) and \( c \).
TABLE 2 - Fracture Initiation Pressures for the Austin Chalk

<table>
<thead>
<tr>
<th>Data Set</th>
<th>Depth ft</th>
<th>$P_{act}$ psi</th>
<th>$P_i$ psi</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Eq. 20 &amp; 21 (Case)</td>
</tr>
<tr>
<td>1</td>
<td>8576</td>
<td>5041</td>
<td>5379 (II)</td>
</tr>
<tr>
<td>2</td>
<td>9074</td>
<td>7098</td>
<td>5878 (II)</td>
</tr>
<tr>
<td>3</td>
<td>9084</td>
<td>6803</td>
<td>6691 (II)</td>
</tr>
<tr>
<td>4</td>
<td>9086</td>
<td>6395</td>
<td>5840 (II)</td>
</tr>
<tr>
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<td>9436</td>
<td>5961</td>
<td>6132 (II)</td>
</tr>
<tr>
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<td>9</td>
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<td>7147</td>
<td>7899 (II)</td>
</tr>
<tr>
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<td>10,820</td>
<td>7663</td>
<td>7455 (II)</td>
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\[
\frac{s^2}{P} = \begin{array}{c}
674 \\
2167 \\
2518
\end{array}
\]
## TABLE 3 - Fracture Initiation Pressures for the Wilcox Data

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<th>$P_i$ (psi)</th>
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</table>

$s_p^2 = \frac{827}{1657} = 0.5046$
CONCLUSIONS

The Mohr-Coulomb failure criterion was combined with equations of equilibrium to obtain equations for predicting failure initiation pressures for two configurations of the principal stresses at failure in which vertical fractures result. These equations were used to predict $P_I$ in analyzing the results of 27 stimulation treatments in which vertical fractures were expected. From this analysis the following conclusions can be drawn:

1. The initiation of hydraulic fractures can be the result of shear failure instead of tensile failure.
2. Values of $P_I$ predicted from Eqs. 20 and 21 correspond well with actual fracture initiation pressures observed in the Austin Chalk and Wilcox formations.
3. With the values assumed for $\phi$ and $c$, Eqs. 20 and 21 seem to be significantly more accurate in predicting $P_I$ than the other two models discussed.

NOMENCLATURE

- $a$: Radius of the borehole
- $b$: Radius of the plastic zone
- $c$: Apparent cohesion
- ISIP: Instantaneous shut-in pressure
- $K_0$: Lateral earth pressure coefficient at rest
- $n$: Number of observations
- $p$: Portion of borehole pressure transmitted to the formation matrix
- $P_{act}$: Actual bottomhole fracture initiation pressure
- $P_b$: Surface pressure at breakdown
- $P_f$: Predicted effective breakdown pressure
- $P_h$: Hydrostatic pressure in the borehole
- $P_I$: Predicted bottomhole fracture initiation pressure
- $P_p$: Friction pressure down the borehole
- $P_{pf}$: Friction pressure through the perforations
- $P_w$: Bottomhole pressure
- $r$: Radius
- $u$: Reservoir pressure, pore pressure
- $z$: Depth
- $\gamma_T$: Unit weight of the overburden
- $\theta$: Polar coordinate around the borehole
- $\sigma$: Total stress
- $\sigma'$: Effective stress
- $\sigma_{11}, \sigma_{22}, \sigma_{33}$: Major, intermediate, and minor principal stresses respectively
\( \sigma_h \) Total horizontal stress
\( \sigma_r \) Effective radial stress
\( \sigma_{re} \) Effective radial stress in the elastic zone
\( \sigma_{rp} \) Effective radial stress in the plastic zone
\( \sigma_z \) Total vertical stress
\( \sigma_t \) Effective tangential stress
\( \sigma_{te} \) Effective tangential stress in the elastic zone
\( \sigma_{tp} \) Effective tangential stress in the plastic zone
\( \phi \) Angle of internal friction

**ACKNOWLEDGEMENTS**

Appreciation is extended to the management of Dresser Titan Division, Dresser Industries for allowing the use of the field data presented in this manuscript. The author also gratefully acknowledges the contribution made by Steven R. Young in compiling the necessary data.
REFERENCES


APPENDIX A - Tabulation of Results Derived for Cases I and II

<table>
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<tr>
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<th>Case I</th>
<th>Case II</th>
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<tr>
<td>$\sigma'_{rp}$</td>
<td>$\frac{1}{\lambda} \left( \frac{1}{\lambda} - 1 \right) + \frac{\beta}{1-\lambda}$</td>
<td>$\frac{1}{\lambda} (\sigma'_{z} - \beta)$</td>
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<tr>
<td>$\sigma'_{dp}$</td>
<td>$\frac{1}{\lambda} \left[ \left( \frac{1}{\lambda} - 1 \right) + \frac{\beta}{1-\lambda} \right]$</td>
<td>$\frac{1}{\lambda} (\sigma'_{z} - \beta)$</td>
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<td>$2 \left[ \frac{K_o \sigma'_{z} + \frac{\beta}{\lambda-1}}{(1 + \frac{1}{\lambda}) b} \left( \frac{1}{\lambda} - 1 \right) \right]$</td>
<td>$2 \left[ \frac{(\lambda K_o - 1) \sigma'_{z} + \beta}{\lambda b} \right]$</td>
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<td>$D$</td>
<td>$\left[ \frac{(\frac{1}{\lambda} - 1)K_o \sigma'_{z} - \frac{\beta}{\lambda}}{1 + \frac{1}{\lambda}} \right] b^2$</td>
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<tr>
<td>$b$</td>
<td>$a \left[ \frac{2K_o \sigma'_{z} + \frac{\beta}{\lambda-1}}{1 + \frac{1}{\lambda}} \right] \frac{\lambda}{1-\lambda}$</td>
<td>$a \left[ \frac{(\lambda K_o - 1) \sigma'<em>{z} + \beta}{\lambda (\sigma'</em>{z} - \beta)} \right] \frac{p - \frac{1}{\lambda}}{\lambda}$</td>
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<td>$p_i$</td>
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APPENDIX B
Tabulation of Data for Fracture Initiation Pressure Calculations

Data for the Austin Chalk

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<th>Depth ft</th>
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Assumed values

- $\phi = 30^\circ \rightarrow \lambda = 3.0$
- $c = 0 \rightarrow \beta = 0$
- $\sigma_z = 1 \text{ psi/ft}$
Data for the Wilcox

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Assumed values $\phi = 30^\circ \rightarrow \lambda = 3.0$

$c = 500 \, \text{psi} \rightarrow \beta = 1732$
Abstract

LLNL has been studying the mechanics of hydraulic fracturing for enhanced gas recovery and is using a minifrac technique for stress measurements at coal mines in the Appalachian and Rocky Mountain regions. These will be used to help design fracturing techniques to recover gas from gassy coal beds.

Research to date indicates that fracture propagation is strongly influenced by existing stresses and that the stress gradient in a non-uniform stress field may stop or turn the fracture.

Field evidence and analyses show that anisotropic rock properties due to rock fabric are a major factor in fracture geometry and calculated stress values.

Fracture growth in non-uniform stress fields

Some aspects of hydraulic fracture propagation in a varying stress field have been theoretically analyzed. These calculations were performed in two-dimensions using modifications of the computer code developed previously. These calculations were completed for a geometry similar to a set of experiments reported below. The two-dimensional analyses were performed in plane strain for an isotropic-elastic material with a Young's modulus of 20 GPa and a Poisson's ratio of 0.25. A loading pattern was chosen so that a hydraulic fracture initiated in the center of the block would encounter an increasing stress along the projected fracture axis as it enlarged. To accomplish this, the block was uniformly loaded in the x-direction but was only loaded for a quarter of its length from each corner in the y-direction. The geometry of the loading is shown on Figure 1. The total load in each direction was equal. Stress conditions resulting from the load shown on Figure 1 are shown as contours on Figures 2 and 3.

Contours of equal stress in the y-direction are shown on Figure 2. The singularities in the loading along the y-faces of the block result in the very high gradients at a quarter of the block length from the corners along the y-faces. Stress parallel to the uniform loading (x-direction) is shown as contours on Figure 3. The discontinuous loading along the y-faces of the block obviously distorts the stresses in both directions. Note the x-y axis chosen is not the principal axis of the stress tensor. Contours of equal stress in the x-direction are shown on Figure 3. The stress state at the center of the block is fairly uniform but changes rapidly as we move away
from the center. Direction of the minimum principal stress is depicted by the short straight lines over the field as shown on Figure 4. We observe that the principal stress axis rotates as a result of the loading conditions on the y-faces of the block.

Further calculations were performed where a pressurized fracture was emplaced at the center of the block. Contours of equal stress in the y-direction for the loading condition shown on Figure 2 and with a pressurized crack located at the center of the block with its axis in the x-direction are shown on Figure 5. The pressurized crack is approximately 1/2 the length of the block. Figure 6 shows contours of the stress in the x-direction with the pressurized crack and the same boundary loading conditions. The latter figures show that the effect of the pressurized crack is localized near the crack. Orientations of the maximum principal axes of stress are nearly identical to those without the pressurized fracture as shown on Figure 4.

These results show that a unilaterally propagating pressurized fracture will move a short distance from the center along the x-axis and then undergo a rotation and propagate diagonally toward the loaded areas on the y-faces of the block. This is what was observed in the experimental results when the blocks were loaded in a similar manner. Correlation of these results with the experiments also confirms that the fracture propagates parallel to the maximum principal initial stress as we have shown in our previous research.²⁻⁵

A preliminary set of calculations were performed to analyze the stress field when an asymmetric load was applied to the block. The geometry of these calculations is shown on Figure 7. As seen from viewing along the borehole one set of parallel faces was uniformly loaded and the adjacent set of parallel faces was loaded along a strip whose width was 1/4 the width of the face. The calculations were performed in plane strain on a material whose Poisson's ratio was 0.25. Figure 8 shows contours of equal stress in the y-direction (direction of load on the non-uniformly loaded surfaces). Here we note the singularity due to the discontinuity of the load on the upper and lower faces. The load spreads toward the center of the block with the maximum stress contour directly below the load singularity. The minimum principal stress orientation for the stress field as shown by the lines in Figure 9. For this load condition the rotation of the axis of principal stress is negligible. Hence, a pressurized crack originating at the borehole in the center should propagate horizontally. This behavior was observed in small scale laboratory experiments.

We have also begun an analysis to determine how hydrofractures shapes are modified by an asymmetrical nonuniform stress field. Initially we determine the displacements of a pressurized crack when the in situ stress field varies locally along the length of the crack. To date, five calculations have been made with a new numerical model.⁶ The new model was used because it is more economical in terms of computer memory and time than the finite element model for this type of problem.
Five cases of pressurized fracturing were examined; each case corresponded to a specific in situ stress. If the pressure in the crack is taken as $P$, then an in situ compressive stress of $0.9 P$ normal to the crack exists over a fraction of the crack length (Figure 10). The remaining in situ stress is zero. Young's modulus ($E$) is specified as $E = 1000 P$ and Poisson's ratio is 0.25. The crack is assumed to exist in an infinite media and the calculations were made in two dimensions (plane strain).

The perpendicular displacements of one face of the pressurized crack (displacements of the other face are symmetric) are shown on Figure 11. The crack length is $l$ and the Cartesian coordinate system originates at the bottom of the crack. When (see Fig. 10) $n$ is zero the displacements have the classical elliptic shape obtained when a pressurized crack is placed in a uniform stress field. When $n = 0.1$ the displacement is only slightly distorted from the elliptical shape. As $h$ increases, the displacements become more strongly distorted from the elliptical shape. Since the stress intensification is related to these displacements it is evident that a crack would be much less likely to extend to the right as the nonzero in situ stress extends further along the crack.

**Hydraulic Fracturing and Rock Fabric**

In many cases rock anisotropies, i.e., fabric elements like joints, control the direction of hydraulic fractures and the breakdown and closing pressures. This can be shown both by theory and by analyses of field data. When extension joints are the result of the existing rock stresses, the interpretation of hydraulic fracturing data can yield good results of minimum stress direction and value. In most other cases, we can expect the fracture both at the drillhole wall and away from the influence of the hole to trend at an angle with the maximum principal stress direction and to yield estimates of minimum principal stress that are different from the true value.

Where it can be reasonably assumed that rock fabric reflects the stresses that are responsible for local structures such as folds and faults, the fabric and the estimated stresses vary with position relative to the local structure. When the fabric affects the hydraulic fracture process, the stress information obtained will also vary with structural position, reflecting local rather than any regional state of stress.

Because of this, the minifrac technique should be valuable in designing large hydraulic fracturing jobs and in predicting the fracture geometry. We here present a simplified analysis of the interaction of the stresses around a drill-hole in a rock with anisotropic strengths.

We have previously discussed the relations between rock joints on other fabric elements, past and present stress, and the geometry of hydraulic fractures.5
In all cases, we have studied the direction of induced fractures (and the stress directions deduced therefrom) are strongly influenced by the fabric of the rock.

Fabric of the rock causes anisotropic physical properties, and analysis of the mechanics that treat the rock as isotropic neglects these fabric-induced anisotropic properties. As an example, apparent directions of regional stress may often be distorted by the rock fabric.

A simple and relatively easily determined parameter is the value and direction of minimum tensile strength in a rock. The following discussion is an example of its effect on fracture direction. Joints, ubiquitously present in rocks, are fabric elements that directionally reduce tensile strength to close to zero in many cases. They are probably the fabric element that produces the major effect.

Referring to Figure 12, $\sigma_A$ and $\sigma_B$ are principal stresses at a distance exterior to the hole.

$\sigma_\theta$ is the tangential compressive stress at the side of the hole at point $r$, $\theta$ with $\theta$ measured from the direction of $\sigma_A$.

Following the method of Miles and Topping,$^7$ $\sigma_\theta$ can be expressed as:

$$\sigma_\theta = \sigma_A (1 - 2 \cdot \cos 2\theta) + \sigma_B (1 - 2 \cdot \sin 2\theta)$$

In hydraulic fracturing, break down pressure will be the value at which borehole pressure $P_D$ can overcome the stress plus the tensile strength $t_0$ of the rock, and the effective stress on the borehole wall is the tectonic stress $\sigma_\theta$ minus the pore pressure $P_f$. Then:

$$P_D = t_0 + \sigma_\theta - P_f$$

With a uniform horizontal stress $\sigma_H$ then, breakdown pressure $P_D = t_0 + 2\sigma_H - P_f$, and the fracture should initiate anywhere $t_0$ is a minimum. (We assume a vertical fracture.)

A more general formulation for $P_D$ with two unequal principal horizontal stresses, where $\sigma_A$ is the lesser and $\sigma_B$ the greater, and if $t_0$ is equal in every direction:

$$P_D = t_0 + 3 A - B - P_f$$

In an anisotropic case, fracturing will occur at a point where $P_D$ is a minimum determined by the ratios of $\sigma_A$ and $\sigma_B$, and the anisotropy of $t_0$. With an anisotropic $t_0$ that point is not necessarily at the direction of the greatest horizontal stress. (We are assuming here that the least principal stress is in the horizontal plane.)
One example will suffice to show the inter-relationship between unequal stresses, anisotropic tensile strength and direction of initial fracture propagation.

In the Piceance Basin, a minifrac technique\(^3\) was used to determine the following values in one test:

- **Vertical principal stress, \(\sigma_z\)**: 1025 psi
- **Horizontal principal stress, \(\sigma_2\)**: 1248 psi
- **Horizontal principal stress, \(\sigma_3\)**: 821 psi
- **Pore pressure \(P_f\)**: 330 psi
- **Tensile strength \(t_0\)**: 250 psi
- **Break-down pressure, \(P_D\)**: 1141 psi

To show effects of anisotropic tensile strength, a sample calculation was made with the following values:

- **Horizontal stresses**: \(\sigma_A = 800\) psi, \(\sigma_B = 1200\) psi
- **Tensile strength, anisotropic**: 250 to 2000 psi \(*\)

\((*\ oriented \ so \ \sigma_0 = 250\) psi across a line at \(\theta = 45^\circ\) and 2000 psi across a line at \(\theta = 135^\circ\).\)

- **Pore pressure**: \(P_f = 300\) psi

After calculating \(\sigma_0\), solved for minimum break-down pressure \(P_D\),

\[ P_D = 1800\) psi at \ = 75^\circ.\]

If \(\sigma_0\) were uniform (isotropic), then

\[ P_D = 1150\) psi @ 90^\circ\) for \(\sigma_0 = 250\) psi

and

\[ P_D = 2900\) psi @ 90^\circ\) for \(\sigma_0 = 2000\) psi.

\(P_D\) could have any number of values and the initial fracture a number of orientations depending on the values and orientations of the various parameters. Should tensile strength be essentially zero, for instance, on a joint at the least strength line, the azimuth of the initial fracture would move toward the fracture, as it would as the horizontal principal stresses approached equality with each other. In a tectonically relaxed environment,
as for instance in the Piceance Basin\(^9\) or the Wattenberg Field in the Denver Basin,\(^9\) induced hydraulic fractures appear to follow closely the planes of weakness due to the rock fabric.

Away from the stress concentrations around the drill hole (refer to figure 13), the closing stress \(\sigma_\theta\) normal to a joint or other plane of weakness at azimuth \(\theta\), is

\[
\sigma_\theta = \sigma_A \sin \theta + \sigma_B \cos \theta
\]

Substituting, this becomes

\[
P_D = t_0 + \sigma_A \sin \theta + \sigma_B \cos \theta - P_f
\]

In our example, with anisotropic strength, this becomes minimum

at \(\theta = 45^\circ\): \(P_D = 250 + 800 \sin 45 + 1200 \cos 45 - 300\)

\(P_D = 1364\) psi

At the azimuth of the beginning fracture,

\(\theta = 75^\circ\): \(P_D = 1634\) psi

And normal to the least principal stress

\(\theta = 90^\circ\): \(P_D = 1650\) psi

Discussion

There are several consequences:

(1) Initial azimuth of the fracture at the borehole wall may not be indicative of azimuth away from the hole.

(2) Initial and final azimuth may not be normal to the direction of minimum principal horizontal stress.

(3) Depending on the anisotropy of the in situ stress and of rock strength, the measured initial shut-in pressure may not measure minimum horizontal stress. Rather, it is related to the azimuth of the fracture, and measures \(\sigma_\theta\), where \(\sigma_\theta = \sigma_A \sin \theta + \sigma_B \cos \theta\).

Together with other work relating local structure and fabric to stress and fracturing, we see here an indication that local fabric and apparent values of in situ stress can be useful in fracture design and prediction, even in cases where they cannot be related to regional values.
References:


FIG. 1. Geometry and loading of the problem.

FIG. 2. Contours of constant stress in the y-direction for problem geometry and geometry shown in Fig. 1.
FIG. 3. Contours of constant stress in the x-direction for problem geometry and load shown in Fig. 1.

FIG. 4. Orientations of the minimum principal stress for problem geometry and load shown in Fig. 1.
FIG. 5. Contours of constant stress in $y$-direction for a constant pressure crack, the problem geometry and loading shown in Fig. 1.
FIG. 6. Contours of constant stress in the x-direction for a constant pressure crack, the problem geometry and loading shown in Fig. 1.

FIG. 7. Asymmetric nonuniform loading pattern.
FIG. 8. Contours of equal y-component of stress for loading configuration in Fig. 1.
FIG. 9. Minimum principal stress orientation for loading configuration in Fig. 1.

FIG. 10. Geometry showing the position of the step change under \textit{in situ} stress ($\sigma_{x}^{0}$) along the crack length.
FIG. 11. Perpendicular displacement (in the x-direction) of the crack face for various positions of the step change in initial stress.
Fig. 12. Stresses around a drill-hole.

Fig. 13. Stresses away from a drill-hole.
THE USE OF BOR(EHOLE BREAKOUTS IN THE STUDY OF CRUSTAL STRESS

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Abstract

The theory of stress concentration near a circular borehole, in an anisotropic horizontal stress field, is summarized. Results of application of the Mohr-Coulomb theory of brittle shear fracture are reported. Breakouts are explained as elongations of the borehole through shear fracturing in the amplified stress difference near the hole. Such shear fractures will be wide enough to be detected by the four-arm dipmeter calipers as elongations of the borehole section, whereas tensile fractures are not detectable in this way. Azimuthal concentration of breakouts is shown to require unequal horizontal principal stresses, and appreciable initial shear strength in the rock. Notes are next given on the four-arm dipmeter, on the identification of breakouts and on the measurement of their azimuths. Data on borehole elongations and inferred horizontal-stress orientations are noted from five stress provinces in North America, with references to papers. Finally some suggestions are offered on future developments in the application of four-arm dipmeter data and inferred breakout orientations to the elucidation of the stress field in the Earth's crust.

Introduction

The walls of many boreholes spall so as to produce intervals with non-circular cross sections whose long axes share common average orientation (Cox, 1970; Babcock, 1978). Such intervals are defined as breakouts in cases where the shorter diameter of the borehole corresponds to the drill-bit diameter. Breakouts exhibiting well-grouped azimuths have been reported in several parts of North America (Cox, 1970; Babcock, 1978; Schafer, 1980; Brown et al., 1980; Gough and Bell, 1981; 1982; Springer and Thorpe, 1981). The known areas exhibiting coherent azimuths cover the range from a single oil-well to more than $3 \times 10^5$ km$^2$. 
Theory

Babcock (1978) and Brown (1978) attributed breakouts to the intersection of pre-existent vertical fractures by the boreholes. Bell and Gough (1979) put forward the hypothesis that breakouts are caused by shear fracturing in the zone of stress amplification close to the borehole wall, and proposed that the azimuthal grouping of breakouts is a result of unequal horizontal principal stresses in the rock intersected by the borehole.

Kirsch (1898) found an analytic solution of the problem of the stress field near a small hole of radius $a$ in a large plate under uniaxial compression $S$:

$$
\begin{align*}
\sigma_r &= \frac{S}{2} \left( 1 - \frac{a^2}{r^2} \right) + \frac{S}{2} \left( 1 + \frac{3a^4}{r^4} - \frac{4a^2}{r^2} \right) \cos 2\theta \\
\sigma_\theta &= \frac{S}{2} \left( 1 + \frac{a^2}{r^2} \right) - \frac{S}{2} \left( 1 + \frac{3a^4}{r^4} \right) \cos 2\theta \\
\tau_{r\theta} &= -\frac{S}{2} \left( 1 - \frac{3a^4}{r^4} + \frac{2a^2}{r^2} \right) \sin 2\theta
\end{align*}
$$

where $\theta$ is measured from the direction of $S$ and $\sigma_r$, $\sigma_\theta$ and $\tau_{r\theta}$ are the radial, tangential and shear stresses respectively. At the hole boundary, $r = a$,

$$
\begin{align*}
\sigma_r &= \tau_{r\theta} = 0 \\
\sigma_\theta &= S - 2S\cos 2\theta
\end{align*}
$$

so that the tangential stress has a maximum value of $3S$ when $\theta = \frac{\pi}{2}$ or $\frac{3\pi}{2}$ at the ends of the diameter perpendicular to $S$.

At the ends of the diameter aligned with the compression, $\sigma_\theta$ has a minimal value of $-S$ and is tensile. A vertical borehole in the Earth's crust will in general be under biaxial compression with horizontal principal stresses $S$ and $s$, $S > s$, though $s$ may be tensile near the surface. In the biaxial case superposition gives, at the wall,

$$
\begin{align*}
\sigma_r &= \tau_{r\theta} = 0 \\
\sigma_\theta &= S + s - 2(S-s)\cos 2\theta
\end{align*}
$$

This case is illustrated in Fig. 1. Near the points $P$ and $Q$
\[ \sigma_\theta = 3S - s, \quad \sigma_r = 0 \]

and the stress difference

\[ \sigma_\theta - \sigma_r = 3S - s. \]  

The stress amplification is very local and the stress difference approaches \((S - s)\) one radius from the wall (Gough and Bell, 1982). A hole is an isotopic horizontal stress field, \(s = S\), has tangential stress \(2S\) at its wall and if it spalls, will do so without preferred azimuth provided the rock is isotopic. Gough and Bell (1982) have applied Mohr-Coulomb brittle fracture theory to the case of anisotropic stress and found that shear fracturing will begin at the walls near \(P\) and \(Q\) (Fig. 1) and may proceed to elongate the borehole section by forming wide, opposed excavations quite unlike the narrow tensile fractures in hydraulic fracturing. Figure 2 illustrates the breakout geometry and Fig. 4 shows an actual breakout as observed by a camera. The Mohr analysis shows that with typical rock parameters breakouts will initially extend the hole by 8-10 percent of the original diameter; and that appreciable initial shear strength (cohesion) \(\tau_0\) is necessary to give tightly grouped azimuths of breakouts. Weak rocks will tend to spall omnidirectionally, whereas strong rocks will fail only near the diameter parallel to the lesser horizontal compression (Gough and Bell, 1982). Azimuthally aligned breakouts are therefore to be expected on the two conditions that the horizontal principal stresses are unequal, and that the rock has appreciable shear strength. The field evidence from Alberta shows that aligned breakouts occur in the more competent rocks in the basin - in sandstones, siltstones, limestones, dolomites and one shale (Babcock, 1978).

Pore-water pressure and drilling-mud pressure effects have been discussed by Gough and Bell (1982). In general they will modify the magnitude of the stress difference at the borehole wall without affecting the azimuthal orientation of breakouts, assuming these are caused by the proposed mechanism of shear fracturing.

While the amplification of stress difference is independent of the hole radius \(a\), the elastic strain energy stored behind unit area of the wall increases as \(a^2\). Larger holes will therefore spall to a greater extent than small ones, as drillers observe.

Identification and measurement of breakouts

Ideally, the best tool for identifying breakout zones in a borehole and measuring their orientation would be an optical or acoustic imaging device which could operate in a borehole and record its azimuthal orientation. In the late sixties, the Mobil Research and Development Corporation developed a borehole...
televiewer and published acoustic images which may show breakout zones (Zemanek et al., 1969). Subsequently, Amoco undertook further development of the tool (Wiley, 1981) and have also published images of fracture zones in borehole walls which may correspond to breakouts (Fig. 3). The most spectacular pictures of breakouts known to the authors, however, are those described by Springer and Thorpe (1981). Figure 4 is reproduced with their permission and portrays a breakout photographed at a depth of 315 m in a test hole of diameter 2.4 m at the Nevada Test Site. The photograph was taken by a downhole ciné camera with an orientation recording system developed by Brugman (1979).

In deep oil-wells full of drilling mud optical devices have limited use. For most breakout investigations one is therefore obliged to rely on the orthogonal magnetically oriented calipers of commercial four-arm dipmeter tools. Such tools led to the original discovery of breakouts. Although the design of these devices varies somewhat, they all contain electrode pads mounted on hydraulically extendable arms which monitor hole diameter and caliper orientation as the tool is drawn up a borehole. Schlumberger's HDT-E and F tool is representative and is illustrated in Figure 5. The tool can be used in boreholes between 20 and 54 cm in diameter. Each pad is 5.83 cm wide and 41.4 cm long. Generally, the tools are raised at 10 m/minute and the cable is torqued so as to cause the tool to rotate clockwise. This rotation stops if one or both pads of a pair are trapped in a breakout.

The calipers of the four-arm dipmeter measure only the gross shape of the borehole and may miss breakouts which do not exceed the calipers (5.8 cm) in width. The recorded extension of the diameter will often be less than the true value and the azimuth will be only approximate, because the pads are so wide. Despite these limitations, the device is our main source of data. From the discussion above it is evident that shear fracturing can be expected to form excavations wide enough to accommodate the electrode pads, whereas tensile fractures will in general be too narrow to be observed with the dipmeter.

Figure 6 illustrates an uncomputed 4-arm dipmeter record of the type that is supplied to oil companies contracting for this logging service. The curves on the far right of the log record the diameters measured by the two pairs of opposed calipers, with diameter increasing to the left. The four noisy traces display the vertically variable resistivity of the stratigraphic sequence. Planes fitted to features of these traces allow the strikes and dips of sedimentary beds to be estimated. Tool orientation is indicated on the left side of the log, where the solid curve records the azimuth of caliper 1 with respect to magnetic North.

A typical breakout zone is present from 9418 ft to 9527 ft in Figure 6. Over this interval, opposed calipers 1 and
3 record a hole width of approximately 24.1 cm, equivalent to the diameter of the drill bit. Calipers 2 and 4 record a varying borehole diameter which generally ranges between 25 and 28 cm. The curve recording the compass azimuth of caliper 1 near the base of the figure shows that the tool had been rotating clockwise as it was drawn up the well. At 9527 ft, tool rotation ceased, calipers 2 and 4 became fixed with the elongated breakout, and the dipmeter was drawn up the borehole with caliper 1 oriented at an average azimuth of 200°. Adding the magnetic declination (23°) and subtracting 90° because the breakout here trapped calipers 2 and 4, one finds an azimuth of N133°E for the breakout. Most breakouts of any considerable length show variations of azimuth of 10-20 degrees. Some of this is presumably a result of variation in the fit of the pads into the fractures, but some probably reflects true variation in the azimuth of the long diameter. Figure 3 shows a televiewer record with a similar vertical variation in azimuth over a possible breakout zone. Zemanek et al. (1969, Figs. 6 and 25) show fractures which may be breakouts, with similar variation of azimuth.

While breakouts can easily be recognized by inspection of uncomputed 4-arm dipmeter records, accurate measurements of azimuths and caliper extensions are less straightforward to obtain. Logging contractors will provide computer printouts of both quantities at one foot intervals, if requested to do so, but frequently such listings are not available. In such cases we have obtained data either by examining the logs under a binocular microscope fitted with a micrometer eyepiece, or by digitizing analog curves like Fig. 6.

Most boreholes which we have examined have exhibited more breakouts in the upper part of the logged section than towards the base. This relationship appears to hold whatever the depth range or rock types penetrated and suggests that the fracturing which gives rise to breakouts may continue for some time after a hole has been drilled. Direct evidence of time dependent propagation of breakouts can be secured from a few Canadian boreholes which have been logged twice (Gough and Bell, 1981). In these holes, the second log runs recorded increased lengths (depth ranges) of breakouts and, in some cases, also increased long axes. This constitutes strong evidence that the holes cause the breakouts. If the amplified stress difference at the wall is only marginally greater than the strength of the rock, the formation of breakouts may be delayed. Possibly their growth may be promoted by the sidewall impacts and shearing effects of rotating drill pipe.

Breakouts and stress orientations in North America

Examples of azimuthally aligned breakouts have been reported from the Western Canadian Basin (Cox, 1970; Babcock, 1978; Bell and Gough, 1979; Gough and Bell, 1981), from Colorado
(Gough and Bell, 1982), from Nevada (Springer and Thorpe, 1981), from west Texas (Schafer, 1980; Gough and Bell, 1981) and from eastern Texas (Brown et al., 1980; Gough and Bell, 1982). In these five regions, which lie in five stress provinces, the orientations of the horizontal principal stresses have been deduced from the breakout azimuths, and in most cases compared with stress orientations estimated from overcoring measurements, hydraulic fracturing, earthquake mechanisms or evidence of recent movements on faults. In all cases thus far, the stress orientations we infer agree with those from other data, and thus support the hypothesis of Bell and Gough (1979, 1982) of the origin of breakouts. The locations and mean orientations of breakouts and inferred layer principal horizontal stress axes, thus far identified in Canada, are shown in Figure 7. Figure 8 illustrates similar information from Texas, and Figure 9 summarizes measured principal stress azimuths and breakout orientations from the Rangely oilfield in Colorado.

Future Development

At this time no borehole televiewers are commercially available for examination of small-diameter boreholes, though there is a growing demand for such a service. On the other hand, four-arm dipmeters have been widely used by the oil industry since the late sixties. The service is not routine and is expensive, but it is considered worthwhile in structurally or sedimentologically complex areas and in high-cost frontier basins. In Canada most dipmeter logs have been released and can be examined by the public and copied for the cost of reproduction. It is likely that a large body of dipmeter data exists from which information on breakouts, and hence on horizontal principal stress axes, can be derived for many of the world's cratonic sedimentary basins and continental shelves. Similar stress orientation data may be expected from breakouts in water wells, tunnels and mine shafts. Lo and Morton (1976) have accounted for spalling of tunnel roofs in Ontario in terms of large horizontal compressions transverse to the tunnel, and have shown how the stress concentration is further magnified above a tunnel at a small depth. This work was important to us in formulating our hypothesis of the mechanism of breakouts in boreholes.

Breakout orientations in a given stress province should prove useful in predicting the direction in which tensile fractures will form when pressure is applied to water in a borehole. It has been known for many years that such hydraulically induced tensile fractures tend to form normal to $\sigma_3$ (Hubbert and Willis, 1957). If $\sigma_3$ is horizontal, breakouts will form parallel to $\sigma_3$ in holes as they are drilled or shortly thereafter. Hydraulic fractures should then be vertical and at right angles to the breakout azimuth. Gough and Bell (1981) have reported evidence of this relationship in west central Alberta. In oilfields in particular, hydraulic fractures should run at right angles to any
breakouts observed in wells, on our hypothesis. If breakouts are mistaken for existing fractures intersected by the wells, hydraulic fracturing may be predicted incorrectly to propagate in the breakout azimuth.

In hydraulic fracturing experiments for measurement of the stress tensor, the magnitudes of the principal stresses are estimated by well-known methods discussed in other papers in this volume. A difficult problem is posed by the determination of the fracture orientation, though this can be done by existing techniques. If breakouts can be detected and azimuthally oriented in the borehole under test or in nearby boreholes, the resulting azimuths of $S_H$ and $S_h$ may provide the directional data to complete the specification of the stress tensor, without direct measurement of the hydraulic fracture orientation.

While stresses in the crust can arise from a variety of natural and man-induced causes (see Gay, 1980, Figure 1), recent compilations strongly suggest that the lithosphere is divided into regions of common stress orientation (McGarr and Gay, 1978; Zoback and Zoback, 1980). As data accumulate some stress provinces may be found to correspond to microplate terrains defined by paleo-magnetic and stratigraphic criteria (Beck and others, 1980). It is quite possible that horizontal principal stress orientations inferred from breakout azimuths will provide another approach for defining terrains which have experienced discrete geotectonic emplacement histories.

Perhaps the principal importance of stress orientations inferred from breakouts lies in the prospect of detecting and delimiting large areas of the crust with coherent orientation of the horizontal principal stresses. The West Canadian Basin is the best example so far, showing northeast-southwest greater horizontal compression throughout the Basin. The oil-wells which provide the data cover a length of 1900 km and sample an area in excess of $3 \times 10^5$ km$^2$. It seems clear that this direction of $S_H$ has existed from the Laramide thrusting and folding of the Rocky Mountains to the present time. Coherent stress orientation over such a large area, persistent through at least 60 my, seems difficult to account for except in terms of large-scale tractions on the North American plate.
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Imperial Oil Ltd., 1978. The Cold Lake Project, Report to Energy Resources Conservation Board of Alberta, Canada.


Figure 1. Principal stresses near the maximum stress concentration produced by a circular hole in rock under biaxial stress. $S, s$ are the horizontal principal stresses far from the hole. At $P$ and $Q$, $\sigma_r = 0$ and $\sigma_\theta = 3S - s$. At $Q'$, very near $Q$, the stress difference is nearly $3S - s$ compared with $S - s$ far from the hole. After Cough and Bell, 1982.
Figure 2. Shear fractures leading to a breakout. For $\mu=1$, $\psi=22^{1/2}$°, and the maximum diameter, $2\times OM$, is 1.08 (original diameter) (see text). Fractures such as those through N will not intersect the hole. After Gough and Bell, 1982.
Figure 3. A borehole televiewer log of an open hole section in a water injection well drilled in Hockley County, Texas, which illustrates possible incipient breakout spalling with east-west orientation (reproduced by permission of R. Wiley, Amoco Production Co.; the original photograph was published by Wiley, 1981, and the possible incipient breakout identified as a vertical fracture up to 2 inches wide).
Figure 4. A breakout zone in a tuff interval photographed by a downhole movie camera at a depth of approximately 315 m in a 2.4 m diameter borehole drilled at the Nevada Test Site. The small white object at 4 o'clock is a spalled rock chip falling down the hole. (Original photograph published by Springer and Thorpe, 1981, and reproduced with their permission.)
Figure 5. Schlumberger's HDT-E/F four-arm dipmeter logging tool.
Figure 6. Uncomputed four-arm dipmeter log showing a breakout zone between depths of 9418 feet and 9527 feet.
MEAN AZIMUTH OF 5 OR MORE BREAKOUTS.
MEAN AZIMUTH OF 4 OR LESS BREAKOUTS.
MAXIMUM HORIZONTAL PRINCIPAL STRESS ORIENTATION INFERRED FROM INDUCED FRACTURES (E) AND STRESS TENSOR DETERMINATIONS (ES).
FORMER MAXIMUM HORIZONTAL PRINCIPAL STRESS ORIENTATION INFERRED FROM NATURAL FRACTURES.

Figure 7. Mean azimuths of 442 breakouts in 48 wells in Western Canada together with otherwise inferred or measured larger principal horizontal stress orientations. (Information drawn from Babcock, 1978; Gough and Bell, 1981, 1982; Imperial Oil Ltd., 1978; Kaiser and others, 1982; Kempthorne and Irish, 1981; McLeod, 1977).
Figure 8. Mean breakout azimuths measured in 21 wells in southwestern Texas (Schafer, 1980) and 50 wells in eastern Texas (Brown and others, 1980), together with larger horizontal principal stress orientations measured by Hooker and Johnson (1969) and inferred from induced fractures by Strubhar and others (1975).
Figure 9. Structure contour map of the Rangely anticline showing orientations of horizontal principal stresses determined from over-coring (single-stemmed arrows) in surface exposures of the Mesa Verde Formation, from earthquake focal plane solutions (double-stemmed open arrows), from the orientation of an induced hydraulic fracture in the Weber Sandstone (heavy solid arrows) and from mean breakout azimuths measured in the Weber Sandstone (solid line denotes mean breakout axis). The map shows subsea contours drawn on the top of the Weber Sandstone and illustrates faults which offset this surface. Data are from Raleigh, Healy and Bredenbush (1977) and Couch and Rall (1982).
ACOUSTIC EMISSIONS DETECTED BY HYDROPHONE
DURING HYDRAULIC FRACTURING STRESS MEASUREMENT

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ABSTRACT

During hydraulic fracturing experiments for crustal stress measurements, sensitive observations of the acoustic emissions (AE) were carried out in order to study the fracturing process. A number of AE was detected by hydrophones placed in a space between the casing pipe and the injection pipe.

AE were recognized as sonic waves generated during hydrofracturing, based on the wave-form and good correlation with water pressure variation. Statistical analysis revealed the amplitude-frequency relation of AE obeys the Ishimoto-Iida relation; the m-values obtained were 2.0 and 2.5.

Differences in the hydrofracturing process are also discussed from the point of view that patterns of frequency of events depend on well wall conditions.

INTRODUCTION

Observing the acoustic emission (AE) activity associated with hydraulic fracturing is absolutely crucial to detect the origin time and directions of the generated fractures. Studies of AE in relation to the hydrofracturing process have been made in both the laboratory using small rock samples [e.g., Shuck and Keech (1977); and Lockner and Byerlee (1977)] and the field [Power et al. (1975); and Power (1977)]. Field experiments present many unfavorable conditions for detecting AE, such as the high attenuation of AE and great background noise.

Power et al. (1975) reported that AE in the frequency range from 100 to 2,000 Hz were detected by a downhole hydrophone immediately after water injection in hydrofracturing conducted by the El Paso Natural Gas Company. In geothermal energy projects some similar experiments also have been made [e.g., Takahashi et al. (1980)]. These experiments were on large scale hydrofracturing. Hydrofracturing for crustal stress measurements, which is a rather small scale fracturing, is of advantage for analyzing AE detected: for example, (1) the hydraulic fracturing is made in intact rocks avoiding pre-existing fractures as much as possible, and (2) the depth where the fractures is initiated is limited within a small range because the straddle section is generally short.
We will discuss in this paper (1) what kinds of sensor are useful to detect AE associated with hydrofracturing, (2) what characteristics AE has, and (3) whether there is any relation between AE occurrence and water pressure.

EXPERIMENTAL METHOD

Experiments were performed at two sites, Okabe and Nishiizu, both in Shizuoka Prefecture, Japan. Fig. 1 shows the AE measurement system at the hydrofracturing site. Basically, hydrofracturing for in-situ stress measurement consists of pumping water into a selected section in a borehole, fracturing the borehole wall by the hydraulic pressure, and measurement for water pressure. A pair of inflatable packers is used to seal the section.

The sensors used for measuring AE are piezoelectric accelerometers, seismometers and hydrophones. Three piezoelectric accelerometers with resonant frequencies of 8.7 kHz, 22.8 kHz and 45.0 kHz were mounted at the top of the casing pipe by magnets. Three 1 Hz seismometers were placed about 50 to 100 m apart from the experimental well on exposed rocks. One or three hydrophones were placed in a space between the casing pipe and the injection pipe at depths of 1 to 15 m. Two types of hydrophone were used. Fig. 2 shows the frequency response of each hydrophone with internal preamplifier (A-type: 60 dB, and B-type: 10 dB). The signal from the piezoelectric accelerometers was recorded on magnetic tapes after passing through a low-cut filter (300 Hz to 500 Hz) to reduce the engine noise from the pump. In the case of the hydrophones, the signal was also recorded on magnetic tapes using a 14 channel data recorder with frequency response between DC and 10 kHz on a frequency modulating system, and between 500 Hz and 150 kHz on a direct recording system, after passing through a low-cut filter (700 Hz to 1,500 Hz). A time code, the water pressure and the flow rate were also recorded simultaneously on the same tape.

EXPERIMENTAL RESULTS

(1) The Okabe site

Hydrofracturing was carried out in two 100m deep wells at Okabe Town to measure in-situ crustal stresses (Tsukahara, this issue). AE measurements were attempted by using the piezoelectric accelerometers, the seismometers and the hydrophones.

A number of AE was observed in experiments M81 and M83 only by the A-type hydrophone placed at the depth of 10m in the borehole. However, AE could not be observed in the other hydrofracturing tests because of inadequate observation conditions such as cut-off frequency and amplification. In the case of piezoelectric accelerometers mounted at the top of the casing pipe, AE was not detected. The seismometers placed on the exposed rocks also failed to indicate seismic events. AE waves were probably attenuated by the rocks and the background noise levels during pumping were too high.
A typical wave-form of AE is shown in Fig. 3. Average noise level is about 0.02 μbars. The maximum amplitude of most events was saturated on our recording system, and further they were analyzed after passing through a bandpass filter (1.5 kHz and 5 kHz). Therefore, the wave-form shown in Fig. 3 does not show the original wave-form of events. However, it is obvious that the wave-form of the initial motions is very sharp and pulse-like. Fig. 4 shows the relation between frequency of the events (numbers/0.5 sec) and pressure variation for experiment M83. Good correlation is found between the two. AE break out just before maximum water pressure. Microcracks seem to have broken out before the main breakdown. A few AE were detected after a high activity period around the breakdown event regardless of continuing water injection. These facts described above corroborate that these AE originated from hydrofractures of the strata.

(2) The Nishiizu site

Two types of hydrophones were used at the 450m deep Nishiizu well. The data shown in Fig. 5 are the typical wave-forms of AE recorded in an experiment at a depth of 436m (N436). The upper part of Fig. 5 shows AE obtained by the B-type hydrophone placed at a depth of 6m (B-6) in the borehole, and the lower one is the AE obtained by the A-type hydrophone placed at a depth of 15m (A-15) in the borehole. Bandpass filtering between 2.5 kHz and 9.0 kHz was used at playback. Generally, the predominant frequencies of the events range from 3 kHz to 4 kHz. Fig. 6 shows the wave-form examples of noise from the surface. It was estimated to be noise from the surface based on the time lag between hydrophones placed at different depths. It was transmitted along the injection pipes to the hydrophones. We can differentiate AE and the noise based on the wave-form difference of the initial motions and the duration of oscillation.

AMPLITUDE-FREQUENCY RELATION OF AE

Statistical analyses were performed on events obtained from the Nishiizu experiments N263 and N436. It is well known that the relation between the number of seismic events and their maximum amplitude is expressed by a simple empirical equation called the Ishimoto-Iida statistical relation, namely:

\[ n(a)da = Ka^{-m}da \]

where \( n(a)da \) is the number of events with the maximum amplitude range from \( a \) to \( a+da \), and K and m are constants.

The data analyzed here were obtained from the first pressurization cycle of hydraulic fracturing. The pressurizing period was divided into several short intervals according to the pressure variation curves (Fig. 7) in order to investigate statistically the difference among the time intervals. Water injection was started at A in Fig. 7. In intervals A-B and E-F, the number of AE detected was too small to be analyzed statistically. Further, in interval E-F, we did not detect any AE. Therefore, we analyzed the data obtained from intervals B-C and C-D.
Fig. 8 shows the relation between amplitude and frequency of AE occurring in intervals B-C and C-D. The dead time of the discriminator was set at 3/4 msec, so as not to count the same event repeatedly. The relation gives straight lines in various amplitudes from 1 to 2.4 in both intervals B-C and C-D of experiment N263. The m-value in Eq. 1 is 2.5 for both intervals. In the case of experiment N436, while the data are fitted well by a straight line with m = 2.0 in various amplitudes from 0.5 to 3.0 in interval B-C, considerable scatter is shown in interval C-D.

Power et. al (1975) showed a m-value of 2.5 for post-main-fracture signals in the frequency range from 500 Hz to 2,000 Hz detected with hydrophones at several depths. There is a significant difference among the m-values. They will much depend on the lithology and the distribution of natural fractures.

DIFFERENCE IN HYDROFRACTURING PROCESS BETWEEN N263 AND N436

As described above, the m-value of N263 is different from that of N436. There should be some difference in the process of fracturing, which depends on well wall condition. While the lithology at both depths is almost the same, volcanic tuff, there are differences in the velocity of P-wave, 5.3 km/sec at 263m and 5.0 km/sec at 436m. Fig. 9 shows the borehole televiewer records [after Tsukahara et al. (1980)] before and after hydrofracturings at 263m and 436m. A large difference in the state of new fractures can be recognized between experiments N263 and N436. In the case of N263, clear fractures are shown in the picture (Fig. 9, left). This picture suggests that the fractures occurred in a homogeneous well wall, and are typical brittle fractures. This is consistent with the amplitude distributions of AE giving essentially straight lines in both intervals B-C and C-D as shown in Fig. 8.

In the case of N436, the picture shows rough and wide cracks, and the amplitude distribution of AE scatters in interval C-D as shown in Fig. 8. Brittle fracture continues until point C, and after that fractures might grow up in such a way that the surface of the borehole wall exfoliates. Accordingly a different type of event which does not follow the Ishimoto-Iida relation might have occurred. To confirm this process, the rate of AE activity versus pressure under the condition of a constant pumping rate is shown in Fig. 10. It is interesting to note that AE activity decreases temporarily around the inflection point of the pressure record C, and again increases in the time interval from C to D. A good correlation between the water pressure and the AE frequency corroborates that the variation of pressure at point C is not due to accidental leakage but change in the fracturing processes. These detailed observations of AE suggest that the initial breakdown pressure would be at point C.

The frequency range of our recording system, however, was very narrow. It is necessary to improve it (extending dynamic range and widening frequency characteristics) for distinction of main fracture events from micro-fracturing events.
REFERENCES


Fig. 1
Schemes for acoustic emission monitoring system at hydraulic fracturing site. Well depth is 100m at Okabe and 450m at Nishiizu.

Fig. 2 Frequency response of two types of hydrophone with preamplifier.
Fig. 3  An example of wave-form of AE detected by the A-type hydrophone placed at 10m depth (experiment at Okabe, 81m depth).

Fig. 4  Relation of AE frequency to water pressure and to injecting flow rate (experiment at Okabe, 83m depth).
Fig. 5 Examples of the typical wave-form of AE detected by the hydrophones placed at 6m depth (B-6) and 15m depth (A-15) in the borehole (experiment at Nishiizu, 436m depth).

Fig. 6 Examples of noise from surface. The B-type hydrophone was placed at 6m depth (B-6), and the A-type was placed at 15m depth (A-15) in the borehole (experiment at Nishiizu, 436m depth).
Fig. 7 Records of surface water pressure and injection flow rate versus time during hydrofracturing for experiments N263 and N436.
Fig. 8 Frequency of events versus maximum trace amplitude for experiments N263 and N436. The letters B, C and D indicate points B, C and D on the pressure curve in Fig. 7 respectively.
Fig. 9  Borehole televiewer logs before and after hydrofracturing. New cracks created by hydrofracturing are seen in the directions of N25°+10°E and S25°+10°W for both experiments N263 and N436.

Fig. 10  Relation of AE frequency to detailed water pressure variation for experiment N436. A, B, C and D are the same points in Fig. 7.
I. INTRODUCTION

The determination of principal stress directions in a rock mass by means of hydraulic fracturing requires the accurate location and orientation of the generated fracture. The usual practice employs post-fracturing down hole measurements with impression packers or borehole televsioning equipment to determine the orientation of the fracture at the borehole wall. However, such methods do not define the fracture away from the borehole. Inhomogeneities in the stress field and/or the rock mass may produce a different fracture pattern from the often assumed symmetric double-winged vertical crack. As a first step in assessing the validity of this assumption, an experiment was designed to monitor the acoustic emission (AE) activity associated with the hydrofracturing process. Sought were the answers to such questions as: (1) the existence of detectable acoustic emissions associated with the hydraulic fracturing, (2) if they exist, their magnitude and occurrence relative to pressurization, breakdown, and fracture propagation, (3) the character of AE activity, whether as discrete events or as near-continuous swarms, (4) given discrete events, is the signal-to-noise ratio sufficient to apply standard seismological analysis techniques to determine the event location, orientation, magnitude, and source characteristics, (5) lastly, if the answer to question (4) is affirmative, the number of AE sensors needed to apply practically these techniques in a hydrofracturing exercise.

II. PROCEDURE AND RESULTS

A 12 element array of vertical component piezoelectric transducers was deployed in a three-dimensional configuration surrounding a vertical and a horizontal hydrofracture hole as shown in Figures 1a, 1b and 1c. The specifications of the AE sensors and amplifiers are given in Table 1. These instruments are identical in gain and frequency response to ones being used to monitor AE activity in the Climax Stock repository, an experiment also
in granite using similar array dimensions (Majer, et al. 1981). The location of the stations are listed in Table 2. Experience at the Climax Stock site with this equipment (Columbia 5002 transducer and 9021 charge amplifier) has shown that serious noise problems can be introduced due to ground loops, i.e., multiple grounds in the system due to the case on the transducer being a ground. Therefore, to avoid this problem, the transducers were mounted on non-conducting material (epoxy discs) before mounting on the rock. The sensors were mounted in several different fashions. The "surface" stations, (i.e., 2, 4, and 5) were attached to the rock by epoxy cement. The two stations in the vertical boreholes, (1,3), were secured with a plaster compound (Hydrocol). The remaining stations in the horizontal holes were clamped tightly to the rock by using a spring arrangement. Because of the time constraint of five 7-hour days for set up, experiment, and removal, all of the sensors could not be attached using epoxy cement for the best coupling. Consequently, the best data (and in most cases the only usable data) were obtained from the surface stations 2, 4 and 5.

Data were recorded on a Honeywell 5600 C 14-channel tape recorder with frequency response of 300 to 20,000 Hz. Care was taken to properly adjust and balance all tape recorder channels, yielding a 54 dB dynamic range using one channel as compensation. Unfortunately, this primary recorder developed a malfunction after arriving at the Stripa mine and another tape recorder with only 40 dB dynamic range (also 5600 C) had to be substituted resulting in a substantial degradation in the data quality. In addition to the 12 data channels and the one compensation channel, a time code was recorded that was simultaneously being recorded on the pressure logs, allowing correlation between AE activity and various stages of the hydrofracture process.

Figure 2 shows one of the larger events recorded from the vertical hydrofracture hole. Although the signal-to-noise ratio is barely adequate, several significant points can be noted: (1) discrete events occurring during the hydrofracture process, (2) all events recorded were similar in nature, i.e., impulsively beginning P and S waves, (S-P times give reasonable source distances) and (3) time separations between events are enough for us to analyze each one for location, size and source type. If not for the problems with the substitute tape recorder, it appears that the data quality would have been sufficient to adequately define the fracture characteristics. The poor data quality was not due to noise generated by the associated hydrofracture process, but rather the recording instrument.

Figure 3 gives the rate of AE activity versus pumping rate through breakdown. Note that no AE activity was detected during the initial breakdown. The only significant activity occurred when fast pumping (4.5 liters/minute) was underway. From Figure 3 it also appears that AE activity (or rock fracture) occurs several minutes after pressurization. The threshold of AE detection was approximately \(10^{-2}\) g (g = acceleration of gravity) at 10 kHz. Most of the events show predominate frequencies near 10 kHz. Assuming that these events follow scaled theories applied in conventional earthquake source mechanics, the predicted size of a fracture is several centimeters in length. If AE activity is indicative of fracture growth, then it appears that in the hydrofracture process a series of
discrete fractures combine in creating a larger fracture. This indicates that even at a scale of centimeters, the local fracture process is a response to the overall applied stress field.

The mechanism of failure of the detected events seems to be shearing. Pearson (1981) also concluded that the events most likely to be detected during hydrofracture operation are shear failures induced by increased pore pressure. These shear events are not on the same plane as the tensional failures induced by high fluid pressures but are closely associated with the main fracture. Therefore, by locating the shear events the growth of the hydrofracture can in practice be traced. Another indication that the events detected are shear failure due to increased pore pressures is the time lag between pressurization and initiation of the events. Several minutes elapsed between "breakdown" and the beginning of the acoustic emissions. This indicates that a threshold of pressure must be reached in the formation before shear failure is initiated. This lag time is undoubtedly a function of fluid volume, permeability and the stress field. A careful study of this lag time and rate of AE activity may yield important information on these critical parameters.

Unfortunately, accurate source locations and fault plane solutions could not be obtained for most events. Several of the larger events at the beginning, throughout, and at the end of the fast pumping were analyzed for locations as shown in Figure 4. The locations indicate that the fracturing process is not symmetrical. Almost all events occurred in the northeast side of the hydrofracture hole. That is, station 2 was always the location of the first arrival, (station 1 failed). Although the events appear to line up in a NE-trending plane the locations are not of sufficient quality to prove that the fracture propagated in this direction. Impression packer work indicated a double-wing fracture propagation, almost on the axis of the drift. There were not enough good data to detect any change in first motion patterns with time, indicating a fracture "turn-over". The fault plane solution would also help resolve the question of detection of shear failure or tensional failure. Although there was a definite amplitude distribution, the poor dynamic range prevented calculation of a meaningful b-value.

III. SUMMARY AND CONCLUSIONS

The goal of mapping the fracture process in detail was not achieved in this experiment, but several significant results are noteworthy.

(1) If the lack of AE activity during breakdown is characteristic of the process, the initial breakdown represents a single large crack with frequency content much less than 1kHz, or it is a slow (aseismic) process of crack growth, or else it radiates energy too high in frequency to detect with our 20 kHz bandwidth tape recorder, i.e., the crack tip generating the signal in a tensional failure would be on the order of 100 khz.
On the basis of both the strong S-wave generation relative to P-wave amplitude and the time history of AE response, it appears that the observable AE activity is due to shear failure from reduction in effective stress from pressurization.

The AE activity slowly builds during fast pumping after pressurization pumping to a more or less constant level. This lag time may be a function of permeability, in that all the permeable cracks may have to be pressurized before significant fracture activity occurs. Other evidence for this may be in the faster decay of activity after pumping is stopped but shut-in pressure is held. It is not clear how to scale the time-constant with the size of the fracture produced.

The determination of hydrofracture growth and location details by seismological methods appears quite feasible. If not for the severe time and equipment constraints on this project, it seems clear that data quality could have been improved sufficiently that the location and characteristics of the individual fractures could have been calculated.

The few source locations determined are consistent with data from the impression packers, but with the major fracture propagating in an asymmetrical fashion mainly in the NE direction from the hole.

It is hoped that an experiment similar to the one described here can be carried out again. However, it is clear that several modifications to the procedure be made. The data should be recorded digitally with at least 12-bits of resolution. Also, the lower band edge should be reduced to 100 hz. If the initial breakdown is generating lower frequency signals (100 - 500 hz) then it would be possible to detect these signals with conventional high frequency geophones. It is important though, to retain the high frequency content of the signal (10 to 20 Khz), to completely characterize the fracturing process. This high frequency requirement limits the distance at which detectors can be placed from the hydrofracture hole. Ideally, one would like to monitor the hydrofracturing process from the same hole as the fract. This experiment indicates that the noise problems associated with acoustically monitoring the hydrofracture from the same hole are not insurmountable. It appears quite possible to develop a sonde that could be placed beneath the hydrofracture zone that would collect wide band three-component data. Each component would be a small array of sensors tuned to detect signals from the rock formation and ignore unwanted signals from the hole (i.e. noise from pumping, tube waves etc.). If successful on a small scale, this technology could possibly be expanded for use with massive hydrofracturing in commercial applications. The need for determining the hydrofracture path seems to be of critical importance, not only for understanding stress measurements but for determining the success of well stimulation operations. With the recent advance of in-field seismic processing and high speed, low power consumption computers, now is the time to bring all the techniques for fracture characterization used in earthquake seismology to bear upon the problem of hydrofracture monitoring.
Acknowledgements

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TABLE 1

Columbia 5002 Transducer

Sensitivity 13 pCoul/g
Frequency Response 2 Hz to 10 kHz, ± 5%
Resonant Frequency 50 kHz
Capacitance 850 pF
Output Resistance 2 \times 10^{10} \text{ ohms}

Columbia 9021 Charge Amplifier

Source Impedance Capacitive device, 500 pF max
Charge Gain 100 mV/pCoul (40dB)
Output Impedance 125 ohms
Frequency Response 1 kHz to 10 kHz, ± 5%

1 pole RC filter at 10 kHz

\( g = \text{acceleration of gravity} \)

TABLE 2

Stripa Station Coordinates (In meters)

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\( Z = 0 \) is at the Earth's Surface

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REFERENCES


Figure 1a. Plan view of experimental area showing station locations and the vertical (VHF) and horizontal (HHF) hydrofracture holes. Stations 1 and 3 are seven meters beneath the heater drift floor in vertical holes. Stations 6 through 12 are four to five meters below the heater drift floor, emplaced in holes drilled horizontally from an adjacent drift.
Figure 1b. Location of stations 9, 10, 11, and 12, showing locations relative to known fracture patterns and to the heater drift.
Figure 1c. Location of stations 9, 10, 11, and 12, showing locations relative to known fracture patterns and to the heater drift.
Figure 2. One of the better events recorded during the hydrofracture operation in the vertical hole.
Figure 3. Rate of acoustic emission activity (number of discrete events per two second interval) versus time and pressure for a fast pumping operation.
Figure 4. Approximate event locations (shaded area) for the vertical hole hydrofracture.
PART IV: THE RELATION BETWEEN IN SITU ULTRASONIC PROPERTIES OF ROCK AND IN SITU STRESS

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Abstract. In situ and laboratory measurements of ultrasonic velocity have been compared to test for the effect of in situ stress on rock properties. Initial data indicates that the large changes in compressional wave velocity ($V_p$) correlate with large stress changes. A change in $V_p$ anisotropy accompanies stress relaxation and is found to correlate with the relative orientations of the maximum compression.

Introduction

More than half a century ago Adams and Williamson (1923) recognized that hydrostatic compression affected the elastic properties of rock and that the change in properties was associated with the behavior of microcracks under load. Birch (1960) systematically studied the change in compressional wave velocity on reloading rocks in hydrostatic compression. Tocher (1957) and Nur and Simmons (1969) recognized that uniaxial stresses induce velocity anisotropies with the largest velocity change taking place in the direction of the applied stress. All of these studies concern reloading a rock that has been removed from an outcrop. Few data exist on the process of initial unloading and its effect on velocities and velocity anisotropies within rock. Observations presented in this paper indicate that the effect on the elastic properties of rock on unloading, also called strain relaxation, varies with lithology in a manner that cannot be predicted from the classic reloading experiments mentioned above.

Two goals in the development of instruments for the determination of earth stresses are to sample earth stresses using a non-destructive test and to sample using a remote sensing technique. Both goals may be reached with the use of sonic techniques provided that a thorough knowledge of the relationship between sonic properties and in situ stress is developed. The purpose of this paper is to present some data concerning the effect of in situ stress and strain relaxation on sonic properties of several types of rocks.

Sonic properties are important in exploration seismology because the accuracy of the earth structure delineated by seismology depends on the accuracy of the velocity structure. Borehole sonic logging is one of the common methods used in calibrating seismic refraction and reflection data. One possible extension of sonic log measurements is to determine in situ stress using the velocity and anisotropy change on cutting a core from a stressed rock. However, borehole tools to measure in situ azimuthal variation in sonic properties are not yet commercially available. In order to compare the sonic properties of cores with those in situ, knowledge of the change in rock properties accompanying stress relief (coring) and the relation between those
changes and in situ stress is desirable. Relevant information includes the change of velocity ($\Delta V_p$) and velocity anisotropy that accompanies a change in stress. Also, a method must be developed to distinguish between contemporary stress field and in situ sonic properties. In this paper we present data suggesting that it is feasible to use ultrasonic logs to gain information on in situ stress.

Techniques

Our technique was to correlate near surface (depth < 2 m) in situ stress and ultrasonic properties for several rock types in the northeastern United States (Figure 1). The lithologies include sandstone, limestone, diabase, metasediment, hornblende gneiss, and granite. The sandstone and limestone sites are on the Appalachian Plateau (Engelder and Geiser, part V), the gneiss and metasediment sites are located within the Precambrian Adirondack Mountains (Plumb, Engelder, and Sbar, part II), the diabase site is part of the Triassic-Jurassic Newark Basin, and the granites sites are within Pennsylvanian and Jurassic aged plutons in New England (Plumb, Engelder, and Yale, part III).

In situ stress was measured using the "doorstopper" technique where strain gauge rosettes are bonded to the end of boreholes and subsequently overcored (Sbar et al., 1979). On overcoring a strain relaxation of the core is measured; stress is calculated from modulii determined by reloading the cores in laboratory tests. At each site the stress is measured in the near surface (<2 m) in several (3-7) boreholes separated by 1-3 m. The outcrops used for these measurements were selected so that few if any joints intersected a line between the test holes. Because in situ ultrasonic velocity was measured along paths between the 7.6 cm diameter boreholes, joints between the boreholes would interfere with the ultrasonic travel time.

In situ P-wave velocity was measured using transducers in cylindrical anvils which were shaped to fit snugly inside 7.6 cm diameter boreholes (Figure 2). The anvils were loaded against the wall of the boreholes with the aid of hydraulic pistons. The transmitter used a 300 kHz, 2.54 cm diameter barium titinate disk driven at 20 Hz with a 500 volt, 1.5 µsec rise time pulse. The receiver was a matched barium titinate disk whose output was amplified and displayed on a Tectronix 335 portable oscilloscope. All electronics are powered with a 12-volt automobile battery so that the in situ velocity measurements can be made in remote locations.

To measure the time of flight between boreholes a delay box was used so that the outgoing pulse could be matched with the return signal to within one microsecond. The high voltage pulse triggers the oscilloscope sweep as well as a pulse which is delayed and displayed on the oscilloscope superimposed on the return signal. This method gave us greater accuracy in identifying the flight time than did using the delay-time mode of the oscilloscope. The field experiment was calibrated before and after use by means of measuring time of flight in an aluminum rod.

Ultrasonic P-wave travel time was measured along core diameters at 15° or 30° intervals to detect horizontal anisotropy (Simmons
et al., 1975; Engelder et al., 1977). The time of flight through cores was measured using anvils shaped to the outside diameter of the core. Here we used the delay-time mode of a 585A tectronix oscilloscope for measuring time of flight. Measurements were calibrated using an aluminum cylinder the size of the cores to be tested. Velocity was then computed from time of flight of ultrasonic pulses transmitted and received by piezo-electric crystals mounted in the anvils at opposite ends of a core diameter. Travel times were measured to a precision of 10 nanoseconds using an oscilloscope. Velocity magnitudes relative to different cores were accurate to 3% and velocity anisotropy observed in a particular core is accurate to 1/2%.

Both in situ and laboratory apparatus were calibrated using 2024 Aluminum which has a longitudinal wave velocity of 6.22 km/sec. A 10 cm diameter by 75 cm long rod was used for the in situ apparatus with the flight time measured along the length of the rod. A 7 cm diameter by 10 cm long cylinder was used for the lab apparatus with the flight time measured across the diameter. Velocities measured in both pieces of aluminum were less than 1% from 6.22 km/sec.

In situ Velocity versus in situ Stress

Our data is displayed as compressional wave velocity versus azimuth for each experimental site (Figure 3, 4, and 5). In situ velocity data at some sites includes measurements at different depths. Where no depth is indicated the measurement was taken at about 30 cm below the surface. Core velocities include data from several cores taken at each site with velocities from a single core indicated by linking the data points. Also plotted with an arrow is the azimuth of maximum expansion on overcoring for several tests. This azimuth may be regarded as the approximate direction of maximum compressive stress within the outcrop. Common labels designate velocity data from cores for which the azimuth of maximum expansion is plotted.

In all rocks there was a decrease in velocity ($\Delta V_p$) upon strain relaxation accompanying the release of stress. The $\Delta V_p$ varied from 1% in the Tully Limestone to 20% in the Milford granite. This $\Delta V_p$ strain relaxation is a function of magnitude of stress decrease ($\Delta \sigma$) as is predicted by the reloading experiments of Birch (1960). The stress dependent decrease in $V_p$ is best illustrated by comparing the $\Delta V_p$ between the Barretto and Fletcher quarries in the Milford granite (Figure 3). Our sample at the Barretto quarry comes from the quarry floor 50 m below the surface of surrounding land. The sample at the Fletcher quarry comes from less than 5 m below surface level in a zone of sheet fractures. It is intuitive to suppose that the natural process of sheet-fracturing near the land surface would act to relieve some of the stress within the Milford granite. Such is indicated by the magnitude of the in situ stress (see Plumb et. al., Part III) where more highly stressed granite at Barretto shows a larger decrease in $V_p$ on strain relaxation than does the granite at Fletcher.

Each rock exhibits a slightly different variation of in situ velocity with azimuth. Three types of in situ behavior are evident. One type is found within the Tully Limestone which shows no variation
in $V_p$ with azimuth (Fig. 4). Another type is found in the Milford Granite at Barretto quarry which shows a well defined anisotropy (Fig. 3). Rocks such as the Palisades diabase and the hornblende gneiss of the Adirondacks show neither a constant velocity with azimuth nor a well defined anisotropy. The outcrop of Palisades diabase is cut by joints that are responsible for the scatter in the time of flight to give a large scatter in the data. The scatter of velocities in the hornblende gneises is caused by inhomogeneities in stress and microcrack density (Plumb et al., Part II).

The velocity anisotropy for cores varies with lithology. Tully Limestone cores show no anisotropy (Fig. 4). Some cores of the Precambrian metasediment show an anisotropy in $V_p$ but the pattern is not consistent from core to core. The Palisades diabase shows a hint of anisotropy whereas many cores from the remaining sites are strongly anisotropic. The variation in velocity at one azimuth within a suite of cores from one outcrop is also noteworthy. Some cores such as the Conway Granite vary by more than 0.5 km/sec over a suite of seven cores (Fig. 3). In contrast the Tully Limestone, which shows no anistropy, also displays the least variability among cores with velocities clustering within 0.2 km/sec.

Regarding the relation between maximum expansion on overcoring and either in situ or core $V_p$ anisotropy, the rocks divide into three categories: 1) those with maximum expansion parallel to the maximum $V_p$; 2) those with maximum expansion parallel to the minimum $V_p$; and 3) those with no relationship because the rock has no consistent anisotropy. Maximum expansion parallel to maximum $V_p$ in situ, is seen in the Milford granite at the Barretto and Fletcher quarries, and the Conway granite at the Redstone quarry (Fig. 3). In all these examples the correlation between maximum $V_p$ and maximum expansion is not perfect largely because repeated stress measurements show a variation in magnitude and orientation of in situ stress even when experimental techniques are consistent from one measurement to the next (Plumb, Engelder, and Yale, part III).

The maximum expansion correlates with the minimum $V_p$ in the cores of the Machias Sandstone and possibly the Palisades diabase (Figs. 4 and 5). In the Machias Sandstone two in situ samples of $V_p$ were taken with the in situ $V_p$ at location I about 0.2 km/sec faster. The two in situ samples were taken within the same bed, a siltstone about 1 m thick. The samples were taken about 4 m apart with joints separating the two sample sites (Engelder and Geiser, 1980). The cores at location I were correspondingly faster. The difference in velocity also correlates with the in situ stress which was larger at location I.

The Tully Limestone and the Precambrian metasediment show a well clustered orientation for maximum expansion upon overcoring, but the lack of any consistant anisotropy preempts any correlation with $V_p$. Stress and $V_p$ were also measured on a free block of Milford granite (3m x 2m x 2m). The residual stress within the block was very low and had no consistent orientation. Here the relieved block showed no tendancy to store a residual strain related to a velocity anisotropy (Figure 3).
Discussion

Ultrasonic properties change upon strain relaxation and are clearly affected by the state of stress on a rock mass. Our data indicates that ultrasonic properties and the change in ultrasonic properties may be used to infer the state of stress. The data presented here is a first attempt to establish a relation between ultrasonic properties and both orientation and magnitude in situ stress. Figure 6 shows the correlation between mean strain change upon overcoring and change in $V_p$. A trend toward large $\Delta V_p$ with larger mean strain is seen. The large scatter in the data is attributed to inhomogeneities in most rocks causing both the variation of mean strain which is typical for overcoring measurements and the variation in velocities among cores in a single outcrop. The same trend is seen for a plot of differential stress versus $\Delta V_p$; the higher the stress the larger the velocity change on overcoring.

The mechanism for the $\Delta V_p$ as a function of stress is undoubtedly related to either the opening or growth of microcracks although identifying those microcracks that actually participate in strain relaxation is difficult. The difficulty is best illustrated for the granites where the in situ and core anisotropy are the same magnitude. On relaxation there appears a D.C. shift in the velocity curves (the velocity decrease is independent of azimuth) (Figure 3). The anisotropy both in situ and in the cores correlates with a set of transgranular cracks that are readily apparent in thin section (Plumb et. al., Part III). The D.C. shift in the velocity curves implies that microcracks opened on relaxation but no particular orientation of microcrack was favored. The well developed set of microcracks seemed not to participate in the relaxation. If other microcracks are responsible for the relaxation, they are not readily apparent. The $\Delta V_p$ on strain relaxation did not conform to Nur and Simmons' (1969) experiments which predicted that a maximum $\Delta V_p$ should occur in the direction of the maximum stress change.

In contrast, our measurements of $\Delta V_p$ in the Machias Sandstone conformed to Nur and Simmons' (1969) prediction that the maximum $\Delta V_p$ would occur in the direction of maximum stress change. Yet, even $\Delta V_p$ behavior in the Machias differed from the laboratory behavior documented by Nur and Simmons (1960) where the largest anisotropy occurred under load. The largest anisotropy occurred on removal of the load within the Machias whereas in situ and under load, the Machias Sandstone showed little tendency to be anisotropic (Figure 4). Basically relaxation was accompanied by the growth of a set of microcracks which have a preferred orientation. In contrast, the lab samples of Nur and Simmons (1960) had a oriented set of microcracks that closed under pressure and opened without growing on release of pressure.

The Tully Limestone appears to have behaved much like the granite. Microcracks participated in the strain relaxation but they did not have a preferred orientation and so no anisotropy developed upon relaxation.

No trend is apparent for a plot of velocity anisotropy $V_p^{max}/V_p^{min}$ within cores versus strain relaxation anisotropy $\epsilon_1/\epsilon_3$ (Fig. 7). This suggests that strain relaxation causes a
decrease in velocity that is independent of azimuth as is the case for the Milford granite (Fig. 3). In contrast to the general trend, the anisotropy of the Machias Sandstone increases on overcoring with the largest $\Delta V_p$ parallel to the maximum compressive stress. Here the opening of microcracks may be a direct consequence of stress relief.

The inhomogeneity of rock-bodies is illustrated by the variation in in situ velocity of the Machias sandstone between locations I and II (Fig. 4). It is further illustrated by the variation of properties cores from the Conway granite (Fig. 3) or the Precambrian metasediment (Fig. 5). This inhomogeneity is also evident by the variation in magnitude and orientations of the in situ strain relaxation data. Another type of inhomogeneity appears in the comparison of in situ velocities at different depths. Often the deeper measurements show a higher velocity as is the case for two sheets of Milford granite. One odd comparison is that the velocities in the free block of Milford granite at the Fletcher quarry are higher than the in situ velocities. Conventional behavior would be that the free block which is stress relieved should show lower velocities. Here again inhomogeneities in the Milford granite pluton may be the source for the errant behavior of the free block relative to the in situ velocities at the Fletcher quarry. In situ velocities at the Baretto quarry within the Milford granite pluton are even higher than the free block at Fletcher. So the possibility that the free block at Fletcher came from a location of high in situ velocity within the Fletcher quarry is very real.

Other experiments to record a change in compressional wave velocity on stress relaxation include those by Swolfs (1977) in the Barre granite and Swolfs et. al. (1981) in the Castlegate sandstone. These two experiments are plotted in Figure 6 and are consistent with the data presented in this paper.

Conclusions

1) Ultrasonic velocities decrease upon in situ stress relief.

2) The magnitude of change in ultrasonic velocities generally correlates with the magnitude of the stress relief.

3) Three trends are observed between orientation of maximum compression and core anisotropy. Anistropy may be used to infer the direction of in situ stresses axes. But directions of maximum stress can only be predicted if the mechanism causing anisotropy is known.

4) Anisotropy in the Tully Limestone is small compared with the Machias Sandstones; if these two sites are representative of the use of ultrasonic logs to infer state of stress, it is likely to be more successful in sandstones than limestones.

Acknowledgements

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Figure 1: The location of six sites where in situ stress and ultrasonic properties were measured.
Figure 2: A schematic of the apparatus for in situ ultrasonic tests.
Figure 3: Plot of compressional wave velocity ($V_p$) versus azimuth for the Milford and Conway granites. Both in situ and core velocities are shown as well as azimuth of maximum expansion on overcoring. The depth of the in situ velocity measurement is also noted in some cases. The Milford granite was measured at two quarries 20 km apart. At the Fletcher quarry a free block of granite was tested. The core and overcoring labels correspond to data on strain relaxation which appears in Plumb, Engelder, and Yale (Part III).
Figure 4: Plot of compressional wave velocity ($V_p$) versus azimuth for the Machias sandstone. Both in situ and core velocities are shown as well as azimuth of maximum expansion on overcoring. The core and overcoring labels correspond to data on strain relaxation which appears in Engelder and Geiser (1980).
Figure 5: Plot of compressional wave velocity ($V_p$) versus azimuth for the Tully limestone, the Palisades diabase, a hornblende gneiss, and a Precambrian metasediment (Sagamore). Both in situ and core velocities are shown as well as the azimuth of maximum expansion on overcoring. The in situ velocity measurement is also noted in some cases. The core and overcoring labels correspond to data on strain relaxation which appears in Plumb, Engelder, and Sbar (Part II).
Figure 6: Change in average velocity between in situ conditions and the relieved core versus a) mean strain, and b) deviatoric stress. The average velocity is determined adding all available azimuthal data (e.g. 12 $V_p$ measurements per core) and dividing by the number of measurements.
Figure 7: The ratio of maximum and minimum velocity on cores versus the ratio of maximum to minimum strain on relaxation of cores.
THE CORRELATION BETWEEN THE ORIENTATION OF INDUCED FRACTURES AND IN SITU STRESS OR ROCK ANISOTROPY

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ABSTRACT

An inflatable packer was used to further test the hypothesis that internally pressurized boreholes fracture parallel to the maximum compressive stress ($\sigma_1$). A packer was set in shallow (depth < 2 m) vertical boreholes where in situ stress and rock anisotropy had been measured. The pressure inside the packer was increased until axial fractures occurred. Packer fracturing differs from hydraulic fracturing because the packer prevents fluid from penetrating the rock during the fracturing process.

Borehole tests were conducted in fractured igneous rocks of New Hampshire (3 locations), mildly deformed sedimentary rocks of western New York (2 locations), metamorphic rocks from the Adirondack Mountains (2 locations), plus deformed igneous and sedimentary rocks in the vicinity of the San Andreas fault zone near Palmdale, California.

The orientations of packer induced fractures were correlated with either in situ stress or rock anisotropy. Seventy percent of all fracture orientations were within 30° of $\sigma_1$. Less than 10% of the fractures were more than 30° from the orientation of $\sigma_1$ or a microcrack fabric. These experiments show that both rock fabric and stress influence the orientation of fracture propagation. The misalignment of fractures with either $\sigma_1$ or a rock fabric results when the stress axes are not orthogonal to the fabric.

INTRODUCTION

When interpreting hydrofracture stress measurements, an important assumption is that the plane of the induced fracture contains the direction of the maximum compressive stress. The relatively few studies of rock fabric associated with hydrofracture experiments indicate that rock fabric may strongly influence the orientation of induced fractures [Overbey and Rough, 1968; Smith, Holman, Fast, and Corlin, 1978; Abou-Sayed, Brechtel, and Clifton, 1978; Smith, 1979; Dula, 1981]. These studies show that hydraulic fractures may have propagated along existing planes of weakness either parallel with a dominant joint set or preferentially oriented microcracks.

To better understand the potentially competing effects of rock fabric and in situ stress on controlling the fracture propagation direction, borehole fracturing experiments were performed in rock with known in situ stress and microfabric. In order to perform these experiments, an unconventional technique of fracturing boreholes was developed. The experiment required a simple rugged clean method of fracturing boreholes that required few assumptions and was generally described by the existing theories of borehole frac-
turing [Hubbert and Willis, 1957; Haimson and Fairhurst, 1970]. The technique is called packer fracturing, and provides a method of inducing axial borehole fractures at any depth without introducing fluids into the fracturing process. To date fractures have been created in shallow boreholes (d \( \leq 2 \) m) above the water table, so that they can be seen from the ground surface (Figure 1).

THE PACKER FRACTURE TECHNIQUE

Field Procedure

The field procedure consisted of reoccupying and fracturing holes created during previous stress measurement programs. Boreholes selected for the fracturing experiments were cleaned and inspected for preexisting fractures. Once a 12" interval was chosen for the experiment, the packer was inflated with hydraulic fluid supplied by an Enerpac hand pump (Figure 1). Initial pressurization was just enough to support the packer in the desired interval. All pressures were measured by a pressure gauge located at the pump and recorded by hand in a notebook. Next a piezo-electric crystal (PZT) was located as close to the top of the packer as possible. The PZT was used to detect acoustic emissions (AE) associated with rock fracture. In the early experiments (e.g., sites 11, 12, 13), the PZT was coupled to the outcrop using a silicone putty. This method was also used when the top of the packer could not be submerged in water. The most sensitive arrangement is obtained by submerging the PZT in water near the top of the packer. The output of the PZT was amplified by a variable gain (20-60 db) broadband (0.5-4.0) KHz amplifier, and monitored by both a portable oscilloscope and headphones. Pressure inside the packer was increased slowly so that the AE activity could be watched. Borehole fracturing (breakdown) was indicated by the characteristics of AE activity. A characteristic breakdown AE was a loud (headphones) large-amplitude, and relatively low-frequency (scope) signal. In contrast, smaller grain-scale (?) cracking consisted of relatively lower-amplitude, higher-frequency signals. In some cases a swarm of the latter type AE replaced or followed the breakdown AE in crystaline rock [c. f., Zoback et al., 1980; Warpinski et al., 1980] (Figure 2). Fractures were most easily recognized as impressions on the untreated packer surface; some were visible using a downhole light and mirror system. In other cases fractures were enhanced when the borehole walls were dampened with a sponge. Initially as the fracture absorbed moisture, it appeared dry relative to the walls. Later, as the walls dried by evaporation, the fracture remained relatively
wet. This was the most definitive method of locating the fracture. If no fractures were found, the packer was repositioned and repressurized. During the second loading cycle, pressures were usually taken higher than the suspected initial breakdown pressure. This was done to extend and accentuate the fracture. Repressurization was stopped either when loud AE were heard or pressures approached the packer strength (300 bars); to avoid rupturing the packer or forming multiple fractures. Occasionally no strong AE were detected and several fractures formed because the borehole was "overpressurized". Such fractures could have formed at planes of low tensile strength or at one of several circumferential stress concentrations \( \sigma_{QQ} \) that exist in anisotropic materials [Cleary, 1979]. Unlike hydrofractures which are extended by continuous pumping beyond breakdown, additional packer induced fractures will develop instead of extending the primary fractures [Newman, 1969]. If this happens without additional tests, there is no way to know which fracture formed first. Only combined data from stress measurements, rock anisotropy, and fractures formed at different locations in the same outcrop will indicate the dominant stress concentration.

The following are notable peculiarities of the packer fracture technique. In certain circumstances, the propagation of fractures was 'heard'. This occurred when horizontal fractures formed. This extension sounds like punctuated static heard over an AM radio during a thunder storm. When horizontal fractures extended in the absence of pre-existing fractures, no vertical fractures were generated regardless of the pressure applied to the packer.

Sketches of fractures formed at four locations are illustrated in Figure 2. Site 8 is a Miocene sandstone; notice the two fracture sets and their interaction. Site 10 is an Eocene sandstone; notice the irregular fracture trace and the fracture interaction with the pre-existing joints (dotted sinusoidal curves) and steps in the bore produced by the drilling (dotted horizontal line). Site 1 is a Pennsylvanian granite. Site 4 is a Devonian siltstone; notice the vertical fractures are confined between the horizontal bedding plane fractures. Fracture traces at sites 1 and 8 are more continuous, straighter, and tend to be diametrically opposed on the borehole walls compared to those at sites 4 and 10. The example from site 10 shows that packer fractures may be induced in naturally fractured rock. If laboratory tests indicate that the resulting fracture azimuths are not controlled by strength or modulus anisotropies, orientations of local stress trajectories are obtained. It is also possible to qualitatively determine the level of shear stress acting on preexisting fractures by studying the intersections of natural and induced fractures.

Fracture Criteria

According to Hubbert and Willis [1957], hydraulic fractures will be initiated on the borehole wall where tensile tangential stress \( \sigma_{QQ} \) first exceeds the tensile strength of the rock. Assuming the borehole is a smooth and cylindrical cavity in an isotropic, impermeable elastic solid subjected to internal pressure \( P \) and with one principal stress aligned with the borehole axis, the
tangential stress at the borehole wall \((r = a)\) can be written as

\[
\sigma_{\theta\theta} r=a = \sigma_1 + \sigma_2 - 2(\sigma_1-\sigma_2)\cos 2\theta - P \tag{1}
\]

where \(\sigma_1 (\theta=0)\) and \(\sigma_2 (\theta=90)\) are the maximum and minimum principal stresses (compressive stresses are positive). According to equation (1), the minimum tangential stress first occurs at \(\theta=0\), thus

\[
\sigma_{\theta\theta} r=a = 3\sigma_2 - \sigma_1 - P \tag{2}
\]

As the internal fluid pressure increases, the tangential stress at the cavity wall decreases until it equals the tensile strength of the rock \(T_o\). At the pressure \(P_f\), a fracture will form at \(\theta=0\) such that

\[
P_f = 3\sigma_2 - \sigma_1 + T_o \tag{3}
\]

In principle, packer fractures are produced by similar boundary conditions. Instead of a fluid pressure boundary condition \(\sigma_{rr} r=a = P\), packer induced normal stress \(\sigma_o\) must be used

\[
\sigma_{rr} r=a = P_o (1 - \gamma/P_o) = \sigma_o
\]

The term \(\gamma\) is a measure of the packer resistance to inflation. It includes expressions describing packer deformation resulting from internal pressure \(P_o\) as well as the difference in radius between the packer and borehole [Warren, 1981]. Figure 3 is a plot of the three components of borehole stress induced by the inflated packer normalized to the packer induced normal stress \(\sigma_o\). Note that the sign of \(\sigma_{\theta\theta}\) is opposite to \(\sigma_{rr}\) and reaches a maximum value of .97 \(\sigma_o\) near the center of the packer. A packer induced fracture will form at \(\theta=0\) when the normal contact stress induces tensile tangential stress just equal to the tensile strength of the rock

\[
P_o (1 - \gamma/P_o) = 3\sigma_2 - \sigma_1 + T_o \tag{4}
\]

An important difference between hydraulic and packer induced fractures is the absence of fluid-rock interaction for packer fractures. Since the packer isolates fluid from the rock, equation (1) is expected to be a good representation of the packer fracture process even in permeable solids. Laboratory studies have shown that when rock is impermeable to fracturing fluids breakdown pressures are higher than in the case when the rock is permeable [Haimson and Fairhurst, 1970; Haimson and Avasthi, 1975; Zoback et al., 1976; Zoback et al., 1977; Medlin and Masse, 1979]. Since borehole walls all contain flaws, fluid interaction with these flaws will lead to breakdown pressures which are low relative to the assumed impermeable rock [Newman, 1969; Abou-Sayed et al., 1978]. For this reason, packer fracture breakdown pressures would be expected to be more representative of stresses \((3\sigma_2-\sigma_1) r=a\) than hydraulic fracture breakdown pressures in the same rock.
A second difference between packer and hydraulically induced fractures is the radial distance fractures propagate. Packer induced fractures can only propagate as far as tensile stresses can be induced near the borehole. Since $\sigma_{\theta\theta} = a^2/r^2$, fractures will only be expected to extend to several borehole radii. If borehole fractures nucleate at existing flaws, fractures will spontaneously rupture a distance of $r = 1.6a$ [Newman, 1969]. Beyond $r = 0.6a$, the stress intensity factor at the crack tip decreases with length, requiring greater pressure for each additional increment of crack extension. However, the propagation distance of hydraulic fractures is limited mainly by the ability of the pumping system to deliver a critical fluid pressure into the fracture.

THE EXPERIMENTS

Site Locations

Near-surface fracturing experiments were made at 13 sites within the United States (Figure 4; Table 1). These sites were selected because information was available on in situ stress. Sites 1 and 2 are in operating granite quarries near Milford, New Hampshire, and located in the same grey, fine-grained, undeformed Pennsylvanian aged pluton [Aleinikoff et al., 1979]. Site 3 is located in the abandoned Redstone Quarry near North Conway, New Hampshire [Dale, 1923; Plumb, Engelder, and Yale, 1982]. The rock is a coarse-grained pink body of Jurassic aged Conway granite [Fowland and Faul, 1977]. Sites 4 and 5 are located in mildly deformed Devonian sedimentary rock of the Appalachian plateau, western New York State [Engelder and Geiser, 1979]. Site 4 is in the Machias formation, a silty shale located at Belmont, New York, and described in Engelder and Geiser [1980]. Site 5 is in the Tully limestone located at Ludlowville, New York. Sites 6 and 7 are located in the Precambrian metamorphic rock of the central Adirondack Mountains, New York [McLelland and Isachsen, 1980]. Site 6 is in the Blue Mountain formation, a hornblende gneiss located on the northeast shore of Blue Mountain Lake [De Waard, 1964]. Site 7 is in a quartz rich metasedimentary rock near Raquette Lake, New York [Plumb, Engelder, and Sbar, 1982]. Sites 8 through 13 are located in a variety of rock types adjacent to the San Andreas fault, Palmdale, California, and described by Sbar et al. [1979].

At each site, in situ stress data were gathered using 'doorstopper' strain relaxation measurements [Sbar et al., 1979]. In addition to the doorstopper measurements, static and dynamic tests were performed on stress relieved cores to determine the orientation and degree of elastic anisotropy. Dynamic anisotropy tests consist of measuring sonic velocity at 15° or 30° intervals around the axis of doorstopper cores. In Table 1, $\varepsilon_1$, $\sigma_1$, $\varepsilon_2$, $\sigma_2$ are the maximum (1) and minimum (2) principal strain and stress, respectively. The $\theta$ with subscript refers to the horizontal bearing of the subscripted quantity. $\theta_{\varepsilon_1}$ is the bearing of the maximum principal strain relaxation, $\theta_{\sigma_2}$ refers to the least
compressible (stiffest) horizontal direction of the core, $\theta V_p$ represents the dynamically stiff direction, $\theta \text{frac}$ is the bearing of the intersection of the induced fracture with the borehole wall; and $P_f$ is the approximate fracture initiation pressure. When no stress or anisotropy data are available for the interval of borehole fractured, site averaged data are provided for comparison in Table 1.

Experimental Results

The data from Table 1 are summarized in the four histograms of Figure 5. These histograms represent frequency of observations versus the angle between the bearings of each induced fracture and the bearings of: 5a) the maximum principal strain relaxation $\varepsilon_1$; 5b) the maximum principal stress $\sigma_1$; 5c) the maximum sonic velocity $V_p$; and 5d) the minimum compressibility $\beta_2$. For example, if induced fractures were oriented within 5 degrees of $\sigma_1$, they would contribute to the histogram in the first column on the left in 5b. For discussion purposes, a correlation is defined to exist where the angular separation between the bearing of induced fractures and any other parameter (e.g., $\sigma_1$, $V_p$, $\beta_2$, etc.) is less than 30°. For reference to Table 1, each element of the histogram contains a number corresponding to the site of the observation.

Note that the majority of induced fractures have bearings close to the bearings of maximum in situ compressive stress $\sigma_1$. Seventy percent of the fracture traces were oriented within 29° of $\sigma_1$ and about 3/5 of this 70% are within 19° of $\sigma_1$. Notice that the correlations represent the majority of rock types and deformation intensities sampled in this study. An exception is the silty shale at site 4, where all fractures are uncorrelated with measured in situ stress. Notice also that not all fractures from a given site are necessarily correlated with measured stress, for example, sites 2 and 11. The correlation is better in Figure 5b than in Figure 5a. This is a result of rock anisotropy. If rocks are anisotropic, the maximum principal strain relaxation in general will not coincide with the direction of maximum compressive stress $\sigma_1$. When a correction is applied to the strain relaxation data for the effect of rock anisotropy, two changes occur. First, the azimuths of $\varepsilon_1$ and $\sigma_1$ are no longer the same; second, for a given site, the variance of azimuths of $\sigma_1$ is less than for $\varepsilon_1$ [Sbar et al., 1979; Plumb, Engelder, and Sbar, 1982]. The differences between Figures 5a and 5b reflect this correction, but more importantly, they imply that stress and not strain is controlling the fracture azimuth.

The relationship of induced fractures with two measures of rock anisotropy are shown in Figures 5c and 5d. The sonic velocity and compressibility anisotropy are most indicative of an anisotropy caused by a microcrack fabric [Simmons, Todd, and Baldridge, 1975; Engelder, Sbar, and Kranz, 1977; Plumb, Engelder, and Sbar, 1982; Plumb, Engelder, and Yale, 1982]. If rocks contain a single preferred orientation of microcracks, the bearings of both the maximum $V_p$ and $\beta_2$ will be generally parallel to the microcracks. Experience has shown that ultrasonic velocity measurements on rocks
containing more than one microcrack fabric can identify the fabric most nearly aligned with in situ stresses. In contrast, the total crack population responds during biaxial compressibility tests, so that individual populations are hard to identify [Plumb, Engelder and Sbar, 1982; Plumb, Engelder and Yale, 1982].

Figure 5c represents the angular relationship between fast sonic velocity ($V_p$) and the orientation of induced fractures. Some of the scatter in this diagram is attributable to the 15° or 30° interval between sonic velocity measurements. This figure shows that 52% of all packer induced fractures were oriented within 29° of the azimuths of fast sonic velocity ($V_p$). 38% were within 14° of $V_p$. Considering only data from crystalline rocks, 63% of all fractures were oriented within 29° of $V_p$ and 46% were within 14° of $V_p$. All fractures produced in crystalline rocks were correlated with a microcrack fabric detected either with sonic velocity measurements or from observations of thin sections (Table 1, Figure 6).

Figure 5d shows the correlation between biaxial compressibility anisotropy and the induced fractures. This figure shows a bimodal distribution with fractures clustering near the orientations of the stiff and compliant directions in the rock. This bimodal distribution reflects the tendency for measured in situ stresses to be correlated with either stiff or compliant directions in the rock [Sbar et al., 1979; Plumb, Engelder, and Sbar, 1982; Plumb, Engelder, and Yale, 1982]. Figure 5d shows that 48% of all fractures were correlated within 30° of the sample stiff direction ($\beta_2$) and that 37% were correlated within 30° of $\beta_1$. The majority of sites that fracture near $\beta_2$ are crystalline rock, whereas the majority of sites that fracture near $\beta_1$ are sedimentary rock. Considering only data from crystalline rocks, we find that 57% of induced fractures correlate with $\beta_2$ and only 20% with $\beta_1$.

Only 15% of all the fractures produced in this study were not correlated with either $\sigma_1$, $V_p$, or $\beta_2$. Excluding from these data those fractures which are known to coincide with a crack fabric observed in thin section, then only 9% of the data are uncorrelated with in situ stress or measured rock anisotropy. Having made these general observations, we will now focus on some details from several of the sites.

### New Hampshire Data

The most comprehensive data set in Table 1 comes from the experiments performed in the New Hampshire granite quarries (sites 1, 2, 3). These quarries were of interest because they all reportedly possess a similarly oriented microcrack fabric [Dale, 1923; Wise, 1964]. This microcrack fabric is inferred from the regional similarity of quarrying planes used for splitting the granite. The granite splits most easily (rift plane) in the horizontal plane; the next easiest splitting plane (the grain plane) is vertical and strikes about east-west. The most difficult plane to split (the Hardway) is orthogonal to the other two and is vertical north-south. In general, the granites possess an orthorhombic symmetry with the axes of symmetry coincident with the poles to an
orthogonal microcrack fabric. These microcracks impart both a modulus and tensile strength anisotropy to the granite [Dale, 1923; Balk, 1927; Wise, 1964; Douglas and Voight, 1969; Plumb, Engelder, and Yale, 1982].

In situ velocities were measured to further study the relationship between microcracks and strain relaxation in the quarries [Plumb, Engelder, and Yale, 1982]. Results of the in situ velocity experiment indicated that the magnitude and orientation of anisotropy in the outcrop was about the same as in stress relieved cores. The results implied that the same cracks inferred from core velocity tests were open in situ and that the orientation of anisotropy was unchanged as a result of stress relaxation.

Figure 6 shows poles to near vertical microcracks for the three granites studied. These are lower hemisphere plots of poles to quartz and feldspar cracks observed in horizontal thin sections using standard universal stage procedures [Plumb, Engelder, and Yale, 1982]. The plane of each diagram is a horizontal rift plane; data are microfractures measured in a thin section cut parallel to the rift, and plotted on an equal area lower-hemispheric projection. Separate diagrams for quartz and feldspar are prepared for site 2 to illustrate two different preferred orientations of cracks, each having a different effect on the mechanical properties of the granite. Sites 1 and 3 (the Redstone and Barretto quarries, respectively) have only one strong crack fabric which coincides with the grain. Induced fractures in these quarries are aligned with both in situ stress and the crack fabric. Therefore, at these sites, the dominant factor responsible for locating the borehole fractures cannot be identified.

Two vertical microcrack sets are found at Site 2 (the Fletcher Quarry) (Figure 6). One set is associated with predominantly quartz crystals and represents the quarry grain. The other is found only in feldspar and is not recognized by the quarrymen. The maximum compressive stress ($\sigma_1$) in the outcrop is aligned with the feldspar cracks. Also, in situ and core velocity anisotropy are maximum parallel with the feldspar cracks. Three of the four packer fracturing experiments induced fractures that were aligned with $\sigma_1$, $V_p$, and the feldspar cracks. The fourth experiment fractured along the quarry grain direction. Another set of experiments at site 2 was performed in a large oriented block of granite quarried several years earlier [Plumb, Engelder, and Yale, 1982]. This block contained only low level residual stresses (Table 1). Fractures induced in the block nucleated primarily in the quarry grain direction but a few nucleated in the feldspar crack direction.

Results from all three quarries imply that the greatest stress concentration around a borehole occurs at the tips of cracks supporting the least normal stress (Figure 6). In situ $\sigma_1$ at site 2 was maximum nearly parallel with the feldspar cracks, resulting in high normal stress across the quarry grain cracks. Therefore, induced fractures followed the feldspar cracks. Fractures in the stress-free block at site 2 followed the tensile strength anisotropy of the rock, which is dominated by the quartz grain cracks but is also affected by the feldspar cracks.
Adirondack Data

Packer fracture experiments were made at two sites in the Adirondack Mountains of New York. These were the two sites with the most knowledge of the rock properties and in situ stress. All of the in situ stress and mechanical data discussed here are from Plumb [1979] and Plumb, Engelder, and Sbar [1982]. At both sites, fractures have been influenced by in situ stress and by microcracks which are subparallel to $\sigma_1$.

Biaxial and triaxial compressibility tests on rock from site 6 indicate that the orientation of $\beta_2$ is dependent upon the level of confining pressure. This stress dependence is apparently caused by a set of cracks associated with the gneissic fabric. The orientation of maximum $V_p$ made at atmospheric pressure and $\beta_2$ measured at low confining pressures are both parallel to the foliation (N90°W), but at biaxial loads of 60 bars, $\beta_2$ is rotated to N65°W. Results of compressibility tests imply that cracks open in situ are striking about N65°W, coincident with the direction of the induced fracture.

At site 7, the orientations of both in situ stress and rock anisotropy are functions of position. Large differences between the orientations of least compressibility and maximum ultrasonic velocity are indicative of a complex microstructure. The rotation of the orientation of $\beta_2$ with increasing confining pressure means that orientation of anisotropy is also stress dependent. This effect could be caused by low aspect ratio cracks aligned with the foliation and higher aspect ratio cracks oriented $\approx 25^\circ$ clockwise of it. On each core tested for both compressibility and velocity anisotropy, velocity maxima occur parallel to the low pressure orientation of $\beta_2$.

Packer induced fracture orientations are correlated with the bearings of $\sigma_1$, maximum ultrasonic velocity, and the strike of outcrop joints. Fracture strikes are better correlated with the average direction of $\sigma_1$ as determined from all three overcoring techniques than with $\sigma_1$ estimated from the nearest doorstopper measurements. This is reasonable because induced fractures are influenced by stresses acting on volumes of rock much larger than the volume of a single doorstopper core. These results suggest that if small-scale inhomogeneous stress fields are present in a rock mass, the orientation of induced fractures with dimensions much larger than the individual stress fields will be correlated with average direction of $\sigma_1$. Results indicate that here, the packer fracture bearings are primarily influenced by the average in situ stress and, secondly, by a microcrack fabric aligned with $\sigma_1$. This means that the scale of the stress field as well as stress-dependent anisotropy are important parameters influencing fracture propagation.

California Data

Six sites located near the San Andreas fault and Palmdale were reoccupied for packer fracturing experiments, three of the sites near the fault were in sedimentary rock and three sites to the north in the Mojave were in crystalline rock. Figure 7 shows the
bearing of induced borehole fractures, and a summary of stress measurements made in the same region by various methods [Sbar et al., 1979; Dahlgren et al., 1979; Zoback, 1980]. The left-hand figure shows in situ stress measured by selected hydrofracture measurements [Zoback et al., 1980] (triangles); deep overcoring measurements [Dahlgren et al., 1980; Sbar et al., unpublished data] (solid circles); and shallow overcoring [Sbar et al., 1979] (solid circles, dotted lines). The right-hand figure shows the orientation of packer induced fractures; site L is in the same location as site 8. Regional stress directions are defined as the predominant direction of the maximum horizontal compression measured in an area comprising many widely spaced sites. The regional stress near Palmdale, California is taken to be north-northwest. Local stresses are a subset of regional stresses and are observed at one or many closely spaced points.

It is commonly assumed that induced hydraulic fractures propagate in the direction of \( \sigma_1 \). Fracture propagation directions at sites 8-11 show excellent agreement with measured regional stress directions. Sites 12 and 13 do not fracture in the direction of regional stress. However, only sites 8, 9, 12, and 13 fractured in the direction of measured stress. Based on results from deep overcoring measurements [Sbar et al., in preparation] and deep hydrofracture measurements [Zoback et al., 1980], fractures at sites 8 and 9 are aligned with the regional tectonic stress, while fractures at site 13 are aligned with local stresses that exist in the upper 20 m [Dahlgren et al., 1979]. There are no deep stress measurements at site 12, but packer fractures are controlled by stresses measured in these boreholes. Site 10 fractures are controlled by anisotropy but no correlation can be made at site 11. Boreholes of both sites 10 and 11 are subject to very low shear stress (\( \leq 5 \) bars). As a consequence, a tensile strength anisotropy probably controls the fracture azimuth. None of the tests performed on site 11 rock indicated a NNW anisotropy. However, it is conceivable that an unidentified regional fabric exists in these rocks and is responsible for the similarity in fracture azimuths between sites 8, 9, 10, and 11 [Wise, 1964]. These results point out that several different mechanisms are responsible for producing 'regional' similarity in the orientation of induced borehole fractures.

**Western New York**

Borehole tests in the sedimentary rocks of western New York (sites 4 and 5) gave inconclusive results. No simple axial fractures were produced at either site. The fractures that formed were usually short, discontinuous features distributed over a relatively broad azimuthal range \( \pm 5^\circ -30^\circ \) (Table 1; Figure 4). At both sites, vertical fractures formed only after the rock was highly deformed with the packer. This deformation was characterized by horizontal bedding partings, short vertical NNW-trending fractures confined to individual beds, and minor fragmentation. The NNW-trending fractures formed at site 5 resulted only after the hole was rapidly loaded to 300 bars. At slow pressurization rates, the Tully limestone at site 5 'yielded', taking increased packer volume
without allowing pressure to build above 20-30 bars. This limestone is cut by E-W trending planes of solution cleavage spaced at about 1 cm. Upon pressurization to about 20 bars, rock 'yielding' may have resulted from opening of cleavage planes intersecting the borehole and closing of planes exterior to the borehole. The mean trend of the fractures at site 5 is near $\phi_1$, but the fractures at site 4 show no clear relationship to any of the measured rock properties (Table 1; Engelder and Geiser, 1980). Experiments at these sites show that sedimentary rocks under low shear stress fracture differently than other sedimentary rocks subjected to higher shear stress, e.g., sites 8, 9.

SUMMARY AND CONCLUSIONS

Packer induced borehole fractures provide new information regarding the orientation of in situ stress or rock anisotropy and potentially the magnitude of stress contrast ($3\sigma_2-\sigma_1$). Improvements in determining fracture initiation pressure could provide constraints on the calculated value of $\sigma_1$ if compared with hydraulic breakdown pressures in the same well. Since fluids do not influence the fracturing process, this technique could be useful in fractured boreholes.

The unique aspect of this study was that in situ stresses and the orientations of rock anisotropy were measured at the same borehole locations that were fractured. Note that stresses measured in nearby holes may be meaningless for interpreting the fracture azimuths unless many measurements in different locations demonstrate uniformity in the stress field.

The three most important results of this study are: 1) 91% of all fractures produced are correlated with either $\sigma_1$ or rock anisotropy; 2) the majority of fractures (70%) were aligned with the direction of $\sigma_1$; and 3) many of these fractures as well as 15% of those not aligned with $\sigma_1$ were related to rock anisotropy aligned with the packer fractures. The mechanisms responsible for this correlation are not yet understood, particularly for some of the sedimentary rocks. It is believed that microcracks or other stress risers interacting with the in situ stress [Abou-Sayed et al., 1978] or simply stress concentrations resulting from elastic anisotropy play important roles in controlling the orientation of induced fractures [Savin, 1961; Lekhnitskii, 1968; Cleary, 1979].

Anisotropy in crystalline rock caused by microcracks was strongly correlated with the orientation of induced fractures. Every fracture produced in granite quarries (sites 1, 2, 3) followed a known crack population in that rock. Fractures produced in the metamorphic rock (sites 6, 7) followed a microcrack set that was observed or inferred from laboratory tests on the same rock. Because in situ stresses typically are aligned with a microcrack set in these crystalline rocks, it is unclear whether the cracks or the stress controls the orientation of induced fractures. Results from site 2 suggest that the interaction of existing cracks and the prevailing stress field controls the fracture azimuth. That is, the packer induced fracture propagates along the microcrack set having the largest crack tip stress intensity. Finally, velocity
anisotropy tests on oriented cores of crystalline rock may predict the azimuth of wellbore fractures.

Geophysicists commonly assume that a regional similarity in the orientations of hydraulic fractures implies a uniformity in the orientation of stress. While this may be a reasonable assumption, the results presented here show that stress is not the only mechanism responsible for this similarity. It was shown that similarly oriented fractures within that region were controlled by different mechanisms. The common factor responsible for controlling the orientation of borehole fractures in rocks supporting low shear stress may be a regionally uniform rock anisotropy [c.f., Wise, 1964].

Finally, it could be argued that near-surface fracturing experiments are not representative of deep in situ experiments. However, the factors which control the orientation of fractures, namely shear stress and anisotropy, are present at all sites. Stresses measured near the surface might not show the same orientation or magnitude as measurements made at depth but the relationship between fracture azimuth, in situ stress, and rock anisotropy should be comparable where similar levels of shear stress exist. Figure 8 is a plot of shear stress versus depth for several sets of hydrofracture data. Also plotted are shear stresses in rocks fractured in this study. Numbers refer to packer fracture sites of Table 1; geometric symbols are hydrofracture data; filled squares Haimson [1978]; filled circles Haimson [1978]; open triangle Zoback et al. [1977]; open hexagon Raleigh et al. [1973]. Shear stress magnitudes at the packer fracture sites are comparable to most of those observed at depth by hydrofracturing. Therefore the mechanisms affecting fracture propagation discussed in this paper should be applicable to deep hydrofracture experiments where shear stresses are less than about 100 bars.

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Terry Engelder, David Yale, Steve Brown, and Jim Mori all helped with the data collection. I want to thank Terry Engelder and Ted Koczynski for their support and valuable discussions. William Warren of Sandia National Laboratories, Albuquerque, New Mexico, kindly provided an unpublished analysis of packer induced stresses. The Adirondack Museum, Blue Mountain Lake, New York, the Barretto Brothers Granite Company, Milford, New Hampshire, and the Maine and New Hampshire Granite Company, Mason, New Hampshire, all granted permission to use their land and facilities. This work was supported by the U. S. Geological Survey, the Department of Energy, The Nuclear Regulatory Commission, and the New York State Energy Research and Development Agency's contracts 14-08-0001-17703, DEAC 76R04054, and 04-81-180, respectively. Terry Engelder, Chris Scholz, and Barry Raleigh critically reviewed this manuscript. Lamont-Doherty Geological Observatory contribution No. 0000.
REFERENCES


Table 1. Experimental Data

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<th>Breakdown Pressure (bars)</th>
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Note: AE = Acoustic Emission; PZ = Packer; N = North; S = South; W = West; E = East; NS = North/South; NE = North/East; NW = North/West; SE = South/East; SW = South/West; ** = Fracture also formed.
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no distinct breakdown, pressurized to 300 bars

AE most intense at 15 to 25 bara, no fracture formed, borehole "yields"

AE at 15-20 bara

AE at 20 bara,

packer volume increased abruptly at 30 bara
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**Hole average values**

**Bureau of Mines core tested.**

Fig. 1 Schematic diagram of the packer fracturing technique showing the pressurizing system and the acoustic emissions method of detecting rock fracturing.
Fig. 2. Sketches of packer fracture traces seen on borehole walls for different rock types. Each figure shows (360° or 180°) the borehole wall as a plane. Depth from the outcrop surface is indicated at the left of each figure.
Fig. 3. Plot of the three components of borehole stress normalized to packer contact stress $\sigma_0$, (see text) for the borehole interval shown in the lower left side of the figure. Note that $\sigma_{00}$ is maximum near the center of the packer, $(z/L = 1)$. 
Fig. 4. Location map of the thirteen sites.
Fig. 5. Histograms showing the correlation of fracture orientations with the azimuth of in situ strain (5a); in situ stress (5b); maximum ultrasonic velocity (5c); and radial compressibility (5d).
Fig. 6. Lower hemisphere plots of poles to microfractures in quartz and feldspar grains for the three New Hampshire granites. Microfracture orientations were measured from thin sections cut parallel to the horizontal plane using a universal stage microscope.
Fig. 7. Comparison of the orientation of packer induced fractures with the orientation of the maximum compressive stress measured in the same region.
Fig. 8. Comparison of shear stress at near-surface packer fracturing sites with shear stress observed at depth by hydraulic fracturing.
A Note on Downhill Detection of Conductive Fractures

W.F. Brace

Department of Earth and Planetary Sciences
Massachusetts Institute of Technology
Cambridge, Massachusetts 02139

ABSTRACT

Some of the fractures which intersect a borehole in crystalline rocks may be hydraulically conductive and their identification prior to hydrofracture may be crucial. Neither televiewer, core examination, nor conventional geophysical logging methods are currently able to identify these fractures with any consistency. Identification by hydrologic tests (drill stem or packer tests, for example) is the usual method. New acoustic logging techniques based on tube waves appear to be highly promising.

INTRODUCTION

Stress measurement by hydraulic fracturing is usually performed in unfractured intervals, that is, in sections of the borehole which seem to be free of fractures. There are several reasons for this. Fractures may influence the direction which the hydrofracture follows as it leaves the hole. Even if this influence is minor, identification of the hydrofracture trace may be difficult if numerous fractures already intersect the borehole. Finally, interpretation of the pressure-time curve may be complicated if fractures already exist in the vicinity of the interval being tested. The worst fractures, from the standpoint of affecting a stress measurement, are probably those which have an appreciable hydraulic conductivity; such fractures extend some distance away from the borehole and may be part of a network of conductive fractures. A first task in planning hydrofracture must be identification and location of these fractures. One way of doing this is by extrapolation of outcrossing fractures [1, 2]. We will not consider this, however, but rather focus attention on identification of conductive fractures in the borehole itself.

A number of methods are used to detect fractures in a borehole, ranging from those which rely on direct observation to those which sense the fracture indirectly. Few methods
give the actual conductivity of a fracture. The purpose of this note is to review the methods available, particularly the one or two which seem able to discriminate a hydraulically conductive fracture from those that die out a short distance from the borehole. The methods fall into three categories, those that rely on hydrologic measurements in the borehole, those that are based on conventional geophysical logs, and those that involve so-called "tube waves".

Hydrologic methods

It is convenient to distinguish matrix flow from fracture flow, where these terms refer to fluid flow through intact centimeter-sized samples of rock, and flow through natural fractures such as joints, faults, or bedding planes, respectively. In rocks like sandstone, matrix and fracture permeability (the quantitative measure of flow rate per given pressure gradient) may be fairly close [3, 4], but in crystalline rocks they may differ by many orders of magnitude. Several measurements have been collected in Figure 1, in which permeability of a fractured and an unfractured sample of the same rock are compared. Except for the Laramie granite, the difference in these two numbers is a factor of $10^6$ or more. The Laramie fracture was a natural joint, in part clay-filled [5].

Permeability, or its equivalent, hydraulic conductivity, is measured in a drill hole by a number of techniques [6]. An interval in the borehole is chosen for study and the flow across this interval measured. An entire borehole can be "logged" by successive measurements [7, 8]. An example of a fairly complete hydrologic log is given in Figure 2 for a site in granite of the Lac du Bonnet batholith, near Pinawa, Manitoba [9]. Laboratory measurements on small samples are also shown in Figure 2. Clearly the matrix permeabilities are quite close to the permeability measured across the "tight" intervals, a result also reported by Marine [10].

A hydrologic "log" of a drill hole evidently provides the information we need about downhole fractures. For example, the log shown in Figure 2 contains five or perhaps six fractures at least an order of magnitude more conductive than the matrix. Presumably any segment in the interval between 55 and 95 m depth would be suitable for hydrofracture stress measurement.

An even more complete picture of conducting fractures has been provided by Marine [10, 11, 12] from extensive hydrologic investigation of the subsurface at the Savannah River Plant, near Aiken, S.C. Seven individual holes were first logged. Then the connection between certain of the fractures was established by cross-hole and tracer tests.
Figure 1: Effect of a fracture on permeability. Log of permeability in darcies for crystalline rocks containing a fracture (upper bar) compared with the intact rock (lower bar). Small number is the confining or normal pressure of the measurement [4, 5, 8].

LOG K, darcy

- WESTERLY GRANITE ------ 100 bars

- STRIPA GRANITE ------ 138 bars

- GABBRO ------ 100 bars

- RAYMOND GRANODIORITE ------ 100 bars

- LARAMIE

70 bars  30 bars  IN SITU FRACTURE
Figure 2: Comparison of hydraulic conductivity from borehole test with laboratory values. $10^{-3}$ cm sec$^{-1}$ is approximately equivalent to 1 μd. [After 9]. Stippled and cross-hatched bars show range of various in situ measurements across the interval shown along the vertical axis.
As a result, some fractures, or fracture zones, could be traced for distances of nearly 2 km, at depths of 300 to 600 m. Similar three-dimensional mapping of conductive fractures is under way at Stripa [8], the Lac du Bonnet batholith [9], and, on a smaller scale, at the Edgar mine, Colorado [13] and at the Climax Stock at the Nevada Test Site [4, 14].

Geophysical and other conventional logging methods

Geophysical logging is a highly sophisticated technique in the oil industry and still forms the underpinning of well evaluation in oil- and gas-bearing sandstones and shales. Logging methods yield, among other things, a fairly precise measure of permeability of these rocks. In fractured crystalline rocks, however, determination of fracture flow is a poorly-developed art [15]. With the exception of the use of "tube waves", as described in the next section, no present method yields even an approximate value of permeability of a fractured interval.

Marine made a number of geophysical logs of his seven drill holes in the crystalline rocks at the SRP, including gamma, neutron, sonic, temperature and caliper [12]. Since he had surveyed the same holes hydrologically, he could see which logs showed deflections that correlated with hydraulically conductive zones. None gave consistent correlations; sonic were best. Davison has reported a similar lack of correlation for drill holes in Canadian granites [16].

What about fracture density as seen in the recovered core, or in televiewer surveys of the hole? Again results are very disappointing. Marine showed graphic examples of core recovered from holes at the SRP. The number and intensity of fracturing is identical for intervals which are highly conducting and virtually impermeable [12]. At Stripa, RQD was logged in a number of holes [8]; this parameter expresses relative fracture density, where 1.0 is relatively fracture-free, as seen in core. There was no correlation of RQD with permeability (Figure 3).

A comprehensive study was made at Stripa [17] to determine which geophysical borehole method best correlated with fractures seen in the core (hydrologic measurements for the same holes have not yet been published). The following logs were used: neutral, gamma-gamma, gamma ray, sonic, caliper and temperature, point resistance, differential resistance, resistivity, SP, IP and VLP. Compressional and shear wave anomalies usually occurred coincident with open fractures as seen in the core, the correlation being especially good at major fracture zones. Differential resistance and caliper probes tended to confirm these anomalies [17].
Figure 3: Comparison of packer tests and density of fractures [8]. Note lack of correlation between RQD and K. From a drillhole at the Stripa mine.
Fracture density has been measured in the drill hole with television camera, or with a so-called acoustic televiewer [18]. Davison reported little or no correlation between fractures seen with TV and intervals which were highly conductive [16]. It is interesting to compare fracture density (or alternatively, spacing) recorded by the two methods. The acoustic televiewer gave average fracture spacing in wells in South Carolina and in the Mojave desert [18] of 1 to 2 meters, all the way from near-surface to depths of 1000 m. Snow surveyed some 35 damsites [7] in crystalline rock where results of packer tests were available. Holes extended to 100 m or less, but the spacing of conductive fractures at depths below 50 m was never less than about 5 m. Assuming fracture densities to be the same in these areas, this would suggest that most of the fractures seen with the televiewer are nonconducting.

Two new studies show some promise in the use of vertical profiling for detection of spacing and orientation of major fractures in the subsurface. Aki et al. [19] used seismograms from a downhole receiver and source and noted that both attenuation and mode conversion increased as major fractures were inflated by pressure. The spacing and areal extent of the fractures could be estimated from synthetic analysis of the seismograms. A similar test was carried out near Krakemala, Sweden, by Israelson [20] using cross-hole measurements. The size and extent of subsurface fractures could be obtained based on their effect on P-wave velocity. In both of these studies, fracture geometry is the main result; no estimate of the actual hydraulic conductivity of the fractures is possible.

Tube waves and fractures

Huang and Hunter [21] and Paillet [22, 23] reported marked effects on tube wave characteristics due to fractures which appeared to be open when viewed by televiewer in a borehole. Tube waves are a guided wave mode, associated with Stoneley interface propagation, peculiar to the geometry of a fluid-filled hole in an elastic solid. The propagation characteristics of tube waves have been studied by Cheng and Toksöz [24, 25] using velocity dispersion, particle motion and attention. Attenuation is controlled mainly by borehole fluid attenuation for rocks of low porosity. Huang and Hunter found that tube waves were generated at a fracture due to the passage of a compressional wave in the enclosing rock. Detection of the propagating tube wave enabled its source, the fracture, to be located. The compressional wave was generated by an explosive source at some distance from the fracture. Davison later made packer tests in the same holes (in the Lac du Bonnet batholith) and reported [26] an excellent correlation between tube wave amplitude and magnitude of hydraulic conductivity. This is probably the first time that an acoustic measurement yielded a parameter quantitatively correlatable with hydraulic conductivity.
Hydrologic measurements are not yet available to complement the Paillet study; there, comparison of tube wave characteristics could only be made with fractures seen by televiewer. Full-wave recording of the waveforms showed that strong reduction - both in tube wave and shear wave amplitude - took place near open fractures. This result seems very significant. It would appear that acoustic methods hold the greatest promise for detection of downhole fractures which are hydraulically the most conductive.
REFERENCES


TOPIC IV

INNOVATIVE METHODS AND TECHNOLOGICAL IMPROVEMENTS

MODERATOR - D. I. GOUGH
DIFFERENTIAL STRAIN CURVE ANALYSIS

DOES IT WORK?

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ABSTRACT

Differential Strain Curve Analysis (DSCA) is a technique used to predict the in situ, three-dimensional stress magnitude and orientation from data obtained on a core removed from its in situ environment. A series of DSCA results from two field case studies is described in this paper. The results are compared with data generated via other established techniques such as tiltmeter surveys, mini-fracturing, downhole seismic, surface electrical potential systems (SEPS), dipmeter surveys (hole ellipticity) and focused gamma ray logs.

INTRODUCTION

Differential Strain Curve Analysis (DSCA) attempts to determine the spatial orientation as well as the ratio of the principal stress components. It is solely based on the assumption that a rock core retrieved from its downhole confined conditions will expand proportionally to the pre-existing in situ stress field. In other words, randomly oriented microcracks will be induced upon retrieval, the density of which will be proportional to the stress differential (i.e., in situ to zero).

The effect of these microcracks upon reloading the rock specimen will lower the initial Young's Modulus. Therefore, by submitting an oriented piece of rock to hydrostatic laboratory conditions, the differential strain observed in the various directions will bear some relationship to the pre-existing downhole conditions. Because these differences are minute, high accuracy and precision are required to be able to infer some reasonable conclusions.

Details of the exact theory and analysis used in DSCA can be found in the Petroleum Society of CIM Paper No. 80-31-33 (or PSE/DOE 8954). A copy of this paper is attached as Appendix A.

TESTING PROCEDURES

After choosing the section of core to investigate (avoiding any major discontinuity and/or inhomogeneity), the core is carefully cut into four parts along two perpendicular directions. In both case studies, global orientation with respect to true north were available (i.e., oriented core and known borehole deviation). Two samples from each depth were prepared in order to check the reproducibility of the results. These cubes, approximately 1-1/2 in. on a side, were surfaces on a parallel plate grinder to obtain three mutually perpendicular faces. These three faces were then instrumented with rosette strain gages (12 strain gages on each sample, Figure 1) and the entire assembly potted in a flexible jacket. The specimen, along with a fused silica standard used to correct for any instrumental errors, was then loaded into a pressure vessel and subjected to hydrostatic
loading increments of 500 psi during initial stages and 1,000 psi during latter stages. The pressures were held constant until the instrument readings stabilized (usually 5 to 10 min). The maximum pressure used in the Case 1 study was 18,000 psi while the maximum pressure used in the Case 2 study was 16,000 psi.

CASE HISTORIES

DSCA has been used to determine the stress field in six different oil-bearing formations. Overall, the results have been consistent and encouraging. To date, the results have been strictly proprietary but in the two cases disclosed in this paper permission was received to discuss the results as long as specific well location and company names were not given.

Case History 1

The well studied in Case History 1 is located north of Calgary, Alberta, Canada. The well was cored from 2,978 to 3,036 m and from 3,068 to 3,075 m with oriented core being retrieved. Mechanical Property Logs (MPL) and in situ stress determinations by mini-frac were run by Terra Tek, Inc. A dipmeter survey was run to determine hole ellipticity. It should be pointed out that only a limited amount of information was provided by the hydraulic fracturing as it was conducted in a cased hole and no orientation information could consequently be obtained. Therefore, the dipmeter survey is the only data generated to which the DSCA results could be compared.

Samples were selected from the most uniform isotropic sections available -- 2,990 m and 3,034.2 m. Two samples from each depth (four total) were prepared. The results of both the two-dimensional and three-dimensional analyses are shown in Table 1.

Data and Results of Case 1 Study

The following descriptions summarize the laboratory data.

Depth -- 2,990 m -- Sample 1

A statistical two-dimensional analysis for the horizontal plane (Table 1) illustrates a directional and a $\sigma_{HMAX}/\sigma_{HMIN}$ prediction showing a standard deviation of $\pm13^\circ$. This implies accurate predictions. Assuming that the vertical direction is a maximum principal stress direction, the maximum and minimum horizontal principal stresses are respectively predicted to act at N9°W and N81°E in a ratio of 1.2:1.

Three-dimensional analysis correlates relatively well in terms of the directions of $\sigma_{HMAX}$ and $\sigma_{HMIN}$ (N15°W and N75°E) and suggests a stress ratio of $\sigma_{HMAX}:\sigma_{HMIN} = 1.3:1$.

Depth -- 2,990 m -- Sample 2

Strain gage malfunction limited the degree of statistical analyses possible for this sample. However, the two-dimensional analysis predicted $\sigma_{HMAX}$ acting at N51°W and $\sigma_{HMIN}$ at N39°E in a ratio of 1.1:1. Three-dimensional
analysis suggested $\sigma_{\text{HMAX}}$ acting at N45°E and $\sigma_{\text{HMIN}}$ at N50°W with $\sigma_{\text{HMAX}}/\sigma_{\text{HMIN}} = 1.3:1$.

**Depth -- 3,034.1 m -- Sample 1**

This sample shows a relatively large standard deviation ($\pm 16^\circ$); however, basic trends appear. Two-dimensional analysis of the strains suggests:

- $\sigma_{\text{HMAX}}$ acts at N18°W,
- $\sigma_{\text{HMIN}}$ acts at N72°E, and
- $\sigma_{\text{HMAX}}/\sigma_{\text{HMIN}} = 1.3:1$.

Three-dimensional considerations show:

- $\sigma_{\text{HMAX}}$ acts at N30°E,
- $\sigma_{\text{HMIN}}$ acts at N35°W,
- $\sigma_{\text{HMAX}}/\sigma_{\text{HMIN}} = 1:6:1$.

**Depth -- 3,034.2 m -- Sample 2**

This sample also shows a moderately large statistical scatter. However, from analyses in the horizontal plane (two-dimensional):

- $\sigma_{\text{HMAX}}$ acts at N70°W,
- $\sigma_{\text{HMIN}}$ acts at N20°E, and
- $\sigma_{\text{HMAX}}/\sigma_{\text{HMIN}} = 1.5:1$.

Analysis of all strain outputs (three-dimensional) suggests:

- $\sigma_{\text{HMAX}}$ acts at N60°W,
- $\sigma_{\text{HMIN}}$ acts at N30°E, and
- $\sigma_{\text{HMAX}}/\sigma_{\text{HMIN}} = 1.3:1$.

In this case, three-dimensional analysis appears to be most representative and more statistically significant (Figure 2 compares the results of the two-dimensional and three-dimensional analyses). The second sample at 3,034.2-m depth is considered invalid because of the anomalous value of $\sigma_{\text{HMAX}}/\sigma_{\text{HMIN}} (3.3:1)$. This is probably caused by the anisotropy of the sample. The average stress orientation (Figure 2) thereby predicted is that the maximum horizontal stress acts at N30°E (1.3:1). The predicted stress ratio is 1.2:1.6. This concurs moderately well with the in situ stress measurements, despite the inherent uncertainty in the values of $\sigma_{\text{HMAX}}$ calculated by mini-fracing. The relatively large $\sigma_{\text{HMAX}}/\sigma_{\text{HMIN}}$ ratio also suggests that fracture orientation will be quite regular.

**Ellipticity Measurements**

The four-arm-high resolution dipmeter tool has been used with success for investigating the borehole geometry encountered in zones containing natural fractures (Babcock (1978); Smith (1979); Brown and Forgetson (1980)). The micro-conductivity sensors are mounted on pads on four hydraulically powered arms 90° apart. From an overhead view, the four arms are numbered in a clockwise direction. Bedding dip is computed by correlating the micro-conductivity responses from the four sensors. Two independent calipers from
opposing arms 1-3 and 2-4 define the borehole shape. Total tool orientation capability allows the monitoring of the azimuth of the pad array and measurement of the degree and direction of the tool from the vertical.

Dipmeter measurements in the Case 1 study (Figure 3) at a depth of 3,034.2 m suggest a hole which is basically round. However, at a depth of 2,990 m, the hole is elliptical with the maximum axis acting at an azimuth of 135° (this elliptical orientation perseveres along much of the wellbore). This breakout orientation conforms favorably with those outlined by Bell and Gough (1979) indicating \( \sigma_{HMAX} \) acting perpendicular to the maximum axis of the ellipse, in this case at N30°E. Figure 4 compares the elliptical characteristics, ascertained from the dipmeter, and the orientation of \( \sigma_{HMAX} \) as predicted by DSCA analysis.

In Situ Stress Determinations

Terra Tek reported in situ stress determination by hydraulic fracturing for the same site. The stress ratio between max. and min. horizontal stresses is consistently 1.55:1 at different depths (seven different zones reported). This value is comparable to the ratio of 1.3:1 obtained from the DSCA data. The three-dimensional, principal stress analysis in Terra Tek's report indicates that the vertical direction is the intermediate stress (assumed to be at a principal stress) and that the maximum stress is in the horizontal direction.

Although the data are too lengthy to present here, it is interesting to note that from the stereographic plot of the DSCA data, the vertical direction is not the principal stress direction. The stress analysis based on the stereographic plot also indicates that the vertical stress is indeed less than the maximum horizontal stress. Unfortunately, no definite number can be reported because of the scattering of the results on the three-dimensional analysis of DSCA. The Terra Tek report provided no indication of stress azimuth and orientation since the borehole was cased.

Case 1 Conclusions

The following conclusions are drawn as a result of the Case 1 study.

1. The three-dimensional DSCA results compare favorably with values interpreted from the hole ellipticity and other regional studies (Gough, 1979).
2. In situ stress measurements by hydraulic fracturing (1.55:1) are comparable to DSA results (1.3:1).

Case History 2

The well studied in Case History 2 is located in Grayson County, Texas. The well was cored from 5,770 to 5,880 ft with oriented cores being taken. Downhole seismic surveys at 5,330, 5,765 and 5,862 ft, Surface Electrical Potential System (SEPS) surveys at 5,700 ft, tiltmeter surveys and focused gamma ray logs were run. These tests were used to determine and predict fracture azimuth. Because the completion was inside the casing, it was not possible to run impression packers after the fracturing job. The testing was completed in February, 1980. After completion of the fracturing job,
temperature surveys and gamma ray logs indicated the fracture was confined to an interval between 5,821 and 5,839 ft. In September of 1981, DOWELL was given the opportunity to evaluate cores from 5,827 and 5,828 ft using DSCA techniques. Cores from these depths were selected for two reasons: (1) they appeared to be the more uniform samples available, and (2) it was in this zone that post-fracturing analyses indicated the fracture was situated.

Eight samples were prepared for DSCA (two from 5,827.2 ft, two from 5,827.5 ft and two from 5,828.8 ft). The results from these tests for both the two-dimensional and three-dimensional analyses are shown in Tables 2 and 3.

Data and Results of Case 2 Study

From an analysis of these data, the following conclusions are drawn.

1. Two-dimensional Analysis — Making an assumption that vertical is one of the principal stress directions, the maximum and minimum horizontal stress directions will be as follows.

   Maximum — N25°E (predicted fracture direction)
   Minimum — N55°W

2. Three-dimensional Analysis

   Maximum — Vertical with a possible ±20° deviation
   Intermediate — N30°E (predicted fracture direction)
   Minimum — N60°W

A higher percentage of strain gages failed during these tests than normally encountered. It is felt that this was a result of the coarse-grained vugular nature of the rock. As the hydrostatic pressure was increased during the test, the pore spaces collapsed inducing a stress release and strain decrease (see Figure 5, a typical curve). Two recommendations are made to prevent or reduce this in future samples. First, place the strain gages close to the center of the cube to prevent edge effects and, second, use larger strain gages to obtain more representative strain data.

Conclusions of Case 2 Study

Table 4 gives a summary of the data gathered from the different methods used for Fracture Azimuth Determination. From these data, the following conclusions are drawn.

1. All of the techniques used, except the tiltmeter survey, indicated the primary fracture would or did run N35°E.

2. The tiltmeter survey showed a secondary fracture running N27°E with the primary fracture running at N117°E. The report on the tiltmeter survey stated that the data were very erratic and interpretation was uncertain.

3. The focused gamma ray log showed N20°E at 5,818 and 5,843 ft. At 5,820 ft, data indicated a fracture at N99°E. It is felt this is a secondary fracture and not the primary fracture.
4. The DSCA results gave excellent agreement with the other techniques used.

OVERALL CONCLUSIONS

It must be emphasized that Differential Strain Curve Analysis is still under investigation. Some instrumental problems and interpretation difficulties are still present but their occurrences are decreasing as more experience is gained. In both cases studied, the reservoir rocks were either very anisotropic (Case 1) or very coarse grained (Case 2). The following overall conclusions are drawn as a result of the study.

1. The DSCA results seem reasonable both in terms of the predicted stress ratios and the predicted orientation. Good agreement between the DSCA results and other techniques was shown. The only technique not agreeing was the tiltmeter survey.

2. Sample preparation appears to be a key factor in producing consistent value data.

3. The technique has some advantages over stress measurements via hydraulic fracturing.
   • It does not assume vertical as the principal stress direction.
   • It can be used in cased holes to predict orientation.

REFERENCES


**TABLE 1**

**SUMMARY OF CASE 1**
**TWO-DIMENSIONAL AND THREE-DIMENSIONAL ANALYSES**

<table>
<thead>
<tr>
<th>Sample and Analysis Technique</th>
<th>Depth (m)</th>
<th>Geologic Description</th>
<th>Azimuths</th>
<th>Stress Ratio $\sigma_{HMAX}/\sigma_{HMIN}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (2D)</td>
<td>2,990</td>
<td>Metaquartzite (very fine-grained quartz sandstone), some organics, shale and pyrite, extensive quartz overgrowths.</td>
<td>$\sigma_{HMAX}$: N9°W $\sigma_{MIN}$: N81°E</td>
<td>1.2</td>
</tr>
<tr>
<td>1 (3D)</td>
<td>2,990</td>
<td></td>
<td>$\sigma_{HMAX}$: N15°W $\sigma_{MIN}$: N75°E</td>
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</tr>
<tr>
<td>2 (2D)</td>
<td>2,990</td>
<td></td>
<td>$\sigma_{HMAX}$: N51°W $\sigma_{MIN}$: N39°E</td>
<td>1.1</td>
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<tr>
<td>2 (3D)</td>
<td>2,990</td>
<td></td>
<td>$\sigma_{HMAX}$: N45°W $\sigma_{MIN}$: N45°E</td>
<td>1.2</td>
</tr>
<tr>
<td>1 (2D)</td>
<td>3,034.2</td>
<td>Chertarenite (very well indurated quartz sandstone), primarily chert (20%), cemented by extensive quartz overgrowths.</td>
<td>$\sigma_{HMAX}$: N18°W $\sigma_{MIN}$: N72°E</td>
<td>1.3</td>
</tr>
<tr>
<td>1 (3D)</td>
<td>3,034.2</td>
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<td>$\sigma_{HMAX}$: N30°E $\sigma_{MIN}$: N35°W</td>
<td>1.6</td>
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<tr>
<td>2 (2D)</td>
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<td>$\sigma_{HMAX}$: N70°W $\sigma_{MIN}$: N20°E</td>
<td>1.5</td>
</tr>
<tr>
<td>2 (3D)</td>
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<td></td>
<td>$\sigma_{HMAX}$: N60°W $\sigma_{MIN}$: N55°E</td>
<td>3.3</td>
</tr>
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</table>

$^\dagger$Average $\sigma_{HMAX}/\sigma_{HMIN}$ via Hydraulic Fracturing = 1.55.
TABLE 2

DSCA RESULTS FOR CASE HISTORY 2

<table>
<thead>
<tr>
<th>SAMPLE</th>
<th>1 (edge) 5,828.5</th>
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<th>3 (center) 5,828.8</th>
<th>4 (edge) 5,828.8</th>
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<table>
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<th>ROCK TYPE</th>
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<table>
<thead>
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<td></td>
<td>Two Strain Gages</td>
<td>Failed, No Data</td>
<td>For Analysis</td>
<td>2.2:1.4:1.0</td>
<td>N70°E 50°</td>
<td>W25°S 18°</td>
<td>N45°E Horizontal</td>
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<td></td>
<td></td>
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<td>2.7:1.8:1.0</td>
<td>N75°E 35° Plunge Down</td>
<td>N75°W 25° Plunge Down</td>
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<td>1.8:1.4:1.0</td>
<td>Vertical</td>
<td>Vertical</td>
<td>N5°E Horizontal</td>
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<td></td>
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<td></td>
<td>1.8:1.4:1.0</td>
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<td>N40°W Horizontal</td>
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### TABLE 3

**DSCA RESULTS FOR CASE HISTORY 2**

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<tr>
<th>SAMPLE</th>
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<td>5,827.5</td>
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<tr>
<td>No. 2</td>
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**ROCK TYPE**

Vugular Coarse-Grained Sandstone

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<th>Max:Min Ratios</th>
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<th>1.2 ± 0.1</th>
<th>1.3</th>
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<tr>
<td>Max. Stress Direction</td>
<td>N17°E (±1°)</td>
<td>N20°E (± 20°)</td>
<td>N16°E</td>
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<tr>
<td>Min. Stress Direction</td>
<td>N73°W (± 1°)</td>
<td>N70°W (± 20°)</td>
<td>N74°W</td>
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</table>

<table>
<thead>
<tr>
<th>Max:Int:Min Ratios</th>
<th>1.7:1.2:1.0</th>
<th>No</th>
<th>1.5:1.4:1.0</th>
<th>2.4:1.7:1.0</th>
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<tbody>
<tr>
<td>3-Dimensional</td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Maximum Direction</td>
<td>S40°W 50°</td>
<td>Analysis</td>
<td>N15°W 30°</td>
<td>Vertical</td>
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<tr>
<td></td>
<td>Plunge Down</td>
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<td>Plunge Down</td>
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</tr>
<tr>
<td>Minimum Direction</td>
<td>N60°W 12°</td>
<td>Available</td>
<td>S45°E 25°</td>
<td>Horizontal</td>
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<td>Plunge Down</td>
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</tr>
<tr>
<td>Int. Direction</td>
<td>N25°E</td>
<td>Horizontal</td>
<td>N30°E</td>
<td>10°</td>
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<tr>
<td></td>
<td></td>
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<td></td>
<td>Plunge Down</td>
</tr>
</tbody>
</table>
TABLE 4

SUMMARY OF RESULTS FOR DIFFERENT METHODS OF FRACTURE AZIMUTH DETERMINATION

1. Downhole Seismic
   (Test Points = 5,765, 5,862 and 5,330 ft)
   N35°E ± 15°

2. Surface Electrical Potential System (SEPS)
   (On Test Point = 5,700 ft)
   N50°E ± 10°

3. Tiltmeters
   N117°E (Primary Frac)
   N 27°E (Secondary Frac)

4. Focused Gamma Ray Log
   N20°E ± 10° (5,818 ft, 5,843 ft)
   N100°E (5,820 ft)

5. DSCA (5,827 ft -- 5,828 ft)
   N25°E - Two-Dimensional
   N30°E - Three-Dimensional
Figure 1. Cube Gage Pattern.

\[ \sigma_{\text{HMAX}} = N40^\circ W \] (Average of 2-D Measurements)

\[ N30^\circ E = \sigma_{\text{HMAX}} \] (Average of 3-D Measurements)

Figure 2. Case 1 DSCA Two-Dimensional and Three-Dimensional Results.
Figure 3. Case 1 Dipmeter Survey.
Figure 4. Case 1 Ellipticity Results Compared to DSCA Three-Dimensional Results.

Figure 5. Typical Strain Gage Failure in Case 2 Study.
ABSTRACT

Recent developments in energy exploration at depths of 5000 to 25,000 feet have made it necessary to quickly and reliably determine the in-situ stresses acting on the wellbore.

Differential Strain Analysis (DSA) is being investigated as a technique applied to core samples to indirectly determine the in-situ stress state.

Testing is being pursued in three steps. First, field measurements of strain are made in-situ as the core is pulled out of the well. Second, the cores are brought to the lab and DSA is performed under in-situ hydrostatic conditions. Third, the rock is examined microscopically.

These tests have been performed on both oriented and non-oriented cores from Texas, Louisiana and Pennsylvania.

At this point in the investigation, it appears very favorable that a reasonably accurate estimate of the 3-dimensional stress state can be obtained using the strain curve analysis method. It has been demonstrated mathematically that not only the ratio of the stresses can be derived but also the orientation of the stresses in free space. The application of these equations to the data from the latest high quality runs yields results well within the reproducible tolerance of other methods.

REFERENCES AND ILLUSTRATIONS AT END OF PAPER.
drops off rapidly as depth (and therefore, distance from surface) increases. Another attempt at predicting fracture orientation has been to measure directly the in-situ stress field, since this is the primary factor influencing the control of fracture orientation (Koziński et al., 1980; Hubbert and Willis', 1956). The traditional method of determining stress in rock masses has been by the overcoring process (Obert and Duval, 1967). Strain gages are attached to the bottom of a core hole and the differential expansion of the rock is measured as the rock is released from the in-situ stresses by overcoring. The method is relatively accurate but limited to shallow depths (a few hundred feet) and to only one plane of measurement (i.e., 2-dimensional). Another method employed at greater depths is the mini frac-impression packer method (Humeson, 1978). This method isolates a zone by openhole packers and induces a small fracture by pressurizing the interval. An impression packer is then inflated over the fractured interval and the imprint of the fracture opening on the packer can then be oriented. This method's shortcomings are: 1) borehole stresses and skin damage may interfere with the fracture initiation and since the packer only measures the inside surface, any re-orienting of the fracture as the fracture penetrates away from the well bore is not measured, 2) only one plane (that of the fracture plane) is being measured, 3) physical problems are often encountered (packer damage, hole collapse, trip time for impression packer, fractures in certain formations not leaving discernable impression, etc.). In addition, there has been reluctance in industry to use this method because a fracture is being produced in an uncased, open hole with attendant zone isolation and well control problems.

Ideally, a method is needed that can give an idea of the 3-dimensional stress state, as to both the stress directions in space, uninfluenced by the wellbore, and to the relative and/or absolute magnitude of each of the 3 principle directions. This is something no method to date can do at the typical depth encountered in the energy industry.

In this study, a variation of Differential Strain Analysis (DSA) was used in that study because it could determine, 1) the linear and volumetric compressibility as a function of pressure, 2) the strain associated with cracks of a particular closure pressure, and 3) the orientation of cracks with particular closure pressures. For details of sample preparation, data analysis and DSA theory, the reader is referred to the original article (Strickland, et al., 1979), and to the works of Simoons et al., (1974) and Seigfried and Simoons (1978).

During the course of the earlier study, it became apparent that there might be a strong and direct correlation between the amount and direction of crack generation in the rock matrix and the original in-situ stress field. In order to modify DSA to be able to predict in-situ stress, four assumptions needed to be tested. The assumptions are: 1) microcracks are induced in the core matrix as the rock expands in response to release from in-situ stresses, 2) these cracks are aligned primarily by the direction of the original stress forces, 3) the cracks are proportional volumetrically to the corresponding in-situ stress magnitudes, as modified by the rock matrix and, 4) by reversing the expansion of the sample by subjecting it to hydrostatic pressure, the contraction of the rock in any specific direction will be analogous to the original strain in that direction.

Assumption #1 was demonstrated in an earlier paper (Strickland, et al. 1978) and by the work of Simons and Richter (1974)*. A program of field core relaxation, lab compression and microscopic examination was initiated to test the other assumptions.

The Simoons, et al. (1974) study also demonstrated direct correlation between the type and slope of the strain curve and the source of the microcracks. This represents a kind of "fingerprint" imparted by each event in the rock's history.

DIFFERENTIAL STRAIN CURVE ANALYSIS - PRINCIPLE AND CALCULATION

A step-by-step procedure from the one-dimensional case through to the 3-dimensional case showing how the strain analysis method works will now be presented.

It was discovered in earlier work (Simoons et al., 1976)
1974), and confirmed in this work, that for most rocks under compression the strain vs. pressure curve consisted of linear segments separated by distinct slope discontinuities. For a gage representing one direction this generally takes the form shown in Figure 1, where there are two distinct linear segments separated by a zone where the slope is continuously changing.

The position of the transition zone on the strain curve between the linear portions of the strain curve is related to the in-situ pressures. This is a result of the curve passing at that point from a matrix with induced crack damage to a matrix approximating the in-situ state. The sharpness with which this transition takes place is apparently a function of the mineralogy of the rock. Crystalline rocks, as used in the Simmons study, generally produce very sharp breakovers whereas the sedimentary rocks used in this study produced longer transition zones. The deeper samples produced sharper transitions, probably due to the better lithification of the sediments. In the case of sharp breakovers, the strain at that point may be used to directly ascertain the in-situ stress. It is worth noting that the transition does shift position in curves representing different directions on the same rock. This would be expected, with the curve going into transition earlier in curves representing smaller in-situ stresses.

The last linear portion (part 2) represents the intrinsic compressibility of the constituent minerals in the rock matrix after all the cracks and most of the pore space is closed.

The first linear portion has a steep slope because the cracks are only partially closed. Consequently, a corresponding increase in pressure yields a larger strain than in the latter portion where the cracks are closed.

Consider Figure 2. The last linear portion of the strain curve represents the intrinsic compressibility of the sample in that particular direction. It can be expressed either as the ratio of \( \frac{\Delta C}{\Delta P} \) or as the slope (8) of the linear portion of the curve.

Points X and Y lie on the first linear portion of the strain curve and are represented by the values \((P_x, C_x)\) and \((P_y, C_y)\). The values \(C'_x\) and \(C'_y\) are the zero pressure intercepts obtained by projecting lines parallel to the intrinsic compressibility slope through points X and Y.

The quantity \(C'_x - C'_y\) is the strain change that occurs over the pressure range \((P_x - P_y)\) caused by the complete or partial closing of all cracks that were open at \(P < P_x\) (see Simmons et al., 1974).

So: \[
\begin{align*}
\frac{C_y - C'_y}{P_y - P_x} &= \frac{C_x - C'_x}{P_x - P_y} \\
\frac{C_y - C'_y}{P_y - P_x} &= \frac{C_x - C'_x}{P_x - P_y} \\
\frac{C_y - C'_y}{P_y - P_x} &= \frac{C_x - C'_x}{P_x - P_y}
\end{align*}
\]

If the curve between \(C_x\) and \(C_y\) is linear, then

\[
\frac{C'_y - C'_x}{P_y - P_x} = \frac{C_y - C_x}{P_y - P_x} = \frac{C_y - C_x}{P_y - P_x} = \frac{C'_y - C'_x}{P_y - P_x}
\]

This gives the value of strain change caused by the complete or partial closing of all cracks over a unit pressure range on this linear strain region. This is defined as \(c'\).

Next, for the two-dimensional case, refer back to the four assumptions stated earlier concerning crack genesis in relation to the original stress state. The following is a simple example to help illustrate the principles being assumed. In Figure 3 (a) we have a rock under in-situ stress in 2 directions, \(X\) and \(Y\).

- \(P_{max} = 12\, MPa\) in X direction
- \(P_{min} = 6\, MPa\) in Y direction

Upon coring, the in-situ stresses are released and the rock is allowed to expand in all directions (it is at this point the overcoring method is used to measure the relative expansion of the rock). Since the X direction has the greatest stress release, it will expand the most by crack genesis and direction \(Y\) will be the least cracked since it has the least amount of stress release. This is shown schematically in Figure 3 (b). In Figure 3 (c) the rock is subjected to 3 MPa hydrostatic reloading in a pressure vessel. Since the \(X\) direction had the most cracks, it will be the most easily compressed and will, therefore, have the highest \(c'\) (see Figure 3) value at \(P_{max}\). For the same reason \(Y\) will have the lowest \(c'\) value because it compresses the least under hydrostatic pressure. From this it can be predicted that the direction of the original maximum in-situ stress was \(X\) and the direction of the minimum in-situ stress was \(Y\).

Now, let's see how this works on a sample. A three strain gage rosette will give 3 strain curves as shown in Figure 4 (compare with Figure 1). In a homogenous rock \(\sigma_1 = \sigma_2 = \sigma_3\), or in other words, the intrinsic compressibility will be approximately the same. The strain curves have an early linear portion, equation (4) can be applied.

\[
c'_1 = \sigma_1 - \sigma_1 = \sigma_1 - \sigma_2 \quad \epsilon'_2 = \sigma_2 - \sigma_2 = \epsilon'_3 = \epsilon'_3 - \epsilon'_3
\]

\(c'\) is the strain due to crack closure per unit of pressure change.

Set

\[
\epsilon'_1 = \epsilon_1 - \epsilon_1 = \epsilon_2 + \epsilon_2 - \epsilon_{xy}
\]

(See Figure 5)

\[
\epsilon'_3 - \epsilon_y = (c'_4 - \epsilon_{xy})
\]

The strain \(\epsilon\) in the direction inclined at some angle \(\theta\) to \(\sigma_1\) is given by

\[
\epsilon = \epsilon_1 \cos^2 \theta + \epsilon_2 \sin^2 \theta + 2 \epsilon_{xy} \sin \theta \cos \theta
\]

(Jaeger & Cook, 1976)

where \(\tau = 1/2 \epsilon_{xy}\).
Then the principal axes are inclined at:
\[
\frac{1/2 \tan^{-1} \frac{2 \tau_{xy}}{\varepsilon_x - \varepsilon_y}}{} \quad (7) \quad (Jaeger & Cook, 1976)
\]
to \(\varepsilon_{\max}\) and \(\varepsilon_{\min}\) are:
\[
2/\sqrt{1 + 4 (\tau_{xy})^2} + 1/4 (\varepsilon_x - \varepsilon_y)^2 \quad (8)
\]
Finally, consider the 3-dimensional case (Figure 6). The same type of forms can be defined as in equation (5):
\[
\begin{align*}
\sigma'_1 &= \varepsilon_1 - \varepsilon_3 - \varepsilon_2 \\
\sigma'_2 &= \varepsilon_3 - \varepsilon_1 - \varepsilon_2 \\
\sigma'_3 &= \varepsilon_1 - \varepsilon_2 - \varepsilon_3
\end{align*}
\]
(9)

Three pairs are parallel in direction and can be averaged as:
\[
\frac{1}{2} (\sigma'_1 + \sigma'_2) = \varepsilon_1, \quad \frac{1}{2} (\sigma'_2 + \sigma'_3) = \varepsilon_3, \quad \frac{1}{2} (\sigma'_3 + \sigma'_1) = \varepsilon_2
\]
This reduces to six knowns as follows:

Set: \(\varepsilon_1 = \varepsilon'_1 + \varepsilon'_3 + \varepsilon'_6\) \(\varepsilon_6 = \varepsilon'_7\)

In three dimensions, strain \(\varepsilon\) may be described using direction cosines (1, m, n) with angles (\(\theta, \phi, \chi\)) with respect to the three perpendicular axes (ox, oy, oz), by the following equation:
\[
\varepsilon = 1 \varepsilon_x + \mathbf{m} \cdot \varepsilon_y + \mathbf{n} \cdot \varepsilon_z + 2 \mathbf{m} \cdot \mathbf{n} \cdot \varepsilon_{xy} + 2 \mathbf{m} \cdot \mathbf{n} \cdot \varepsilon_{xz} + 2 \mathbf{m} \cdot \mathbf{n} \cdot \varepsilon_{yz}
\]
(Jaeger & Cook, 1976) \(11\)

From the definition of the three principal strains, the following three equations can be established.
\[
\begin{align*}
1 \varepsilon_x - m^2 \varepsilon_{xy} + n^2 \varepsilon_{xz} &= 0 \\
1 \varepsilon_{xy} + m (\varepsilon_y - \varepsilon_x) + n \varepsilon_{yz} &= 0 \\
1 \varepsilon_{xz} + m \varepsilon_{yz} + n (\varepsilon_z - \varepsilon_x) &= 0
\end{align*}
\]
(12) (13) (14)
The three real roots of \(\varepsilon\) will be the three principal strains \(\varepsilon'_1, \varepsilon'_2, \varepsilon'_3\).

By displacing \(\varepsilon_1\) into equations (12) (13) and (14), one set of (1, m, n) can be solved. The same applies to solving \(\varepsilon_2\) for \(\varepsilon'_2\) for \((1, m, n_2)\) and \(\varepsilon'_3\) for \((1, m, n_3)\) and three corresponding sets of angles can be obtained as \((\theta_1, \phi_1, \chi_1); (\theta_2, \phi_2, \chi_2)\) and \((\theta_3, \phi_3, \chi_3)\), respectively.

Two computer programs were written to calculate the two-dimensional and three-dimensional case. By knowing the (ox, oy, oz) orientation and the three sets of principal strain cosine angles, the true orientation of the three principal directions can be plotted using the "Stereographic Projection Method" (Gossett, 1976). The ratio between the three principle stresses can be related to the ratio between \(\varepsilon_{\max}\), \(\varepsilon_{\max}\), and \(\varepsilon_{\min}\). Specific values can be assigned by using the instantaneous shut-in pressure (ISIP) during a fracture treatment as the minimum stress and using the ratios to calculate the other 2 stress values.

**PROCEDURE**

The sample preparation has been standardized after investigating different strain gage configurations. First, the cube is carefully cut (to avoid further crack generations) and hand lapped into a cube approximately 1/2" on each side, being careful to preserve the orientation references. It is important that all faces are at right angles to simplify data analysis. Then on each of three faces surrounding a common corner (o), a full strain gage rosette comprised of 3 gages at 45° apart and a single gage were mounted as shown in Figure 6. The single gage is mounted perpendicular to the middle gage on the rosette. The four-gage configuration allows the selection of 2 or more rosette displays for calculating the stress field in that plane. This increases the accuracy and provides a backup in case one of the gages fail. The rosettes are aligned parallel to the edges of the cube. In order to get point strain, the rosettes were mounted as close as possible to point o, but no closer than 1/2" to the edge of the cube to avoid edge effects. After wiring, the sample was vacuum dried and potted in a clear, flexible and impermeable jacket to exclude the pressure medium from the cracks in the sample. The sample was then cured overnight at 120°F with a slow heating and cooling rate. The cube, along with a fused silica standard, were loaded into the pressure vessel and subjected to a total hydrostatic pressure of 100 to 140 MPa (depending on the depth of the core). The total hydrostatic pressure was achieved by small initial increments of 1-3 MPa and then by larger increments of 5-10 MPa. The recorded stress-strain curves are compared with the fused silica sample and any deviations were removed. Finally, the curves were plotted, numbering 1 to 12.

**FIELD RELAXATION DATA**

An on-site field program was initiated in an attempt to get measurements of the original rock expansion and crack genesis after the rock is released from the in-situ stress field. As soon as possible after the cores were received at the surface, strain gages were applied to the cores (Figure 7) and placed in an oven preheated to in-situ temperatures to reduce thermal effects. The resulting strain data was plotted and compared with the lab compression curves to see if there was a correlation.

**RESULTS**

**Differential Strain Analysis**

Table 2 contains the calculations for a typical sample (#2865.5) from the Pennsylvania well. True co-
pass orientation is known for this sample, but to sim-
plify the example, assume that the X axis = 0°W and Y axis = 0°E (it is easier to apply a single correction to the final stress direction determination than carry the true axis directions through the calculations).

In Table 2, strain curves from each of the twelve strain gages shown in Figure 6 are resolved for 8, 8, and 8. Then using the four gages on each face to form 2 sets of 3 gage rosettes (by interchanging the intermediate gages) two sets of values are obtained for the maximum and minimum stress values. As can be seen, the ratios are very consistent but the directions differ by 8° to 16°.

To extend the data to the 3-dimensional case, first the C' values for the 3 duplicate direction pairs are averaged to give the 6 directions of C' values (Table III). Again, by interchanging the intermediate gages in the rosettes, two sets of ratios and stress directions are calculated (Table IV). The ratios between the maximum, intermediate and minimum stress values for the two rosette patterns are identical. The direction of the maximum principal stress direction varies by 14° azimuth and 10° dip.

In all but one sample, the vertical strain was greatest in the Texas and Louisiana cores and in the Warren and Bradford Formations in Pennsylvania. Samples from the Tiona Formation revealed that in 4 of the 6 samples the maximum and minimum strains were in the horizontal plane and the intermediate was vertical.

Field Data

Technical problems prevented getting useful data for most of the Louisiana and Pennsylvania cores. Once the technical problems were solved, curves similar to Figure 6 were normal. The steep initial slope was due to warming the cores back to downhole temperature after cooling off during gaging the sample. Then a steady decrease was observed, tapering off to essentially no slope after about 12 hours. The trip time to bring cores out of the Louisiana well averaged about 5 hours and in the Pennsylvania well it was between 2-3 hours. By this time most of the relaxation had occurred and we were measuring only a few of the strain curve. When compared to the lab DSA curves, there was generally good agreement, although the field curves were somewhat erratic in nature.

DISCUSSION

From earlier work, it appears that the stress field becomes uniform over larger areas with increasing depth (Haleson”, 1977). This makes it necessary to sample only a few wells in a field to predict the stresses over the entire area. For most sedimentary rocks encountered in the oil field, the solving for the primary horizontal stress vectors (two-dimensional case) would be adequate. This assumes the stresses are not inclined by tectonics or bedding planes and the depth is sufficient so the fracture will be vertical. In all the samples tested, without exception, the minimum strain was horizontal, tending to confirm the vertical fracture assumption in the formations examined.

The reversal of strain observed in the Tiona Formation curve may be due to mineralogy, but at the time of this writing, this was not conclusive. Another explanation may be that there is an actual reversal of stress directions in that formation. This would be difficult to determine by any previous method involving fracture mechanics because the fracture would still be vertical and the minimum stress is still horizontal. Consequently, changes in the other two stress directions would be undetectable. The main affect if the maximum and minimum stress directions were both in the horizontal plane would be an even stronger resolution of the fracture direction due to the larger contrast in values.

CONCLUSIONS

At this point in the investigation, it appears very favorable that a reasonably accurate estimate of the 3-dimensional stress state can be obtained using the strain curve analysis method. It has been demonstrated mathematically that not only the ratio of the stresses can be derived but also the orientation of the stresses in free space. The application of these equations to the data from the latest high quality runs yields results well within the reproducible tolerance of other methods.

Comparison with the relaxation curves obtained in the field experiments has tended to confirm the reliability of the lab measurements, although the quality of the field data was not what we had anticipated.

ACKNOWLEDGEMENTS

The authors would like to express their gratitude to Anadarko Production and Delta Drilling Company for supplying the cores used in this study and allowing access to their well sites to make the field measurements. The author also gratefully acknowledges Carl Montgomery and Jim Wallace for their suggestions, encouragement and help during the course of this work. Thanks are also extended to the management of Dow Chemical for permission to publish this paper.

REFERENCES

<table>
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**NOMENCLATURE**

- $E$ = Intrinsic compressibility - $\mu$/KPa
- $\beta$ = Compressibility at local - $\mu$/KPa
- $\gamma$ = Shear strain
- $\alpha$ = One-half of shear strain
- $(\alpha, \beta, \gamma)$ = Direction cosine
- $(\varphi, \psi, \chi)$ = Angle of direction cosine - (degrees)
- $c$ = Axial strain
- $P$ = Pressure - MPa
TABLE I

<table>
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<th>Rock Type</th>
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TABLE II
Two Dimensional Solution (Sample #2865.5)

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<th>$\epsilon_zy$</th>
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<td>$\phi$</td>
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TABLE III
Three Dimension Data (Sample #2865.5)

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TABLE IV
Three Dimension Solution (Sample #2865.5)

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FIGURE 7  FIELD SAMPLE PREPARATION

FIGURE 8  Field Data
SOME EXAMPLES AND IMPLICATIONS OF OBSERVED ELASTIC DEFORMATIONS ASSOCIATED WITH THE GROWTH OF HYDRAULIC FRACTURES IN THE EARTH

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Abstract

On a number of occasions during the past four years, tiltmeters have been used to monitor the surface deformations associated with the formation of shallow (< 500 m) hydraulic fractures in the earth. In this paper we examine the data with a view toward establishing the broad scale growth and consolidation characteristics of in situ hydraulic fractures. A discussion of the techniques used to interpret the surface deformation data is also presented. The results show that fracture development generally proceeds through a series of one or more phases, each phase being characterized by predominantly in-plane extension of an element of the fracture structure. Where more than one phase is involved, the transition to predominant growth in the new plane is seen to be rapid. Although few 'breakout' events are typically seen during a specific treatment, their occurrence appears to be commonplace during the formation of hydraulic fractures at shallow depths at least. Continued extension of the fracture structure following shut-in of the well is also favoured by the data. Treatments which involve the injection of proppant are estimated to result in residual fracture volumes many times the volume of proppant injected. An estimate of the static shear modulus of Devonian shale appropriate to the scale of hydraulic fractures is also deduced and is found to differ from the laboratory determined value by almost an order of magnitude.

INTRODUCTION

During the four years that the tiltmeter method of hydraulic fracture mapping has been available as a commercial service to the energy industry* a number of the fracturing operations monitored have yielded tilt data of good signal-to-noise ratio. Usually, these fractures were formed in an attempt to stimulate production from hydrocarbon reservoirs, the well treatments typically involving the injection of large quantities of fluid at sufficiently shallow depths that the resulting elastic deformation field could be easily detected by an array of tiltmeters located at the Earth's surface. However, on one occasion, the formation of a series of comparatively small fractures associated with the measurement of in-situ stress about deep mine drift was monitored by a number of tilt sensors located only tens of metres from fractures. The important characteristic of each of these data sets is that the time-history of the evolving elastic deformation field can be deduced from the records. Hence it is possible to infer the time-history of fracture geometry by modelling the observed deformations developed during successive time increments. This is fortunate, for at shallow depths the difference between the overburden and the minimum horizontal principal stress is not generally great and hence the development of complex non-planar fracture structures might be expected. In this paper we shall review

*Fracture Technology Incorporated, 1056 Elwell Ct., Palo Alto, CA 94303.
the shallow fracture datasets collected up to present and discuss features in the data which we feel provide important information regarding the broad scale growth and consolidation characteristics of hydraulic fractures produced in the real Earth. Aside from providing a 'window' to view the geometry of the fracture, the data also contain information regarding the mechanical behavior of the dislocated medium itself and we shall address the problem of determining certain medium mechanical properties by discussing some results. Finally, the tilt-inferred closure behavior of the fractures associated with the hydraulic stress measurements will be discussed with a view to better understand the nature of Initial Shut-In Pressure value (ISIP). Much, although not all, of the data discussed has been previously published in the form of contractual reports and readers interested in obtaining copies should contact Fracture Technology Incorporated.

As a preparation to the discussion of results we first present an overview of the tiltmeter technique in which we will clarify the nature of the observations, discuss the theoretical framework within which the data are typically interpreted and comment on the significance of the various parameters resolved.

Background

On deploying an array of tiltmeters about a treatment well the intent is to obtain as complete a description of the fracture-induced surface deformation field as is possible given some finite number of sensors. In some instances where a-priori information is available regarding the likely azimuth of the fracture, the array can be designed to optimize the model constraining power of the resulting dataset. More often than not, however, all eventualities must be allowed for in array deployment and the subsequent compromise in resolution must be tolerated. A schematic diagram of a typical tiltmeter installation is shown in Figure 1.

The physical quantity measured at each tiltmeter site is the horizontal spatial gradient of vertical displacement. Assuming for the moment that the shallow crust in the vicinity of the well can be adequately represented as a homogeneous isotropic linear-elastic half-space, it follows that the surface deformation field produced by a fracture of arbitrary geometry is a function only of that geometry and the Poisson's ratio of the medium (Converse and Comninou, 1975). Thus, provided Poisson's ratio is known, the surface tiltfield is dependent only on the wall displacements that define the fracture. Fracture fluid pressure and shear modulus of the medium enter into the problem only when the resolved fracture geometry is further interpreted in terms of the distribution of stresses within and about the fracture which govern the fracture's disposition. Consequently, a suitable fracture model for quantitative interpretation of the surface tilt data then might seem to be one in which the fracture is described purely in terms of wall displacements without regard to stress. The approach is certainly possible due to the availability of analytic solutions for the surface displacements induced by a dipping rectangular mode I dislocation (Figure 2a) in an ideal half-space (Davis, pers. communication, 1981). The problem then becomes one of
finding a model dislocation configuration, synthesized from the rectangular elements, which predicts a surface tiltfield consistent with the observations. The drawback with the dislocation theory approach is that it is inherently non-unique: generally a suite of different fracture geometries can be found which fit the data equally well, even when exotic geometries are disqualified in favour of plane, contiguous structures. This is a direct consequence of not imposing the condition that candidate fracture solutions be physically acceptable in that their geometry reflect the mechanical equilibrium that effectively exists between the fracture structure and the medium. In other words, the dislocation solutions do not necessarily satisfy the appropriate stress boundary conditions on the fracture face. In practice, the dislocation theory approach is usually found to be adequate where azimuth, dip, depth-to-centre and volume of fracture are the only parameters of interest. Where fracture dimensions are sought, however, the problem of non-uniqueness renders the approach inadequate and it becomes necessary to include some consideration of stress in order to usefully limit the space of tilt-compatible fracture geometries. Shear modulus of the medium and fracture fluid pressure thus enter the problem.

Within a dislocation theory framework the stress constraint can be incorporated by constraining the dislocation distribution to mimic the wall displacement distribution of a suitable pressurized fluid filled crack model (e.g. Perkins and Kern, 1961; Advani, 1980). An alternative and more convenient strategy, however, is to invert the data using some pressurised fluid-filled crack model that determines the free surface deformation field directly. Two such models, given by Pollard and Holzhausen (1979) for the case of a dipping uniformly pressured plane strain crack (Figure 2b), and by Sun (1969) for a horizontal, uniformly pressured penny-shaped crack, formed the analytical basis for most of the results discussed in this paper. Specifically the models give the displacement field at the surface of a uniform, homogeneous, isotropic, linear-elastic half-space of Poisson's ratio, $\nu$, as a function of crack depth-to-centre, $d$, crack half-height (or radius for the Sun model), $a$, and the quotient, $PD/\mu$. Here $\mu$ is the shear modulus of the medium and $PD$, the driving pressure, is the excess of fluid pressure inside the fracture over the total medium stress seeking to close the fracture [pore pressure plus rock matrix stress normal to the plane of the fracture (Hubbert and Willis, 1957)]. It is important to note that within the framework of these models, shear modulus appears only as the denominator of the independent model parameter $PD/\mu$ and it is the value of this quotient together with the parameters $d'$ and 'a', which are constrained, within the framework of the model, by the surface tilt data.

A further constraint on the crack model solution may be applied by consideration of the mode I fracture toughness, $K_{IC}$, of the medium. Assuming the fracture to be well represented as an equilibrium crack then the theory of linear elastic fracture mechanics yields a geometry dependant relationship between the constant $K_{IC}$ and the model parameters of driving pressure and radius (or height in the plane strain case) given by $K_{IC} = CPD_{a}^{1/2}$, where $C$ is a
geometry dependant constant.

The uniformly pressured fluid filled equilibrium crack models were used extensively in deriving the results discussed in this paper. Consequently it is pertinent to enquire as to their limitations in providing a suitable analytical framework within which the data may be quantitatively interpreted. Several points are of note. First, during the fluid injection phase, viscously induced pressure gradients in the fracture distort the geometry from that of a uniformly pressurised equilibrium crack. Only on shut-in of the well are uniform conditions likely to prevail. Consequently shut-in tilt data are used exclusively in constraining the models, the datasets used in each inversion consisting of the set of tilt vectors developed at each site during the course of injection up to the time of shut-in. Second, there is the question of the degree to which homogeneous, isotropic, linear elastic conditions prevail in the vicinity of the treatment well. Certainly this can be a problem in some localities, especially where severe stratigraphic contrasts in shear modulus are found. The results discussed in this paper, however, are not considered to suffer excessively from structural complexity and the reader is referred to the various reports cited for validation of the assumptions. Finally, there is the question of reliability (or rather, interpretation) of the values of length, height or radius which result from the analyses. This is essentially equivalent to questioning the validity of the crack models used, both of which assume primary fluid penetration to the periphery of what is essentially a Griffith crack. We wish to make clear that the surface tilt data provide only smoothed information regarding the fracture wall displacements: the data are practically incapable of resolving details in cross-sectional geometry that distinguish various crack models, even in situations where the idealisations of plane strain or penny-shaped geometries are strictly realised. For example, a peripheral region within which the fracture is extremely narrow, such as might be appropriate for a crack with a Barenblatt tip (Barenblatt, 1962) would contribute very little to the surface deformation field and hence go undetected. We consider it unlikely, however, that such a periphery region would be spatially extensive (Biot et al., 1981). Thus, although the estimates of containment that result from the analysis cannot strictly be taken as indicative of the position of the fracture tip, any underestimate, we feel, is likely to be small.

FRACTURE GROWTH AND CONTAINMENT

In this section we shall discuss the geometrical development of hydraulic fracturing during the period of fluid injection as inferred from observations of the evolving pattern of surface deformation. A general feature of deformation field development, common to all datasets studied, is that the fields evolved via a series of one or more successive phases, each characterised by a period during which the tilt rates were constant on all array records (Figures 4, 6, 8, and 10). The implication is that, during each phase, the pattern of tilt field development remained essentially constant. These patterns in most cases were suggestive of a predominantly planar fracture source
from which we infer that in-plane fracture growth was taking place throughout each phase. The marked difference in tilt-development patterns between phases is taken as indicative of a change in the plane of fracture growth. Although we speak of planar fractures we remind the reader that the data generally do not permit the distinction to be made between the development of a single fracture or that of a localized zone containing a labyrinth of sub-parallel plane fractures such as reported by Mahoney et al. (1980) and Aki et al. (1982). All the data can resolve is that the width of such a zone is small compared to the dimensions of length, height and depth. Neither is it possible to discern between growth through previously unfractured fracture of the rock and growth along pre-existing fractures.

Knox County, Ohio: An example of tilt records showing clear evidence of change in the plane of fracture growth is presented in Figure 4. The data were collected during the injection of nitrogen gas into the 335 m deep Black #1 well located in Knox County, Ohio (Figure 3). The well was uncased for the lower 30 m and penetrated the top of the Devonian shale at a depth of 168 m. A clear change in the pattern of tilt development was seen to occur after some two thirds of the fluid had been injected. This event can be recognised in the raw tilt records shown in Figure 4 by the abrupt change in the tilt-rate detected by all instruments at the end of the period denoted phase 1. During both this phase and the following period until shut-in, the tilt-rates remained reasonably stable. Snapshots of the tiltfields developed from the start of injection up to the time of the change (denoted phase 1) and in the ensuing period up to shut-in (phase 2) are shown in Figure 5a and 5b, respectively. Clearly, a significant change in fracture growth characteristics occurred between phases although the wellhead pressure and flow rate records (Figure 4) show no indication of this. Using both dislocation (Davis, 1981) and plane strain crack (Pollard and Holzhausen, 1979) representations of the fracture to model the tilt fields shown in Figures 5a and 5b, Evans et al. (1982) inferred that during phase 1 a bilateral subvertical fracture of orientation N62°E grew from the wellbore. By the end of the phase the fracture extended upward over 150 m to a depth near the interface between the shale and the overlying well-cemented Berea sandstone. Breakout then occurred into what was clearly a near horizontal plane. The quality of the data fit afforded by this interpretation can be judged in Figure 5 by comparing the model-predicted tilt vectors (dashed arrows) with the observed (solid arrows). Analysis of the state of earth stress in the vicinity of the well using the observed initial shut-in pressure and a laterally confined homogeneous earth model indicated that the final 100 m or so of upward propagation of the sub-vertical fracture took place in a stress regime progressively favouring horizontal fracture propagation. Upon breakout, further upward propagation was negligible. This breakout behavior would have gone undiscovered but for the tilt data.

Vernal, Utah: A similar example of sudden changes in fracture growth characteristics can be seen in Figure 6. The data were collected during the injection of 239m³ of proppant-laden fluid into the Rimrock Sandstone (Cretaceous Mesaverde formation) through a slot
in the casing at a depth of 154 m. Seven tiltmeters surrounded the well, located near Vernal, Utah (Figure 3), and the records from all seven show the same characteristics: a reasonably constant tilt rate, roughly proportional to injection rate, during the first 91 minutes sharply succeeded by a period punctuated by numerous changes in tilt rate which lasted until shut-in. The latter period, it was found, could be decomposed into four sub-phases during which the patterns of tilt development were stable across the entire array. The transitions between sub-phases were rapid and often coincident with sharp features in the pressure record. The patterns of accumulated tilting developed during the initial 90 minute phase (8:11 – 9:42) and during the succeeding period until shut-in (9:42 – 10:16) are shown in Figure 7a and 7b respectively. The latter, of course, is the sum of the individually complicated tilt fields developed during the four sub-phases. These two tilt fields were analysed by Holzhausen using a plane strain crack model. Although the model was not well suited to the task, for the fields are clearly three dimensional in form, subsequent modelling using a three-dimensional dislocation model upheld Holzhausen's results on fracture strike and dip. Specifically, a fracture strike of N10°W, steeply dipping to the ENE was required to explain the tilt field shown in Figure 7a whereas a fracture of identical strike but of different dip, this time toward WSW, was suggested by the field shown in Figure 7b. Thus it would seem that a significant change in the plane of fracture growth occurred after 91 minutes of fluid injection. Nine minutes prior to this breakout event wellhead pressure began to rise, attaining a peak value corresponding to an increase of 350kPa at the time of breakout, after which time it began to fall to the previous level (Figure 6). The mechanism underlying this increase in the hydraulic impedance of the well, which presumably prompted the breakout event, remains unresolved. However, it is clear that fracture development during the ensuing period was influenced to some degree by the behavior of the injected proppant. For the dominant spike in the pressure record, which climbed from ambient levels to breakdown values in only one minute was co-temporal with one of the identified transitions in tilt sub-phase. The pressure transient is most likely reflective of the development of proppant obstruction to fluid flow. Hence we suggest that proppant consolidation within the fracture system accounts for the somewhat complicated fracture development behavior following breakout, at least in part. It is noteworthy that the amplitude of tilting continued to increase at most sites throughout the transient pressure spike (Figure 6) from which we infer that the obstruction occurred within the fracture system rather than the wellbore.

Lake Gregoire, Alberta: As a final example of sudden fracture breakout, albeit in perhaps a different sense, we present in Figure 8, 24 hours of data collected during a five week cycle of steam injection into the unconsolidated oil sands of the cretaceous McMurray formation. The well, located near Lake Gregoire, Alberta, was cased and perforated over the interval 308 m to 317 m and was surrounded by a circular array of eight tiltmeters (Figure 9). Analysis of the wellhead pressure and injection rate data together with the tilt observations indicated that steam penetration into the formation took place.
episodically, at least in part, through the generation of transient horizontal fracture-like structures, each episode being accompanied by an increase in the hydraulic conductivity of the formation (Holzhausen et al., 1980). The characteristic signature of these uplift events as manifest in the tilt and pressure records can be observed in Figure 8 where two events of different amplitudes are present. The pattern of tilting typical of these events, as shown in Figure 9, suggested the development of a horizontal, approximately radially symmetric fracture. It is of note that no uplift episodes were observed during the first three weeks of steaming. One week prior to the start of steam injection the well was hydraulically fractured using cold water and the resulting pressure data indicated a vertical fracture to have formed. The three week delay in horizontal event onset is consistent with the notion that during this period significant modification of the initial in situ stress field took place as a result of formation heating. This modification resulted in a change in minimum principal stress at the depth of injection from a horizontal to the vertical direction as heating progressed (see also W.F. Bawden, this volume).

Lorain County, Ohio: In the previous examples, inadequate sampling of the surface deformation field prevented useful estimation of the lateral extent of the fractures from the surface tilt pattern alone. To do so generally requires a broad distribution of high quality data. This is because the dependence of the surface deformation field on lateral extent of the fracture is comparatively weak, especially where near vertical fractures are involved. The requisite data quality was, however, achieved in the treatment of Columbia Gas Well #20148-T which penetrated the Devonian shale in Lorain County, Ohio (Wakefield #1 Well in Figure 3) (Evans et al., 1982). Typical records from one of an array of eleven tiltmeters operated about the wellbore throughout the experiment are presented in Figure 10. During a five day period a total of seven separate treatments were administered to the well through the same perforation interval of 268-345 m in depth. The first six treatments were comparatively small (< 10 m³) dilute HCl fractures all of which were performed within a 24 hour period. The purpose of the preliminary series was both to break down the well and to facilitate the collection of acoustic emission data. The final treatment, some four days later, involved the injection of 219 m³ of a sand-CO₂-water mix, 11% of which was proppant. The comparative size of the treatments can be appreciated from Figure 10. The shut-in tilt patterns developed during each of the first six injections are shown in Figure 11. On inverting each of the tiltfields using the model of Sun (1969) with depth-to-centre, fracture volume and quotient value P_0/µ (or equivalently, within the framework of the model, radius) as free parameters, the values labelled on the appropriate tiltfields in Figure 11 were deduced. The results strongly suggested the continual reworking of the same horizontal fracture at a depth somewhat shallower than the uppermost perforation. Furthermore, the indicated fracture volumes were found to agree closely with the fluid volumes injected during the individual treatments from which it was inferred that fluid leakoff was negligible. This result was supported by results from analyses of cores taken from a nearby well which confirmed the population density
of natural fractures in the area to be low.

The main treatment was studied more rigorously than the previous six and four snapshots of the tiltfield taken at various times up to and including shut-in are shown in Figure 12. All four snapshots represent the tiltfield developed since the start of injection. Also shown in Figure 12 is a vector plot of the tiltfield developed during the three hours following shut-in. On taking suitable averages and estimates of the uncertainties in the data, the four co-injection tilt fields (Figures 12a-d) were inverted subject to the assumption of negligible fluid leakoff so as to constrain depth-to-centre and radius. The resulting space of acceptable solutions, shown in Figure 13, again indicated a fracture shallower than the top perforation and strongly favoured the suggestion that the treatment simply extended the unpropped fracture produced during the previous six treatments. The consistency between the results for the large-scale and small scale fractures was taken to suggest that fluid leakoff during injection was indeed small. On assuming a depth-to-centre, compatible with all treatments, of 230 metres, a fracture radius at shut-in of between 100 and 117 metres was deduced (Figure 13).

The conclusion that the horizontal fracture communicated with the wellbore via a near vertical natural fracture of orientation N3°E was suggested by acoustic emission data and supported by the wellhead pressure record (Evans et al., 1982). It remains curious, however, that no evidence of a subvertical fracture can be discerned in the tilt pattern at any time during series of treatments. We take this as an indication that the fracture was limited in extent and localised near the wellbore.

**POST SHUT-IN BEHAVIOR AND CONSOLIDATION OF FRACTURE STRUCTURE**

We now address the behavior of shallow hydraulic fractures following shut-in of the well. A general feature of the data is that the surface deformation pattern is seen to continue to evolve for some time following the termination of fluid injection. This is most certainly diagnostic of continued changes in fracture geometry controlled by three possible mechanisms; continued fracture advance, diffusive fluid leakoff into the formation (pores, vugs, microcracks) and fluid leakoff into extensive natural fracture systems of substantial fluid-consuming capacity, the distinction between the latter two categories being drawn principally for convenience of exposition. As each of these mechanisms result in closure of the hydraulic fracture, some relaxation of the deformation field developed during fluid injection operation might be anticipated. Although this is often the case, it will become evident from the examples considered that simple closure of the fracture structure developed up to the time of shut-in, due to fluid leak-off alone, cannot wholly account for the observations.

Knox County, Ohio: Continued active tilting was observed for some six minutes following shut-in of the Black #1 well. The initial pronounced response to shut-in was manifest as a stable trend in tilting which resembled the reverse of that established during phase I (Figure 4). After two minutes, however, a marked change in the
pattern of development took place corresponding to the time when well-head pressure had stabilised. Almost certainly the initial phase of stable tilting reflected closure of the initial sub-vertical fracture (Evans et al., 1982). The nature of the active tilting which persisted for a further four minutes, however, remains unresolved.

Vernal, Utah: A somewhat simpler example can be seen in the data collected near Vernal (Figure 6). Figure 7c shows a plot of the tilt-field developed during the one hour period following shut-in. Comparison with Figure 7a clearly reveals the tilting to be due to the gradual partial collapse of the fracture. The inferred smoothly decaying closure persists for at least five hours after which time the tilt signal cannot be discerned from tidal and thermally-induced noise (Figure 6). It is likely that the prolonged closure is a reflection of proppant consolidation. Analysis of the residual tilt-field persisting some five hours after shut-in suggested the width of the propped fracture to be about half that of the fracture prior to deflation (Holzhausen, 1979a).

Lorain County, Ohio: Perhaps the most interesting post shut-in data collected to date are those from the Wakefield #1 well (Figure 10). The data are unique in that the fracture appears to have grown without intersecting any extensive pre-existing 'bleed-off' system. On applying a predictive filter to the data to remove the Earth tide component, Evans (1981) was able to show that the obvious initial partial collapse of the pre-shut-in deformation field which decayed exponential-like over a period of three hours (Figures 10 and 12) was, in fact, smoothly succeeded by very low rate tilting in the same sense which persisted for a further seven hours at least. The two phase behavior was taken as indicative of the transition from predominantly hydraulic support of the fracture walls during the first three hours of shut-in to proppant support thereafter. The initial quasi-exponential decay was most likely a consequence of closure of the fracture walls facilitated by one of two mechanisms; continued fracture advance in response to stress corrosion cracking (Anderson and Grew, 1977) or fluid leakoff into localised cracks and vugs which, as a consequence of low formation saturation could have acted as a localised fluid volume sink. The inferred low fluid leakoff during injection, however, suggests that continued fracture advance following shut-in was indeed significant. Unfortunately, the data were insufficient to permit discrimination between these two mechanisms, though additional data from a few high stability instruments would have made this possible. At the end of the seven hour period of low rate tilting the deformation field established at shut-in had decayed by only 25% resulting in an estimate of propped fracture volume of $164m^3$. It is remarkable that this structure was supported by only $24m^3$ of proppant.

ESTIMATION OF IN-SITU SHEAR MODULUS

A by-product of applying fluid filled crack models to the interpretation of surface tilt data is an estimation of the effective static shear modulus of the medium at the scale of the fracture. These estimates are, by themselves, not well constrained for the
actual value of shear modulus sought is bound up in the model parameter \( P_D/U \). However, where a sufficiently accurate estimate of mode I fracture toughness, \( K_{IC} \), is available, the range of acceptable values of driving pressure, \( P_D \), can be limited to yield useful bounds on the value of shear modulus. The outcome of applying this procedure to the range of \( P_D/U \) values identified by the shut-in solution for the Wakefield #1 Well main frac (Figure 13) is shown in Figure 14. The strip bounded by the bold horizontal lines encloses all combinations of radius and driving pressure compatible with the main frac shut-in tilt data. The dotted lines denote contours of constant shear modulus predicted for a uniformly pressured penny-shaped equilibrium crack in an idealised medium of Poisson's ratio 0.25 (Perkins and Kern, 1961; Sun, 1969), and the two continuous lines identify the area of driving pressure-radius space for the same model geometry which is compatible with the fracture toughness bounds (U. Ahmed, personal communication). The shaded trapezoidal area identifies all combinations of radius, driving pressure and shear modulus which are compatible with both the fracture toughness data and the main frac shut-in tiltfield. The resulting estimate of effective static shear modulus of \( 1.4 \times 10^6 < \mu < 8.4 \times 10^5 \) kPa is certainly less than the values \( 10.5 \times 10^6 < \mu < 14.0 \times 10^5 \) kPa (measured normal to the horizontal bedding plane) determined from laboratory testing of cores (Ahmed et. al., 1981) but not substantially so. However, if we accept the likelihood that the same fracture plane was being reworked throughout the seven Wakefield treatments, then we can appeal to the depth solutions found for the initial six fractures to further constrain the estimate of shear modulus. From Figure 11 it can be seen that these solutions do not permit the fracture to be shallower than 230 metres. Thus, revising the upper bound on fracture radius to comply with this depth constraint results in an estimate of effective static shear modulus of \( 1.4 \times 10^6 < \mu < 4.2 \times 10^6 \) kPa, considerably less than the laboratory determined value. A similar result was found by Holzhausen (1979b) in modelling surface tilt data associated with fluid injection into oil shale. In this case a shear modulus of \( 3.5 \times 10^5 \) kPa was required to fit the data whereas laboratory tests revealed a value of \( 13.3 \times 10^5 \) kPa. The reduction in shear modulus value with increasing scale is well known to rock mechanists and engineers (Bieniawski, 1981) and is attributed to a class of cracks and imperfections in the bulk rock which are not represented in laboratory sized specimens. We note, however, that data pertaining to the appropriate value for static loads applied on the scale of hundreds of metres are few (Heuze, 1980).

In evaluating the accuracy of the above estimates two factors are of note. Firstly, the model used in the analysis (Sun, 1969) specifically assumes the fracture radius to be short compared to its depth thereby permitting the effect of the free surface on the fracture geometry to be neglected. Hence, the geometry assumed in generating the parameter bounds shown in Figure 13 is that appropriate to a deeply buried crack. However, as the depth-to-radius ratio of the fracture in question was only two, some investigation of the validity of the bounds is clearly warranted. Pollard and Holzhausen (1979) have investigated the mechanical interaction between a fluid
filled plane strain crack and the earth's surface. As a plane strain crack is more susceptible to free surface effects than one of penny-shaped geometry, their results provide upper bounds on the significance of the interaction. The results demonstrate that for cracks of depth-to-radius ratios of two, the form in cross-section remains essentially that of some equilibrium crack in an infinite medium. Hence, our estimates of fracture radius and depth remain valid because they were constrained within a purely geometrical framework by the surface tilt data. However, the value of the quotient, \( P_D/\mu \), required to produce this resolved geometry is reduced by near surface proximity for as a consequence of geometry it becomes physically easier for the horizontal crack to displace the overburden. In the plane strain case the resulting overestimate of \( P_D/\mu \) is less than 8\% (Pollard and Holzhausen, 1979) and hence the values of the contours of constant shear modulus shown in Figure 14 are too high by 8\% at most. In support of the contention that the crack in cross-section was adequately represented as an equilibrium crack in an infinite medium we investigated the change in crack tip stress intensity factors due to the proximity of free surface. The plane strain results of Pollard and Holzhausen (1979) for a horizontal crack of length-to-depth ratio similar to that resolved for the Wakefield main fracture show the mode II stress intensity factor to remain negligible and the mode I factor to increase in value above that for an infinite medium by less than 10\%. Thus, this increase is an overestimate for the penny-shaped geometry and in view of the uncertainty in estimating model fracture toughness, \( K_{IC} \), we consider the associated bounds shown in Figure 13 by continuous lines to be essentially valid. The lower bound on fracture toughness was taken from laboratory tests of Devonian shale cores under atmospheric pressure (Ahmed, personal communication). The upper bound represents an attempt to correct for the effects of confining pressure. Schmidt and Huddle (1977) and Abou-Sayed (1978) have both reported substantial increases in the measured mode I fracture toughness of Indiana limestone under confined conditions. Although both estimates of fracture toughness under atmospheric conditions were in approximate agreement, the reported increases under a confining pressure of 7MPa differed by their order of 50\%. In estimating our correction for Devonian shale we have assumed a dependence on confining pressure equivalent to the greater of the two reported (Abou-Sayed, 1978). It must remain for future investigations of Devonian shale to validate this assumption.

FRACTURE CLOSURE AND INITIAL SHUT-IN PRESSURE

Nevada Test Site, Nevada: An opportunity to study the relationship between fracture closure and fracture fluid-pressure following shut-in was provided by data collected during the formation of a suite of small hydraulic fractures in volcanic tuff at the Nevada Test Site during March 1979. The data consist of high quality pressure records (Warpinsky et al., 1982) and tilt records (Holzhausen, 1979c) derived from a number of tilt sensors located only tens of metres from the induced fractures. The experiment formed part
of a larger study, reported in detail by Warpinski et al. (1982), in which excavation was used to reveal the exact fracture geometries. The fractures were formed by pumping 0.6 m$^3$ of coloured water into adjacent, packed-off, 1.5 m intervals along a borehole, inclined 16° from horizontal, drilled 28 m into the well of a 457 m deep tunnel (Figure 15). Unfortunately, only three tiltmeters, located in a single vertical hole in the tunnel floor (Figure 15), were used to monitor fracture-induced deformations and hence resolution of fracture geometry is poor. On the other hand, the underground environment proved to be exceptionally quite from the viewpoint of tilt measurement and unusually high instrument stabilities were obtained.

The tilt and pressure time series shown in Figure 16 were collected during the formation of fracture #3 (Figure 15). A single channel of output from the lowermost tiltmeter is shown. Positive tilt indicates rotation in a plane containing the borehole CFE-1 such that the top of the instrument moves away from the fracture. The pressure and flow-rate data were obtained from sensors located at the borehole collar. Flow-rate during each cycle was constant at 30 litres/min and only one brief episode of flow-back took place in order to seal a packer leak which resulted in a premature shut-in at the end of cycle 3. The absence of a detected tilt response to the first pump cycle is ascribed to the small ($\approx$ 1 gallon) volume of fluid injected following breakdown (Figure 16). The tilt reference datum during the period spanned by the tilt time series is believed to be stable to better than 5 x 10$^{-9}$ radians; hence the offset in tilt remaining after each pump cycle is real. We attribute the offset to self-propping of the induced fracture and remark that the progressive increase in offset with pumped volume during successive cycles is not necessarily indicative of an increase in permanent set with the scale of the fracture. Rather, it may merely reflect the increase in fracture surface area (and hence total propped volume) with scale.

Provided that continued fracture extension following shut-in was at most small, the tilt decay curve following shut-in can be taken as a direct, albeit uncalibrated, indicator of the time-history of fracture volume decline and hence fracture closure. The 'calibration' however, will in general vary from one cycle to the next depending upon the geometry change achieved during each successive cycle. Clearly high-rate closure began immediately upon shut-in and persisted for a minute or so before decaying, exponential-like in most cases, to essentially zero at the permanent-set asymptote (Figure 16). The time for closure was largely independent of the volume of fluid injected during the cycle, ninety five percent of closure taking place within two minutes of shut-in. The high closure rate was in most part facilitated by substantial fluid leak-off into an intersected bed of highly fractured vitric tuff lying near the top of the Transition Zone (Warpinski et al., 1982). Leak-off into this system, as evidenced during mine-back by dye decorated fracture surfaces, also accounted for the flattening of tilt response to continued fluid injection during cycle 4. The shut-in pressure decline curves show similar exponential-like characteristics to the closure curves although a portion of the initial steep decline is most certainly due to equilibration of fluid pressure gradients in the borehole-fracture
system upon shut-in. As the fracturing fluid is water, however, this equilibration time is likely to be short. We shall return to this point shortly. The similar forms of the closure and pressure decay curves result from the stiffness of the closing hydraulic fracture whereby support of the fracture walls (or rather, in-situ stress component normal to the fracture plane) is progressively transferred from the fluid to the increasing number of asperities which come into contact as the fracture closes. In Figure 17 we present a photograph of a section of the pressure strip-chart corresponding to the breakdown cycle (boxed area in Figure 16). The tick marks at the foot of the chart define ten second time intervals. A transient arrest of the initial steep pressure decline can now be discerned as a consequence of the expanded time scale. Both the initial pressure drop and the transient plateau are of the order of one half-second duration which explains the absence of the feature in Figure 16. This plateau-like disturbance was a common feature of the pressure decline curves from all five fractures shown in Figure 15 and generally appeared most pronounced during the first few pump cycles. The feature almost certainly announces the attainment of hydraulic equilibrium conditions and hence, by definition, corresponds to the initial shut-in pressure (ISIP). When interpreted in this way, a spatially consistent description of in-situ minimum horizontal stress variations about the borehole CFE-1 was obtained (Warpinski et al., 1982). As an explanation of the transient feature we suggest that it represent a period of closure immediately following the hydraulic equilibration phase when the fracture briefly continues to display low stiffness characteristics. That is, during this period, the crack walls are almost exclusively supported hydraulically and the fluid pressure is thus identical to the component of in-situ stress normal to the plane of the fracture. Unfortunately, owing to a slow chartspeed and the absence of synchronous timemarks on the tilt records it is not possible to infer the closure behavior of the fracture during the first few seconds after shut-in (microseismic noise makes this tricky in general). Nonetheless, it is certain that substantial fluid leak-off was taking place throughout the period and hence it seems reasonable to conclude that closure was taking place continually from shut-in onward. The implied abrupt transformation to high stiffness characteristics associated with the resumption of steep pressure decline is, perhaps, a little surprising. Quite generally it was found that the resumed decline in pressure tended to be less steep when larger volumes of fluid had been injected during the cycle. We attribute this to the degree of mismatch of contacting asperities on closure. That is, where a good match in asperity topography exists between closing walls, a sudden stiffening in the fracture compliance characteristics would be expected. A greater mismatch would result in a more gradual stiffening with closure. A dependency of mismatch on injected volume could come about through translation of the fracture walls in response to non-zero shear stresses acting across the plane of the fracture. For, provided shear stress and effective shear strength of the fracture are uniform over the fracture face, the resulting shear displacement will be scale dependent. In this case a scale dependent increase in permanent set might also be anticipated to
a degree dependent upon the statistics of the fracture surface. This is most certainly consistent with the observations although, as we have previously stated, we were unable to determine whether the observed scale-dependent residual tilting could be explained entirely by the increase in fracture surface data. The example shown in Figure 17, where the resumed decline is as steep as the initial equilibration-induced decline, is typical of the breakdown cycles. In such cases the brief 'hiccup' in the pressure decline curve presented the only clear indication of the required ISIP. During the final (comparatively large volume) cycles, however, the resumed decline was noticeably less steep than the preceding equilibration-induced decline and the ISIP could be precisely identified by the resulting knee in the decline curve. Thus, coarse sampling of the pressure data (1 second sampling period or less) would result in a well-defined ISIP value for the final cycle only. Experiments involving careful digitisation and resampling of the pressure record suggested that a minimum of five samples per second were required in order to 'capture' the transient arrest in decline. We also note that it is unlikely such a short-period (< 1/2 second) pressure perturbation would have been detected by a pressure sensor located more than tens of metres from the fracture.

Conclusions

From the observations of surface deformations produced by the development of hydraulic fractures, the following conclusions are drawn:

(1) At all times during the treatments studied the pattern of change in surface tilting indicated the on-going fracture development to be occurring predominantly through the extension of some localized planar fracture structure.

(2) Occasional abrupt changes in the on-going pattern of tilt field development during several of the treatments testified that sudden changes in the plane of fracture growth (breakout or branching) is a common phenomenon, at least in the shallow crust.

(3) Significant changes in fracture geometry were often inferred to have occurred for some tens of minutes to hours following shut-in. Although fluid leak-off into the formations certainly accounted for some of the changes, there is evidence in some cases that significant post-shut-in fracture advance was occurring.

(4) The residual 'propped' volumes of fractures produced using proppant-laden fluid were many times the volume of proppant injected.

(5) The estimated value of the static shear modulus of Devonian shale appropriate to loading on a scale of hundreds of metres was found to be $1.4 \times 10^6 < \mu < 4.2 \times 10^6$ kPa.

(6) Comparison of the shut-in fracture closure and pressure decay
responses shows that some seconds after shut-in the fracture was closing as a stiff feature. There is evidence to suggest that during the first second, the fracture was closing as a compliant feature. The transformation from high compliance to high stiffness characteristics was abrupt rather than gradual.

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REFERENCES


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FIGURE CAPTIONS

Figure 1: Schematic diagram of a typical sensor installation used to collect the data. The sonde is a biaxial bubble-type tiltmeter capable of resolving tilts of the order of $5 \times 10^{-9}$ radians. Disturbances in the Earth's shallow surface layer, primarily of meteorologic origin generally limit instrument resolution to below this value.

Figure 2: Schematic representations of the three models discussed in the text.

Figure 3: Location of three of the experiments discussed in the text. The fourth experiment took place 50 km southeast of Fort McMurray, Alberta, Canada, at a site near Lake Gregoire.

Figure 4: Tilt time series recorded during the treatment of Black #1 well, Ohio. The wellhead pressure and volume flow rate records are also shown. The vertical dashed lines denote the times of breakdown and shut-in. The vertical continuous lines identify the times of sudden changes in tiltfield development pattern. The location of the sensors is shown in Figure 5.

Figure 5: Patterns of surface tilting developed during each of the three phases of fracture development identified in Figure 4.

Figure 6: Unfiltered tilt records from site #2 together with wellhead pressure and flow-rate data recorded during the Vernal, Utah experiment.

Figure 7: Patterns of surface tilt developed during each of the three time windows identified by the vertical lines in Figure 6. The bold lines represent contours of zero tilt predicted by the best-fitting plane-strain model.

Figure 8: 24 hours of radial tilt and wellhead pressure data simultaneously recorded during a five week period of steam injection into oil sand at Surmont, Alberta. The arrows mark the onset of the uplift episodes discussed in the text.

Figure 9: Typical pattern of tilting at the onset of collapse of an uplift event. The crossed circles denote the location of instruments.

Figure 10: Tilts recorded by a tiltmeter during the Wakefield #1 Well series of experiments. The initial cluster of six injections is marked by the short vertical lines beneath the
record. Each line corresponds to one pump. The smooth undulations on the records are the Earth tides.

Figure 11: Snapshots of the tilt patterns at shut-in for the initial six treatments administered to the Wakefield well. The specified model solution parameters refer to the horizontal, penny-shaped uniformly pressured, fracture model which best fits the tilt pattern.

Figure 12: Snapshots of the evolving tilt pattern taken at various times during the Wakefield well main treatment of October 7th. The final frame is the tiltfield developed during the three hours following shut-in as estimated from the tide-free data.

Figure 13: $P_p/\mu$ versus depth-to-centre solution space for each of the four co-injection snapshots shown in Figure 12. Each of the closed figures encloses all pairs of $P_p/\mu$ and depth-to-centre values which are consistent with the tilt data to within some chosen margin of error (see Evans, 1981). The vertical bold lines mark the maximum and minimum bounds on fracture depth imposed by the pattern of shut-in tilting. The dashed vertical bold line indicates the shallowest depth consistent with the solutions from the preliminary series of six treatments. The arrows denote the bounds on shut-in fracture radius which result from the depth constraints.

Figure 14: The space of combinations of radius, driving pressure and shear modulus values which are consistent with the shut-in tilt pattern assuming the fracture to be given by a uniformly pressurized penny-shaped equilibrium crack is in an infinite idealized medium of fracture toughness between 1333 and 2000 kPa-m$^{1/2}$. The area between the two bold horizontal lines represents all combinations which are consistent with the shut-in tilt data alone. The bold dashed horizontal lines denote the upper bound on radius imposed by the preliminary treatment solutions. The dashed lines identify contours of constant shear modulus predicted for a penny-shaped crack of volume 219m$^3$ in an infinite idealized medium of Poisson's ratio 0.25 as a function of driving pressure and crack radius. The continuous lines bound the area of parameter space consistent with the adopted range of permissible fracture toughness values. The total hatched (single and double) area denotes the region constrained from the shut-in data alone and the double hatched area is the region which has been further constrained by the solutions for the preliminary treatments.

Figure 15: Cross-section through tunnel showing location of induced fractures, tilt sensors and principal lithologic
boundaries.

Figure 16: Pressure and tilt records obtained during the formation of Fracture #3. The arrows denote the location of the ISIP indicating feature, an example of which is shown in Figure 17.

Figure 17: Section of pressure strip chart corresponding to the boxed area of Figure 16. The arrow indicates the 'kink' feature discussed in the text. The ticks at the foot of the chart define 10 second time intervals.
CAPPED CASING (12"
SAND (Tamped)

DATA LOGGER
(30sec Sample Rate)

CABLE

SENSOR

Figure 1
Figure 2
FRAC PHASE I
30 OCT 79
FLUID VOLUME INJECTED: 717 m³ (at 10000 psi, 10°C)
MAX INJECTION RATE: 15 m³/min
DURATION OF PHASE: 16 minutes

FRAC PHASE 2
30 OCT 79
FLUID VOLUME INJECTED: 159 m³ (at 10000 psi, 10°C)
MAX INJECTION RATE: 14 m³/min
DURATION OF PHASE: 8 minutes

FRAC PHASE 3
30 OCT 79
POST SHUT-IN RESPONSE
DURATION OF PHASE: 6 minutes

Figure 5
SITE 2: VERNAL, UTAH
INJECTION DEPTH: 143 m
INJECTED VOLUME: 239 m³

RADIAL TILT
(+VE = TOWARD WELLBORE)

TANGENTIAL TILT

WELLHEAD PRESSURE

INJECTION RATE

LOCAL TIME: 25th MAY 1979

Figure 6
Figure 8
EVENT OF 29th AUGUST 1979: SURMONT ALTA.

Figure 9

WELL 11-20

131m.

MICRORADIANS
7X: TANGENTIAL TILT ABOUT WELLHEAD

7Y: RADIAL TILT (UP = TOWARD WELLHEAD)
Penny-shaped equilibrium crack in an infinite medium:
Fracture Volume $= 219 \text{ m}^3$
Poisson's Ratio $= 0.25$

$\mu = 1.4 \times 10^7 \text{kPa}$
$\mu = 7 \times 10^6 \text{kPa}$
$\mu = 3.5 \times 10^6 \text{kPa}$
$\mu = 1.4 \times 10^6 \text{kPa}$

$K_{IC} = 1333 \text{kPa} \cdot \text{m}^{1/2}$
$K_{IC} = 2000 \text{kPa} \cdot \text{m}^{1/2}$

Figure 14
NEVADA TEST SITE
DRIFT: EV-6
DEPTH: 457 m

HYDRAULIC FRACTURES
(Fluid Volumes: 0.6 m³)

DISTANCE FROM SENSOR HOLES (m)
24.8 13.7 9.5 5.8 3.4

TILT SENSOR DEPTHS (m)
5.2

Figure 15
CFE-I: FRACTURE 3

Figure 16
STATE OF THE ART AND FUTURE PLANS ABOUT HYDRAULIC FRACTURING STRESS MEASUREMENTS IN SWEDEN

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Background and Aims

Virgin rock stresses are one of the critical parameters to be determined in the site investigation program for radioactive waste disposal in crystalline rocks. The author has suggested that a repository should be located in a large homogeneous block of rock (about 4x4x4 km$^3$) and surrounded by extensive weak zones, like faults or shear zones. Any large-scale loading or displacement that is going to affect the area of the repository must then be absorbed or released by the weak zones, and the storage will be protected. An application of this technique of storage demands rock stress measurements inside the block, along the weak zones and in the surrounding rock mass.

Theoretical analysis has shown that the heat load from a repository at 500 m depth gives tensile stresses at the ground surface above the repository. To avoid the development of new fractures or opening of existing joints, due to thermal loading, and minimize the risk of migration of radioactive material to the biosphere, a repository should be located in an area with an excess of horizontal stress. Summarizing our present knowledge of rock stresses in Sweden, we find the suitable excess of horizontal stress in areas of Precambrian rocks. However, the generality of this situation has to be verified by more field measurements.

Recent seismic studies in Sweden have proven the existence of shallow earthquakes with low magnitude. Focal mechanisms and fault plane solutions seem to indicate that the intraplate earthquakes are localized to existing discontinuities. A causative mechanism or critical characteristics for the shallow earthquakes have not yet been identified. Rock stress measurements will add new knowledge and help to sort out the major problem, whereas the stresses are anomalous or the rock strength is low in areas of shallow earthquakes.

In summary, the hydraulic fracturing stress measurements in Sweden will be directed toward stress determination in glaciated, jointed and faulted crystalline rocks, with special emphasis to determine rock stresses in potential areas for storage of radioactive waste. Rock stress measurements will also be undertaken in areas of shallow intraplate earthquakes of low magnitude in Sweden.
Equipment for Field Hydraulic Fracturing Tests

Equipment for field tests in 500 m deep, 3 inch boreholes is designed to fracture crystalline rocks at a maximum hydrofracturing pressure of 40 MPa. The basic principal for hydrofracturing and fracture orientation is shown in Fig. 1. Originally, the system of multi-hose linked to a hydraulically driven feeder and drum was developed for hydrological tests in deep boreholes of the Swedish nuclear waste program (KBS). A slightly modified version of the system will be used for the hydraulic fracturing tests. The multi-hose consists of a bearing wire, three hoses, two cables for signals and an additional supplement of plastic in order to obtain a displacement equal to the density of water. Hence, the multi-hose will float in the borehole and the capacity of the wire, 3.3 tons can be used to pull out the down-hole equipment if it gets stuck.

The down-hole equipment during hydrofracturing consists of a transducer for pick up of acoustic emission (45-1,000 Hz) from fracturing, straddle-packer and a pressure transducer to record the down-hole fracturing pressure, pore pressure and shut-in pressure. The straddle-packer and the fluid pressurization are operated by means of two air pumps. The pressure and flow in the fracturing interval is monitored on the surface and the down-hole pressure is going to be double checked by a pressure transducer below the upper inflation element. The data acquisition system consists of a signal conditioner and a chart recorder with three channels.

The fracture orientation will be determined by a TV-camera which displays the walls of the borehole and the orientation of the camera by means of a compass. This device along with the emission transducer will be connected to the multi-hose. If the fracture orientation for any reason cannot be detected by the TV-camera, an impression packer with a thin layer of soft rubber will be used.

The pressurization unit and data acquisition system is manufactured and the feeder system, drum and multi-hose is in production at the time of the presentation of this paper. The first field tests will be run in 1982 at the Kiirunavaara iron ore mine, Northern Sweden or at the nuclear power plant of Forsmark, Central Sweden. For both sites, the state of stress is known from stress measurement by overcoring technique.
Theories of Induced Stresses from Packers

Kehle (1964) investigated the conditions of normal fracture initiation near the ends of a pressurized borehole. His model assumes that a packer is a rigid cylinder in full contact with the wall of the borehole. Due to an axial load at the end surface of the packer shear stresses are applied to the surrounding rock mass which might create fractures perpendicular to the axis of the borehole.

Haimson (1974) opposed the suggestion by Kehle and stated that as rubber used in the packers is a "liquid-solid" elastomer the axial load mentioned by Kehle would probably apply a similar radial load to the rock.

The axial and circumferential stresses generated by the normal stress imposed by the inflated packer on the rock mass might be quite large as demonstrated in a theoretical analysis by Warren (1981). The rock mass is modeled as an unbounded linear elastic material containing an infinitely long circular cylindrical cavity, and the packer is modeled as a semi-ininitely long, thin walled cylindrical shell with elastic modulus much less than that of the surrounding formation. Results of the analysis show that the maximum circumferential stress $\sigma_{\theta\theta}$ and the maximum axial stress $\sigma_{xx}$ induced in the rock mass by the inflated packer are dependent upon the difference between the packer pressure $P_{p}$ and the hydraulic fracturing pressure $P_{H'}$, i.e., $\Delta P = P_{p} - P_{H'}$. These maximum stresses occur in the sealed off or fracturing region of the borehole and very close to the point of contact of the packer. Warren (op. cit.) states that for typical packer parameters and pressures, the circumferential stress is of the order of $\sigma_{\theta\theta} \approx 0.7\Delta P$ while the axial stress is of the order of $\sigma_{zz} \approx 9.5\Delta P$ which may be enough to initiate fracture of the borehole before any hydraulic fracturing pressure is applied.

Typical stresses resulting from Warren's numerical evaluation for the specific case of $P_{p} = 7\text{MPa}$ and $P_{H} = 3.5\text{ MPa}$ are shown in Figure 2. Notice that the theory does not take into account the reduction of stresses from the mandril of the straddle packer. For the example of Figure 2 the maximum tensile stresses are $\sigma_{\theta\theta} = 1.76P_{H}$ and $\sigma_{zz} = 0.48P_{H}$ which are substantially in error compared to the assumption in hydraulic fracturing where $\sigma_{\theta\theta} = P_{H}$ and $\sigma_{zz} = 0$ along the entire pressurized region.
Figure 2. Thoretical packer induced stresses along borehole surface for $P_o = 7$ MPa and $P_H = 3.5$ MPa. After Warren, 1981.
Laboratory Tests of Packers in Tubes

Independent of the theoretical analysis of Warren packer induced stresses have been tested in the laboratory of the University of Lulea. A set of straddle packers manufactured by Lynes and with an outer diameter of 68mm and a length of 935mm were tested in a long steel tube with outer diameter 89mm and inner diameter 76mm. The packers were connected by a 530 mm long steel mandril with diameter 32 mm. Pressurization of the packers and the sealed-off section between the packers was conducted with two separate air-driven Haskell pumps, with a capacity of 4 and 9 liters per minute respectively. The system for testing the straddle packers is shown in Figure 3.

Figure 3. System for testing straddle packers.
pressurization cycle the straddle packers are moved 0.5-2 cm inside the tube and a new cycle of pressurization and strain recording is conducted. The movement of the packer is limited to 0.5 cm in positions where large stress gradients are expected. By means of the technique to move the packers inside the tube a large number of strain readings have been taken and the continuous strain distribution over the packers and the sealed-off portion has been determined. Applying the extended Hooke's law for plane stress condition the circumferential stress $\sigma_{\theta \theta}$ and the axial stress $\sigma_{zz}$ have been calculated from the recorded strains.

The recorded circumferential and axial strain on the outer surface of the tube from pressurization of a single packer to $P = 20$ MPa is shown in Figure 5A. Several strain gradients are obtained at the edge of the packer and the axial strain, $\varepsilon_{zz}$, has a complicated distribution along the tube. The calculated stresses are shown in Figure 5B where maximum tensile stress appears at about 3, 5 cm from the edge of the packer and a maximum compressive stress appears in the axial direction at the edge of the packer. It is important to realize that the results presented in Figure 5 are valid only for the outer surface of the tube. This means that they can not be directly compared to the results presented by Warren (1981). However, we notice that the change of stress and the stress gradients over the packer edge looks a lot different in the experimental test results, cf. Figure 2 and Figure 5. Further, the theoretical analysis gives compressive stresses in the axial direction over the packer, whereas the experimental results show tensile stresses over the packers all with the assumption of plane stress conditions.

Tangential and axial strain from the edge of a pressurized straddle packer with steel mandril in a tube is shown in Figure 6. The test was performed at constant packer pressure $P = 20$ MPa and varying pressure in the sealed-off portion, $P_R = 0$, 10.0, and 19.5 MPa. As the pressure increases in the sealed-off portion the tangential strain as well as the tangential stress increases. At the same time the axial strain is reduced and the strain concentrations at the edge of the packer are evened out.

The strain data for $P = 20$ MPa and $P_R = 10$ MPa in Figure 6 have been used to study the influence of the steel mandril to the overall stress distribution of a straddle packer. For that reason the strains were also recorded for a test configuration, whereby two packers without a mandril were placed in the tube. The calculated stresses for the outer surface of the tube in
Strain rosettes were mounted on the circumferential of the tube to allow for recording axial strain, $\varepsilon_z$, and circumferential strain, $\varepsilon_{\theta\theta}$. The rosettes are of type Kyowa, KFC-5-D16-11 L30 and contain two strain gauges. Rapid adhesive Z70 of HBM was used to apply the strain rosettes to the tube. A 12-channel data logger with a resolution of ±1με was used as a recording unit. The position of the strain rosettes on the tube is shown in Figure 4.

![Figure 4. Testing of packer induced stresses in a tube; A, tube with mounted strain rosettes. B, straddle packer. C, arrangement of strain rosettes close to the end of the pressurization area.](image)

A typical testing procedure is as follows. The straddle packer is installed and a zero recording of the strain gauges is taken. Then the packers are inflated to a pressure $P_0 = 20$ MPa and a new recording is taken. As the inflation pressure is kept constant the pressure in the sealed-off section, $P_H$, is increased in steps of 5 MPa and the strains are recorded for each step. When the pressure in the sealed-off section approaches the packer pressure leakage appears. The limit for water leakage is found to be $P_H < P_0 + 0.4$ MPa for the particular packer-tube configuration. When leakage appears the pressure $P_H$ is decreased in steps of 5 MPa back to the starting point and $P_H$ strains are automatically recorded for each step. After completion of one
Figure 5. State of strain and stress from the end of a pressurized single packer in a tube. A, axial strain, $\varepsilon_{zz}$ and circumferential strain, $\varepsilon_{\theta\theta}$ at pressure, $P_o = 20$ MPa. B, axial stress $\sigma_{zz}$ and circumferential stress $\sigma_{\theta\theta}$ at pressure $P_o = 20$ MPa. C, test arrangements.
the vicinity of the packer with and without mandril are shown in Figure 7. Peaks of tangential and axial stresses are found to be shifted in the direction of the packer with maxima 5-6 cm from the edge. We also notice how the axial stresses are reduced by a factor 2/3 as the steel mandril is introduced. These results speak in favor of hydraulic fracturing and reduces the anticipated risk of fracture initiation to occur very close to the packer end.

From these results it appears quite clear that in order to reduce the effect of packer induced stresses during hydraulic fracturing for rock stress determination the pressure difference, \( \Delta P = P_0 - P_1 \), should be kept small. The laboratory tests indicate that \( \Delta P \) can be as small as 0.4 MPa before leakage starts. The mandril helps to reduce the stresses as indicated in Figure 7 and stiff steel mandrils are, therefore, recommended to be used in hydraulic fracturing for rock stress determination.

According to the theory of thick-wall cylinders with internal pressure, \( P_i \), and zero outer pressure, the circumferential stress has the form

\[
\sigma_{\theta \theta} = \frac{a^2 P_i}{b^2 - a^2} + \frac{a^2 b^2 P_i}{(b^2 - a^2) \cdot r^2}
\]

where \( a \) is the inner radius, \( b \) the outer radius and \( r \) is the radius of the cylinder. For \( r = b \) equation (1) becomes

\[
\sigma_{\theta \theta} = \frac{2 P_i}{b^2 - (b^2 - 1) a}
\]

This equation might be used to check the results of the stress determinations over the packers and over the sealed-off section between the packers. The average value of the inner and outer radii of the tube was found to be

\[
a = 38.08 \text{ mm} \\
b = 44.39 \text{ mm}
\]

which gives the relationship

\[
\sigma_{\theta \theta} = 5.57 \cdot P_i
\]
Figure 6. Tangential and axial strains from the end of a pressurized straddle packer with steel mandril in a tube.
Figure 7. Tensile stress from the end of a pressurized straddle packer with and without mandril. The mandril helps to reduce the axial stress in the sealed-off portion.
Insert \( P_1 = P_2 = 10 \text{ MPa} \) in eq. (3) for the sealed-off portion gives \( \sigma_\theta = 55.7 \text{ MPa} \). The calculated value of circumferential stress from experimental strain measurements is \( \sigma_\theta = 55.5 \text{ MPa} \) as shown in Figure 7.

If we apply eq. (3) to the portion over the packer for \( P_1 = P_2 = 20 \text{ MPa} \) it gives \( \sigma_\theta = 111.4 \text{ MPa} \). The calculated value from applying the results of the strain measurements is 109.6 MPa. The discrepancy is due to the reduction in radial pressure at the packer-tube interface due to the stiffness of the packer. According to Warren (1981, eq. 33) the magnitude of the normal contact stress at the mid of a packer is

\[
\sigma_o = P_o \left(1 - \frac{4\beta^4 D \delta}{P_o}\right)
\]

where \( P_o \) is the packer pressure and \( \beta, D \) and \( \delta \) are constants depending upon the geometry and material properties of the packer. For \( o = 19.7 \) - calculated for the Lynes packer and inserted in equation (3) the tangential stress at the surface of the tube will be \( \sigma_\theta = 109.2 \). Hence, from this analysis of the circumferential stresses we might conclude that there is good agreement between theoretical and experimental results.

The axial force in the sealed-off portion of the tube due to pressurization is \( F = A \cdot P \). This force can be transmitted via the tube and the mandril. If we assume the force to be transmitted via the tube the axial stress will be

\[
\sigma_{zz} = \frac{F_H}{A_H} = \frac{\pi a^2 P_H}{\pi (b^2 - a^2)} = \frac{a^2 P_H}{b^2 - a^2}
\]

With measured values of \( a \) and \( b \) for the tube and inserted in eq. (5)

\[
\sigma_{zz} = 2.79 \cdot P_H
\]

Insert \( P_H = 10 \text{ MPa} \) in eq. (6) gives \( \sigma_{zz} = 27.9 \text{ MPa} \). The calculated value of axial stress of the sealed-off portion in the experiment without mandril is found to be \( \sigma_{zz} = 27.8 \text{ MPa} \), cf. Figure 7.
Preliminary Results from Laboratory Testing of Hydraulic Fracturing of Jointed Rocks

Hydrofracturing specimens were prepared from drill-cores of a medium grained, jointed porphyry with granitic composition from the foot-wall of the Kiirunavaara orebody, Kiruna, Sweden. The specimens had a diameter of 72 mm and a length of 20 cm. A centrum hole with a diameter of 10 mm was drilled to a depth of 2/3 of the specimen length.

Specimens were placed in a biaxial chamber and loaded with a circumferential pressure that varied from 7 to 40 MPa. The axial load was applied by a stiff rock mechanics loading machine in the range of 5-40 MPa. The hydraulic fracture pressure was introduced through the upper piston and critical pressure was obtained in the range of 12-40 MPa. Altogether, 16 samples were tested. Axial and radial strain gauges were mounted on the surface of 8 specimens at a distance of about 5 cm from the end surface containing the central hole. The idea was to check the deformations due to variations in loading and to calculate the theoretical strains for assumed values of the elastic constants of the rock.

Four samples out of 16 did show tensile failure in a radial direction, according to the theory of hydrofracturing. Two samples show tensile fracturing perpendicular to the axis of the central hole. Critical hydraulic pressure led to failure and/or leakage along existing discontinuities for the remaining 10 samples. Plotting of critical pressure $P_c$ versus confining pressure $P_c$ indicate a uniaxial tensile strength of 14 MPa. The slope of the curve for $P_c$ versus $P_c$ is 0.5; i.e. one quarter of the theoretical value, Fig. 8. The low values of the critical pressure as a function of confining pressure is due to the pore pressure built up in the vicinity of the borehole and along preexisting discontinuities. In order to separate the effect from each of these agencies, fracturing will be performed with a mixture of water and a fluorescent liquid. Preliminary tests have indicated a possibility to separate these effects from analyzing the samples with a fluorescent lamp after hydrofracturing. This work will continue along with determination of fracture toughness of natural joints in the laboratory.
Figure 8. Laboratory experiments of critical breakdown pressure as a function of confining pressure to Kiirunavaara prophyry.
BIAXIAL TESTING AND HYDRAULIC FRACTURING OF PORPHYRY FROM KIRRUNAVAARA, KUJCA.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Axial Stress $\sigma_z$ (MPa)</th>
<th>Conf. Stress $P_0$ (MPa)</th>
<th>Critic. Pressure $P_C$ (MPa)</th>
<th>Total Strain at $P_C$</th>
<th>Radial Strain $\epsilon_r$ ($\mu$e)</th>
<th>Axial Strain $\epsilon_z$ ($\mu$e)</th>
<th>Young's Modulus $E$ (GPa)</th>
<th>Poisson's Ratio $\nu$</th>
<th>Fluid Volume $V$ cm$^3$</th>
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<td>Conf. Stress ( \sigma_0 ) (MPa)</td>
<td>Critic. Pressure ( p_c ) (MPa)</td>
<td>Radial Strain ( \varepsilon_r ) ( \mu \varepsilon )</td>
<td>Axial Strain ( \varepsilon_z ) ( \mu \varepsilon )</td>
<td>Young's Modulus ( E ) (GPa)</td>
<td>Poisson's Ratio ( \nu )</td>
<td>Fluid Volume ( V ) cm(^3)</td>
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<td>1</td>
<td>Rough, tensile failure. Strain gauges applied at 5 cm from the top of the sample.</td>
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<td>Tensile failure at top two parallel fractures at the top and an incline 40° at the trunk.</td>
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<td>3</td>
<td>Tensile failure ( \perp ) axis; no sign of radial fracture.</td>
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<td>4</td>
<td>Tensile failure lined up to an existing joint. Intersection of hole and the joint caused the failure.</td>
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<td>5</td>
<td>Tensile failure. Branches of joints at top surface.</td>
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<td>6</td>
<td>Tensile failure. Clear cut fracture 7 cm deep and symmetric.</td>
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<td>7</td>
<td>Tensile failure across the core along joint surface with pyrite coating. Failure at the intersection between hole, flat planar surface and steep set of joint.</td>
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<td>8</td>
<td>Tensile failure with strong branching at the top surface. Failure stopped along the axis at an inclined filled joint.</td>
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<td>9</td>
<td>New fracture from hole to inclined joint. Leakage started at $P_i = 12$ MPa</td>
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<td>10</td>
<td>Opening of existing joint of a length about 1/2 circumference. Leakage started at $P_i = 11$ MPa</td>
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<td>11</td>
<td>Failure along existing joint dipping 70° and interconnected to the bottom of the central hole. Variation of $P_o$, $C_2$ and $P_i$. Increase of $P_o$ increases $P_i^c$.</td>
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<td>12</td>
<td>Tensile fracture according to theory. Very thin fracture along the axis of the core. Minor branches parallel to the main joint. Leakage started at $P_i = 35$ MPa</td>
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<td>13</td>
<td>Tensile fracture along with a branch perpendicular to a long fracture. Minor leakage before failure.</td>
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<td>14</td>
<td>Leakage of two filled joints dipping 60°. Leakage started at $P_i = 16$ MPa</td>
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<td>15</td>
<td>Leakage of two filled joints dipping 60°. Leakage started at $P_i = 16$ MPa</td>
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<td>16</td>
<td>Leakage of one joint at the top and a minor leak at one joint at the bottom of the sample.</td>
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Concluding Remarks

Hydraulic stress measurements in Sweden is in progress and a new equipment will be tested in late 1982. Measurements will be directed towards stress determination in jointed and faulted crystalline rocks. Within the Swedish-American cooperative program on radioactive waste storage in mined caverns in crystalline rocks, hydraulic fracturing has been used to determine the state of stress at the Stripa mine, in central Sweden, DOE (1982).

Hydraulic fracture measurements in G Tunnel, Nevada Test Site has brought attention to the problem of packer-induced stresses on the borehole. Observations made during mineback of the fractures show they were initiated at or near the packers, Smith et al. (1981). Theoretical analysis by Warren (1981) demonstrated the possibility of significant tensile stresses at the end of the packers. Results from laboratory experiments of pressurization of a straddle-packer in a long tube are somewhat contradictory and do not indicate tensile stress concentrations at the ends of the sealed-off portion of the tube. To minimize packer-induced stresses the difference between packer and hydrofrac pressure should be kept as low as possible during the entire fracturing operation. This is evident from both theoretical analysis and experiments. Results from packer tests in tubes also show that a steel mandril helps to reduce packer induced stresses during hydraulic fracturing for rock stress determination.

Preliminary results from laboratory testing of hydraulic fracturing of jointed rocks indicate that only one quarter of the samples show tensile failure in a radial direction according to the theory of hydrofracturing. The rest of the samples show leakage and/or aperturing of existing joints. This effect and the fact that a pore pressure is built up in the vicinity of the borehole indicate the need of a fracturing technique that exclude liquids as a fracturing agency. The packer fracturing technique developed by Plumb (1982) might be one solution to the problem. Here the packer isolates fluids from the rock and the packer induced fracture will form parallel to the maximum principal stress when the normal contact stress induces tensile tangential stress just equal to the tensile strength of the rock.

Another technique under development, Stephansson (1982), is called sleeve fracturing, whereby the fracturing pressure is generated in a Adiprene membrane. For low pressures the technique is equivalent to rock deformability measurements by
means of the so-called CSM-cell, Hustrulid (1975). As the pressure is increased in the cell, the normal contact stress increases and a fracture will form when the tensile tangential stress just equal the tensile strength of the rock. Laboratory tests on limestone and sandstone blocks under uni-bi- and tri-axial loading show that fracture initiation is followed by a pressure drop in the sleeve. The break down pressure and the pressure for opening the fracture allow for calculation of the principal stresses perpendicular to the axis of the borehole. Direction of maximum principal stress is determined by impression of the fracture on a thin plastic tape wrapped around the sleeve.

Acknowledgment

The author is indebted to Arne Torikka for conducting the laboratory tests of the packers, and to Bengt Leijon for helpful discussions. I want to thank Rebecca Marsh for typing the manuscript. This work was supported by the Swedish Board for Technical Development.
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