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25 July 1986

David Tiktinsky - SS623
U.S. Nuclear Regulatory Commission
Division of Waste Management
Washington, D.C. 20555

"NRC Technical Assistance for Design Reviews"
Contract No. NRC-02-85-002
FIN D1016

Dear David:

Enclosed is the review of the document "The State of In-Situ Stresses Determined by Hydraulic Fracturing at the Hanford Site" by Kunsoo Kim, Steven A. Dischler, James K. Aggson, and Michael P. Hardy (RHO-BW-ST-73P). Please call me if you have any questions.

Sincerely,

Roger Hart

Roger Hart
Project Manager

cc: J. Greeves, Engineering Branch
Office of the Director, NMSS
E. Wiggins, Division of Contracts
DWM Document Control Room

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ITASCA DOCUMENT REVIEW

<u>File No.</u>: 001-02-21

<u>Document</u>: "The State of In-Situ Stresses Determined by Hydraulic Fracturing at the Hanford Site" by Kunsoo Kim, Steven A. Dischler, James K. Aggson, and Michael P. Hardy (RHO-BW-ST-73P)

<u>Reviewer</u>: Itasca Consulting Group, Inc. (M. Board)

<u>Date Approved</u>:

<u>Date Review Completed</u>: 25 July 1986

<u>Significance to NRC Waste Management Program</u>

This document reviews the present knowledge of the in-situ state of stress at the Hanford site and recommends values to be used in future design and performance assessment activities. The state of stress at the site and its implications toward pre- and post-closure repository performance is one of the critical issues identified in the Final Environmental Assessment (FEA). The values recommended in this report will control, to a great extent, the structural design of the repository and its performance.

<u>Summary of the Document</u>

This document reviews the hydraulic fracturing stress measurements made at depth from surface boreholes drilled within the Hanford site. In addition, in-situ stress-related phenomena (such as microearthquake swarms, core disking, and borehole spalling which occur within the Pasco Basin) are reviewed with reference to the hydraulic fracturing measurements.

The general conclusions of the report are as follows.

1. The maximum principal stress is horizontal and oriented in a north-south direction. The intermediate stress is horizontal in an east-west direction, and the minimum principal stress is vertical.

2. The stress directions are consistent with focal plane solutions from shallow seismicity, east-west spalling of boreholes, and the fold structures at the site.

3. The estimated maximum and minimum horizontal stress magnitude at the repository horizon for all hydraulic fractures at the candidate horizon are:

$$\sigma_{Hmax} = 61.1 \pm 5.4 \text{ MPa}$$

$$\sigma_{Hmin} = 33.8 \pm 2.7 \text{ MPa}$$

The estimated maximum and minimum horizontal stress magnitude at the repository horizon from tests within the Reference Repository Location (RRL) are:

$$\sigma_{Hmax} = 61.1 \pm 4.9 \text{ MPa}$$

$$\sigma_{Hmin} = 33.4 \pm 2.7 \text{ MPa}$$

4. The stress ratios resulting from this data are consistent with theoretical calculations of the core disking phenomena on the site.

5. Hydraulic fracturing measurements within the RRL and within the Grande Ronde candidate horizon indicates little variation laterally with depth or basalt flow.

6. The recommended design stress levels for the Cohassett Flow are:

$$\sigma_{Hmax} = 61.5 \text{ MPa, north-south}$$

$$\sigma_{Hmin} = 32.8 \text{ MPa, east-west}$$

$$\sigma_{V} = 24.2 \text{ MPa}$$

Several problems exist with the measurements or methods of analysis. Stress measurements were made only in areas with minimal fracturing and without the presence of sidewall slabbing. The presence of sidewall slabbing could indicate higher stresses. Therefore, choosing unfractured or spalled intervals may bias the results. Fracturing was performed only in the entablature of the Cohassett Flow. No data are obtained for colonnade or the interior vesicular zone. Only five measurements of stress in three holes have been performed in the Cohassett Flow. Although the standard deviation of these measurements is low, five measurements do not provide a statistically valid base of data. There is some controversy as to whether or not pore water pressures should be accounted for in the analysis of hydraulic fracturing data in hard rocks (Pine, 1983; Hickman and Zoback, 1982). The use of a pore water pressure term decreases the calculated value of the principal stresses. Therefore, the values reported in this document for design may not be conservative. The stresses have been recalculated assuming a zero pore pressure and are presented in Appendix C of the report. With this assumption, the average of the maximum horizontal stress in the Cohassett Flow is roughly 10 MPa higher—or over 70 MPa. The horizontal-to-vertical stress ratio also increases within the range of 2.5 to 3.0.

In general, the report does a good job of presenting the available data, both quantitative and empirical, which relate to in-situ stress at the Hanford site. The data base for measurements is small and primarily relates to the Grande Ronde flows within the Reference Repository Location (RRL). Only seven tests from holes DC-4 and DC-12, outside the RRL, are given. In addition, many measurements have been discarded for which no data are presented. For the data given, the standard deviations in the magnitude are quite small, and the directions are consistent. Thus, even though the data base is small, there is a high probability that the calculated stresses are representative of those at the repository level within the RRL. It is not possible to make conclusions concerning the validity of the stresses either laterally or vertically within the RRL or across the Hanford site. There is nothing geologically to suggest, however, that the stresses should vary radically, laterally, across the site. There is also no data to support Rockwell claims that the horizontal stresses should be significantly less within the interior vesicular zone (DOE, 1986).

We suggest that additional stress measurements be conducted in existing holes across the site, if possible. These measurements would establish the lateral and vertical variability of the stress state. As a minimum, additional stress measurements should be conducted within the Cohassett Flow to examine vertical variability of stresses across intraflow structures.

Investigation of hydrofracturing of the interior vesicular zone should also be made. This is of particular importance since the decision in the FEA (DOE, 1986) to allow excavation within the vesicular zone is based, to a certain extent, on a lower horizontal stress within this zone (Barton, 1986).

Finally, the suggested design values of stress are inappropriate because they represent the <u>average</u> stresses calculated from reopening pressures, including the pore pressure. It is prudent for conceptual design to use conservative ranges of stress. This involves assuming the zero pore pressure presented in Table C-2 of the report. From the five Cohassett measurements given in this table, the following statistics may be calculated:

$$\sigma_{Hmax} = 70.3 \pm 6.0 \text{ MPa}$$

$$\sigma_{Hmin} = 32.8 \pm 2.2 \text{ MPa}$$

$$\sigma_V = 24.2 \pm 1.1 \text{ MPa}$$

Therefore, for a 2 standard deviation spread, the most conservative range is:

$$58.3 < \sigma_{Hmax} < 82.3 \text{ MPa}$$

$$28.4 < \sigma_{Hmin} < 37.2 \text{ MPa}$$

$$22.0 < \sigma_V < 26.4 \text{ MPa}$$

$$2.2 < \sigma_{Hmax}/\sigma_V < 3.7$$

$$1.5 < \sigma_{Hmax}/\sigma_{Hmin} < 2.9$$

The present conceptual design may be significantly different if the above values are used. The BWIP program should re-evaluate their conceptual design based on these more conservative values. This further points to the necessity for reliable stress measurements by overcoring within the ES facility.

Problems, Limitations and Deficiencies

Intense interest in measurement of the in-situ stress state at the Hanford site has occurred since observation of extensive disking of diamond drill core in the late 1970s. The presence of core disking and spalling of borehole walls has lead to the speculation that construction of the repository may be troubled by severe ground control problems such as excessive deformation, spalling and rockbursting. In the following discussion, the observational background which supports the stress measurements is discussed, as well as the measurements themselves and their implications for repository performance.

OBSERVATIONAL INFORMATION RELATING TO STRESS

The document reviews the indicators of stress present at the Hanford site, including shallow seismicity, core disking, and borehole wall spalling.

As a rock mass undergoes yield, seismic energy is released. The rate and magnitude of these releases are indicators of the intensity of the applied stress field. Seismicity at the Hanford site is characterized by shallow microearthquake "swarms" consisting of up to 100 earthquakes of magnitude 1 to 3.5. These swarms typically last a few days to several months and occur within a rock volume roughly 5km on a side. About 75% of all microearthquakes occur at depths of less than 4,000m. Figure 1 shows the location of swarm activity, which is greatest in the Saddle Mountains and Frenchman Hills structures as well as Wooded Island at the eastern boundary of the site. Focal mechanisms indicate a major north-south horizontal compression with a vertical minimum compression. This is consistent with east-west trending fold axes and sub-parallel thrust or reverse faulting.

Core disking is a phenomenon in which the stub of a diamond drill core at the base of a borehole will split perpendicular to the hole axis, thus forming thin disks. There are several differing views on the mechanical phenomenology of disking, but there is unanimous agreement that it is an indicator of high horizontal stress (Obert and Stephanson, 1965; Jaeger and Cook, 1979).

Spalling of borehole walls along an east-west plane has been documented by downhole television, impression packers and acoustic televiewers (Paillet, 1985). Spalling of borehole walls occurs as a result of shear failure of the rock at a position 90° to the major principal stress direction. This is consistent with the direction surmised from geologic evidence as well as the empirical evidence of a high stress magnitude and large stress deviation in the horizontal plane. The report presents evidence that disking and spalling can be correlated to a certain extent in boreholes DC-4, RRL-6, and DC-12. The fact that both phenomena are not continuous with depth in the hole can be a result of a non-lithostatic stress field or the effects of changes in the rock properties.

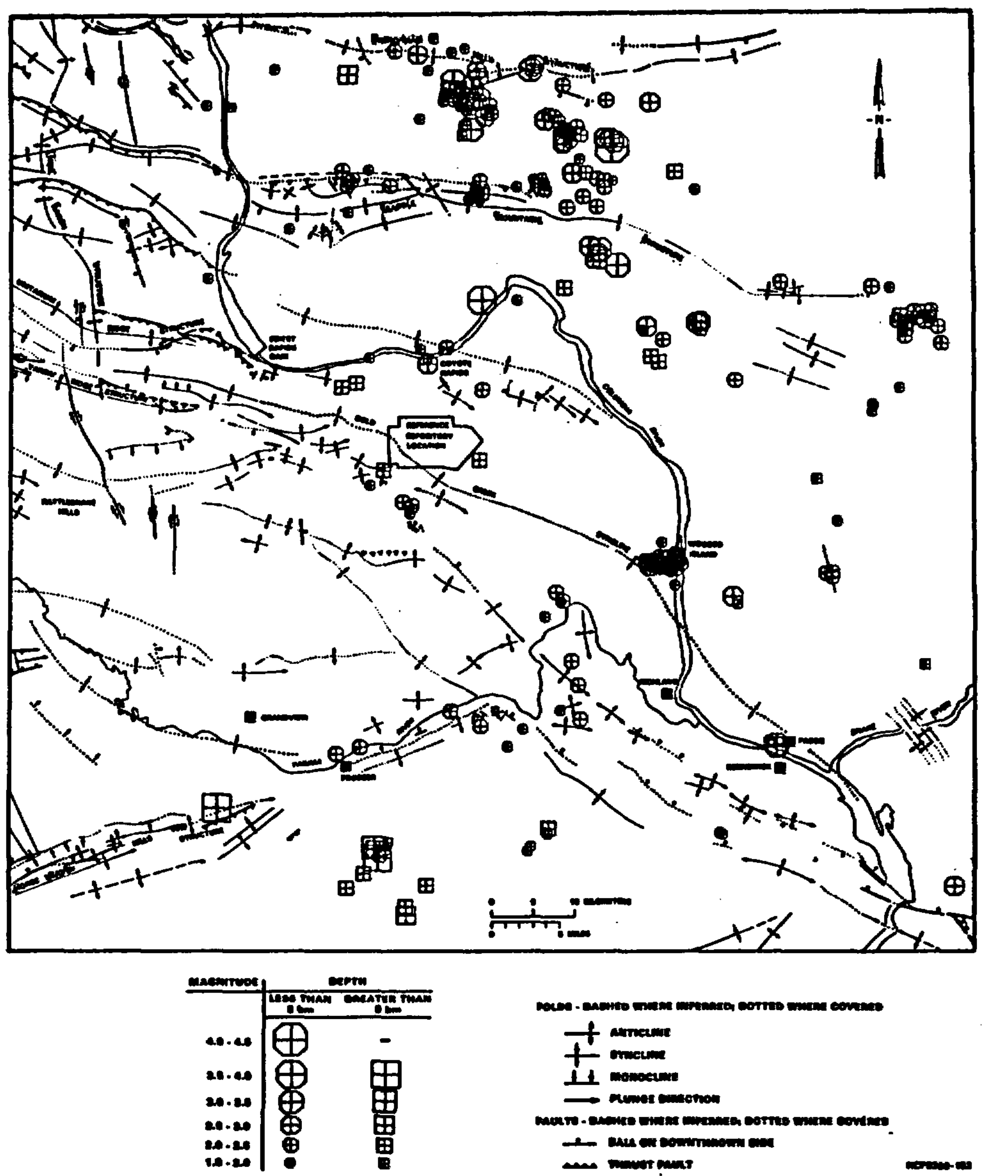


Fig. 1 Seismicity in the Pasco Basin as Determined from the Site Monitoring System

IN-SITU STRESS MAGNITUDE

The authors discuss various methods of treatment of the hydraulic fracturing data. With all methods, the assumption is made that the vertical stress is a principal stress direction and, therefore, for a vertical surface borehole, the tests will determine the principal horizontal stresses. The vertical stress magnitude is determined from the weight of the overlying rock mass. This is the usual assumption and is most reasonable. The minimum horizontal stress is determined by the instantaneous shut in pressure, as it is always assumed that the fracture propagates perpendicular to it.

There are several methods used for determining the horizontal stress component, including (1) the initial breakdown method; (2) the fracture re-opening method; and (3) fracture mechanics methods. The initial breakdown method is rejected, here, in favor of the fracture re-opening technique, because the former requires a value of the in-situ tensile rupture strength of the material. The re-opening method is most widely accepted, and it is our opinion that it is a reasonable choice for this study.

A controversial element of the stress determination is the inclusion of pore pressure in the calculation of maximum horizontal stress, σ_{Hmax}, which is given by

$$\sigma_{Hmax} = 3\ \sigma_{Hmin} - (P_{f2} + P_n) - P_o$$

where σ_{Hmin} = minimum horizontal stress

P_{f2} = re-opening pressure = applied pressure

P_h = hydrostatic pressure in the borehole (fracturing done with water)

P_o = pore water pressure at depth of measurement

Figure 2 is a schematic which illustrates these quantities. In a crystalline rock with low permeability, it is questionable whether communication between fracture and the water pressure is established. The conservative assumption is that the pore pressure term be neglected from the above equation. The authors argue that the pore pressure be included in the analysis, citing work by Hickman and Zoback (1982) as supporting their position. Although we agree in principle with this approach, it is felt that, for re-

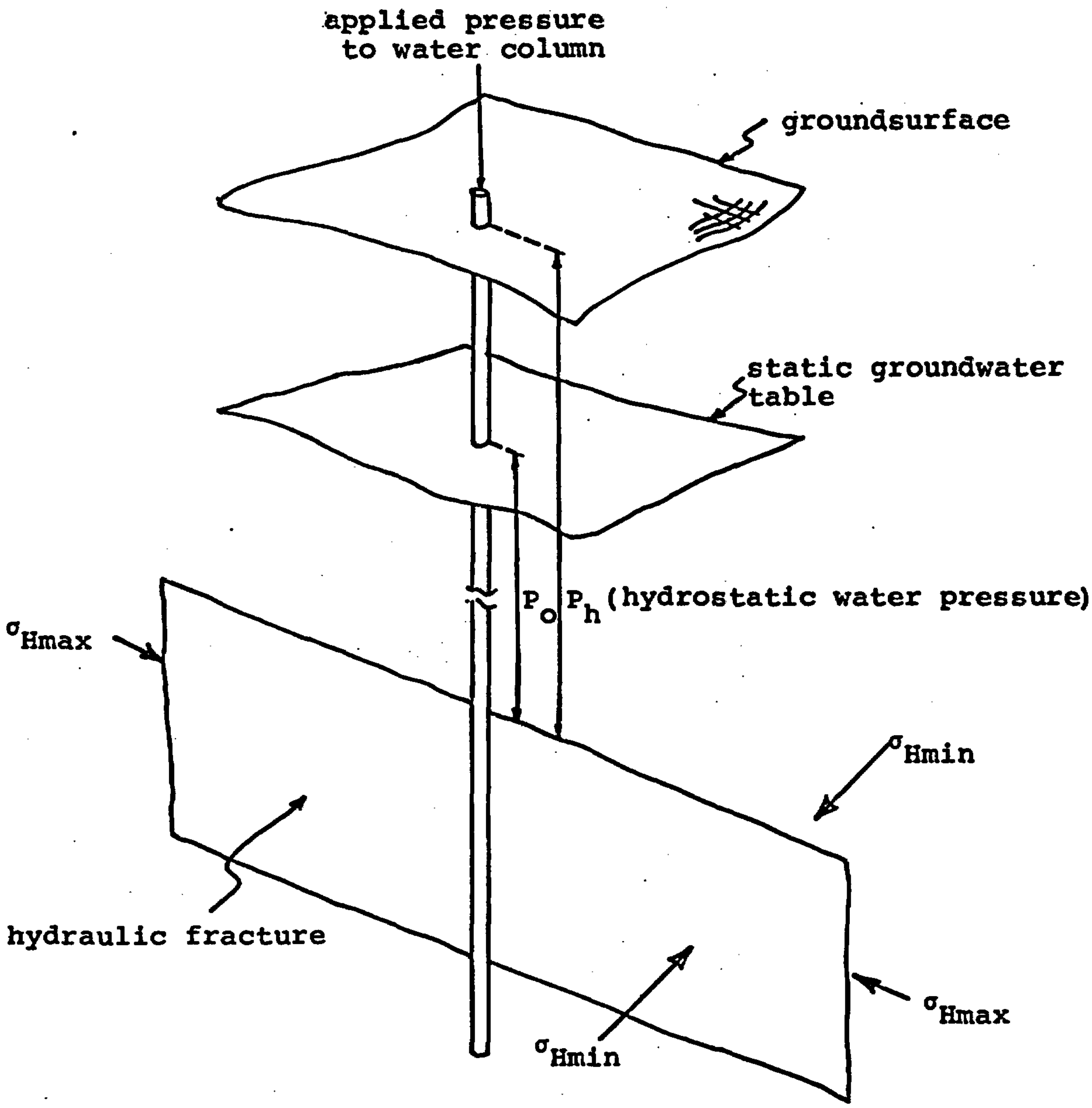


Fig. 2 Illustration of Hydraulic Fracturing Terminology (In the present case, $P_o \approx P_h$.)

pository design, the most conservative method (i.e., zero pore pressure) should be chosen.

The test techniques, field data from successful tests, and interpretation methods are given in detail in the report and represent acceptable practice. The five generally used methods (Zoback and Haimson, 1982) for determining shut in pressure from pressure-time records are discussed in Appendix C of the report and are used to examine sample test data. The inflection point method (Gronseth and Krey, 1982) was consistently conservative and chosen for use in this document.

A number of tests were rejected based on the presence of inclined fracturing or excessive spalling in the hole. Based on the uncertainty of these test results, it is probably reasonable to reject them, although data is not given to make this judgement independently.

The resultant calculations of the stress magnitude, orientation, and ratios are summarized in Table 1 of the report. These results appear to indicate the following.

1. There is no consistent change in horizontal stress with depth nor is there a consistent change in stress ratio with depth (Fig. 3). It appears that any variation with depth is masked by scatter in the data.

2. Measurements of horizontal stress from the Umtanum Flow within and outside the RRL indicate little lateral variation in stress; however, there is too little data to make this assertion confidently. It is reasonable to assume that the stresses are consistent laterally within the RRL.

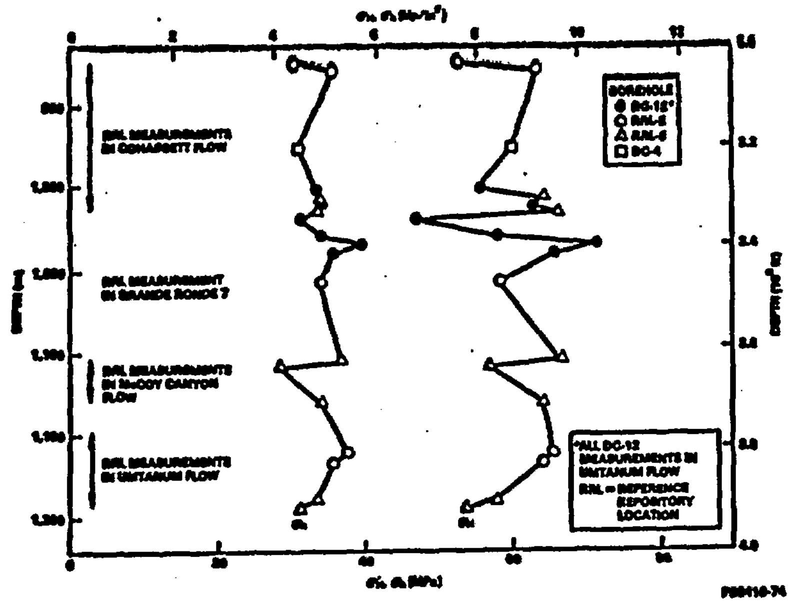
3. The data consistently indicates a north-south major horizontal compression. This correlates well with geologic data discussed earlier.

Although the total number of stress measurements is small, particularly within the Cohassett Flow, the consistency of the data presented in addition to the good correlation to geologic evidence gives a reasonable confidence in the data. It can safely be stated that few civil or mining construction projects enter the exploration development stage with details of stress measurement in any greater detail than in the present case.

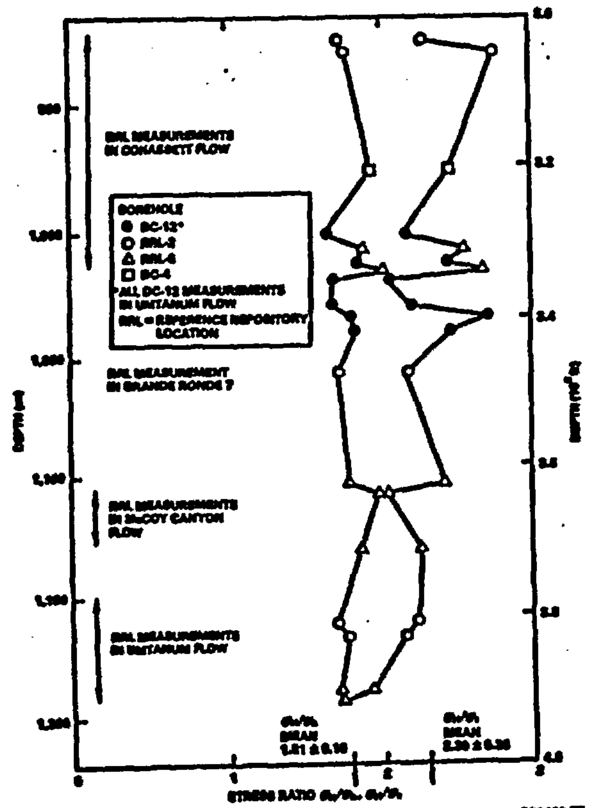
Table 1

IN-SITU STRESS MEASUREMENTS AT THE HANFORD SITE
(compiled from Kim et al, 1986)

Borehole	Number of tests	Mean depth, m (ft)	Mean, σ_h[a], MPa (lbf/in^2)	Mean, σ_H[b], MPa (lbf/in^2)	Mean, σ_v[c], MPa (lbf/in^2)	Mean fracture orientation	Mean stress ratios	
							σ_H/σ_h	σ_H/σ_v
Cohassett flow RRL-2 RRL-6 DC-4	5	970 ± 44 (3,181.6 ± 144.3)	32.8 ± 2.2 (4,756 ± 319)	61.5 ± 5.7 (8,917.5 ± 826.5)	24.2 ± 1.1 (3,509 ± 159.5)	N.06°E.[d] ± 17°	1.87 ± .12	2.54 ± .18
Grande Ronde 7 RRL-2	1	1,058 (3,470.2)	33.6 (4,872)	58.1 (8,424.5)	26.6 (3,857)	N.08°W	1.73	2.19
Umtanum flow RRL-12 RRL-2 RRL-6	10	1,085 ± 81 (3,558.8 ± 265.7)	34.6 ± 2.6 (5,017 ± 377)	60.8 ± 6.0 (8,816 ± 870)	27.6 ± 1.8 (4,002 ± 261)	N.09°E.[e] ± 22°	1.78 ± 0.6	2.21 ± .27
McCoy Canyon RRL-6	3	1,115 ± 14 (3,657.2 ± 45.9)	33.1 ± 4.3 (4,799.5 ± 623.5)	62.6 ± 5.1 (9,077 ± 739.5)	27.9 ± 0.3 (4,045.5 ± 43.5)	N.00°E. ± 27°	1.90 ± .10	2.24 ± .18
DC-4	1	976 (3,201.3)	30.8 (4,466)	59.9 (8,685.5)	24.4 (3,538)	N.12°E.[d] ± 16°	1.95	2.45
DC-12	6	1,024 ± 15 (3,350.7 ± 49.2)	34.8 ± 2.7 (5,046 ± 391.5)	61.2 ± 6.8 (8,874 ± 986)	26.2 ± 0.4 (3,799 ± 58)	N.23°E.[e] ± 21°	1.76 ± .08	2.33 ± .24
RRL-2	5	1,047 ± 119 (3,434.2 ± 390.3)	34.5 ± 2.7 (5,002.5 ± 391.5)	60.6 ± 5.3 (8,787 ± 768.5)	26.3 ± 3.1 (3,813.5 ± 449.5)	N.03°W.[f] ± 14°	1.76 ± .03	2.32 ± .23
RRL-6	7	1,174 ± 74 (3,850.7 ± 242.7)	33.0 ± 2.4 (4,785 ± 348)	61.6 ± 5.4 (8,932 ± 783)	27.8 ± 1.9 (4,031 ± 275.5)	N.05°E. ± 22°	1.87 ± .11	2.24 ± .33
All RRL holes including DC-4	13	1,074 ± 96 (3,522.7 ± 314.9)	33.4 ± 2.7 (4,843 ± 362.5)	61.1 ± 4.9 (8,859.5 ± 710.5)	26.9 ± 2.5 (3,900.5 ± 362.5)	N.02°E.[g] ± 17°	1.83 ± .11	2.29 ± .28

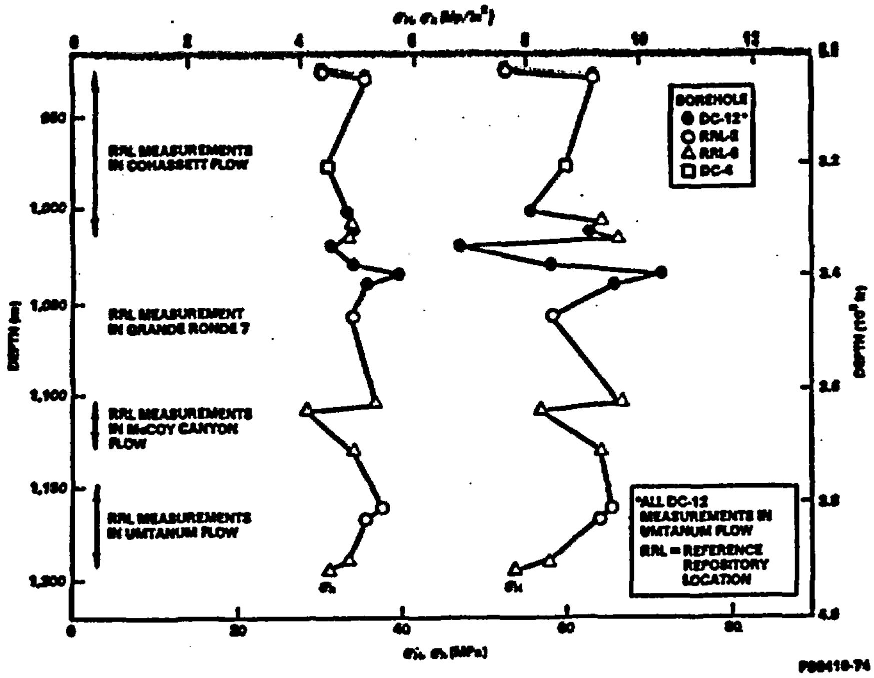


(a)

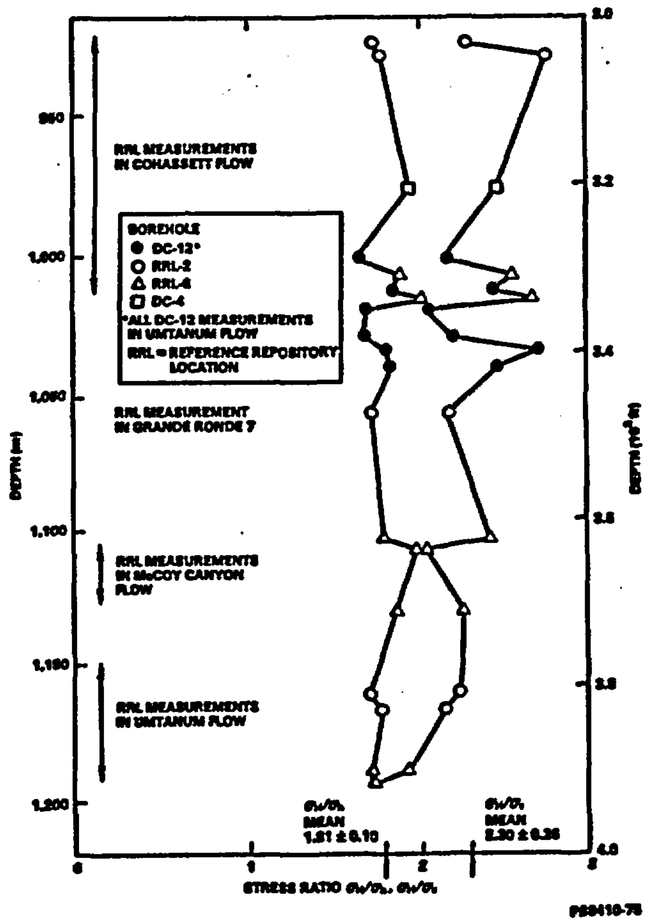


(b)

Fig. 3: (a) Horizontal Stresses as a Function of Depth
(b) Stress Ratios as a Function of Depth



(a)



(b)

Fig. 3: (a) Horizontal Stresses as a Function of Depth
(b) Stress Ratios as a Function of Depth

OBSERVATIONS ON DESIGN RECOMMENDATIONS

The report has recommended preliminary stress design values as the average Cohassett measurement ± 2 standard deviations (Table 2).

Table 2

ROCKWELL RECOMMENDED DESIGN STRESSES

Stress	Direction	Design Range (MPa)	Average (MPa)
σ_{Hmax}	N6°E	$50.1 \le \sigma_{Hmax} \le 72.9$	61.5
σ_{Hmax}	N84°W	$28.4 \le \sigma_{Hmin} \le 37.2$	32.8
σ_{V}		24.2	24.2

The above range is in conflict with the final recommendation given on p. 61, which suggests a design value of the average values of 61.5 and 32.8 MPa for the maximum and minimum stresses, respectively.

We feel that, at this conceptual stage, the design should be based on the most conservative assumptions. The assumption of zero pore pressure should be used in the calculations, resulting in the design range given in Table 3.

Table 3

RECOMMENDED DESIGN RANGE BASED ON ASSUMPTION OF ZERO PORE PRESSURE

Stress	Direction	Design Range (MPa)*
σ_{Hmax}	N6°E	$58.3 \le \sigma_{Hmax} \le 82.3$
σ_{Hmin}	N84°W	$28.4 \le \sigma_{Hmin} \le 37.2$
σ_{V}		$22.0 \le \sigma_{V} \le 26.4$

*mean ± 2 standard deviations

Rockwell recently formed an expert panel for reviewing the in-situ stress data (St. John and Kim, 1986). This panel has recommended a design stress range of

$$50 \le \sigma_{Hmax} \le 75 \text{ MPa}$$

$$30 \le \sigma_{Hmin} \le 40 \text{ MPa}$$

Finally, to best of our knowledge, the architect engineer has used

$$\sigma_{Hmax} = 58 \text{ MPA}$$

$$\sigma_{Hmin} = 33 \text{ MPa}$$

$$\sigma_{V} = 23.2 \text{ MPa}$$

in its most recent repository design activities (RKE/BP, 1985). All of these recommendations are summarized in Table 4.

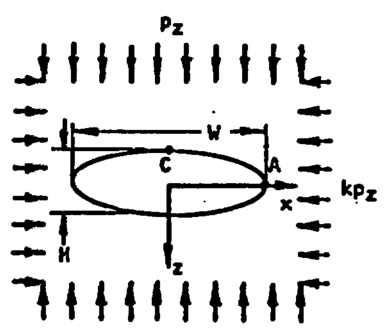
Table 4

SUMMARY OF REPOSITORY DESIGN STRESS RECOMMENDATIONS

Group	Stress Range or Values (MPa)	Comment
RKE/PB (1985)	$\sigma_{Hmax} = 58$ $\sigma_{Hmin} = 33$ $\sigma_V = 23.2$	Data used in 1985 repository design
Kim et al (1986)	$\sigma_{Hmax} = 61.5$ $\sigma_{Hmin} = 32.8$ $\sigma_V = 24.2$	Present document recommendations
St. John and Kim (1986)	$50 \le \sigma_{Hmax} \le 75$ $30 \le \sigma_{Hmin} \le 40$	Recommendations of BWIP expert review panel
Itasca Review for NRC	$58.3 \le \sigma_{Hmax} \le 82.3$ $28.4 \le \sigma_{Hmin} \le 37.2$ $22.0 \le \sigma_V \le 26.4$	Extreme stresses from Kim et al (1986) assuming zero pore pressure and ± 2 standard deviations

Our purpose in reviewing all of these measurements is to point out the significant effect they will have on the conceptual repository design. The RKE/PB (1985) design is based on a numerical analysis of induced excavation and thermal stresses assuming the in-situ stresses given above. The induced stresses are compared to the estimated rock mass strength as derived from the Hoek and Brown (1980) empirical failure criterion. Rock mass strength values of 200 and 152 MPa have been determined for the emplacement hole and room periphery, respectively. The waste canister pitch, room dimensioning, and spacing were derived, in large part, from the estimated in-situ stresses. Therefore, any deviations from the Table 4 RKE/PB values must result in a re-assessment of the conceptual design. The two most conservative cases (i.e., the expert panel and the present recommendation) result in increases in the maximum horizontal stress by 29% and 42% over the RKE/PB design values, respectively.

The effect of these higher design stresses can be seen by examining the stress concentrations around the emplacement room periphery. If it is assumed that the emplacement room is an ellipse, effects of these new stresses can be easily examined from an analytic solution. Hoek and Brown (1980) give the solution for the tangential stresses at the boundary of an ellipse in a bi-axial stress field:



$$\sigma_A = P_z\ [1 + 2(W/H) - K]$$

$$\sigma_C = Pz \left[K\ [1 + 2(H/W)] - 1\right]$$

where W/H = 2

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$$\sigma_A = P_z \, [1 + 2(W/H) - K]$$

$$\sigma_C = Pz \left[K \, [1 + 2(H/W)] - 1\right]$$

where W/H = 2

Using the RKE/PB (1985) values, these stresses are:

$$\sigma_A = 23.2\ (1 + 2*2 - 2.5)$$
$$= 58 \text{ MPa}$$

$$\sigma_C = 23.2\ [2.5\ (1 + 2*0.5) - 1]$$
$$= 92.8 \text{ MPa}$$

If, however, the extreme values from Table 5 assuming no pore pressure are used, we have

$$\sigma_A = 22.0\ (1 + 2*2 - 3.7)$$
$$= 27.6 \text{ MPa}$$

$$\sigma_C = 22.0\ [3.7\ (1 + 2*0.5) - 1]$$
$$= 141 \text{ MPa}$$

The stress at the roof is now nearly at the design criteria without addition of thermal loads.

Even with the extreme values suggested by the expert review panel, the room periphery stresses are:

$$\sigma_A = 23.2\ [1 + 2*2 - 3.2]$$
$$= 41.7 \text{ MPa}$$

$$\sigma_C = 23.2\ [3.2\ (1 + 2*0.5) - 1]$$
$$= 125 \text{ MPa}$$

The analysis of possible fault or joint slippage under the influence of the in-situ stresses presented on pp. 57-59 must also be examined in the light of the extreme data ranges. This analysis indicates that in-situ joint friction would have to be as low as 33° (with no cohesive strength) for slip to occur. It is suggested that microseismic events at the site are related to small areas of joints where the friction angle is low or at joint intersections. A plot of the Mohr envelopes for the extreme data ranges is given in Fig. 4. Here, the range of extreme values for the σ_{Hmax}, σ_v envelope are plotted for all stress values discussed in this review. As seen, the ranges of stress recommended by Kim et al (1986), the review panel, and this review are fairly similar and suggest slip on surfaces with a friction angle of about 40° for cohesive joints.

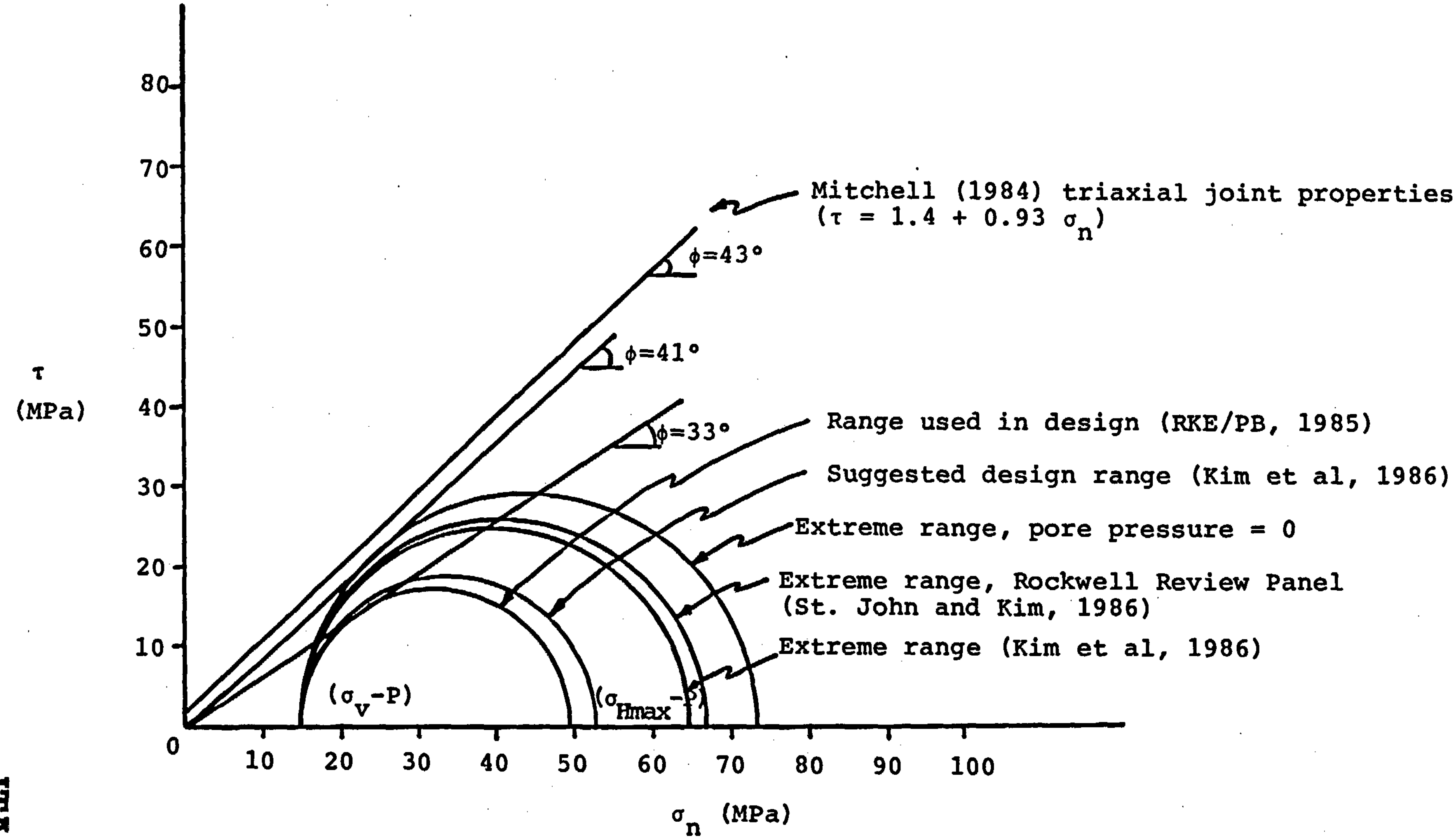

Fig. 4 Mohr Envelope Plot of Stress States Given in Table 4
(This plot shows that for the extreme stress states, slip could occur on favorably-oriented joints with friction angles of 40°±.)

CONCLUSIONS

The in-situ horizontal stresses at the Hanford site appear to be exceptionally high. The many indicators of high stress, including core disking, borehole spalling and microearthquake activity confirm the high magnitude as well as orientations of the stresses. The techniques used for stress measurement by hydraulic fracturing and stresses appear to be satisfactory. One controversial point encountered was the assumptions regarding the use of pore pressure in the stress calculations. Although calculations were presented for both cases (assuming pore pressure and no pore pressure), the authors adopted those values from the non-conservative assumption of pore pressure in the determination of σ_{Hmax}.

The average of the range of non-conservative stresses were chosen as recommended design values. Rockwell's own expert panel has suggested the extreme values (i.e., average ± 2 standard deviations) be used for design. This alone results in about a 23% increase in the value of σ_{Hmax}. If the most conservative case is used (i.e., assuming the pore pressure is zero), the design values increase by roughly 35%. Changes in design stress values of these magnitudes can have significant effects on the repository design since, at these levels, the stress concentrations are fairly close to the design criteria strength of the rock mass.

Recommended Action

It is our recommendation that additional hydraulic fracturing measurements be conducted in existing site boreholes, if possible, within the Grande Ronde flows. A detailed program of overcoring measurements must be conducted within the ES facility at depth during construction. NRC should review the latest BWIP conceptual design documents to confirm the stress values assumed. If these values are outdated, the implications regarding the repository design must be determined.

References

Barton, Nick. Final Report: Rock Mass Quality and Support Recommendations for Basalt at the Candidate Repository Horizon, Based on the Q-System. Rockwell Hanford Operations SD-BWI-ER-012 (Rev. 1). April 1986.

Gronseth, J. M., and P. R. Krey. "Instantaneous Shut-In Pressure and Its Relationship to the Maximum In Situ Stress," Proceedings of the Workshop on Hydraulic Fracturing Stress Measurements (Washington, D.C., 1982). USGS Open File Report 82-1075, 1982.

Hickman, S. H., and M. D. Zoback. "Interpretation of Hydraulic Fracturing Pressure-Time Data for In Situ Stress Determination," Proceedings of the Workshop on Hydraulic Fracturing Stress Measurements (Washington, D.C., 1982). USGS Open File Report 82-1075, 1982. 1982

Hoek, E., and E. T. Brown. Underground Excavations in Rock. London: Institute of Mining and Metallurgy, 1980.

Jaeger, J. C., and N.G.W. Cook. Fundamentals of Rock Mechanics (3rd Ed.). London: Chapman and Hall, 1979.

Mitchell, S. J. "Direct Shear Testing Date Reduction," Rockwell Hanford Operations RHO-BWI-Comp. Brief No. CB-00066, 1984.

Obert, L., and D. E. Stephanson. "Stress Conditions Under Which Core Disking Occurs," SME Trans., 232, 225-227, 1965.

Paillet, F. L. "Acoustic Televiewer and Acoustic Waveform Logs Used to Characterize Deeply-Buried Basalt Flows, Hanford Site, Benton County, Washington," USGS Open File Report 85-419, 1985.

Pine, R. J., P. Leadingham, and C. M. Merrifield. "In Situ Stress Measurement in the Carnmenellis Granite — II: Hydrofracture Tests at Rosemanowes Quarry to Depths of 2000 m," Int. J. Rock Mech. Mining Sci., 20(2), 63-72 (1983).

RKE/PB. "Task V, Engineering Study No. 9: Underground Repository Layout, Project B-301," Rockwell Hanford Operations SD-BWI-ES-023 (Rev. 0), January 1985.

St. John, C. M., and Kunsoo Kim. Geoengineering Design Parameters Workshop (Proceedings of a Workshop Held in Rapid City, South Dakota on June 25, 1985). SD-BWI-TI-299. January 1986.

U.S. Department of Energy. Nuclear Waste Policy Act (Section 112) Environmental Assessment Reference Repository Location, Hanford Site, Washington (Vols.I-III plus Overview). May 1986.

Zoback, M. D., and B. C. Haimson. "Status of the Hydraulic Fracturing Method for In-Situ Stress Measurements," Issues in Rock Mechanics (Proceedings of the 23rd Symposium on Rock Mechanics, Berkeley, August 1982), pp. 143-156. New York: American Inst. of Mining, Metallurgical and Petroleum Engineers, 1982.