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A REVIEW OF THE PROPOSED WIPP UNDERGROUND MINE DESIGN

A REPORT TO

D'APPOLONIA CONSULTING ENGINEERS

BY

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1. Introduction

This report represents an independent review of the proposed WIPP underground mine design. This review is based on material made available by the TSC as detailed in Appendix A. In reviewing the proposed design, its functional suitability and stability have been assessed. Principal emphasis in these areas has been placed on ground control and stability, since these are the major areas of expertise of the reviewer.

Certain criteria have been defined by the reviewer based on his understanding of the requirements for the WIPP. These are summarized below:

- A reasonable utilization of the available site is required, however demonstrable stability both of the underground design, and its effect on surface structures is considered to be more important than the development of high extraction ratios. The demands for safety and stability are of overriding importance, and the trade-offs between these aspects and economic requirements common in a mining operation do not apply. The use of trial and error in the development of early panels which is common in mining is not acceptable for the WIPP. The result of this criteria is that any design should be conservative.
- Information supplied by the TSC states that storage rooms should be designed to allow a ten-year waste retrievability period, but encapsulation of the waste should occur within 25 to 50 years.
- Failure of any of the storage rooms by roof fall or extensive pillar slabbing is presumed to be unacceptable.

- Main entries should remain usable at least until waste encapsulation occurs. They therefore should be designed for lower closure rates and greater stability than the storage rooms.
- Shaft stability should be assured for at least the life of the repository.
- Subsidence effects in the shaft area are unacceptable.

Due to time constraints, it has not been possible for the reviewer to develop a totally independent functional design, nor is it understood that this is required. For this reason, this review depends heavily upon the design calculations provided by the TSC and relies, in large part, on a critical examination of the current design.

2. Design information

One of the strongest criticisms of the proposed design lies in the scarcity of well-documented design data, and the poor use of the data which does exist. Any mine design should rely heavily on empirical data available from mines in similar lithologies and at similar depth. This empirical data may be backed up and extended by careful use of numerical stress analysis techniques (particularly finite elements).

Empirical data for mine design in bedded salt is not as freely available as for other bedded deposits, particularly coal. Nevertheless, data is available. Reference is made in the documents to experience in the Saskatchewan potash mining district. However, this is not detailed, nor is the applicability to the current region investigated. Some information is included from the Carlsbad area (A 46, A 47)*, but this is not apparently

*References A1-A47 refer to the documents received from the TSC and are listed in Appendix A.

included in the design, although the intent of doing so is stated in several places. Indeed, the current design in several areas goes against local experience (see Sections 4, 5, 6).

Considerable reliance is placed on finite element analyses. However, there are serious questions about the validity of the models used and the results obtained from them. The starting point for any numerical analysis has to be a reasonable representation of the expected lithology and of the mechanical properties of the rock. These are examined in the following sections.

3. Choice of horizon

The horizon chosen appears the best available. The presence of clay seams in the roof and floor may be troublesome, and an agreement on definitions and identification should be sought between the AE and TSC (A 25).

Comments made at the meeting of April 24, 1979 (A 24) regarding the differences between WIPP-12 and ERDA-9 should be noted. If WIPP-12 is anomalous in its clay content development to the south and east would be advisable. Later developments to the north could be considered after some experience of the mining and data on ground movements and stability has been obtained.

4. Storage area

The storage area is in two panels each developed in a series of four 33-foot wide by 300-foot long rooms, with 25-foot yield pillars and 300-foot abutment pillars separating each group of rooms. This design is unconventional. Thus, at two local mines panels are developed with 28-foot rooms with 44-foot square pillars at a depth of 1,070 feet (A 46) and 26-foot rooms with 54-foot square pillars at a depth of 1,750 feet, these designs being reasonably typical of practice in other areas (see e.g. 1). No reasons for this design are given, and the only design analysis is given in the Stability Analysis (A 45). It is presumed that the design is an application of the Stress Control Technique published by Dr. Serata.

4.1 <u>The Stress Control Technique</u>

Comparison of the present design to an experimental system used at the Kerr McGee mine (A 46) and a consideration of the proposed order of main entry development (A 3) suggest that this is an application of Serata's "Time-Control Method" (2). However, consideration of the mining concept schematics (A 15 - A 17) suggest that this may be an application of the "parallel room method" (2).

Both of these methods are intended for use in weak or failing ground with clay 'separation' seams in the roof. In the "parallel room method," a second room is driven parallel to a failing room with a minimal yield pillar between. The new room is thus protected by the development of stress-relieved ground around this room. A third and fourth room can be added, the final configuration being stable except for the first room which will continue to fail. In the "time control method," two outer rooms are developed and the central rooms are driven between in "strain hardened" ground. This is essentially a similar technique to that suggested by Baar (3), although he states that the intervening ground is "stress-relieved" rather than strain-hardened. Success has been claimed for these methods at various mines in the Canadian potash region (2). It must be emphasized that the use of narrow yield pillars in the design goes against normal mining practise for this type of deposit. This being the case, the onus of proof must rest with this design. It should be demonstrated that the yield pillar concept is a necessary modification of 'conventional' designs and that the resulting storage areas will have the required stability. For a number of reasons, this reviewer does not feel that this proof has been provided. Thus

- The presence of strain-hardened ground in evaporite mines has been questioned (3, 4), although a similar result develops if stress-relief is invoked.
- In the only case of which the reviewer is aware of the use of these methods in the WIPP area, the design did not function as intended. Thus, at the Kerr McGee mine, a trial system showed rapid failure of the outer rooms, and early failure of the central rooms (A 46). The system was discontinued.
- The reviewer is not aware of any well documented field verification of the methods. This is essential.
- Numerical simulations of the proposed room layout (A 45), although themselves suspect (Section 4.2) do not agree with the projected behavior. Thus, according to Serata, closure rates should be highest in the outer rooms, while numerical simulation shows similar closure rates throughout, rates being slightly higher in the center rooms (A 45, Figs. 3.4.8, 3.4.9). A different result

is obtained by Sandia for the RH level rooms, though the geometry here is different (A37).

More conventional pillar analysis suggests that the yield pillars may be detrimental to stability. The Bechtel analysis suggests large initial vertical stresses in the yield pillars, decaying somewhat in time, and comparatively small horizontal stresses. After two and one-half years, the stresses in the center of the yield pillars are of the order of 3,000 psi vertically and 500 psi horizontally, compared to 3,000 psi vertically and 1,000 psi horizontally in the abutment pillars. This indicates a poorly confined core in the yield pillars and the potential for serious slabbing or even failure of these pillars.

Other questions arise in the storage area design. One of the major reasons put forward for the use of yield pillars is to allow the relaxation of horizontal stresses above and below the rooms and thus to avoid roof and floor slabbing due to buckling. Baar (3) has suggested that intersections, where horizontal creep can occur in more than one direction, may lead to greater stability in these conditions. The length of the pillars (300 feet) relative to their width should be considered. Finally, the panel entries require attention. These are 33-foot rooms separated by 300-foot abutment pillars. Presumably, these are expected to be stable, although no calculations are given. If these entries are stable, the need for narrow yield pillars in the storage rooms is not obvious.

4.2 Stability Analysis Review

This subsection reviews the Bechtel Report on Stability Analysis of the Underground Openings, WBS No. 51, Sections 3 and 4 (A 45).

The stability analyses are based on runs made using the MARC-CDC program with a viscoplastic material property subroutine added. Questions arise over the use of this code. It is stated that very small time steps are required under high strain rate conditions. For this reason, runs on the CH level were limited to 0.4 years for the model with clay seams and 2.5 years for the model without clay seams. Extrapolation was used to give ten-year creep closures. The extrapolations are highly suspect; they indicate a cessation of creep closure after about five years, a result in contradiction with field evidence (3, 1), which indicates continuing creep closure at a constant rate. It is clear that the MARC code is unsuitable for the current applications, and another more suitable code should be sought. It is stated that the REM code of Serata Geomechanics will be used in future analyses. This code is proprietary and without documented evidence of its suitability, including field verification in similar lithologies, it cannot be judged.

Neither of the CH level idealizations (Figs. 3.4.1, 3.4.16) show a reasonable representation of the site geology based on the ERDA-9 core logs. A detailed comparison of the ERDA 9 log and the model geology is included as Fig. 1. The most notable differences are summarized in Table 1.

Table 1

	Horizon	Core log (A 23)	Model (A 45)
a.	MB 136	13.6 ft. thick	17.8 ft. thick
b.	MB 138	0.7 ft. anhydrite with basal parting	absent
c.	anhydrite	0.9 ft. at 2120.9; clay parting at base	absent
d.	MB 139	3 ft. thick anhydrite; top at 2154.3, 4.3 ft. below CH level	8 ft. thick; top at 2150, floor of CH level
e.	anhydrite	absent	2233-2249.8

Clay seams are included in the model as "clay with partings" in beds of appreciable thickness. This 'lumping' technique is invalid since the importance of the clay seams is most likely to be in their lack of tensile and shear strength and the resulting contribution to potential bed separation. The "clay with partings" beds do not correlate well with the identified clay partings in any case. The omission of the clay partings and anhydrite in the near roof and floor of the CH level are particularly serious.

Even with a verifiable and useful computer code, and a realistic idealization, any numerical investigation is limited by the knowledge of material properties and the ability to represent them mathematically. The properties used for the stability analysis appear to not be based on the Sandia experimental work (A 41), nor is any source for the properties available. The quoted source (reference 2 of the document) was not available to the reviewer.

The viscoplastic law used for the simulation is apparently a simplification of the generalized law reported by Serata (2). The constants for the law -- octahedral shear strength K_o and viscoplastic V_4 -- have also been reported as generic values (5), although the value for V_4 is closest to that reported for "weak salt" while that for K_o is closest to that reported for "strong salt." No evidence for the temperature or stress exponents are presented, nor is this evidence available in the literature, to the reviewer's knowledge. The creep strains predicted by this viscoplastic law for a simple constant stress triaxial test can be compared to those predicted by Sandia laws developed from test data of the WIPP salt (A 35, A 36). This shows the Bechtel equation consistently predicts different secondary creep strains than the Sandia law (Appendix B). It must be noted, however, that the more recent Sandia law is based on short-term creep data in which the existence of a true secondary creep region is not established, so that the creep rates predicted by this law will tend to be overestimated. Finally, no account is taken of the time sequence of excavation of the rooms which could have a significant effect on the deformation and stress fields.

4.3 Storage area design

Before the storage area design can be agreed to, or even properly reviewed, the following data must be made available:

- Numerical simulation using a verified, and verifiable, code, reasonable geology incorporating the clay seams in a realistic manner, and material properties developed from the Sandia data on SENM salt. These simulations should be run for

    a) storage rooms (Section B-B, Drg. 51-W-001)
    b) panel entries (Section E-E, Drg. 51-W-001)
    c) alternative 'conventional' room designs

- Comparisons of the proposed designs to mining experience in similar geologies.
- Documentation of the proposed design by actual mine experience.

Even with this data, it must be appreciated that a final design cannot be developed until field data from the early mine development is available.

5. Main entries

Four 25-foot main entries with 20-foot yield pillars are planned.

All comments made in relation to the storage area apply to these entries. However, main entries must be stable throughout the projected life of the storage area, and detailed stability calculations must be reported. Failure of the thin yield pillars and excessive creep closure in these entries could have serious consequences. Again this design does not follow normal practices (Table 2). In particular, the comparison of the pillar W/H ratios should be noted.

Table 2 (Main entries) (A 46)

Mine	Entry		Pillars		Depth
	Width	Height	Width	W/H ratio	
Nash Draw - mains	20 ft.	-	44 ft.	--	1070
-declines	20 ft.	7 ft.	35 ft.(1)	5	
Kerr McGee	25 ft.	5 ft.	100 ft.	20	1100 - 2000
National Potash	26 ft.	6 ft.	54 ft.	9	1750
WIPP (proposed)	25 ft.	12 ft.	20 ft.	1.7	2150

Before any agreement on, or proper review of, the main entry design can be made the following data must be provided:

- Numerical simulation of the proposed entry design.
- Numerical simulation of an alternative design with wide entry pillars.
- Comparison to field experience.

It should be noted that this design is more critical than that for the storage areas for the following reasons:

- It concerns the main entries which must have a higher degree of stability.
- Main entry design will influence the early development which will precede most of the preliminary stability data to be obtained from the experimental area.

6. Shop area

The stability requirements for the shops are even greater than for the main entries, since these will, presumably, be used for future repository developments, and since they will include equipment and facilities which will be sensitive to gross ground movements.

Twenty-foot wide pillars are again used in this area. These are unacceptable for the following reasons:

- The heights are greater than elsewhere. Thus, in Drg. 54-U-001 a room height of 28'2½" is indicated, a pillar W/H ratio of 0.7
- Only two entries are indicated for each pillar. This does not even agree with the stress control yield pillar concept.

- A high degree of stability is required.
- No design data is given using wider pillars.

It should also be noted that a height of 28'2½" gives a roof at 2121.8 feet coincident with the base of a 0.9 foot anhydrite. Extensive roof bolting will be necessary.

The sloped rib pillars shown on Drg. 54-U-001 could give spalling problems. Note that Duvall use vertical ribs at the Nash Draw mine for this reason (A 46). The sharp corners in some of the storage area pillars (e.g. southwest of the SE shaft) will lead to high stress concentrations and should be eliminated.

Consideration should be given to moving the shops out of the immediate shaft area. An alternative shop layout should be developed.

7. Shaft pillar

The size of the shaft pillar will depend largely upon the angle of draw for the particular overburden. There is little available evidence on this angle for these strata. Local practice is summarized in Table 3.

Table 3 (A 46)

Shaft Pillars

Mine	Depth ft.	Shaft pillar radius ft.	angle	Plant(Shop)pillar radius ft.	an
Nash Draw	1070	550	27^{0}	1070	4
Kerr McGee*	1100	600	29^{0}	1600	5
WIPP	2150	1000	25^{0}	--	-

*Kerr McGee has noticed some subsidence at its main office buildings.

The 25° angle proposed for the WIPP shaft pillar is in accordance with the various formulae used from the Mining Engineering Handbook (6) (Drg. 51-W-013). It is also in line with normal coal mining practice (7) and with other mines in the area. However, for minimal subsidence an angle of draw of 35° is indicated in coal mining (7, page 13-108), while local mines use at least 45° for this subsidence pillar. A more conservative design would be to use a 1000-foot shaft radius pillar and a 2150-foot radius subsidence pillar. Development in the shaft pillar would be limited to shaft entries. Shops would be sited in the subsidence pillar (1000 feet - 2150 feet) and the experimental and storage areas placed outside of the subsidence pillar.

8. Design instrumentation

Preliminary design layouts and calculations must, of necessity, be based on information on the detailed lithologies from core holes and the extrapolation of field data from other mines in similar rocks. However, it must be recognized that design modifications may be needed based upon early experience in the mining horizon. These could occur because of the limited knowledge of the geology, the difficulties in correlating laboratory mechanical data and in-situ behavior and the fact that current field experience is limited to other areas and stratigraphies or to different horizons in the same area. At the very least, early verification of the suitability of the proposed design and of the validity of design prediction is needed.

The field data necessary for design verification, and/or modification, can only be obtained by a comprehensive instrumentation scheme. Moreover, since time is an essential factor in the behavior of openings and structures in salt, this instrumentation scheme must be implemented as soon as practical after opening the horizon to be mined. The instrumentation scheme should be designed for the following general objectives:

- To verify design data and methods (rock mechanical properties, computer codes and empirical extrapolations);
- To assess the stability, and hence the suitability, of the preliminary design;
- To provide data for design modification, if this is shown to be necessary.

The collection of field data should be continuous from first breaking ground to abandonment of the repository. Early data will be used for continuous design updating; later, data will be invaluable in the design of later facilities. The instrumentation scheme should give information on all parts of the design; that is, the shafts, shaft pillars, shops, entries and storage area. More specific recommendations are included in the following sections.

8.1 Instrumentation layout

8.1.1 Shaft instrumentation

As a minimum, the following should be monitored:

- Shaft deformation, both in terms of changes in shaft diameter and in terms of deformation within the rocks surrounding the shaft
- Lining strain
- Pore water pressures in water bearing horizons

Information from these measurements is essential for the verification of shaft lining design. Data collection should be started as soon as possible, preferably concurrently with shaft sinking. It is understood that the first shaft is to be blind bored and that early instrumentation will be difficult or impossible. The operational advantages of this procedure should be weighed carefully against the need for early information on shaft behavior to be used in the verification or modification of design for the later shafts.

8.1.2 Subsidence monitoring

Shaft pillar verification will require precision surface subsidence monitoring, as will general environmental considerations. Monitoring should not be restricted to the shaft pillar area, but should extend over the whole site. Information on any surface subsidence caused by deformations in the storage area will aid in shaft pillar evaluation. Monitoring can be by standard surveying techniques.

8.1.3 Shop areas, entries

These permanent openings should be instrumented as soon after opening as possible and monitoring should be continued throughout their life. As a minimum, the following should be monitored:

- Room convergence (horizontal and vertical)
- Pillar deformation (horizontal and vertical)
- Roof and floor deformation away from the opening.

Monitoring stations should include intersections.

8.1.4. Storage area

Design verification - Test rooms with dimensions the same as planned for the storage area should be driven. Alternative room designs should be planned in case these should prove necessary. Measurements should include those specified for the shops and entries (Section 8.3). In addition, stress changes at various depths in the roof, floor and pillars should be monitored.

Monitoring instrumentation should be included in some of the actual storage rooms. Measurements should include those specified for the shops and entries with the possible addition of some stress measurements.

8.1.5 The experimental area

Part of the experimental area will be used for the design verification work (Section 8.4). The remainder is devoted to Sandia experiments and instrumentation for that area is not considered here.

8.2 Measurement techniques

No attempt is made to detail techniques in this section. However, some general comments on design of the techniques are given.

8.2.1 Deformation measurements

The deformations of interest to the design verification include any resulting from the openings. Surface deformation (closure) must be monitored, as well as the movement of points inside the rock mass. The first measuring point in any instrument boreholes should be as close as possible to the surface and measurement points near the rock surface, where deformations are greatest, and should be more closely spaced than those further from the surface. It is important that anchors be placed outside the zone of influence of the openings in at least some of the instrument holes. These will give a 'stable' reference point for the other data.

Instrumentation for deformation measurements can be either conventional, mechanical, extensometers (using tapes for convergence and wires for boreholes) or the more sophisticated electrical type. On balance mechanical extensometers are preferred for reasons of economy and ruggedness. If electrical instruments such as LVDT's are used for borehole measurements, the transducers should be sited at the mouths of the borehole to allow easy replacement. The borehole measurements should be grouped in a single borehole for each station and direction, as far as possible, by using multi-anchor assemblies (see e.g. 8).

Anchor design should be kept simple. Mechanical anchors (e.g. 8,9) are preferred to the hydraulically activated type for reasons of long-term reliability.

8.2.2 Stress measurements

Stress changes, where these are repaired, should be measured at various depths into the roof, floor and pillars. Both vertical and horizontal stress changes should be measured if possible, but the number of stress measuring boreholes should be kept to a minimum.

Stress measurement in viscoplastic materials must be approached with care. Rigid inclusion stress meters are not satisfactory. Soft inclusion (rubber, salt, etc.) are preferred (10). Again, instruments should be chosen for simplicity and ruggedness.

8.3 Determination of the primitive stress field

A knowledge of the primitive stress field (magnitudes and directions) is important to a full understanding of the mine stability, and to the ability to predict this stability. It represents the initial condition from which all mining induced stress and deformation fields are developed.

Unfortunately, the measurement of this stress field in salt is full of uncertainties related to the mechanical properties and crystalline nature of the material.

Standard techniques such as borehole pressurization and overcoring have been applied with mixed success. Serata (11) has claimed success in the use of a pressurization technique, though this relies upon the validity of the REM code (Section 4.1). Recently success has been claimed using overcoring of a rubber inclusion stress meter (12). Hydrofracting is attractive since salt behaves in a nearly elastic manner under tension (9) and the geometry of the method is simple. It is recommended that measurements of the primitive stress field be conducted with more than one technique and compared to numerical simulations of room closures and pillar deformations. In this way, an assessment of the validity of the data will be possible.

9. Conclusions

The following are the major conclusions of this review:

- The storage area design is unconventional. No acceptable calculations are given to support this design and no comparison to mining experience is made. No comparison is made to conventional designs as justification for this design approach.
- The finite element code used for calculations on the stability of the storage area is unsuitable. Highly questionable extrapolations are used. The idealization used is erroneous and not representative of the geological section as it is known. The analysis takes no

account of the time sequence of development. The equation used for salt behavior differs from those based on SENM salt tests and is not a result of these tests.

- The main entries use a similar unconventional design principle. No supporting calculations or evidence is given. This is particularly serious in view of the high degree of stability (including low closure rates) required in the main entries.
- The shops use 20-foot "yield" pillars. These are unacceptable because of the high degree of stability (including low closure rates) required in the shops, and the greater height of the pillars which leads to a very low W/H ratio. The use of sloped ribs could cause problems. No design calculations are given for these areas.
- The shaft pillar dimension is in line with practice in the area; however, most mines in this area exclude the shops from the shaft pillar and place them in a "plant subsidence pillar" area. This practice should be seriously considered.
- The development of a comprehensive, early and continuing, instrumentation plan for the shafts and for the mining area is essential. The information from such a plan is absolutely necessary for design verification and/or modification.

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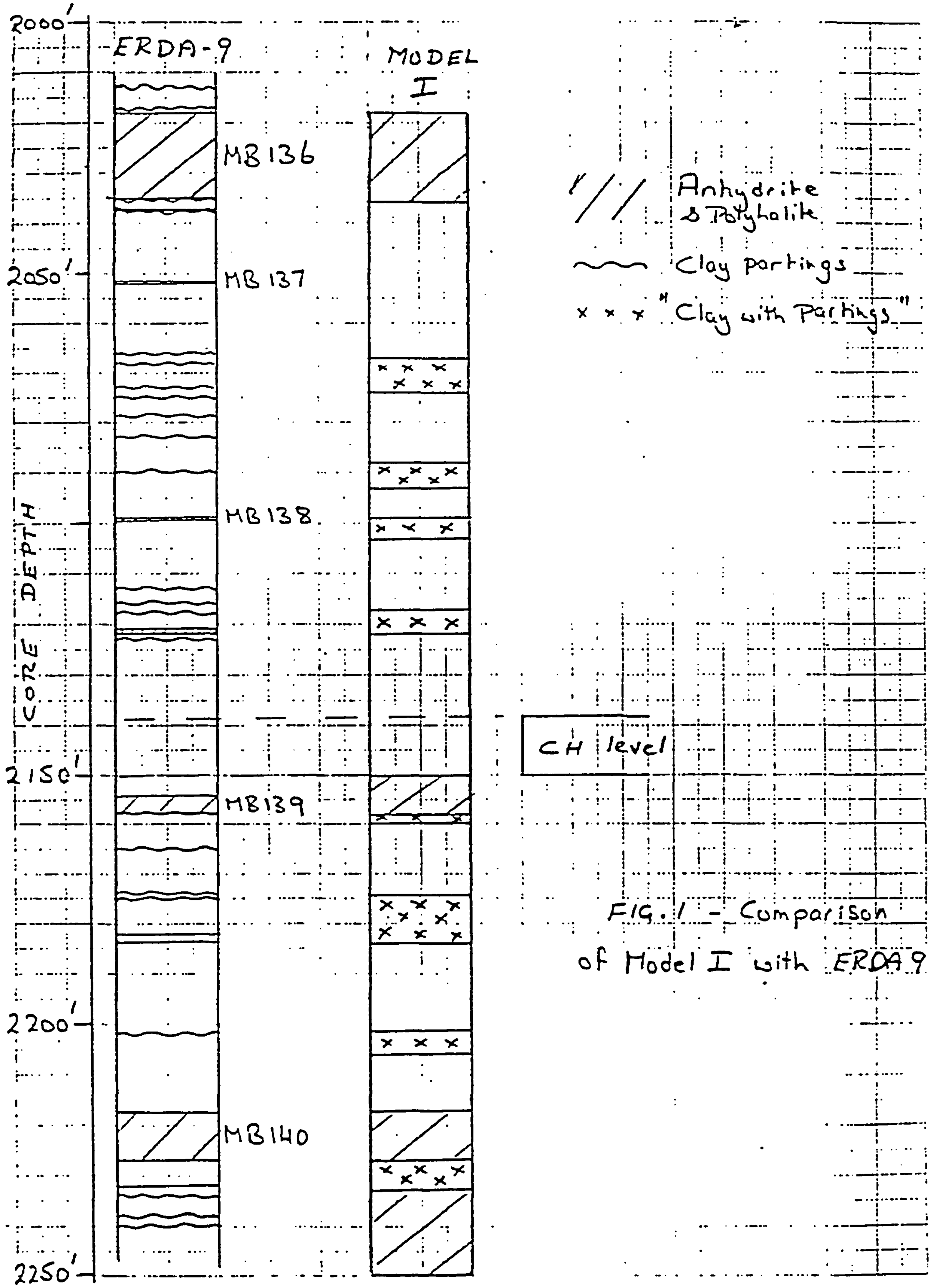

FIG. 1 - Comparison of Model I with ERDA 9

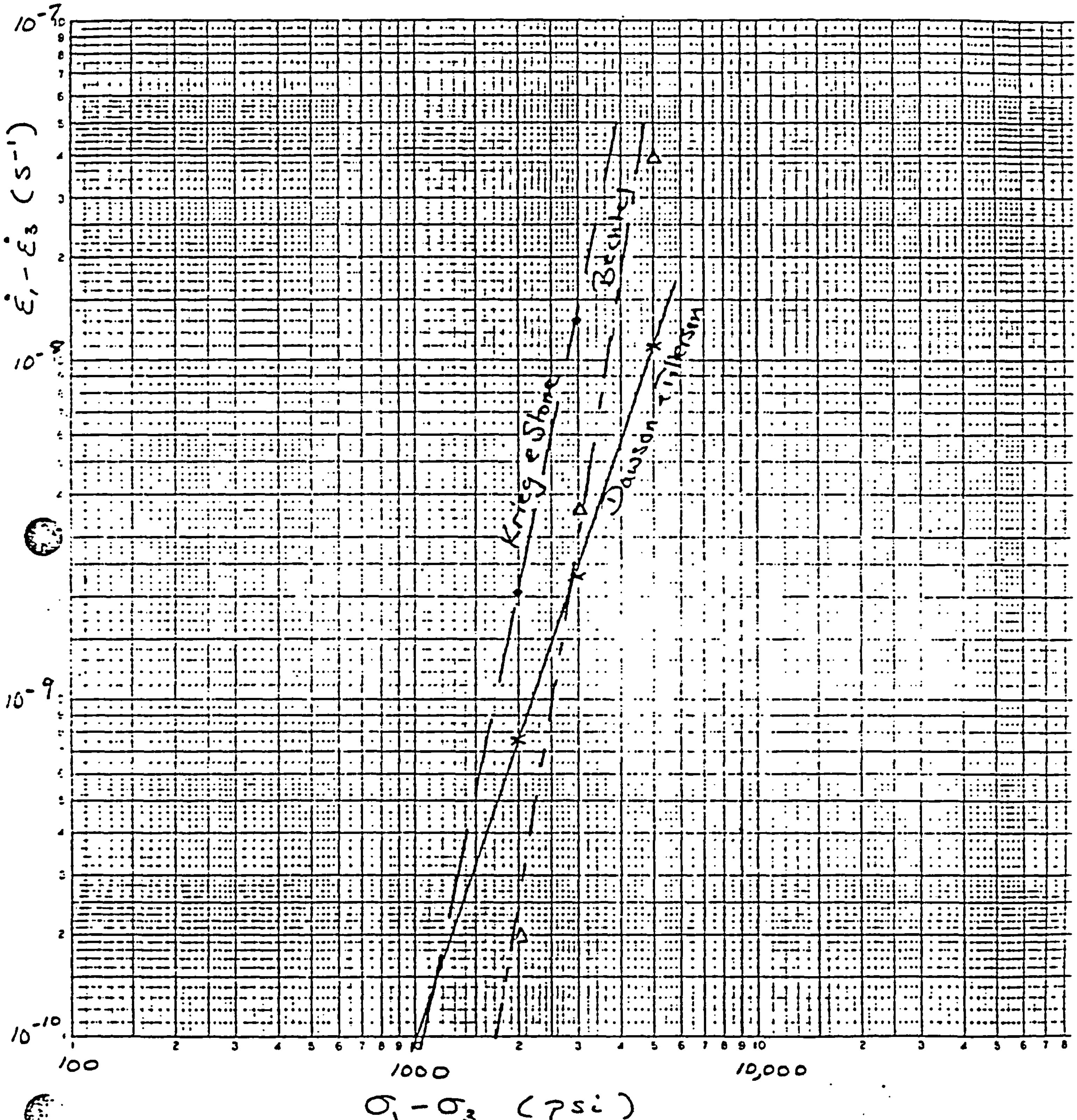

FIG 2. Secondary creep laws

Full Logarithmic, 3 × 3 Cycles

APPENDIX A

LIST OF ENCLOSURES

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2. Drawing #24-C-001 Rev. D
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<u>Underground Plans and Sections</u>

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5. Drawing #51-W-002 Rev. D (dated 9/18/79)
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Underground Design Basis

18. Design Basis D-51-W-01 Rev. 2
    Underground Excavation

19. Design Basis D-54-T-01 Rev. 3
    Underground Shops and Warehouse

Geological Section

20. Document 22-V-510-02 Rev. 0
    Geologic Data and Hole History for Borehole B-25

21. Document 22-V-510-01 Rev. 0
    Geologic Data and Hole History for Borehole WIPP-12

22. ERDA-9 Crib Sheet (USGS)
    Abridged History of Borehole ERDA-9

23. Letter - Twenhofel, W.S. "Lithologic Description." 3/6/79

24. BSCN-125 - ERDA-9/WIPP 12 Correlation meeting - 4/24/79

25. BSCN-132 - ERDA-9 Core Examination Single Horizon WBS 22V Meeting 7/20/79.

26. CN-114 - Examination of Geological Cores WBS 76 - 12/12/78

27. CN-146 - WBS No. 22 Meeting with Charlie Jones USGS to discuss ERDA-9
    Drill log - 1/24/79

28. Geological Characterization Report (WIPP) Sand 78-1596 Chapter 4

ROCK PROPERTIES

29. Document 22-V-510-04 Rev. 1
    Geology Laboratory Results of Rock Testing

30. Interim Summary of Sandia Creep Experiments on Rock Salt
    from WIPP Study Area, Southeastern New Mexico. Sand 79-0115
    by Wawersik, W. R. and Hannum, D. W.

Rock Properties and Size Effects - Lab Testing vs. In-Situ Properties

31. Geological Characterization Report (WIPP), Sand 78-1596 - Chapters 4 and 9.

32. BWCN-44 - Pre-Design Review Meeting
Geotechnical Subjects WBS No. 22V
Meeting Date 9/17/79.

SANDIA CREEP DATA

33. BDCN 195 - Shaft and Underground Excavation Computer Code Analysis - Creep Closure of Opening. WBS 51 and 61. Meeting Date 9/21/79.

34. BDCN 217 - Salt Creep Law - WBS 51 & 61. Meeting Date 11/13/78

35. Constitutive Models Applied in the Analysis of Creep of Rock Salt.
Sand 79-0137 - Dawson, Paul R.

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37. Letter Report - Stone, C. M. and Krieg, R. D. - "Results of Thermal-Structural Analyses of Bechtel's Scheme 4 for the WIPP Project." (5/8/79)

38. Letter Report - Wayland, J. R., "Room Closure Calculations for Title I." 3/29/79.

(See Rock Properties and Size Effect Sections for the Following)

39. BWCN-44 - Pre-Design Review Meeting
Geotechnical Subjects WBS No. 22V
Meeting Date 9/17/79.

40. Geological Characterization Report (WIPP)
Sand 78-1596 - Chapter 9.

41. Interim Summary of Sandia Creep Experiments on Rock Salt from WIPP Study Area, Southeastern New Mexico, Sand 79-0115 by Wawersik, W. R. and Hannum, D. W.

Empirical Data Supporting Design

42. BSCN-115 - Trip Report to Rocanville Mine - WBS 76. Date 2/14/79.

43. BSCN-116 - Trip Report to Cory Mine - WBS 76. Date 2/14/79.

44. CN-150 WBS No. 76 Trip report - Rocanville Mine Date 11/30/78.

Bechtel Underground Analysis

45. BDL-620 - Reference Report Stability Analysis of the Underground Openings - WBS N.51 of 9/7/79 with TSC Comments.

<u>Trip Reports Not Given to Bechtel During Design</u>

46. Trip Report 9/12 - 9/13 to Duval, Kerr McGee and National - (Bechtel present during trip).

47. Trip Report - Kerr McGee Potash Mine March 14, 1979, WIPP:AKK:79:1646

48. <u>Reference Size Effects</u> - Baar, CA, Applied Salt - Rock Mechanics (Not included)

APPENDIX B

Comparison of Sandia and Bechtel Creep Laws

Two secondary creep laws have been proposed by Sandia based on data for SENM salt. These are:

1. Dawson and Tillerson (A 35 - see also ref. A 33)

$$\dot{\varepsilon}_{II} = 1.232 \times 10^{-23} \exp(-5200/T) s_{II}^{3} \qquad \text{-- B1}$$

where

$$\dot{\varepsilon}_{II} = (\tfrac{2}{3}\, \dot{\varepsilon}_{ij}\, \dot{\varepsilon}_{ij})^{\frac{1}{2}} \text{ ; the effective deformation rate in sec}^{-1}$$

$$s_{II} = (\tfrac{3}{2}\, s_{ij}\, s_{ij})^{\frac{1}{2}} \text{ ; the effective deviatoric stress in Pa.}$$

$$T = \text{temperature in } {}^{o}K.$$

Now

$$s_{II} = \frac{1}{\sqrt{2}} \left[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)\right]^{\frac{1}{2}}$$

$$= \frac{3}{\sqrt{2}} \tau_0$$

where τ_0 is the octahedral shear stress.

Also, if the volumetric strain rate $\dot{\varepsilon}_{kk}$ is zero,

$$\dot{\varepsilon}_{II} = \sqrt{2} \left[(\dot{\varepsilon}_1 - \dot{\varepsilon}_2) + (\dot{\varepsilon}_2 - \dot{\varepsilon}_3) + (\dot{\varepsilon}_3 - \dot{\varepsilon}_1)\right]^{\frac{1}{2}}$$

$$= \frac{3\sqrt{2}}{2} \dot{\gamma}_0$$

where γ_0 is the octahedral strain rate

Hence, we may write B1 as

$$\gamma_0 = 1.848 \times 10^{-23} \exp(-5200/\theta)\ \tau_0^3 \qquad \text{-- B2}$$

For a triaxial test with $\sigma_2 = \sigma_3$ and $\varepsilon_2 = \varepsilon_3$ equation B2 becomes

$$\dot{\varepsilon}_1 - \dot{\varepsilon}_3 = 1.232 \times 10^{-23} \exp(-5200/T)\ (\sigma_1 - \sigma_3)^3 \qquad \text{-- B3}$$

2. Krieg and Stone

$$\dot{\varepsilon}_{II} = 3.65 \times 10^{-36} \exp(-5400/T)\ s_{II}^{4.06} \qquad \text{-- B4}$$

where s_{II} is in Pa and $\dot{\varepsilon}_{II}$ in sec^{-1}.

For the triaxial test and the same assumptions as before, this becomes

$$\dot{\varepsilon}_1 - \dot{\varepsilon}_3 = 3.65 \times 10^{-36} \exp(-5400/T)\ (\gamma_1 - \gamma_3)^{4.06} \qquad \text{-- B5}$$

3. Bechtel

Bechtel uses a viscoplastic law (A 45):

$$\dot{\gamma}_0 = \frac{K_0}{V_4} \left[\frac{T+273}{293}\right]^{9.7} \left[\frac{\tau_0 - K_0}{K_0}\right]^{2.62}$$

where K_0 is the octahedral shear strength, given by

$$K_0 = K_A + (K_B - K_A) \left[1 - \exp(-0.00156\sigma_m)\right],$$

$$K_B = 800 \text{ psi}$$

$$K_A = K_B/3$$

$$\sigma_m = \text{"confining pressure"}$$

and V_4 = viscoplastic viscosity = 1.5×10^6 psi. day

For the triaxial test this gives:

$$\dot{\varepsilon}_1 - \dot{\varepsilon}_3 = 6.499 \times 10^{-9} \left[\frac{T+273}{293}\right]^{9.7} \left[\frac{\tau_0 - K_0}{K_0}\right] \sec^{-1}$$

Table B.1 and Fig. B.1 give strain rates calculated from these three models.

Table B.1

$\sigma_1 - \sigma_3$	$\dot{\varepsilon}_1 - \dot{\varepsilon}_3$ (sec^{-1})		
(psi)	Dawson & Tillerson	Krieg & Stone	Bechtel
1000	9.5×10^{-11}	7.5×10^{-11}	--
2000	7.6×10^{-10}	22×10^{-10}	2.1×10^{-10}
3000	2.6×10^{-9}	16×10^{-9}	3.8×10^{-9}
5000	1.2×10^{-8}	19×10^{-8}	4.2×10^{-8}

The following should be noted

- Dawson and Tillerson's model is earlier than Krieg and Stone's.
- Krieg and Stone base their model on several data points for SENM salt. However, the existence of true secondary creep in many of these tests is uncertain due to the short duration of the tests. This could lead to an overestimate of strain rates.
- Bechtel ignores viscoelastic creep due to the faster rate of viscoplastic creep. This is reasonable at high stress differences however, the 1000 psi value for $\sigma_1 - \sigma_3$ does not exceed the octahedral shear strength.

J J Peshel

From : Engineering
WIN :
Date : August 5, 1983
Subject: Vertical vs. Horizontal Emplacement of Defense High Level Waste in Layered Salt

To : D. Rasmussen

SUMMARY

The WIPP will receive two categories of Remote Handled Waste. RH TRU waste and Defense High Level (Experimental) Waste. The present plan is to place RH TRU Waste into horizontal drill holes and the experimental waste into vertical drill holes in storage room floor. It would be cheaper to place both wastes horizontally and save the cost of special equipment for vertical emplacement and recovery. However, there are some strong reasons to place the experimental waste in the vertical holes in the floor, as shown in this report. The main reason is the long range stability of the underground opening when subjected to high level radiation and especially the heat from experimental waste.

The heat and radiation weaken the surrounding salt. The heat affects the whole adjacent opening, the radiation affects only the immediate surroundings of the canister. If the canisters with experimental waste are placed horizontally in the walls it accelerates the creep rates in the pillars. An axial load on the roofbed causes a buckling type of accelerated sag of the roof.

This accelerated sag is, therefore, seen as a result of the disturbed stress field caused by the superimposition of thermal stresses onto the preexisting overburden and mining stresses. The sag could cause collapse of the opening, unless reinforced by roof bolting.

Another stability problem is that the salt in the pillars around the horizontal canisters is losing some of its compressive strength. These portions of the pillars next to the opening do not carry their share of the overburden load. It effectively increases the span of the opening, making it less stable.

Discard Date:

By placing the experimental waste in the vertical holes in the room floor the source of heat and radiation is kept at the maximum distance from the layered roof "beam" and the pillars. The stability of the opening is better secured.

The heat output of the experimental waste is expected to be 250W - 810W per canister. The need for the vertical placement becomes less important if the heat output of the waste canisters to be stored is lower than 250 W.

There are some other factors influencing the decision on canister location: The hot canister, if placed in the pillar, will move from horizontal position and it will be more difficult to retrieve it. Drilling and heavy lifting equipment works better in vertical direction. Vertical hole can be more reliably backfilled.

The conceptual designs for commercial spent fuel repository in salt use vertical emplacement holes ("AGMES Report"). The experimental or permanent storage of DHLW in salt should use them also.

DISCUSSION

Stability of Underground Openings in Layered Salt

The underground openings of the WIPP depository were designed using experience from a number of potash mines in the Carlsbad Area in comparable lithology and stratigraphy. Room and entry width as well as pillar spacing for WIPP were developed based on the experience with those mines. In addition, the selected room and pillar sizes were checked by different state-of-the-art rock mechanics methods for design of openings in a layered rock.

The expected stresses around an underground opening at WIPP are relatively high (see Enclosure 1: "Stress Distribution Around a Horizontal Opening" by Bechtel). In situ stresses in the undisturbed ground are on the order of 2000 psi. Stress around an opening just after its excavation are in the range of 3,000 to 4,000 psi. These stresses are higher than the unconfirmed compressive strength of salt and, therefore, local spalling with associated stress redistribution have to be expected and provided for.

In conjunction with retrieval high stresses, high plastic creep deformation and resulting large convergence of underground openings typically occur in rock salt.

Discontinuties between the rock strata caused by the clay seams in proximity to roof or floor adversely affect the immediate roof or floor of the opening. To diminish the effect of the discontinuity induced into the layered deposits by clay seams, the room elevation has to be carefully chosen to ensure, that the roof beam and floor beam of the opening were adequate. The quality of the roof beam is the most important, especially for an opening with a larger span. Because of the non-uniformity of the rock mass, the occurrence of zones of weaknesses or cracks should be taken into consideration. The cracked roof beam behaves like an arch. This behavior is different from that of a solid beam.

Once a crack has developed, such a beam can fail in three ways. (See Enclosure 1 - Arching Action of Roof Beams by Bechtel). If the Horizontal Thrust (T) is not great enough, the blocks of rock could simply slide down and the roof would colapse. A second possible mode of failure is by the rock crushing at points of high compressive strength, permitting the rotation of blocks and consequent collapse. A third possible mode of failure is by elastic buckling where the rock at the abuttments and center could deform to such a degree that blocks can rotate without exceeding the crushing strength of the rock at point of rotation.

The Two Types of Remote Handled Waste at WIPP

There are two types of remote handled waste to be placed into drill-holes. The size of the canisters and the size of the drill-holes is similar. But the two wastes differ substantially in the amount of the heat and radiation they emit. Also, the weight of the canisters differs.

Following table shows the difference in the two types of wastes and their emplacement as designed:

DHLW (Experimental)	RH TRU Waste
Radiation Output:	
300 R/Hour - 7,000 R/Hour	25 R/Hour - 1,000 R/Hour
Heat Output:	
250W - 810W/canister	60W/canister
Temperature:	
150°F - 320°F	85°F - 95°F
(65°C - 160°C)	(29°C - 35°C)
Weight of canister:	
2,000 lbs - 20,000 lbs	7,000 - 8,000 lbs
Length of Canister:	
10' to 11'	10'5"
Diameter of canister	
24" to 31" O.D.	26" O.D.

Drill Hole Casing
None

2" Thick Steel Sleeve
30"I.D. - 36" O.D.

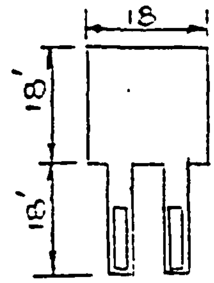
Drill Hole Length (Depth):
18'

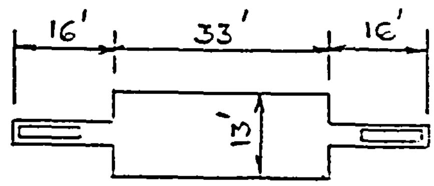
16'

Drill Hole Diameter:
27" - 36"

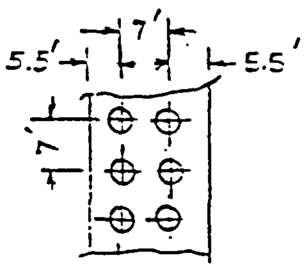
36"

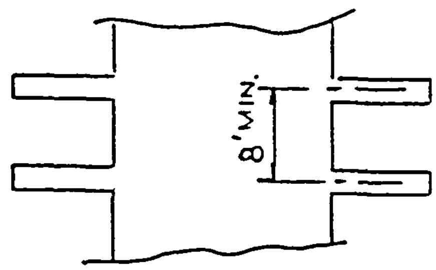
Storage Room Geometry:
Cross Section:



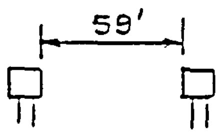


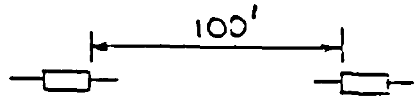
Plan:





Pillar Cross-Section





Effect of Radiation and Heat on Strength and Creep Rates of Salt Rock

The results of "Project Salt Vault" indicate that the radiation effects of DHLW on physical properties of salt will be minimal. Uniaxial compression tests were carried out at the Oak Ridge National Laboratory with irradiated salt specimens (see Enclosure 2, by ORNL). The tests were conducted at room temperature and the effect of heat in this case was excluded. Doses of more than 10^8r are needed to lower the compressive strength of salt by 10%. That would not affect the structural integrity of the opening, especially if the canister is placed in the floor:

> "... Integrated salt doses as high as 5×10^8 rads would not accumulate at distances of more than 1 ft. from the waste containers. Since the floor does not have to support the overburden pressure and the dose reaching the pillars is insignificant, radiation would not be expected to affect the structural stability of the rooms."

The heat connected with the radiation has much more significant effect on rock salt properties than the radiation itself. The salt has good shielding effect on radiation (similar to concrete).

The significant influence of heat on stability of underground openings was demonstrated on model pillar tests by ORNL (see Enclosure 3). The graph shows the deformation with time of models at various temperatures and stresses. It shows, for example, that the deformational behavior of the salt pillar tested at 4,000 psi at 60°C is approximately the same as the behavior of the sample tested at 6,000 psi and 22.5°c (room temperature). This strongly suggests that the net effect of elevated temperature is essentially the same as that of increased pillar stress.

<u>The Influence of the Location of the Heat Source on Stability of the Opening</u>

At the Lyons Mine in Kansas, the full scale tests have shown the difference in emplacing the heat source in the floor and in the pillar. (see Enclosure 4 - Project Salt Vault, by ORNL). The picture explains how the stresses from the canisters in vertical holes (in the floor center) affect the laminated roof of the opening.

The next picture (Enclosure 5) shows the heated pillar experiment, where heaters were placed in the floor along each side of the pillar:

> "...The measured rock deformations around the heated pillar were similar to those around the array rooms but considerably larger,...."

This is a good example that the heat generating DHLW could endanger the stability of the opening if placed in horizontal holes in the pillars.

The Sandia National Laboratories are preparing a $12W/m^2$ mockup experiment to test the effect of heat from DHLW on salt. One of the objectives is to determine how the structural stability of a proposed repository configuration will affect its operation. The experiment schematic shows the heater in place of future DHLW canister (see Enclosure 6).

The expected results are shown in Enclosure 7. It is a temperature profile as expected at 3 years after the heat start up. A temperature increase of 20°C at the center of the pillar is calculated at 5 years, and at 10 years the temperatures in the bulk of the pillar will have increased by 40°C.

The next figure by SNL (Enclosure 8) shows expected results for overtest for simulated DHLW at 3 years.

Some Other Factors Influencing the DHLW Emplacement

In Favor of Vertical Emplacement

- Vertical drilling, heavy lifting, overcoring is easier than the same tasks in horizontal direction.

- If the hot 10 ton canister is inserted into a horizontal hole it will, with years, change position with the flow of salt in the pillar. It will sink unevenly down and the locating, overcoring and retrieval will be more difficult.

- The backfilling of the vertical hole with salt is more reliable than that of a horizontal hole.

- The roof bolts under heat expand differently than the salt. That could weaken the roof support in some cases, where the canisters are at a close distance to roof bolts (such as when placed horizontally in the pillar).

- Horizontal holes collapse easier than vertical holes because of gravity, or vicinity to clay seams along the hole.

- The height of the 18'x18' rooms can be increased, without causing any serious stability problems. That could happen if more room is needed for equipment, which is still being designed. The emplacing and retrieving equipment might require as much as 22' overhead room.

- ONWI is preparing a conceptual design for repository in salt. They place the commercial high-level waste into the vertical drill-holes. The rooms are 18'x21' high (see Enclosure 9 - storage in tuff and salt is the same geometry). In basalt (Enclosure 10), they use horizontal holes and minimum height of the storage rooms (20' wide x 10' high). It is because of high horizontal stresses in deep basalt at BWIP.

In Favor of Horizontal Emplacement

- The advantage of horizontal placement is the wider and lower storage room, which could be better used for contact-handled waste storage later. This is the reason to place RH TRU waste horizontally. Those canisters do not generate high heat and are lighter weight to handle.

- Vertical holes in the floor are obstructing vehicular traffic through the drift.

SOURCES

1. Peter Frobenius, Boler Chytrowski, Dale Roberts, and Ching L. Wu, Bechtel, "Exploratory Shafts and Underground Test Facility for the WIPP" 1983 RETC Proceedings.

2. R. L. Bradshaw and W. C. McClain "Project Salt Vault" 1971 Oak Ridge National Laboratory.

3. Sandia Reports:

   WIPP R&D Program: In Situ Testing Plan, March 1982;
   Stresses Near Waste Canisters Buried in Salt, March 1983; and
   Test Plan: 12-W/m^2 Mockup for DHLW, Draft, Feb. 1983.

4. Stearns-Rogers Services, Inc. "AGMES Report" (Alternative Geologic Media Engineering Studies), 1983, ONWI.

John Peshel

J. J. Peshel
Engineering

JJP/dk

ENCLOSURE 1

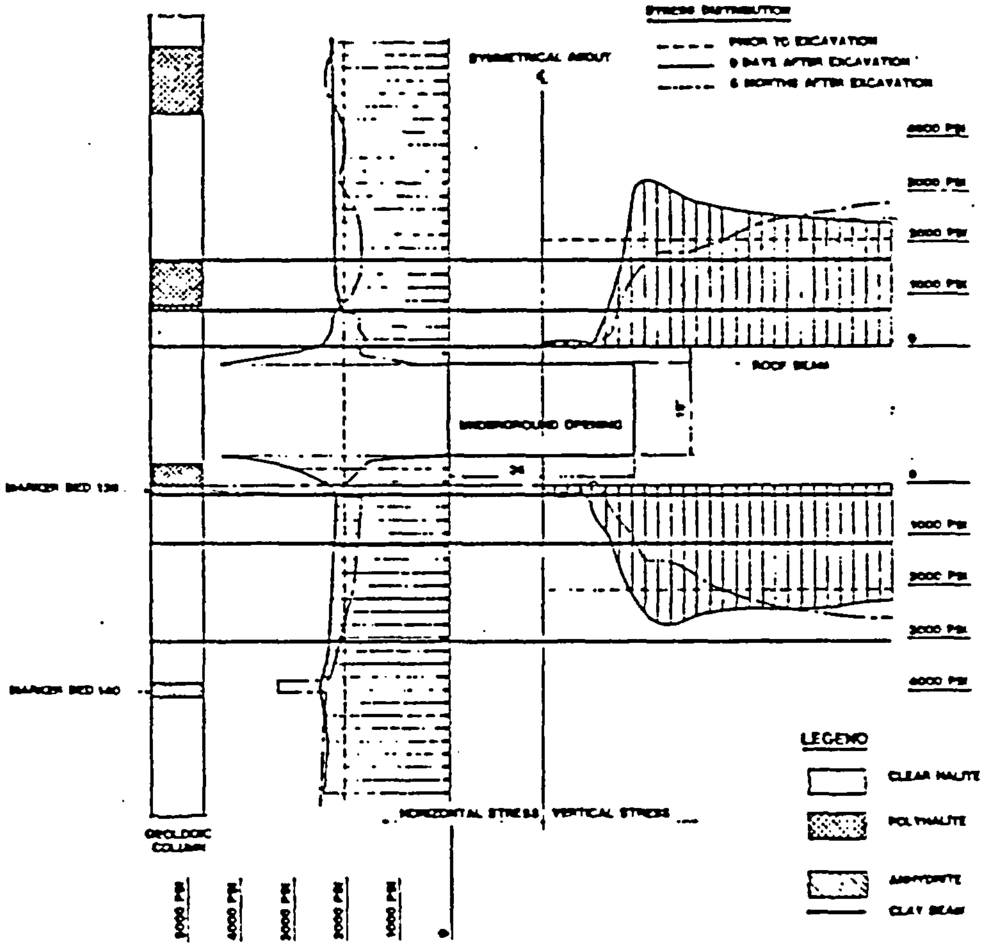


FIGURE 10 STRESS DISTRIBUTION AROUND A HORIZONTAL OPENING

valid.

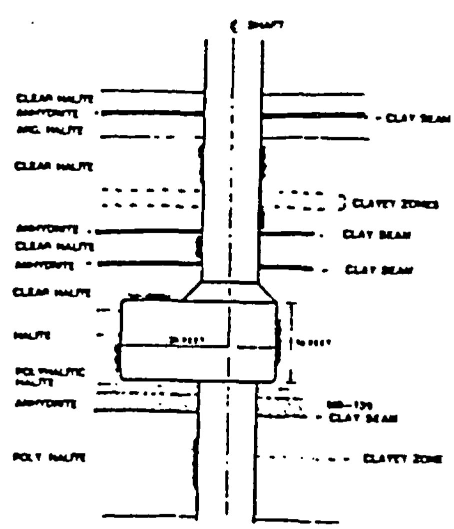


FIGURE 11 DISCONTINUITIES CAUSED BY CLAY SEAMS AROUND AN UNDERGROUND OPENING

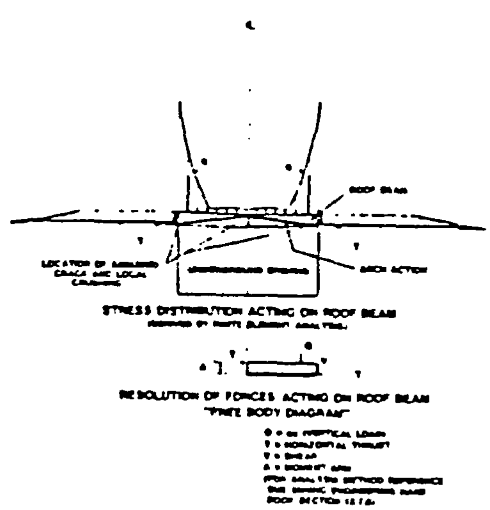


FIGURE 12 ARCHING ACTION OF ROOF BEAMS IN AN UNDERGROUND OPENING

ENCLOSURE 2

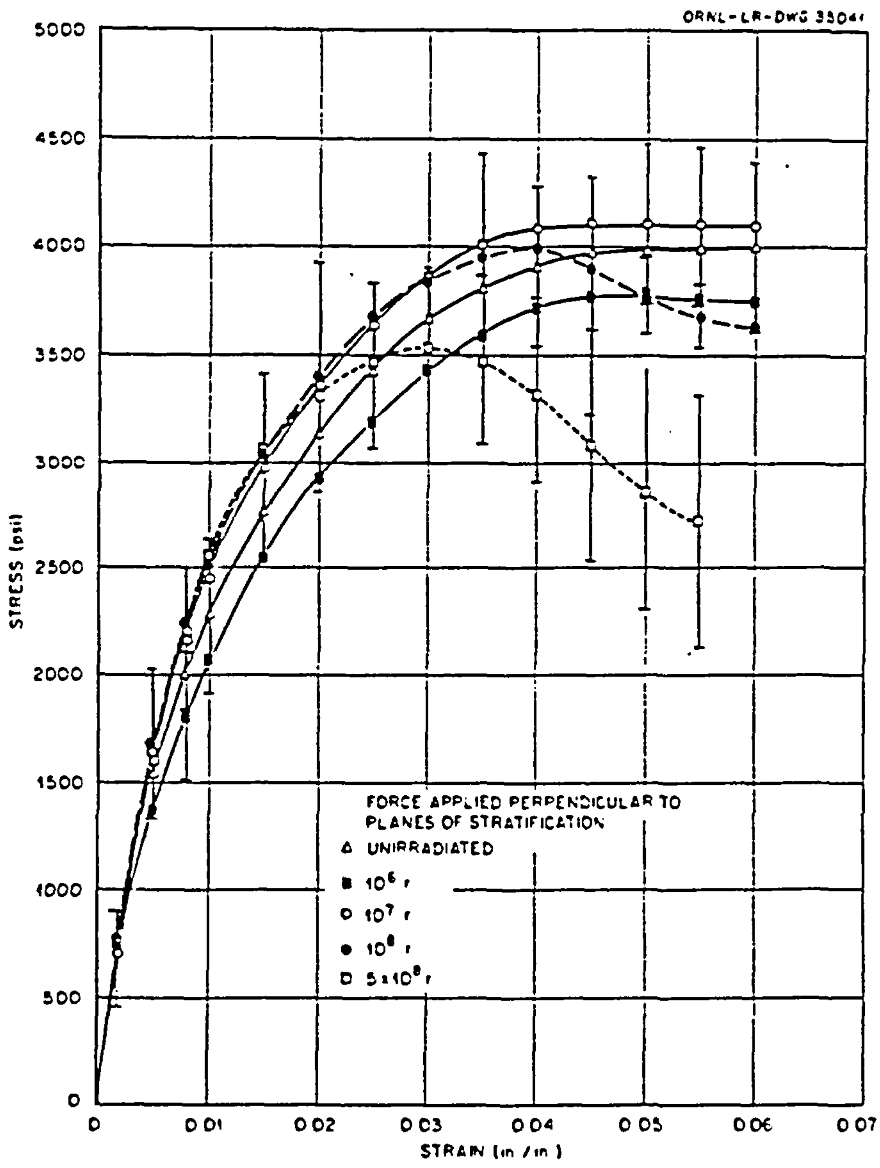


Fig. 2.4. Compression Test of Bedded Salt at Room Temperature.

ENCLOSURE 3

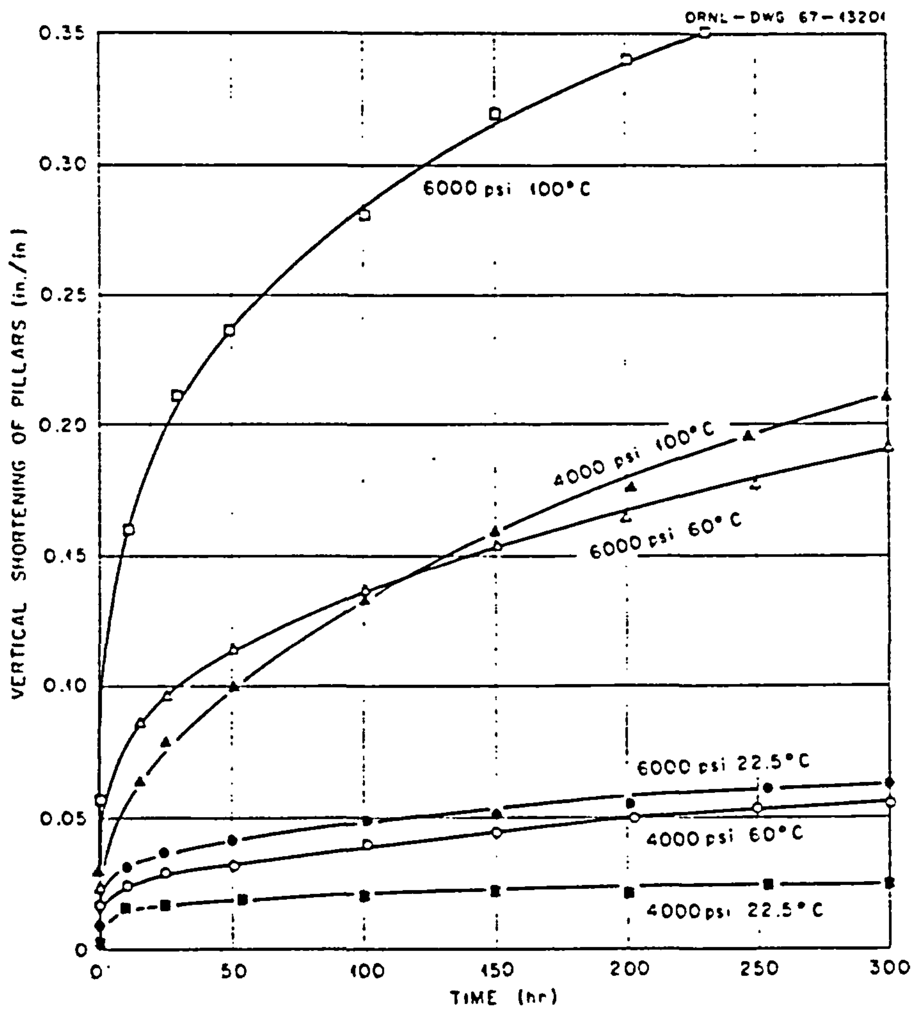


Fig. 12.16. Deformation of Pillar Models at Various Temperatures and Pressures.

ORNL-DWG 68-7583

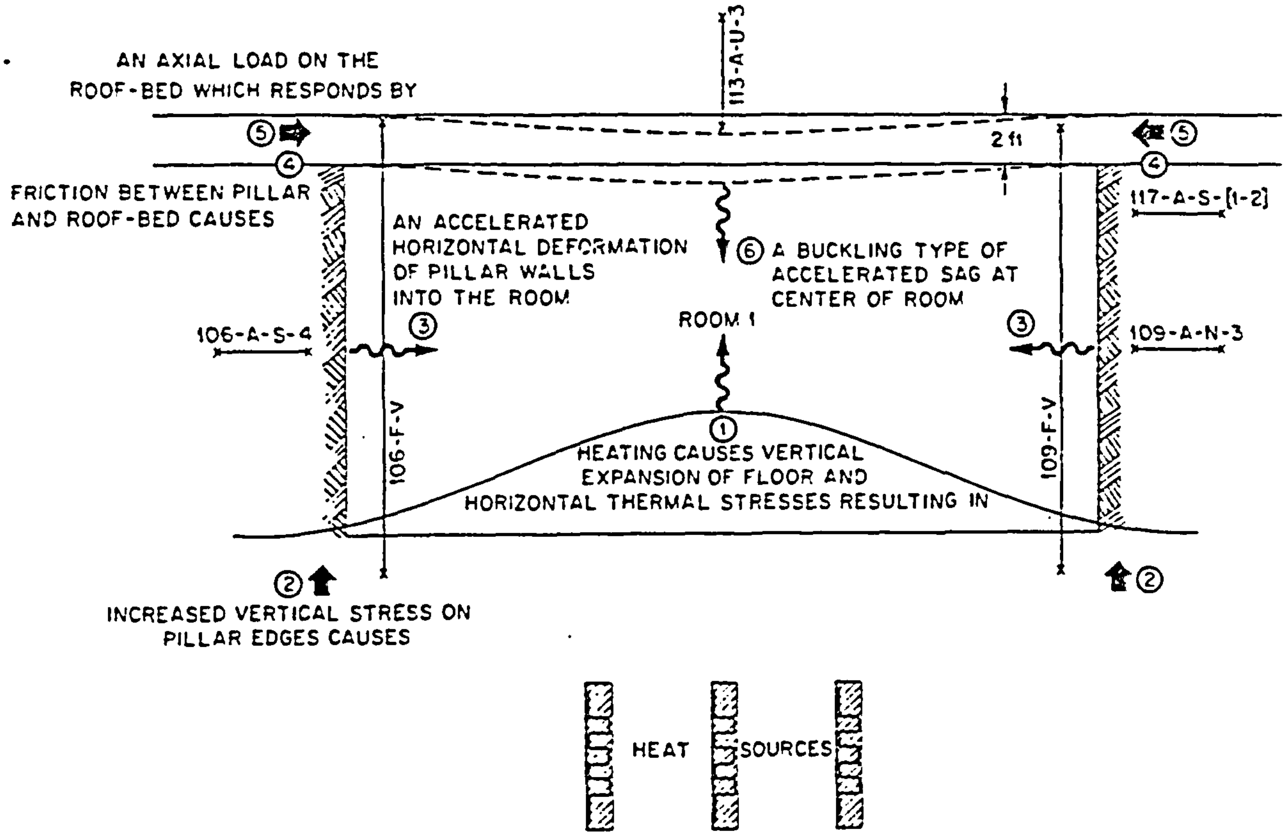


Fig. 14.5. Mechanism of Stress Transference from Floor to Ceiling.

ENCLOSURE 5

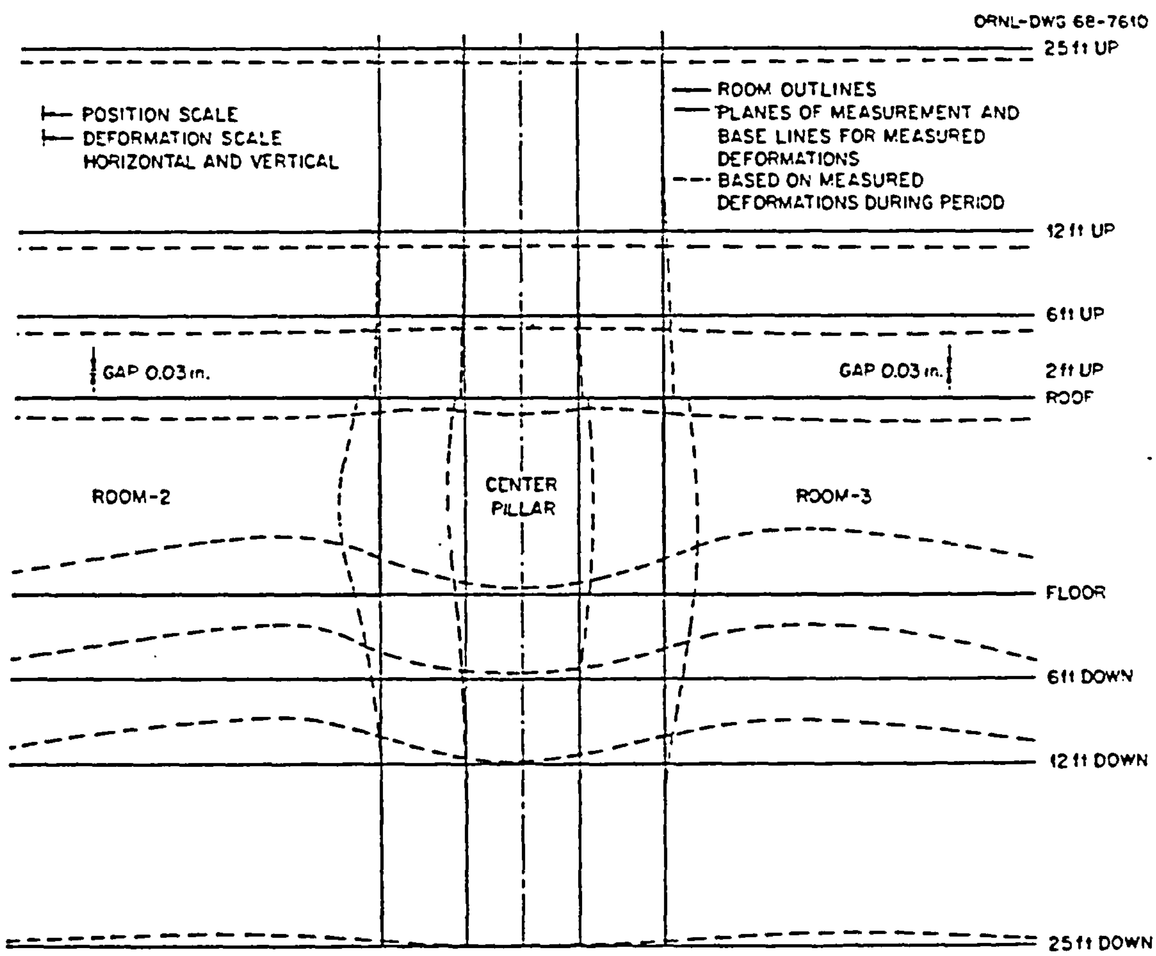
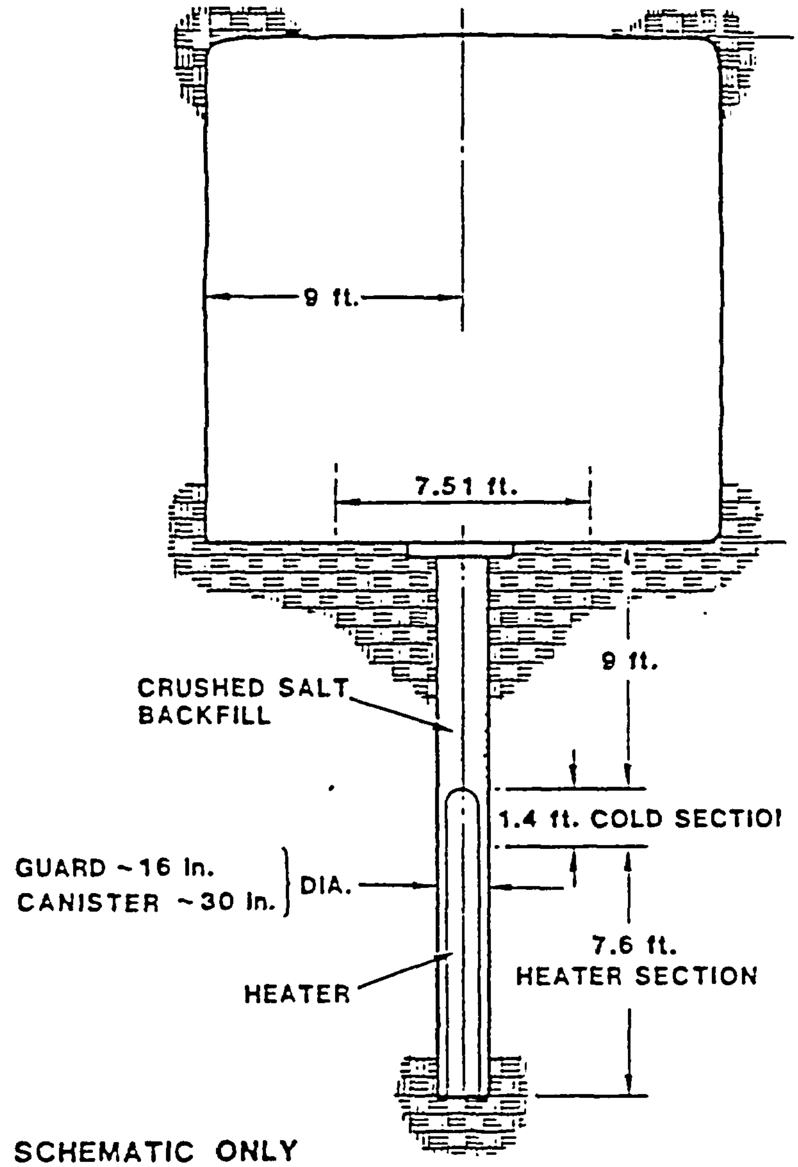


Fig. 14.7. Summary of Deformations Around Center Pillar and Rooms 3 and 4 During Heating.



NOTE:

SCHEMATIC ONLY

GUARD HEATERS
CENTERED IN ROOM, BUT
CANISTER HEATERS
IN DOUBLE ROW

Figure 9.8. Schematic of Canister Emplacements in 12 Mockup of DHLW Test (Note: Canisters Row and Guard Heaters in Single Row)

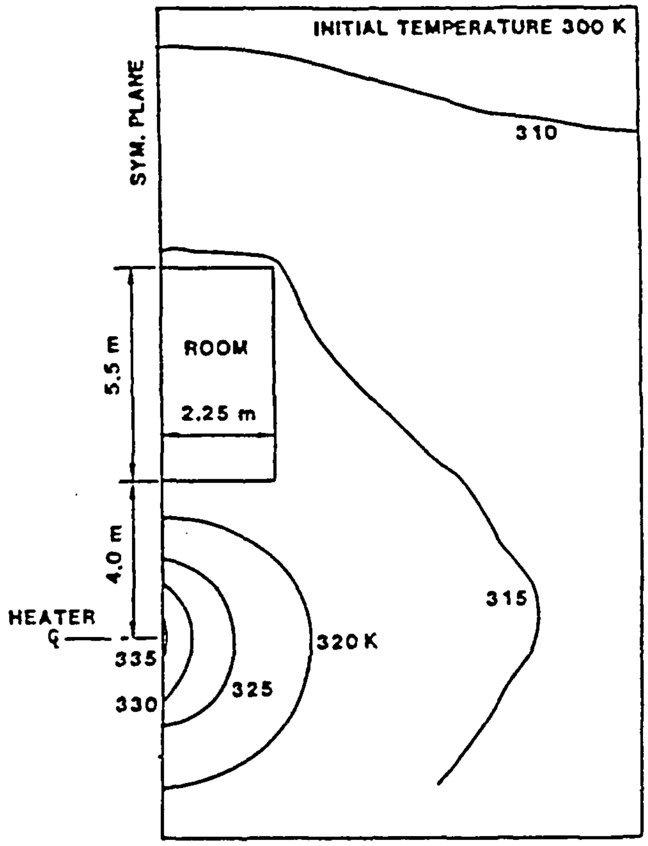


Figure 4-10. Temperature Contours (in K) for the 12-W/m^2 (47-kW/Acre) RRC Mockup Experiment at 3 Yr

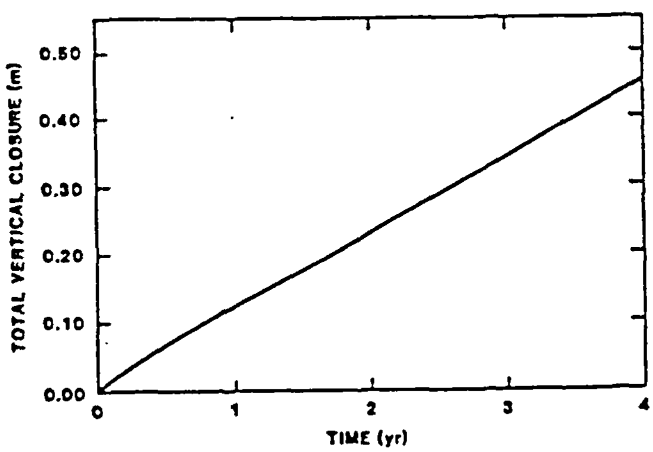


Figure 4-11. Expected Room Vertical Closures as a Function of Time for the 12-W/m^2 (47-kW/Acre) RRC Mockup Experiment

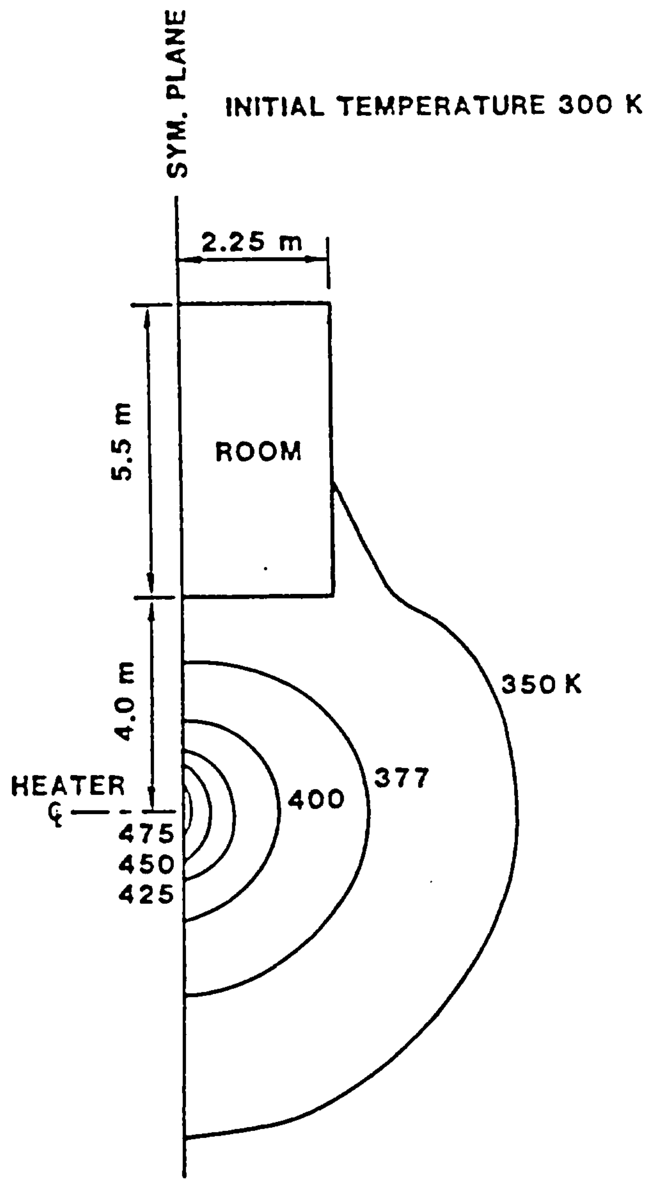
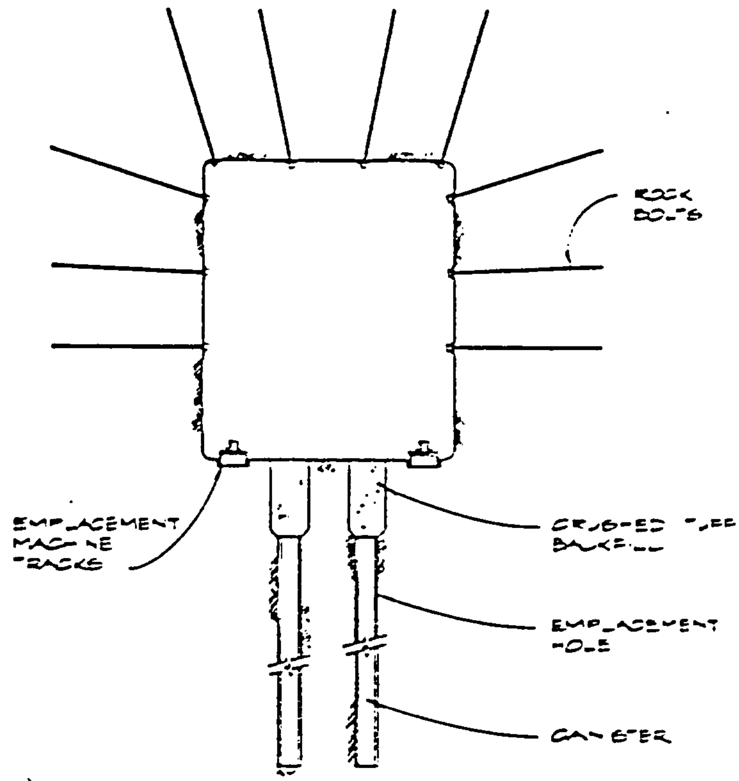
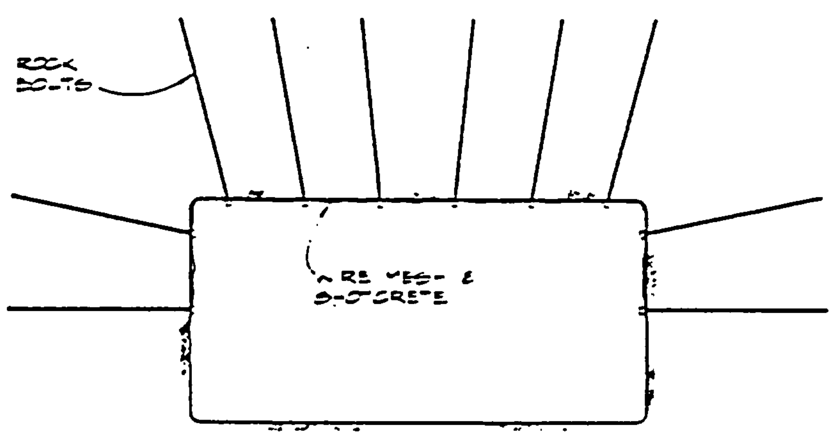


Figure 4-16. Expected Temperature Contours (in K) for the Overtest for Simulated DHLW at 3 Yr

ENCLOSURE 9



STORAGE ROOMS



PASSAGEWAYS

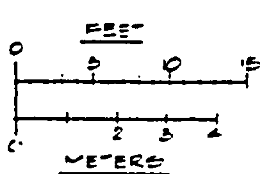
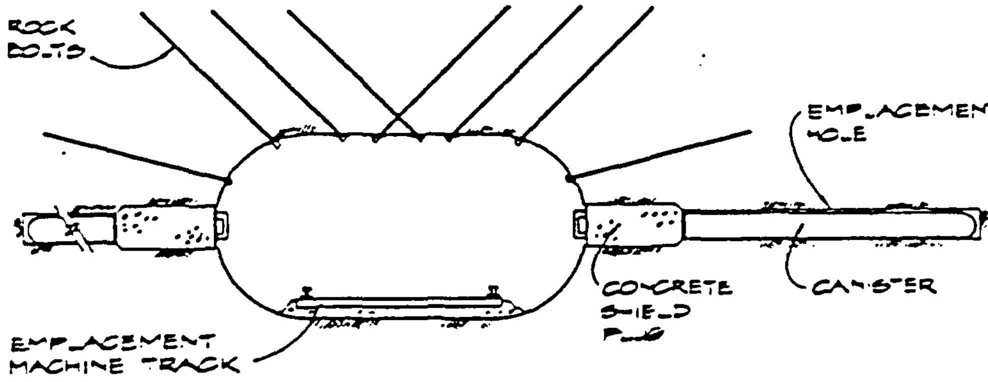


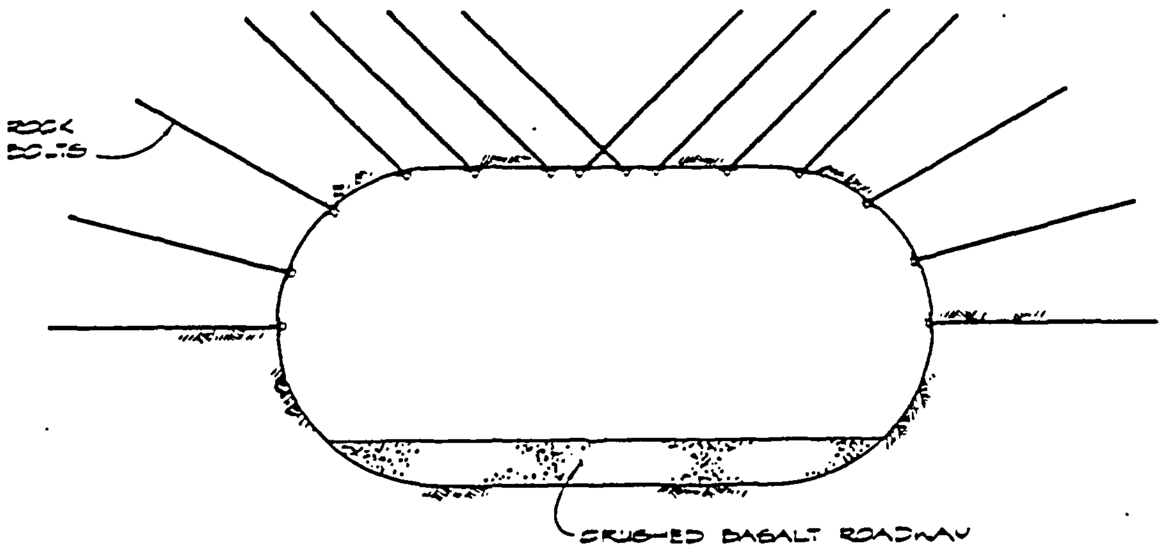
FIGURE 4-10
STORAGE ROOMS AND PASSAGEWAYS
IN TUFF

DATE: 7·6·82 SK 3040-1



STORAGE ROOM AND EMPLACEMENT HOLE

ROOM CROSS SECTION = 16.6m² (179 FT.²)



PASSAGEWAYS

EXCAVATED CROSS SECTION = 50.8m² (547 FT.²)

OPEN NET CROSS SECTION = 44.6m² (480 FT.²)

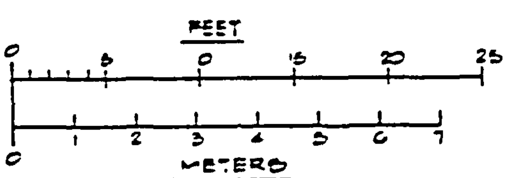


FIGURE 4-11

STORAGE ROOMS AND PASSAGEWAYS

IN BASALT

DATE: 7-14-82 SK.33.0-2

Dravo

October 16, 1979

Prof. John F. Abel, Jr.
310 Lookout View Court
Golden, Colorado 80401

Dear John:

Attached for your review is the package that we have discussed over the past few weeks. This package represents the WIPP underground mine design as it exists today, and the type of review that is desired is defined in the scope of work that is included in the package.

The Department of Energy has requested that your review be maintained as confidential. Any questions you have about the package, or any additional information needed, should be directed through me.

Please forward, by October 24, 1979, a written estimate of your fee and associated expenses for performing the review. As you are aware, other consultants are also reviewing this same package and are trying to complete the review within the period specified in the scope of work. Please advise me, at the time you send your estimate, if you see any problems in completing your review by November 15, 1979.

John, I am verypleased that the Department of Energy has specifically requested your review. I think your recommendations will have a significant impact on the WIPP design, and I personally look forward to working with you on this matter.

Very truly yours,

Ken Beall

Ken Beall
Mine Design Manager
WIPP Project

KB:bbb

Attachment

cc: R. F. Harig
R. McCoy
J. Jimenez

SUMMARY ASSESSMENT OF THE TITLE I UNDERGROUND DESIGN

This document is a summary of several assessments and studies of the Title I underground design developed by Bechtel. Summarized herein are results of the following:

Consultant's Reports

- John F. Abel, Jr., "Review of Proposed WIPP Underground Mine Design", Nov. 15, 1979
- R. Kenneth Dunham, "A Review of the Proposed WIPP Underground Waste Disposal Facility", Nov. 21, 1979
- T. William Thompson, "A Review of the Proposed WIPP Underground Mine Design", Nov. 16, 1979.

D'Appolonia Study of the SCT Method

- A. K. Kuhn and R. D. Ellison, "The Stress Control Technique -- Its History and Suitability for WIPP", Dec 7, 1979

Computer Stability Analyses

- o D'Appolonia
  - o T. Harrington and A. K. Kuhn, "TSC Single Room Concept - Room Closure Analysis", Dec. 7, 1979
- o Sandia
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- A conventional design is preferred to a yield pillar design by JFA, RKD, TWI and D'Appolonia. In addition to the aforementioned issues of the SCT per se, the recommendations for a conventional design are made for the following reasons:

  1) The 15 ft. of good salt beam above the roof obviates the need for yield pillars, allows successful use of full-load bearing pillars (JFA, TWT, RKD, D'App.).

  2) Mine design should aim for simplicity and flexibility (JFA). A yield pillar design is inherently complicated, being sensitive to relative dimensions of rooms and pillars, mining sequence and rates, geologic details, and load capacity of deep roof strata

(TWT, RKD). In contrast, a conventional design allows variations in these factors and permits election of other geometries at a later date (D'App.).

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The Sandia analysis predicts greater closure rates for the yield pillar design than for the single room (conventional) design. The Sandia and D'Appolonia analyses, with corrections to account for the differences in the geologic models used, predict approximately the same closure rates for single rooms.

Other Concerns with the Title I Design

The following specific opinions were expressed in the several assessments:

- o Roof spans should be minimized everywhere, but especially at intersections (RKD).

- o A uniform grid mine pattern should be adopted for WIPP (RKD; JFA in telecon on 12/6/79).

o Better use of Sandia lab data should be used in the design analyses (TWT, RKD).

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TSC - Room Closure Analysis, Single Room Concept

The TSC room closure analysis was made using the Finite Element technique. A single room, 12 feet high by 32 feet wide with pillars 112 feet wide between rooms was used in the analysis. The model had clay seams at 16, 26, 38 and 51 feet above the roof of the room and a three feet thick anhydrite seam four feet below the floor of the room. Clay seams were located below the room at the base of the anhydrite and at 15 and 34 feet below the room floor.

The analysis started with an elastic response to mining of the room. This elastic response resulted in a maximum horizontal closure of 0.82 inches. The analysis then proceeded to show approximately a one to two years of rapid stress-relief creep. This period was characterized by rapidly decreasing creep rates. At the end of this period total room closure was approximately 4.0 inches in both the vertical and horizontal sections. The remaining analysis out to ten years time showed nearly constant loading conditions. The horizontal creep closure occurred at 0.76 inches per year while vertical creep closure occurred at 0.65 inches per year. The final total room closures at ten years were 10.41 inches in the horizontal direction and 9.82 inches in the vertical direction.

The analysis showed that vertical room closures were heavily influenced by the anhydrite seam four feet below the floor of the room. This anhydrite acted as a rigid barrier preventing creep closure from the floor of the room. This stabilizing effect was more than offset by the increased creep closure from the roof of the room due to the location of clay seams. The roof was responsible for 84% of the total vertical closure.

Sandia Room Closure Analysis

The Sandia modeling effort included three models, two of which are summarized herein.

The first problem was a finite element model of a single room 13 feet high by 33 feet wide with pillars 94 feet wide between rooms. This model had clay seams 64 and 118 feet above the room. The floor of the room had an anhydrite three foot thick two feet below the floor of the room with a clay seam underlying the anhydrite seam and a combination polyhalite-anhydrite-halite layer 22 feet thick below the anhydrite.

The analysis started with a one to two year period of stress-relief creep during which the creep rate reduced rapidly. The closure at the end of this period was approximately 2.0 inches. The remainder of the closure occurred at a nearly constant rate of approximately 0.38 inches per year in the vertical and 0.55 inches per year in the horizontal direction. The final closures at ten years were 5.6 inches vertically and 6.8 inches horizontally.

The second problem was a finite element model of a four-room yield pillar layout with the same room size and yield pillars of 25 feet - abutment pillars of 300 feet. The geologic features were the same as those in the first model.

The closure analysis did not show as definitive a change from stress relief creep to constant creep as the previous analysis did. At ten years time the vertical closures were 17.3 inches in the inner drift and 15.0 inches in the outer drift. The horizontal closures were 31.5 inches in the inner drifts and 23.6 inches in the outer drifts.

Closure Criterion

The closure criteria for the WIPP storage rooms has been proposed by Sandia Laboratories as follows:

Criterion 1 -- The storage rooms must be designed to produce a safe, stable environment in which to work and store waste.

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Criterion 4 -- The storage room should be designed to produce a calculated creep closure reduction of room cross section of 40% in 150 years.

Criterion 5 -- The main and secondary haulage ways must be sealed with plugs to limit the interconnected storage rooms to nominally 500,000 ft^3 of waste.

The stability and creep rates predicted by D'Appolonia and Sandia models indicate that criteria 1, 2, and 3 can be satisfied by the single room design approach, but closures of the four-room panel (yield pillar design) could exceed the limits of criteria #3. Satisfaction of criterion #4 is [illegible] ~~mediated~~ by the Sandia model for the four-room design, but the D'Appolonia and Sandia single room models indicate some doubt about that design satisfying criterion #4.

SUMMARY ASSESSMENT OF THE TITLE I UNDERGROUND DESIGN

This document is a summary of several assessments and studies of the Title I underground design developed by Bechtel. Summarized herein are results of the following:

Consultant's Reports

- John F. Abel, Jr., "Review of Proposed WIPP Underground Mine Design", Nov. 15, 1979
- R. Kenneth Dunham, "A Review of the Proposed WIPP Underground Waste Disposal Facility", Nov. 21, 1979
- T. William Thompson, "A Review of the Proposed WIPP Underground Mine Design", Nov. 16, 1979.

D'Appolonia Study of the SCT Method

- A. K. Kuhn and R. D. Ellison, "The Stress Control Technique -- Its History and Suitability for WIPP", Dec 7, 1979

Computer Stability Analyses

- D'Appolonia
    - T. Harrington and A. K. Kuhn, "TSC Single Room Concept - Room Closure Analysis", Dec. 7, 1979
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R. K. Dunham agrees with TWT in principle, although his comments are less specific. He urges design of the shaft pillar in accordance with local experience.

The TSC believes that an acceptable approach to shaft pillar sizing must match the tolerable total and differential settlement of surface structures with conservative empirical methods of shaft pillar design and subsidence predictions.

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- The SCT was developed to increase extraction of ore and increase stability in locations where both were inadequate (for mining objectives) using the more conventional room-and-pillar designs. However, in WIPP maximum extraction is not a goal, and conventional mine designs have proven to be stable in the Carlsbad area (RKD, TWT, JFA, D'App.)

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DISTRIBUTION LIST

WIPP: T. Dillon
J. Gonzalez
G. Pendell
J. Reese
M. Rodich

Return to K. Beall

From WIPP Office

Date March 26, 1979

Subject Professor Abel's Report on Mine Design

To D. I. Hulbert
A. Kuhn
R. Rudolph
J. Treadwell

Attached for your information is Professor John F. Abel Jr.'s, report on the mine design review meeting held on March 13, 1979.

K. Beall

G. K. Beall

GKB/tj

Discard Date:

G-382

Internal Correspondence

To	Ken Beall	Location	Albuquerque	Date	3/21/79
From	R. N. Datta	Location	Denver	Page No.	1

Subject PROF. ABEL'S REPORT ON WIPP MINE DESIGN

cc: J. L. Soma
R. F. Harig

Enclosed please find the report by Prof. John F. Abel, Jr. of the Colorado School of Mines following the meeting on March 13, 1979 held at the Craddock Building to discuss mine design for the WIPP.

In the bottom paragraph of page 2 in his report John refers to "pages 60 and 61 in the 4th section of the blue covered handout". Copies of these pages are attached. In Table II various design features of actual salt and other mines are listed. Seven of these mines are identified with Dravo. This relates to field data in the Dravo study in 1974 on "Analysis of Large Scale Non-coal Underground Mining Methods" for the U.S. Bureau of Mines.

I shall be able to submit my report by the end of the month. My report will be in accordance to your memo of March 14, 1979 to John L. Soma.

R. N. Datta

R. N. Datta

dp

Encl.

RECEIVED
MAR 26 1979
WIPP PROJECT
DRAVO-ALBUQUERQUE N.M.

Dravo

JOHN F. ABEL, JR. • *Mining Engineer*

310 *Lookout View Court, Golden, Colorado* 80401 • 279-4901

March 19, 1979

Dr. R.N. Datta
Dravo Corporation
1250 Fourteenth Street
Denver, Colorado 80202

Dear Robin:

The following letter report on the WIPP review meeting March 13, 1979 is submitted pursuant to your request.

The use of an unconventional mine design for the WIPP facility appears to me to be highly questionable when successful conventional mines are operating in the Carlsbad district. It is my belief that the instances of single entry instability described by Dr. Shosei Serata are primarily the result of the site specific geology. A mining operation does not have the luxury of moving up or down in the geologic section to achieve stability for extraction openings. This luxury, actually, this necessity for WIPP to select an horizon with maximum opening stability should permit stability for single or multiple openings.

The a priori assumption by Dr. Serata that narrow single openings are less stable than multiple openings did not prevent him from employing single entries at two critical locations, (1) in the shaft pillars and (2) for storage area access on the CH level. In fact, the single entries running from the access entries to the storage rooms and between panels of storage rooms traverse abutment loaded (more highly stressed) barrier pillars. It seems important to the eventual defense of the mine design before the Nuclear Regulatory Commission (RC) that a more rational justification be prepared if this design is adopted.

The short discussion of a creep law for salt appears to me to be a side issue. Table I attached to the end of this report presents the results of various investigators. Their results have been compared to the Salt Vault results in at least two instances and found to produce reasonable predictions of mined opening creep at Lyons, Kansas. As was brought out in the discussion the Lyons constants are unlikely to exactly match those eventually determined at the WIPP site. Instrumentation in the WIPP shaft(s) and in the WIPP openings will permit site specific constant determination. Increasing depth and thermal gradient in the shaft will permit evaluation of pressure, temperature and time constants.

The assumption that limited site specific experience from Saskatchewan potash mining can be applied to WIPP salt extraction without modification was not vindicated by the discussions at the meeting. The critical information for CH and RH level selection at the WIPP site will only be available when the initial openings are driven out into shaft pillar. These openings will have to be instrumented to measure convergence and holes drilled up into the roof to detect bed separations. Holes should also be drilled into the floor to detect any bed separations, or potential bed separations in the strata below. Ramping down into the floor to obtain additional salt between the roof and a parting that is opening up overhead obviously cannot be done blindly. I do not have the degree of confidence in the stratigraphic continuity described by the geologists at the meeting. Two drillholes are only an indication. The experience of Cleveland Potash Ltd. at Boulby demonstrated the hazard in connecting bedding intercepts from hole to hole, even when you have 18 holes.

The use of yield pillars in mining is a venerable method of limiting subsidence while obtaining reasonable extraction. This method is being employed in potash mining in Alsace. Pages 60 and 61 in the 4th section of the blue covered handout I sent you show deformation observations and the full panel extraction achieved there. The French have also mined panels with small yielding pillars. An excellent paper on the yield pillar technique is Barrientos and Parker (1974) which describes the White Pine Copper Company experience in Michigan. The critical part of yield pillar design is dimensioning the panel pillars to yield under tributary area loads but to support the column of rock under the arch. The size of yield pillars can actually decrease toward the outside of the panel for large arch widths which occur at greater depths. The column of rock beneath the arch increases toward the center of the panel. This decreases outer room stability, however.

The main load carrying barrier pillars at the side of each panel must be large enough to carry the loads transferred from the panels on both sides. Obviously this latter point is more important where extraction is the object, which is not the case at WIPP.

I don't believe enough emphasis was given to the more severe roof problems which occur over the outer rooms of a yield pillar panel. The roof over the outer rooms is flexed more than the internal rooms. In retreat mining this is not a problem because no one needs enter these rooms after they are mined. The roof can collapse later without affecting the mining operation. However, in the case of WIPP, either the roof must be prevented from collapsing or no waste can be stored in the outer rooms of a panel. The height of collapse must be known in the case of WIPP. We must be able to demonstrate that any collapse will be limited and not open the possibility of communication with an aquifer. The 25 year reentry and retrieval requirements require stability. This problem resulted in the consideration of stub drifts beyond the outermost storage rooms in the Fenix and Scisson, Inc. WIPP conceptual design. The pillar design philosophy used by F&S was a uniform lowering of the roof across the entire repository. The F&S roof lowering was the result of applying Lomenick's creep equation to the pillars.

The placement of the CH level immediately beneath a 2.5-ft thick anhydrite bed is untenable. A strong but thin roof member subjected to a high axial load would not "eliminate roof bolting" as indicated by Bechtel but would tend to buckle into the 44-ft wide rooms. The "major separation" indicated at the base of the anhydrite will tend to release the top of pillars and allow them to expand into the rooms. I believe Bechtel should come up with a new stratigraphic location.

The core logs shown at the meeting appear to indicate that any depth below 2580 ft may be satisfactory geologically. Therefore, the shallower the better in order to reduce the creep rate by reducing stress and ambient temperature. The level separation requirement, currently 400 ft, would appear to be the governing factor.

The logic of wider rooms having more stable roofs than narrower rooms escapes me. The fact that most salt mines employ room widths of less than 100 ft must have a basis (Table II). No miner enjoys the prospect of roof collapse and if faced with the reality will attempt almost anything to avoid another roof fall. I include widening the rooms and introducing yield pillars in the alternatives to be tried. A zone of vertical (radial) tension must occur above any flat-roofed opening in rock. This zone extends to a greater height above a wider room. In jointed hard rock this results in arched backs. Arched roofs are mined to produce a more stable roof. In many cases if they are not mined that way the arch forms naturally, i.e. it falls out.

I appreciated the opportunity to participate in the meeting. WIPP has to produce the safest salt mine ever.

Sincerely,

John F. Abel, Jr.

JFA/mr

Att.

TABLE I. Creep constants for salt.

$\epsilon = CT^a \bar{\sigma}^b t^c$

ϵ = Cumulative strain at time - t (in./in.)

T = Absolute temperature ($^oK-273^oK = 0^oC = 32^oF$)

$\bar{\sigma}$ = Average pillar stress (psi)

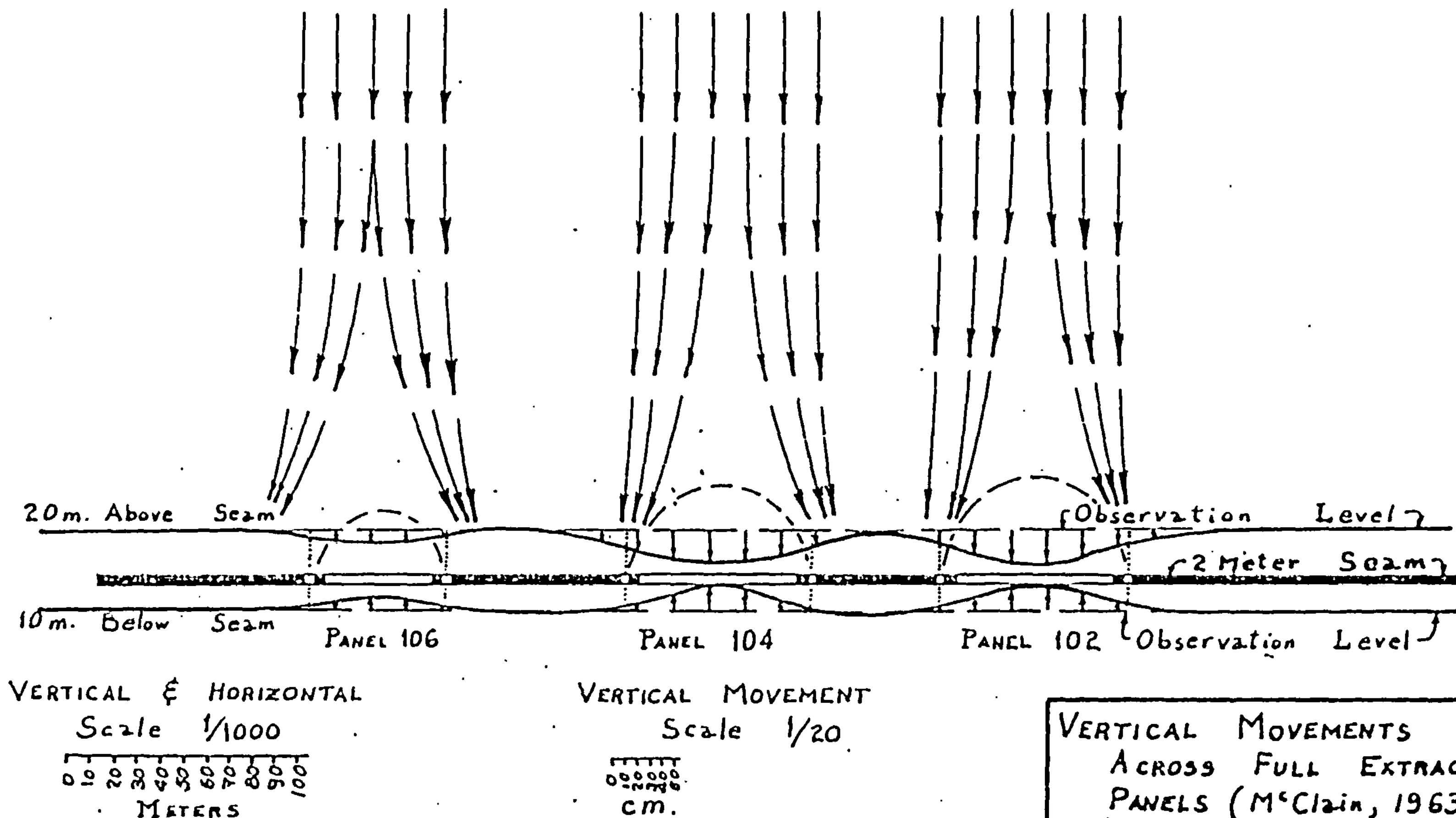
t = Elapsed time (hr)

Constant	Lomenick (1968)	Starfield & McClain (1973)	Hardy & St. John (1977)	Headley (1967)	Obert (1965)	Bradshaw, Boegley & Empson (1964)
C	$1.3x10^{-37}$	$1.3x10^{-37}$	$0.65x10^{-36}$			
a	9.5	9.5	9.5			
b	3.0	3.0	3.0	2.7	(3.0 KA salt (3.1 Ml salt (3.3 NM potash	3.1
c	0.3	0.3	0.4			0.4

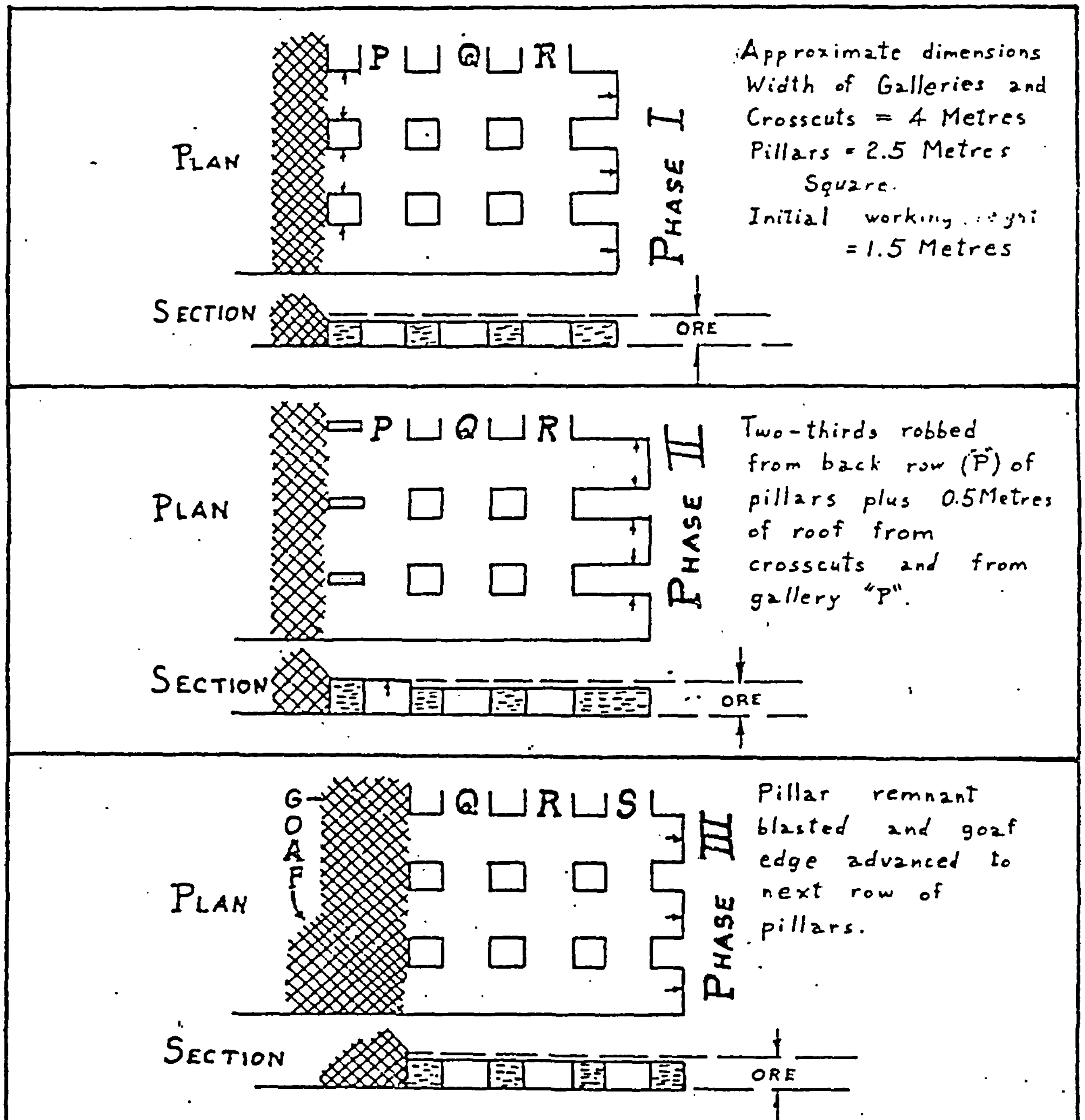
Table II. Indicated factors of safety f

Mine Identification and Type		Product	Depth (ft)	Pillar Width (ft)	Pillar Lengt (ft)
1977	Cote Blanche - Dome	Salt	1290	100	100
1977	Belle Isle - Dome	Salt	1200	40	---
1964	Winsford - Bedded England	Salt	480	90	90
				100	100
1972			580	65	65
1974	Dravo (1)	Salt	1980	80	130
	Dravo (2)	Salt	1300	110	110
	Dravo (3)	Salt	1060	60	60
1971	Headley, Canada - Bedded	Salt	1760	210	210
				150	150
1970	Hutchinson - Bedded	Salt	1024	50	---
				50	50
				50	50
				40	40
1970	Goderich, Canada - Bedded	Salt	1760	200	200
1974	Dravo (4)	Evapor.	1000	60	60
	Dravo (5)	Evapor.	1070	42	42
	Dravo (6)	Evapor.	800	25	25
	Dravo (7)	Evapor.	3140	126	4000
1965	Barr, Germany - Bedded	Potash	2690	23	820
1971	Barr, Canada - Bedded	Potash	3140	54	---
1973	Esterhazy - Bedded	Potash	3150	90	---
~~1958~~	~~U.S.~~ Potash - Bedded	Potash	1000	58	58

Notes: (1) Design strength based on $\gamma = 135$ lb/ft^3; ¢
(2) TAL = Tributary area load --- halfway to a
(3) Pillar deterioration indicated.
(4) Long rib pillars of unspecified length.



Vertical Movements Across Full Extraction Panels (McClain, 1963)



Working Cycle – Retreat Panel
(McClain, 1963)

Use of the Pressure Arch in Mine Design at White Pine

ABEL J.F.

by Gonzalo Barrientos and Jack Parker

The most realistic mine pillar design is based on observations and measurements in situ. Low-cost reliable instrumentation has made this approach possible. At the White Pine mine, with its thousands of pillars of all shapes and sizes under all kinds of loading conditions, practical pillar design criteria have been developed. The pressure arch theory is explained and work in two test panels of the mine is described. The pressure arch mechanism, which relies on lateral stress for support, can be applied to puzzling roof and pillar conditions in other mines.

The art of pillar design has for many years depended upon assumptions that the pillars must support the weight of the overlying rock, and that the load must not exceed the measured strength of the rock. Because of unknowns and uncertainties, and because some case histories do not fit the theory, it is customary to incorporate large safety factors into pillar design. Safety factors of 2 to 4 are, of course, admissions of gross ignorance, and the practice of measuring rock strength in the laboratory (perhaps 20,000 psi in compression) and then using a "design" strength one-quarter of that (5000 psi), is highly questionable.

A better understanding of pillar behavior is obtained by working with larger specimens, by investigating the effect of height:width ratio, and by measuring rock behavior under mine conditions, rather than under warm, dry laboratory conditions, however, the most realistic design will be based upon observations and measurements of real pillars in real mines. Low-cost, reliable instrumentation has made this approach possible, and at the White Pine mine—with its thousands of pillars of all shapes and sizes under all kinds of loading conditions—practical pillar design criteria have been developed.

About 22,000 tons of ore are mined underground each day by trackless room-and-pillar methods. The geology of the ore body has been described in detail by Ensign et al.[1]

This paper describes how simple observations and measurements were used to define the limits of the "pressure arch," a concept wherein pillars do not have to carry the total weight of the overburden, but can be designed to shed load onto special "abutment" pillars. The concept has important implications for ground control and mine design.

The Pressure Arch

The principle of the pressure arch is simple, as shown in Fig. 1.

The existence of the arch has long been recognized under various names: arch, Voussoir arch, beam and dome. Evans,[2] describing British coal mine experience, observed that certain layers of rock would stand unsupported over much wider spans than beam-theory would predict. He also described "yield-pillar" techniques used in coal mines, whereby roof conditions could be improved by allowing the pillars to yield, provided that the zone of yielding did not exceed some critical width.

Wardell[3] has described how the pressure arch concept allows high extraction ratios at great depth—within panels of limited width, and how surface subsidence can be controlled by limiting the width of the caves underground.

Coates[4] formulated a pillar design approach which recognizes that pillar load will depend upon width of the area mined and upon distance from the edge of the area mined.

Other intriguing evidence comes from descriptions of natural caverns, unbelievably wide openings in some mines, deteriorated pillars which should have collapsed—but did not—and pillars which appeared to be sound but which collapsed suddenly after a critical width of mining was exceeded. Information of this kind accumulated at White Pine. In the early efforts in rock mechanics, for example, beginning in 1964, it was observed with concern that some pillars were falling apart, and failure was predicted. The pillars were measured and the

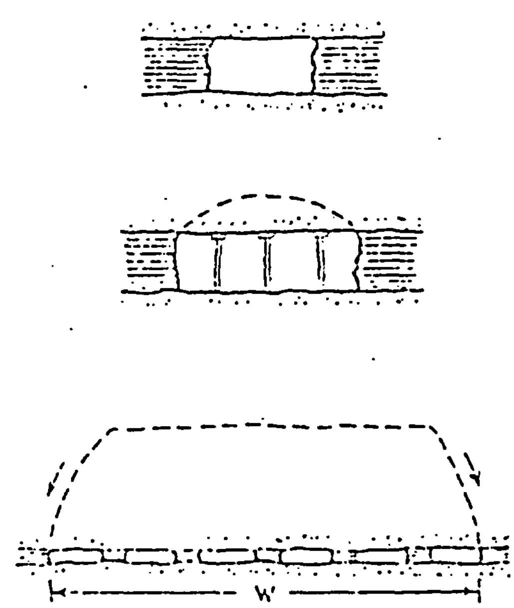


Fig. 1—Principle of the pressure arch. A, top: A narrow opening will stand without support. B, center: A wider opening may need some support, but the full weight of overburden need not be supported. C, bottom: Within some critical width, W, small pillars can yield and shed most of the weight of the overburden onto adjacent abutments, thus forming a pressure arch.

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probable load was calculated, using tributary area theory. That load was found to be much higher than the design strength. The pillars were cracked throughout and it seemed that they could support very little load, yet, despite the predictions, the roof did not collapse. It was puzzling.

The first good clues came from convergence measurements. Points were installed in and around the zones of failing pillars, and they were checked regularly (using a Reed-type Invar extensometer). Convergence was slow but continuous, and the activity slowly spread, then suddenly, when the width of the active zone reached 300-400 ft, convergence rates would accelerate and failure would follow, unless the area was quickly backfilled.[7]

Similar evidence was gathered from pillar-robbing operations. It was found that a major collapse could not be induced even if both roof and pillars were blasted, unless the area affected was about 300 ft wide.

It became possible to document many of these underground failures, some not planned but most induced by pillar robbing. It became clear that, within some critical width, pillars could fail without there being a major collapse, and that this critical width increased slightly with depth. This is the direct evidence of arching which was used to construct Fig. 2.

Significance of the Pressure Arch

The ore body at White Pine is a blanket like, sedimentary deposit. It outcrops and dips gently to the northeast, and it has been developed to a depth of about 2000 ft. It has been proven at a depth of 3400 ft and may go deeper yet. Near surface it is not difficult to attain 70-80% extraction, but complications arise at greater depths. Pillar strength by itself is not a major problem, because beyond certain dimensions (based on height:width ratio) pillar strength is practically infinite, but when depth and extraction combine to give a pillar load around 4500 psi, the pillars then tend to punch into the roof, necessitating much roof repair. Strangely enough, the roof around robbed, failing pillars often looked better than it did in the original headings.

The evidence from the pillar failures suggested that it might be possible to get around the deep-mining problem, that it might be possible to mine at great depth, with high-efficiency room-and-pillar equipment and with good roof conditions, provided that pillars were designed to yield. Mining would have to be confined to narrow panels, separated by load-bearing abutment pillars; otherwise widespread collapse could be expected.

Measurements of surface subsidence over caving operations had shown a relationship between depth of mining, width of cave, and amount and rate of subsidence. There were indications that subsidence could be controlled, that there need be no excessive or violent subsidence, and no open cracks and water ingress, provided that the underground workings were correctly designed.

White Pine Findings Compared with Other Evidence

An effort was made to relate the direct evidence of arching at White Pine with experiences in British and German coal mines and French iron mines, with doming theory and with a clamped-beam analysis (Denkhaus). As shown in Fig. 3, there is some agreement, but not enough for further design work at White Pine, so a decision was made to mine some full-scale test panels and to instrument them.

The Test Panels

Two experimental panels were mined, beginning July 1966, to test the theory of the pressure arch and especially to define the maximum width of the arch. Maximum width is a most important factor: if a panel were mined too wide it would collapse; if a panel were too narrow the mining would be constricted and inefficient. The layout is shown in Fig. 4. The west panel

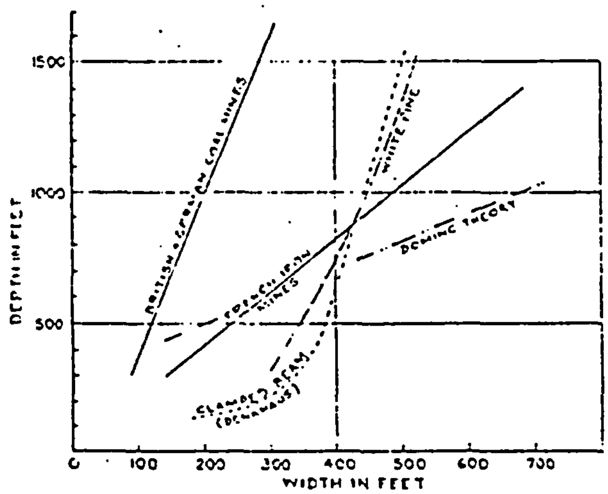


Fig. 3—The pressure arch, theory and experience.

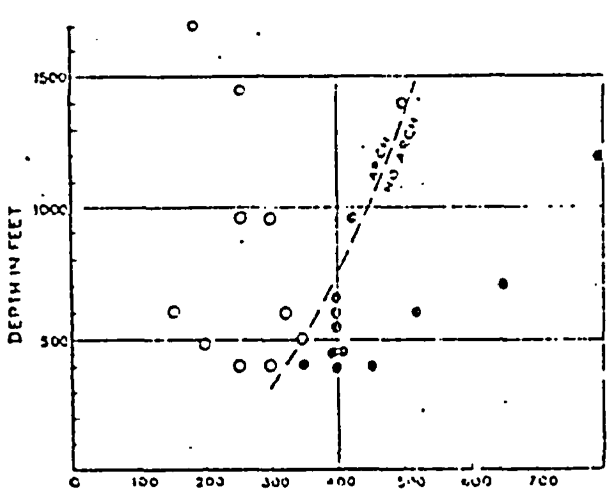


Fig. 2—Evidence of the pressure arch at White Pine. Filled circles, underground collapse which appeared at surface, open circles, underground collapse which did not appear at surface

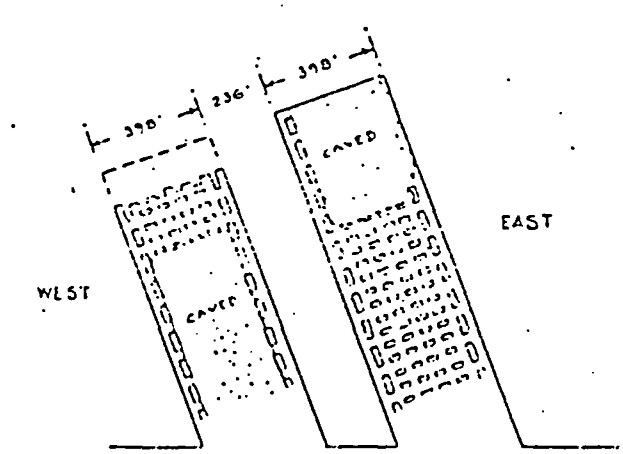


Fig. 4—Layout of the test panels. Depth from surface, 550-650 ft. Main headings, 28 ft wide. Crosscuts, 32 ft wide. Pillar height, 6½ ft. Pillars within the panels, 14 x 38 ft. Extraction within the panels, 82%. Pillar strength, 8700 psi. Pillar load, 4350 psi. Nominal pillar safety factor, 2.0.

was to be caved as the mining front advanced; the east panel was to be caved on the retreat; and the center abutment was also to be retreated after completion of the outer panels.

The panels were aligned to accommodate local geology and stresses; spans were designed according to experience and to suit the mining equipment and pillars were designed according to depth from surface, percent extraction, and pillar strength, which was deduced from a White Pine adaptation (see Fig. 4) of the height-width theory by Holland.*

Note: The observed and measured performances of pillars of various heights and widths, under various loads, led to the conclusion that the strength of pillars at White Pine could be approximated by the formula

$$\text{Strength} = 10{,}000 \text{ psi} \frac{\text{Width in In.}}{\text{Height in In.}}$$

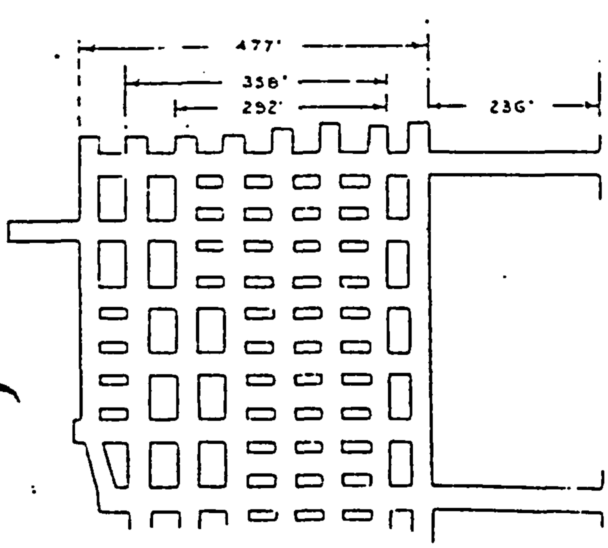
Where width is less than height, the pillars has essentially no strength, and where width is about seven times height, the strength is essentially infinite.

No Pressure Arch in the First Panel-100 Feet Wide: Stress measurements were made in a row of pillars across the panels after the advance mining had been completed. The problems associated with stress measurement and borehole deformation gages were known and allowed for, but there was no evidence of arching. There were the conclusions:

1) Pillar pressures were higher than expected.
2) The "abutment" pillars were subjected to no abutment loading.
3) No pressure arch was induced.

It was assumed that the panel must be too wide, causing the small pillars to bear the full weight of the overburden, so the layout of the second panel was modified. The width would initially be only 226 ft, but it could be increased in 66-ft increments to 292, 358, and 424 ft., as shown in Fig. 5.

No Pressure Arch in the Second Panel: Stress measurements were made in a row of pillars after the second panel had advanced far enough. Again the evidence indicated no pressure arch. Although the orientation of principal stresses showed redistribution of load toward the abutments, the load on the intermediate pillars was some 36% higher than tributary-area theory would predict. There was no significant overload on the abutments, as shown in Fig. 6.



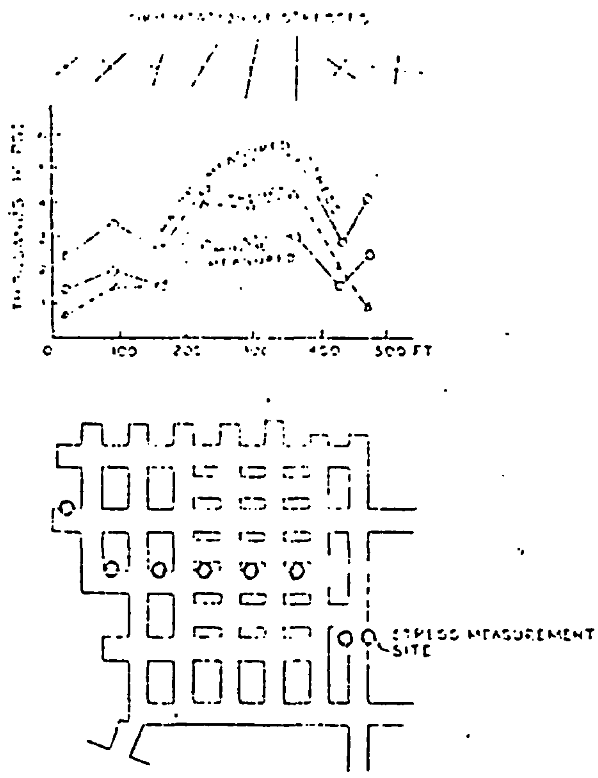


Fig. 6—Stress profile across second panel.

A review of the facts found in the test panels and in the rest of the mine brought out the design fault—the pillars within the panels, although only 14 ft wide, were not yielding. They were not shedding load into the abutment pillars; therefore no arch formed. Plans were then made to reduce the size of the pillars until they did yield, and to measure their behavior.

Inducing Pillar Yield: Twelve of the pillars were drilled so that they could be blasted and destroyed in stages, as shown in Fig. 7. All 12 pillars were instrumented, as shown in Fig. 8, to give stress:strain information. Stress increase was to be measured with Horstman glass photoelastic stressmeters cemented in holes in the pillars; strain was to be measured at convergence stations close to the pillars.

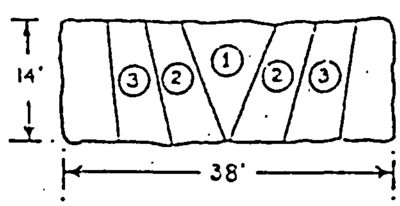


Fig. 7—Pillar destruction sequence.

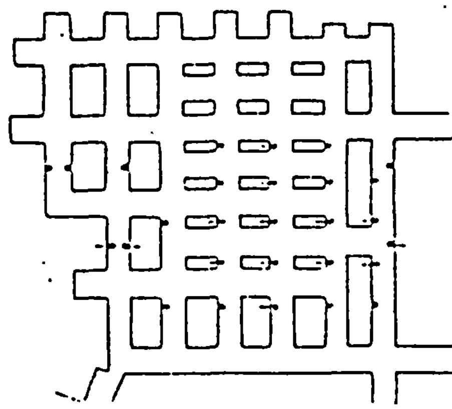


Fig. 8—Instrumentation of the pillar-yield test. Dot outside block, convergence point; dash in block, stressmeter.

The Pillars Did Yield, and a Pressure Arch Did Form: After each segment of the pillars had been blasted, the remaining stubs were measured and the theoretical change in load was calculated. This was compared with the change in load shown by the stressmeters. It was found that the actual load increase was much less than the tributary-area theory would predict, meaning that the load had shifted elsewhere. Fig. 9 shows the predicted and the measured load changes.

Modulus of Deformation of the Yielding Pillars: Some instruments did not survive the blasts, and [illegible] the stressmeters crushed at about 2000 psi, but they indicated that the effective modulus of the pillars was less then half of the value measured on small, intact rock specimens in the laboratory. In the small pillars the modulus varied from 1.3 to 3 million psi, whereas laboratory tests gave a value of about 6 million psi. These results indicate that some caution is needed when assigning values for calculation purposes. Fig. 10 shows some of the measured stress:strain relationships.

An Area 226 Ft Sq Stood Unsupported: After nine of the pillars had been completely destroyed, there was still no major collapse, only a few local falls of roof. The roof was blasted in an attempt to get a cave started, but only 7 ft of rock came down—up to the roof bolt anchor horizon. There was then an unsupported span of thinly laminated shale about 226 ft sq, Fig. 11. Obviously there was some mechanism at work other than the gravity-loaded beam; something else had to be supporting that 500 ft of overburden.

Arching as the Mining Front Advanced: In the second panel the pillars were robbed as the mining front advanced, with only two or three crosscuts between the cave and the mining front so that equipment could easily be trammed from one job to the other. The roof usually collapsed as soon as a row of pillars was destroyed, and the collapse was predictable. Convergence points were installed at every pillar, and convergence was measured at least once a day, both for cave-control and research purposes. The network of convergence stations is shown in Fig. 12.

Convergence profiles and stress-change profiles both indicated that an arch was formed across the panel as the cave advanced, with the weight of the rock above the cave being transferred to the abutment pillars, Fig. 13. Chain pillars on the east side were only 25 ft wide; they did not offer as much resistance as the 38-ft-wide pillars on the west side, and they allowed the abutment load to extend much further.

Convergence profiles along the length of the panel also showed an arch forming, Fig. 14. Note that the total convergence (hence pressure) on the small front abutment pillars increased as the cave grew longer, until the cave was about 400 ft long. At that time there was much deep-seated rumbling and cracking above the cave, as if the cave were working higher; then the front-abutment load decreased. Apparently a critical width was exceeded at about 400 ft.

Surface Subsidence as an Indication of Pressure Arches: The best evidence of arch formation is visual observation in the mine. Surface subsidence measurements are almost as conclusive, so a line of points was installed over the cave-to-be, leveled and checked before mining began, then checked after each major collapse underground. When the cave was 226 ft wide, there was no subsidence. The width of the cave was then increased by mining and destroying additional rows of pillars. With a width of 292 ft, there was still no subsidence; with a width of 358 ft there was 4 cm of subsidence, and when the cave was 424 ft wide, there was a total subsidence of 5 cm. Apparently the overlying rocks were bending, but they did not collapse. It should be noted that at this stage the other plan dimension of the cave, 303 ft, was now the controlling factor, which is probably why the increase in width from 358 to 424 ft had

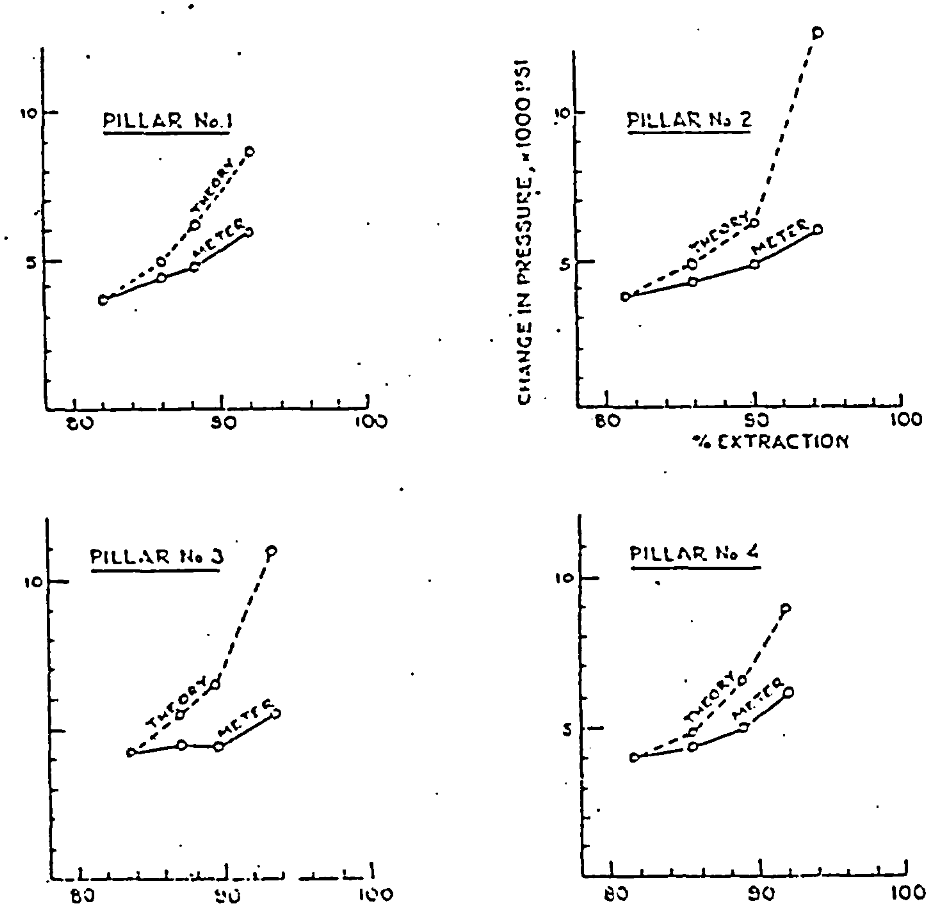


Fig. 9—Stress changes in the yielding pillars.

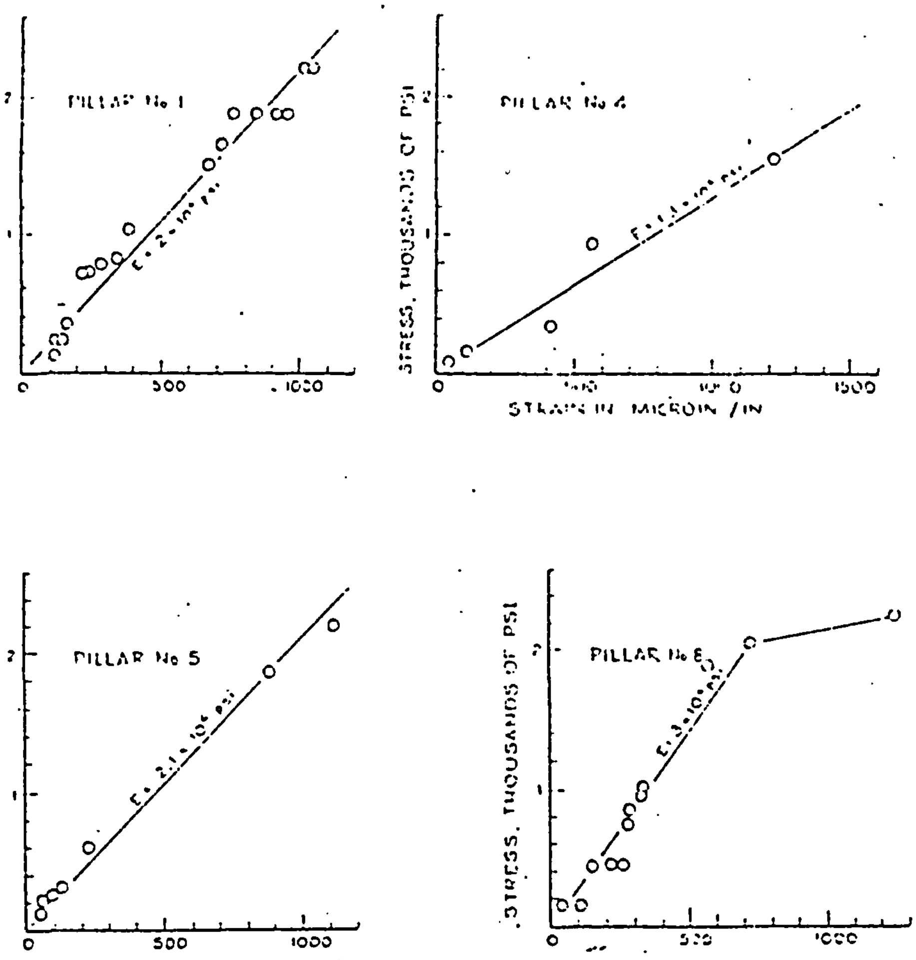


Fig. 16 Stress-strain relationships in the small pillars

so little effect. The caves and the surface profiles are shown in Fig. 15.

Conclusions From the Test Panels:

1) A pressure arch can be formed.

2) The maximum width of the arch at a depth of 550 ft was at least 308 ft.

3) Pillars within the panel *must* be small enough to yield, or no arch will form.

4) Load was tranferred to abutment pillars.

5) Distribution of load could be arranged through design of pillar stiffness.

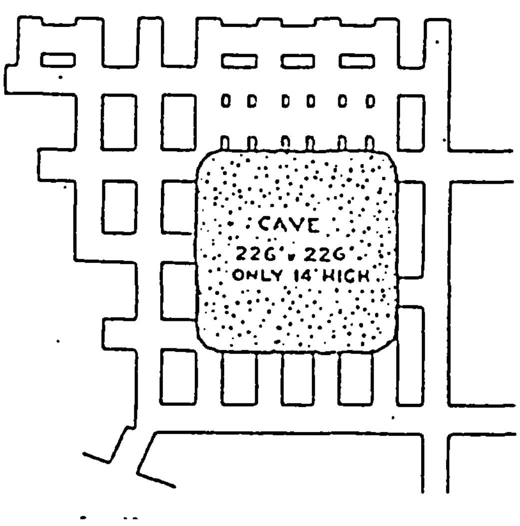
6) High extraction (in this case 92%) can be attained within the narrow panel.

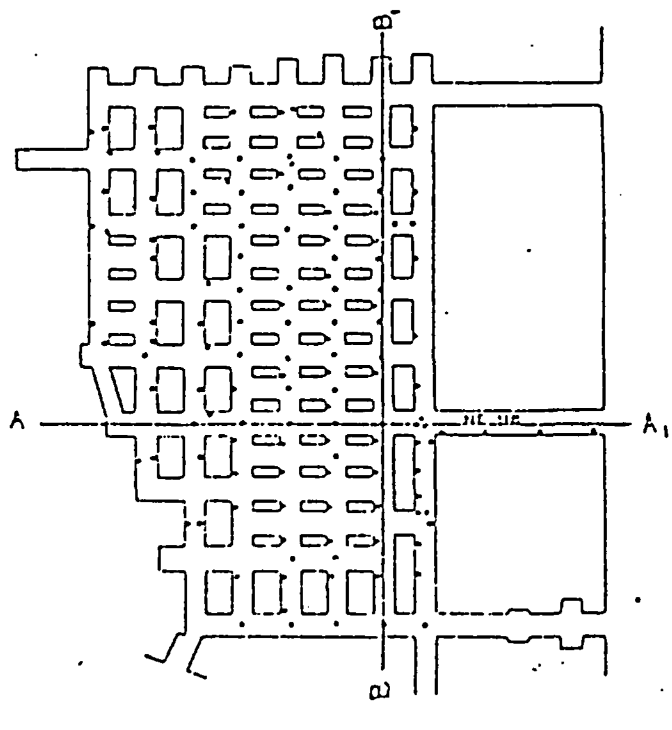
7) Effective modulus of deformation of small pillars was less than half of the laboratory-measured value.

Explaining the Pressure Arch

Experience with roof design at White Pine could be extrapolated to explain the pressure arch.

Although small, intact rock specimens tested in the laboratory exhibit some tensile strength, it is obvious in the mine that the rock mass, cut by numerous joints and faults, cannot exert any significant resistance to tensile stress. On the other hand, there were many





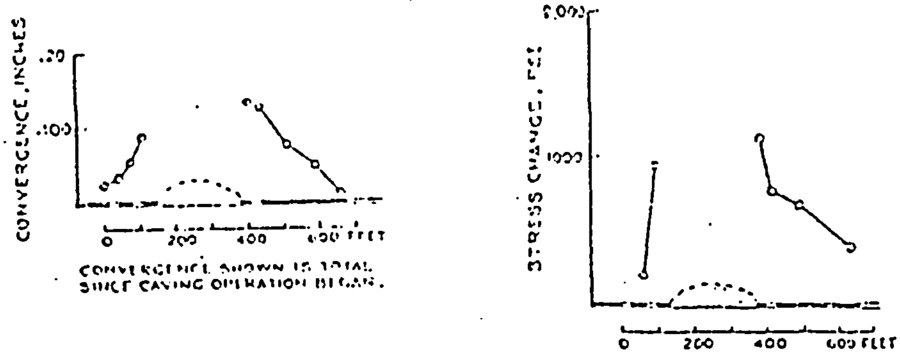


Fig. 13—Convergence and stress-change profiles across the cave, both showing arch abutment loading. These profiles are on Section A-A, Fig. 12.

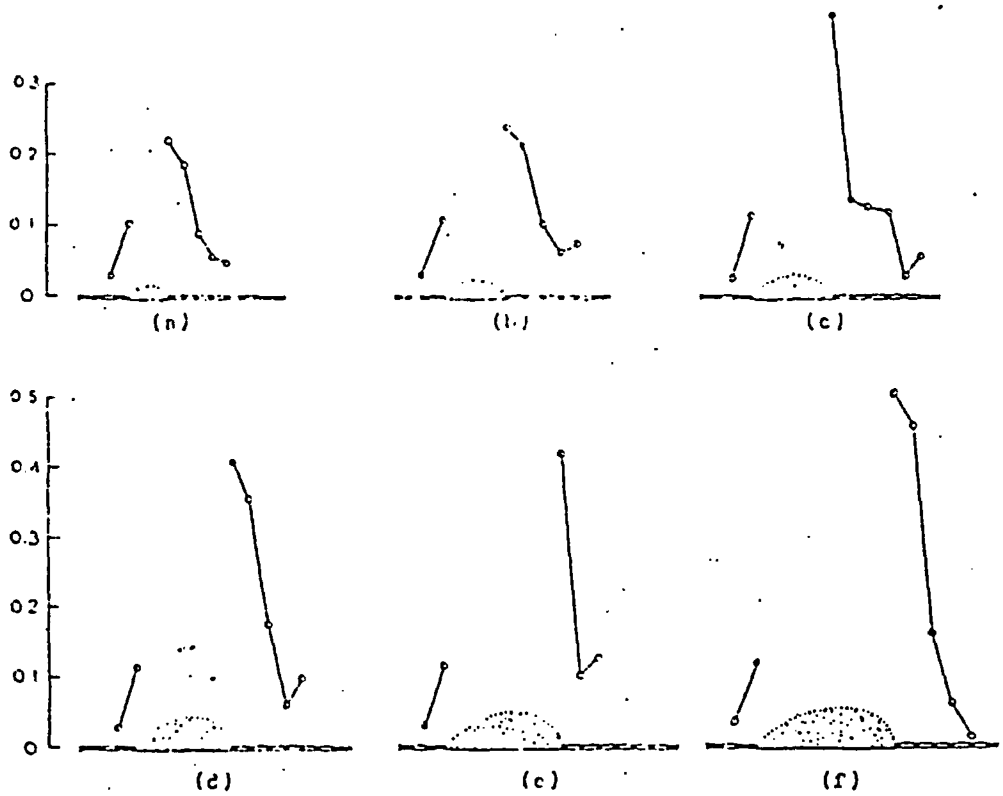
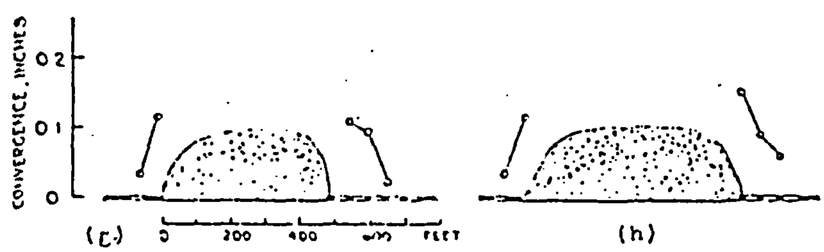


Fig. 14—Front and rear abutment loads as shown by convergence measurements. These profiles are on section B-B₁, Fig. 12.

At this stage, when the cave was between 400 and 500 feet long, there was much deep-seated rumbling above the cave, as if it were working higher.

Notice that the front abutment load then decreased.



measurements and observations of the effects of horizontal compression, and there can be little doubt that it is these lateral compressive forces which hold the jointed rockmass in place in the roof. Crude measurements show that the coefficient of friction on the joint plane is around 0.8 and it can be shown that a lateral stress of 3 psi will support a 1-in. square beam of White Pine rock 8 in. long.

It follows that the weight of the overburden above a cave, or above yielding pillars can be supported from the sides if the lateral stress is great enough, and if the weight of the rock is not too great. The weight of the rock mass, per square inch of side-support area, depends upon the span. Therefore for each lateral stress condition there will be a critical span, or width of arch.

The lateral stress will be made up of two parts: a primitive stress and the Poisson component of the vertical stress. The primitive stress will exist close to surface and will probably not change much with depth, but the Poisson component of vertical stress *will* change with depth. Therefore, the width of the pressure arch can be expected to increase with depth. Fig. 16 illustrates this explanation.

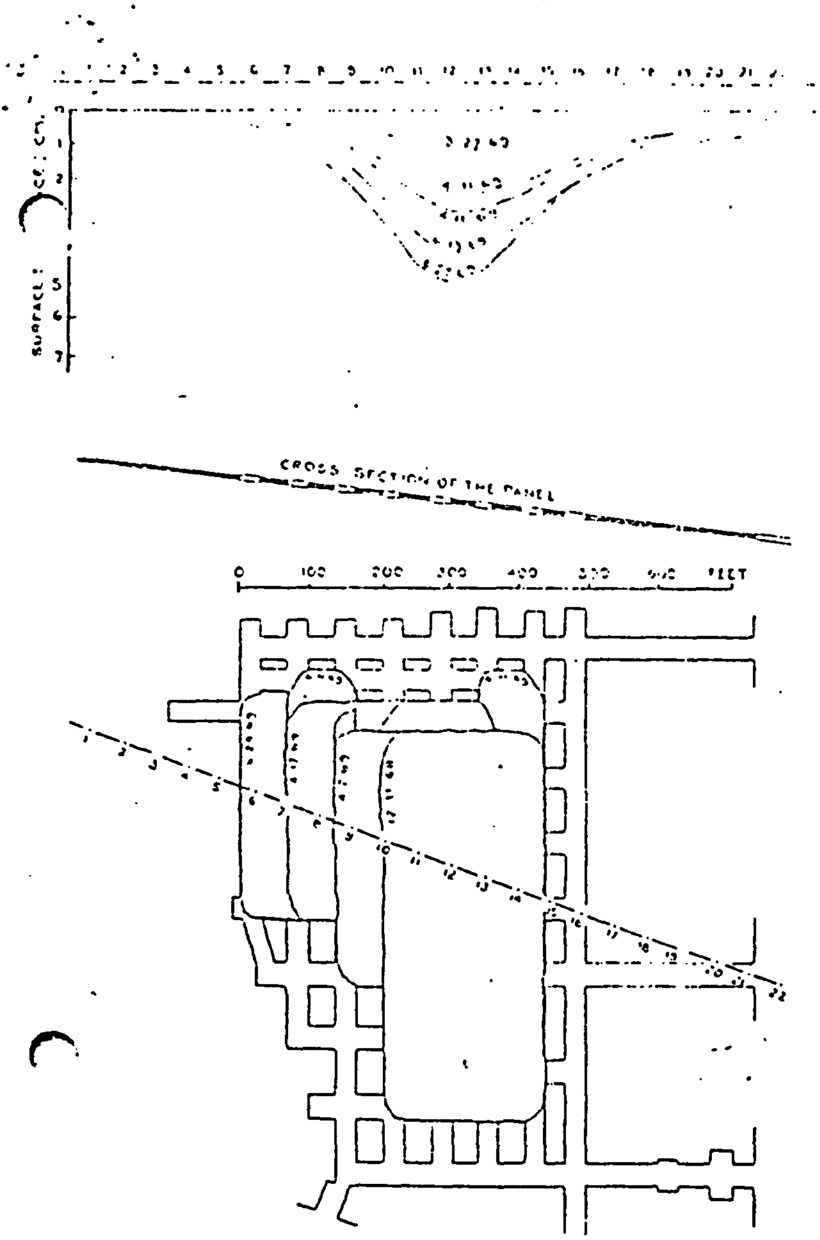


Fig. 15—Surface subsidence after successive caves in the panel.

Designing Yield Pillars and Stiff Pillars

The important pillar dimensions are height and least width. Something can be learned in the laboratory by testing specimens with various height:width ratios, but the best information will come from the mines. At White Pine, for example, laboratory tests show an unconfined, compressive strength between 20,000 and 30,000 psi. However, more significant facts come from the mine, where observations and measurements show that under heavy load a pillar with width less than two times height will yield; a pillar with width around seven times height will be infinitely stiff and strong; and between these limits there is a predictable degree of stiffness. This understanding permits unsophisticated yet simple and reliable design of yielding, abutment, and intermediate pillars. Fig. 17 illustrates pillar behavior. This behavior will, of course, by modified locally by variations in fault and joint density.

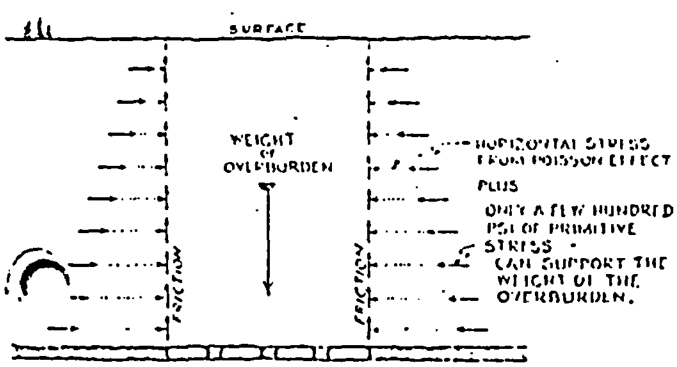


Fig. 16—Weight of overburden supported by lateral stress and friction.

Using the Pressure Arch

An understanding of this mechanism of ground support, and its limitations, gives new solutions to ground-control problems.

Pillar load depends in part upon the width of the area mined, and this width can now be manipulated to benefit the mine operator, to prevent a collapse or to induce a cave when he wants one.

Roof conditions deteriorate as depth increases; they can now be improved by using pillars which yield slightly, but the width of the yielding zone must be

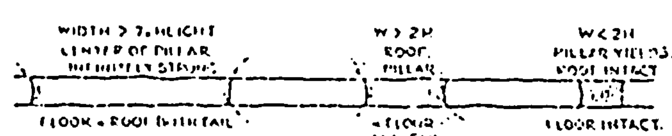


Fig. 17—Behavior of pillars under heavy load, showing the effect of height-width ratio.

limited so that the weight of the overlying rock will be supported on stiff abutment pillars. The overall amount of the ore yield recovered can be increased at depth by using such narrow panels.

Pillar robbing is better understood and results improve, because it is now known that in zones of limited width the extraction percentage can go into the 90s, with little danger of major collapse. It is also understood that blasting the roof to get a major collapse started is a waste of effort, unless the [illegible] of the zone is close to critical.

Surface subsidence can now be controlled by managing the underground cave activity. If the width of the cave is less than the width of the pressure arch, there will be practically no subsidence. If a critical width collapses in the mine, there will be pluglike failure to surface, with a large amount of subsidence and steps and open cracks at surface. Surface effects can be diminished by leaving some stiff pillars within the cave, less than the critical distance apart, so that multiple arches will form.

It is believed that this pressure arch mechanism, relying upon lateral stress for support, can explain many of the puzzling roof and pillar conditions in other mines, and that the understanding can be applied to improve safety and mining efficiency.

Acknowledgments

Assistance from the other members of the White Pine Rock Mechanics group, and permission from the parent Copper Range Co. to present and publish this paper are gratefully acknowledged.

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Significance of Mixed Potentials in Eh Measurements with Platinum Electrodes

by K. A. Natarajan and I. Iwasaki

The influence of dissolved oxygen and the Pt — Pt-O reaction on the measurement of redox potentials in solutions containing ferrous-ferric couple was examined. Current-potential curves were used to illustrate the shift in measured potentials toward the oxygen potential in oxygenated and deoxygenated systems at low concentrations of ferrous and ferric ions. The ranges of concentrations of the ferrous-ferric couple, in which the interference from oxygen and the platinum-oxygen reactions becomes significant, are illustrated through the mixed potential mechanism.

Redox potential (Eh) measurements are often used in the metallurgical industry to indicate oxygen levels and concentration ratios of redox couples, for example, ferrous and ferric ions. Unless the redox species sought are present in sufficient concentrations, the measured potential may be influenced by the presence of oxygen. Such an interaction, referred to as "mixed potential," presents another problem in redox potential measurements in addition to electrode poisoning.[1] This is because platinum electrodes are not strictly inert and, depending on the pH, Pt — Pt-O and Pt-Pt(OH) type reactions are possible at the electrode surface.[2]

A proper understanding of the role of oxygen, the electrode reactions involved, and the concentration range of the redox species in minimizing their interactions with oxygen, therefore, becomes important in interpreting measured potentials in metallurgical systems. In this paper, these points in redox potential measurements will be examined by taking the ferrous-ferric system in the presence and in the absence of dissolved oxygen as an example. The term "mixed potential" will be explained and its significance portrayed through polarization diagrams.

K. A. NATARAJAN and I. IWASAKI, Member SME, are Research Assistant and Professor, respectively, Mineral Resources Research Center, University of Minnesota, Minneapolis. TP 428215. Manuscript, Feb. 10, 1972. Discussion of this paper, submitted in duplicate prior to June 15, 1974, will appear in SME Transactions, September 1974, and in AIME Transactions, 1974, Vol. 256.

Experimental Procedure and Results

Experimental setups and procedures for redox potential measurements and for tracing current-potential curves have been described elsewhere.[1,3]

Fig. 1 shows the potential readings as a function of time in a deoxygenated 1 M sulfuric acid containing ferrous and ferric sulfates in concentrations ranging from 10^{-2} and 10^{-6} M and in the concentration ratio of unity. As evident in the figure, there was no significant change in the potential readings with time at 10^{-2} M. As the concentration of the iron ions decreased, the magnitude of the potential change with time increased. A similar set of results was also obtained in oxygenated solutions.

If a potential reading is that of a mixed potential there will be a shift in the reading upon rotating the electrode at steady state[4] due presumably to the disturbance of concentration gradient at the electrode sur-