Department of Energy
Albuquerque Operations Office
Waste Isolation Pilot Plant Project Office
P. O. Box 3090
Carlsbad, New Mexico 88221

JUN 14 1991

Ms. Judith M. Espinosa, Secretary
New Mexico Environment Department
P.O. Box 26110
Santa Fe, NM 87502

Dear Ms. Espinosa:

Enclosed is a copy of the "Report of the Geotechnical Panel on the Effective Life of Rooms in Panel 1," June 1991, DOE/WIPP91-023. The report was requested by the New Mexico Environment Department (NMED) in your May 9, 1991 letter. The ten draft reports of the expert panel members reviewing the stability of Panel 1 in the WIPP underground were provided to the NMED with a May 24, 1991 letter. The final versions of these reports are included in the enclosed report.

If you have any questions or comments regarding this matter, please contact Jerry Carr of my staff.

Sincerely,

for Arlen Hunt
Project Manager

Enclosure

cc w/enclosure:
C&C File (ATS910036)

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Report of the Geotechnical Panel on the Effective Life of Rooms in Panel 1

June 1991

Waste Isolation Pilot Plant

691B:0438

DOE/WIPP 91-023

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This document is issued by Westinghouse Electric Corporation, Waste Isolation Division, as the Managing and Operating Contractor for the Department of Energy, Waste Isolation Pilot Plant, Carlsbad, New Mexico, 88221 .

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REPORT OF THE GEOTECHNICAL PANEL ON THE EFFECTIVE LIFE OF ROOMS IN PANEL 1

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List of Acronyms

CH - contact-handled
EEG - Environmental Evaluation Group
SPDV - Site and Preliminary Design Validation
TRU - transuranic
WID - Waste Isolation Division
WIPP - Waste Isolation Pilot Plant

EXECUTIVE SUMMARY

An evaluation of the effective life of underground rooms in Panel 1 of the waste storage area of the Waste Isolation Pilot Plant (WIPP) was performed during April 1991 by a panel of geotechnical experts. The evaluation addressed concerns regarding WIPP's ability to complete a test program proposed for Panel 1. This program currently requires bins containing controlled quantities of contact-handled (CH) transuranic (TRU) radioactive waste to be placed in rooms in the panel. The bins will be monitored to obtain data on the potential generation of gases from the degradation of wastes emplaced in the WIPP underground facility. The purpose of the evaluation was (1) to provide an estimate of the life expectancy of the rooms in Panel 1; and (2) if necessary, to recommend additional remedial actions that would improve the longevity of Panel 1 rooms to allow the testing to be successfully completed.

Panel 1, the first panel to be mined in the waste storage area, was developed to receive waste for a demonstration phase that was scheduled to start in October 1988. Mining of the panel began during the second half of 1986 and was completed to final dimensions in June 1988. The original plan was to store drums of CH TRU waste in rooms for a period of 5 years. The demonstration phase was changed to an experimental program that will use CH TRU waste in bin scale tests which will be located in Panel 1. For the purposes of this report, a nine-year test period beginning July, 1991, was assumed to be necessary to complete these bin scale tests.

The panel members were able to reach positions that were reasonably consistent. They agreed on the qualitative mechanisms identified as the principal causes of the failures found in the roof of excavations in the WIPP underground test areas and established that similar fracture development could be expected in other WIPP underground areas. They concluded that if no additional remedial measures are taken, the rooms in the panel are likely to have a total life from seven to eleven years from the time of excavation using the currently installed roof support system, consisting of rockbolts. They indicated that the rockbolts had some beneficial effects, but agreed that it was not possible to measure their effectiveness. Estimates made by individual panel members of room life extension due to the bolting varied from a few months to several years. In conclusion, the panel believed that modifications, enhancements, and regular maintenance would be required for the rooms in Panel 1 to perform satisfactorily over the assumed nine-year test period starting July 1991.

The panel indicated that techniques were available that would extend the life of the rooms to varying degrees. They indicated that the rooms were currently stable but added that continuous access into the rooms would probably require remedial measures of some kind during the test period, and these measures should be undertaken. Techniques currently used in mining that would improve conditions were suggested by the panel members and included the following:

- The use of full column resin or resin anchored bolts.
- Grout anchored cables with loops, lace, and mesh covering the roof to contain and control roof rock failure.
- Relief of the lateral stresses to prevent roof and floor failures by slotting and/or relief entries.

- Yielding support.
- Rely on currently installed support and upgrade when necessary, based on the results of the geomechanical monitoring program.
- Roof trusses.
- Drive new rooms through existing pillars in Panel 1.

The panel recommended that the project evaluate these alternatives and determine which would be the most effective for improving ground conditions in the waste storage area for the period of the bin scale tests.

The panel members also stated that the geomechanical monitoring program currently in place at the site was satisfactory and would provide adequate warning of deteriorating conditions in the underground. They did suggest that additional instrumentation should be installed to provide an even stronger monitoring program, and they were satisfied with the revised geomechanical instrumentation proposed by project personnel at the second meeting. Installation of this equipment was initiated in May 1991.

The measures recommended by the panel constitute a series of positive actions that should extend the life of the rooms in the panel to the required total of 14 years. The geomechanical instrumentation program and the understanding derived from the test areas of the facility will be used to alert project personnel to changing conditions to allow the remediation and stabilization activities to be undertaken as needed during the testing program.

In summary, the panel agreed that measures could be taken in Panel 1 that would give a reasonable assurance that the bin scale tests could be carried out to completion. The panel members suggested a number of alternative actions that could be taken. They recommended that the WIPP project evaluate the alternatives and select one, or a combination, of the measures that would assure continued use of the rooms over the period of the tests. They also indicated that these additional measures should be augmented by an enhanced monitoring program that would regularly assess the geomechanical conditions and that maintenance should be carried out as a routine activity in the rooms as they aged.

1.0 INTRODUCTION

An evaluation of the effective life of underground rooms in Panel 1 of the waste storage area of the WIPP was performed during April 1991 by a panel of geotechnical experts. The evaluation addressed concerns regarding WIPP's ability to complete a test program proposed for Panel 1. This program currently requires bins containing controlled quantities of CH TRU radioactive waste to be placed in the rooms. The bins will be monitored to obtain data on the potential generation of gases from the degradation of wastes emplaced in the WIPP underground facility. The purpose of the evaluation was (1) to provide an estimate of the life expectancy of the rooms in Panel 1; and (2) if necessary, to recommend additional actions that would improve the longevity of Panel 1 rooms so that the testing could be successfully completed.

The Waste Isolation Division (WID) formed a panel of experts to provide an independent assessment of the projected useful life of rooms in Panel 1 at WIPP and to provide advice on ground control measures. This group of eleven experts made a preliminary assessment of the stability of the Panel 1 rooms, especially Room 1. This report describes the process by which the panel of geotechnical consultants arrived at an evaluation of life expectancy of the rooms in Panel 1 and presents the findings of the panel.

The panel met twice as a group. The first meeting took place on April 9 - 10, 1991, in Carlsbad, New Mexico. At this meeting, geotechnical information was presented to the panel by project personnel, and panel members toured the WIPP underground. The panel members were then given seven days to review the information and submit a draft report based on a series of prepared statements provided to them. The panel reconvened in Carlsbad on April 23 - 24, 1991, at which time the individual panel members made presentations that summarized their views. At the conclusion of the meeting, a consensus was reached, which is included in this report.

The panel members concluded that if no additional remedial measures are taken, the rooms in the panel are likely to have a total life of seven to eleven years from the time of excavation using the currently installed roof support system, consisting of rockbolts. Mining of Room 1, Panel 1 began during the second half of 1986. Therefore, the remaining life of this room is anticipated to be between two and six years. However, the panel agreed that measures could be taken in Panel 1 that would give a reasonable assurance that the bin scale tests could be carried out to completion. The panel members suggested a number of alternative actions that could be taken. They recommended that the WIPP project evaluate the alternatives and select one, or a combination, of the measures that would assure continued use of the rooms over the period of the tests. They also indicated that the measures should be augmented by a monitoring program that would regularly assess the geomechanical conditions and that maintenance should be carried out as a routine activity in the rooms as they aged.

1.1 CHARTER FOR THE PANEL ON GEOTECHNICAL STABILITY OF PANEL ONE

Prior to the selection of the geotechnical experts to evaluate the stability of Panel 1, a charter was established that defined (1) the scope of work for the geotechnical panel; and (2) the tasks that were to be accomplished. The charter is as follows:

Purpose

The purpose of the Panel on Geotechnical Stability is to establish a position regarding the anticipated useful life of the rooms in Panel 1 of the waste storage area.

Scope

The scope of the activities for the panel is the review of current and historical geotechnical data and observations from the WIPP underground. Based on this review the requirements for maintaining the Panel 1 rooms will be evaluated to enable the successful completion of the Bin Scale Test Program.

Document Review

The panel members will review existing documentation of the geomechanical performance of the WIPP underground openings. This documentation will be made available prior to the site visit.

Underground Evaluation

An inspection of the underground excavations will be conducted in order to familiarize the members of the panel with the existing conditions of the openings, the roof support system currently in use, and the repository stratigraphy.

Questions to be Addressed

The members of the panel will combine the results of the document review and underground evaluation to develop a technical position on the future performance of the waste storage panel. This position will specifically address the following questions:

- What is the useful life span of the storage rooms as they are currently configured?
- Is the current roof support system adequate for the term of the Bin Scale Tests?
- If the current system is not adequate, what type of roof support system should be installed?

These questions were formulated into five statements that were presented to the panel members and were addressed by each panel member.

1.2 BACKGROUND

1.2.1 Panel 1

Panel 1 was the first panel to be mined in the waste storage area. The Panel entry in S1950, Room 1, and parts of Rooms 2 and 3 were excavated during the second half of 1986 and the first 3 months of 1987. Mining restarted in January 1988, and the panel excavation was completed to final dimensions in June 1988.

The original design for the waste storage rooms at the WIPP provided a limited period of time during which to mine the openings and to emplace waste. Each panel, consisting of seven storage rooms, was scheduled to be mined and filled

in less than five years, before being sealed. Field studies, as part of the Site and Preliminary Design Validation (SPDV) Program, showed that unsupported openings of a typical storage room configuration would remain stable and that creep closure would not impact equipment clearances during at least a five year period following excavation. The information from these studies provided the validation of the design of openings for the permanent disposal of waste under routine operations.

Panel 1 was developed to receive waste for a demonstration phase that was scheduled to start in October 1988. Although rockbolt support was installed in Panel 1 in 1988, the rockbolt design was based upon the requirements for the demonstration program in place prior to 1988. The original plan consisted of the storage of drums of CH TRU waste in rooms for a period of 5 years. During this time and immediately following it, the rooms were to be inaccessible, but the option to reenter was to be maintained so that the waste could be removed, if required. To assist with the possible reentry, ten-foot rockbolts were installed in all rooms to enhance roof stability.

The demonstration phase was deferred and an experimental program that uses CH TRU waste in bin scale tests is now planned for Panel 1. The decision to use Room 1 of Panel 1 for these bin scale tests was made in June 1989 and was based on waste receipt in 1990. Further delays to the test program have currently revised the date for waste receipt to July 1991. For planning purposes and this report, on the order of nine additional years of useful life are required for the test rooms in Panel 1. This is the projected time, including a one year allowance to reflect uncertainties, required to initiate, conduct, and retrieve test bins for the bin scale tests. The current test program requires much greater access into the rooms, leading to more stringent requirements for roof stability.

1.2.2 SPDV Test Rooms

A significant part of the basis for this assessment is the geomechanical performance of the four SPDV Test Rooms that were mined in 1983 and additional data gathered from instruments installed in drifts and rooms of Panel 1 itself. The SPDV Test Rooms were instrumented and monitored for rock movement and creep closure over successive years since excavation. This monitoring program validated the use of rooms of this geometric configuration for emplacement of waste in the storage areas.

At eight years after mining, a roof fall occurred in SPDV Test Room 1. Roof deterioration was first observed and commented upon more than two years before this fall. As the excavation aged, the potential for roof collapse in the room was reassessed several times. About fifteen months prior to the failure, an estimate of the size and timing of the fall was made. The size estimate proved reasonably accurate. However, the time of the fall was predicted for the summer of 1990, and the actual fall occurred in February 1991.

SPDV Test Room 4, which was mined at the same time as the remaining test rooms, has not undergone the same degree of deterioration and is still open for daily access. This room is rock bolted and geomechanical conditions are regularly checked. There are no indications that this room will be closed in the immediate future. The differences between the performance of SPDV Test Rooms 1 and 4 indicate the significant variations that can occur in the effective life of rooms excavated in very similar geologic conditions. The differences may be caused by geologic variations across the site or exposure to different stress histories.

2.0 QUALIFICATIONS OF PANEL MEMBERS

A primary consideration in the selection of panel members was to include technical personnel who have hands-on professional experience in, or who have provided consulting services to, evaporite mines at depths in excess of 2000 feet. It was anticipated that these experts would (1) have knowledge and practical experience of the strata movements that develop at the WIPP; and (2) recommend measures that might be used to alleviate deteriorating ground conditions. In addition, experts with a general background in rock mechanics were selected to provide expertise in engineering, geology, and numerical analyses.

The following general criteria were used in selecting the panel members:

- Academic and industrial experience in rock mechanics
- Experience designing and monitoring excavations in deep evaporite strata
- Experience mining in the Carlsbad Potash Basin
- Experience in the investigation or design of roof support systems
- Experience with numerical analyses

The specific qualifications of individuals for their selection as members of the geotechnical panel are as follows:

Dr. George Griswold is an independent consultant based in Albuquerque, New Mexico. Formerly, he was head of the Mining and Geological Engineering Department at the New Mexico Institute of Mining and Technology. He has been involved with the WIPP project since 1977, when he was associated with the initial geological investigations for the site while working for Sandia National Laboratories. As an independent consultant, he has also carried out assignments for the Environmental Evaluation Group (EEG) that has an oversight role for the WIPP Project.

Dr. Ian Farmer is the chairman of Farmer and Partners, a geotechnical engineering company based in Newcastle, England. Formerly, he was head of the Mining and Geological Engineering Department at the University of Arizona. He is the author of several books on rock mechanics and engineering geology and has published over one hundred technical papers in these fields. Dr. Farmer has carried out research projects related to the time dependent constitutive relationships for salt rocks, the mechanical performance of full column resin anchored rockbolts, field instrumentation, and roof support systems.

Mr. Tony Iannacchione is the supervisor of the Rock Mechanics Group at the U.S. Bureau of Mines, Pittsburgh Research Center. He has conducted research on mining related problems for over 16 years and is the author of over 35 technical papers on the subject. Currently, he is responsible for managing research projects concerned with the design and reinforcement of pillars, rock mass characterization, rock burst control, mine-wide monitoring, and rockfill characterization. He has also had considerable experience evaluating gas outbursts within Louisiana and New Mexico salt and potash mines.

Dr. Stephen McKinnon is a geotechnical engineering consultant working for the Itasca Consulting Group, based in Minneapolis, Minnesota. Previously, he was the head of the Mine Design Section at the Chamber of Mines Research Organization in South Africa. While in South Africa, he investigated various mine collapses and made recommendations on remedial actions and monitoring programs to predict field conditions. Dr. McKinnon is presently involved in numerical modelling and field studies for Itasca.

Dr. Hamish Miller is the principal of International Mining Services, Inc., based in Vancouver, Canada. Previously, he was Professor of Mining at the University of British Columbia. His main field of research was concerned with the design and stability of excavations in salt and potash mines. In addition to six years in the deep, hard rock mines in South Africa, Dr. Miller has spend more than 15 years as a consultant to the salt and potash industries in the USA, Britain, and Canada. During this time he analyzed, in detail, field data from nine evaporite mines. Dr. Miller was a member of the peer review panel for the Design Validation Final Report for the WIPP Project prepared by the Architect/Engineer, Bechtel.

Dr. Parviz Mottahed is the head of the Mining Technology Section at the Canada Center for Mineral and Energy Technology, based in Elliot Lake, Canada. Previously, he was the head of the Earth Sciences and Mining Department for the Potash Corporation of Saskatchewan, where he provided technical services in the fields of rock mechanics, geology, and geophysics to four potash mines. He has published over twenty papers in the fields of rock mechanics and mine design in potash and gypsum rocks.

Mr. Jack Parker is the principal of Jack Parker and Associates, based in White Pine, Michigan. His qualifications include 45 years working in and around mines, with the last 20 years as a consultant working primarily on mine design and ground control problems. He has worked in many mines, including 11 salt mines, 2 trona mines, 3 potash mines and 3 gypsum mines. Mr. Parker has published a series of papers describing the practical aspects of rock mechanics for the miner.

Dr. Bill Thompson is a senior scientist specializing in geotechnical problems for SAIC based in Golden, Colorado. Previously, he was an Associate Professor at the University of Texas, Austin. He acted as a consultant to D'Appolonia Consulting Engineers for the WIPP project during the early site investigation phase. He has worked in salt and potash mechanics and mine stability for over 20 years, performed laboratory and field experiments for a gas storage feasibility study in England, evaluated solution cavity development and stability, and investigated crushed salt behavior for the sealing of a high level nuclear waste repository. He is presently involved in a major project evaluating mine flooding and stability in a potash mine in Saskatchewan, Canada. Dr. Thompson has published a number of papers on rock mechanics.

Mr. Tod Burrington is the Manager of the Advanced Repository and Technology Department at the WIPP for the Managing and Operating Contractor, Westinghouse, during which time he has held the position of Manager of Mining Engineering.

Dr. Roy Cook is the Manager of the Geotechnical Engineering Section at the WIPP for the Managing and Operating Contractor, Westinghouse. He has worked on the WIPP project for 4 years. Formerly, he was with the high level nuclear waste program studying potential sites for a repository in salt. He also has experience with mining in deep evaporite deposits.

Dr. Joe Tillerson is the supervisor for the Rock Mechanics, and the Plugging and Sealing groups for the WIPP Project for Sandia National Laboratories, based in Albuquerque, New Mexico. Previously, he worked in the rock mechanics program on the site investigations for a high level nuclear waste repository at Yucca Mountain in Nevada. He has published papers and reports on nuclear waste repository design and the creep behavior of underground openings in salt.

3.0 TECHNICAL STATEMENTS

Five technical statements prepared by the Managing and Operating Contractor, Westinghouse, were provided to the panel members at the start of the first meeting in Carlsbad. The purpose of the statements was to focus the attention of the panel members on specific technical questions related to the overall issue of life expectancy of rooms excavated in the first waste storage panel at the WIPP. Each panel member was requested to respond to these statements individually. Assumptions, and the factors to be addressed were provided for each statement. The statements, assumptions, and factors were expected to undergo modifications as the meetings progressed in order to be more effective in addressing the issue under consideration. Although modifications were made during the meetings, they did not change the underlying intent of the statements.

The final statements are given in Table 1. Changes from the original are included. Additions are shown in bold type and deletions have been lined through.

The purpose of the first statement was to establish an estimate for the period of time that rooms designed for waste storage could be expected to remain accessible on a daily basis. Since actual performance depends on the extent to which the room is supported and maintained on a regular basis, a series of different cases relating to support and maintenance were to be addressed. The panel members were also asked to provide upper and lower bounds for their estimates, and to address the question of uncertainty in their responses.

The second statement addressed the effectiveness of the rockbolt system currently installed in Panel 1. The panel members were requested to evaluate the assumptions used in the design and to consider the adequacy of the safety factor for the overall system.

The third statement considered the uncertainties associated with the design of structures in rock. The panel members were asked to address the design approaches currently used in mining.

The purpose of the fourth statement was to provide modifications that could enhance the performance of the rooms in Panel 1 such that the bin scale tests could be successfully completed. The panel members were requested to propose alternative support systems and to recommend maintenance activities that would keep the rooms open.

The fifth statement addressed the adequacy of the geomechanical monitoring in the underground facility and, in particular, its ability to provide early warning of deteriorating conditions in the rooms of Panel 1.

Table 1

STATEMENTS FOR CONSIDERATION BY GEOTECHNICAL PANEL

STATEMENT	ASSUMPTIONS	FACTORS TO BE ADDRESSED
1. An estimate can be established for the period of time that Panel 1, in particular Room 1 remains accessible on a daily basis beyond July 1991. (Revision 1) The following cases should be considered: • ~~No maintenance in terms of scaling of roof, milling of floor or installation of additional support.~~ • Limited maintenance without moving bins. • Extensive maintenance on an as required basis, with bins removed from room, if necessary during maintenance activities.	1. Room height on July 1, 1991, 13.5 feet and minimum room height needed to support equipment clearances, 10.0 feet. 2. Room initially excavated in July/August 1986. 3. **Falls of lumps of roof or side wall rock that might damage bins or instruments should be prevented.**	1. The ability of the Panel to address Statement 1 based on the available information. 2. Best estimate for life of Room 1. 3. Lower and upper bound estimates for the life of Room 1. 4. Levels of uncertainty associated with estimates. 5. Reasons for the levels of uncertainty. 6. **Additional information that would be needed to improve estimates.** 7. **Potential pillar (side wall) spalling.**
2. The rockbolt system as currently configured, is sufficiently effective to ensure that the test program in Panel 1, in particular Room 1 can be completed. (Revision 1)	1. The test program will start in July 1991. 2. ~~The test program will be completed in July 1996.~~ 3. ~~Retrieval from Room 1 can be accomplished between July 1996 and July 1997.~~ 4. The bins CANNOT be disconnected and moved to facilitate maintenance of the rooms. 5. **The test program including retrieval will be completed by July 2000.**	1. The affect that the changes associated with the test program have on support requirements for Room 1, Panel 1. 2. The rock load to be supported is approximately the full weight of the roof beam up to the anyhydrite "b" layer in the middle third of the span, and half this weight over the outer two thirds. 3. The adequacy of the factor of safety of the bolting system used in Room 1, Panel 1 to support the design rock load. 4. The salt above the anhydrite "b" will remain competent. 5. Slippage of anchors provides an acceptable approach to supporting the rock load while accommodating roof closure, with daily access to the room. 6. The mechanism by which the bolt anchors will accommodate the movement of the salt while supporting the immediate roof beam.

STATEMENT	ASSUMPTIONS	FACTORS TO BE ADDRESSED
3. The level of confidence that can be placed in the estimate of the life for Panel 1 provided in the response to Statement 1 is in accordance with accepted mining practices. (Revision 0)		1. The extent to which a probabilistic basis for determining risk assessment is presently applied in mining. 2. The qualitative nature of geologic information. 3. The extent to which a database or experience is available in the mining industry from an operations point of view to provide the meaningful judgements at the probability levels used in the nuclear industry (i.e. probabilities of less than 1 in 10^6). This is not to be applied to an assessment of the long term (10,000 year) performance of a repository. 4. The adequacy of the geomechanical database developed at the WIPP and the methods currently in place to evaluate the performance of openings.
4. Modifications to the support system in Panel 1 can be implemented to ensure that access is maintained to the rooms on a daily basis until the tests are completed. (Revision 0)		1. The modifications and additions to the support system needed to ensure the completion of the tests. 2. The maintenance activities that will be needed in the room. 3. The need to remove the cables for the bin scale tests in order to install additional support.
5. The geomechanical monitoring program and the routine observations in Panel 1, can provide sufficient warning to allow the timely retrieval of the waste from the Panel. (Revision 0)	1. In an emergency, all waste can be removed from the room within a 6 month period.	1. The adequacy of the geomechanical database developed at the WIPP provides an adequate basis to predict and provide early warning of deteriorating conditions in Room 1. 2. The adequacy of the present geomechanical instrumentation, installed in Room 1 is adequate to provide early warning of deteriorating conditions. 3. The adequacy of the proposed additional geomechanical instrumentation to be installed in Room 1 to provide early warning of deteriorating conditions. 4. The criteria to determine when removal of waste becomes necessary.

4.0 TECHNICAL MEETINGS

The geotechnical panel met on two occasions in April to evaluate the life expectancy of Panel 1. Both meetings were held in Carlsbad, New Mexico. Documentation of the meeting is given in Appendix I.

The first meeting was held on April 9, 1991, and was attended by the panel members and observers from various organizations associated with the WIPP Project. The purpose of this meeting was to provide an overview of the project to the panel members, to provide geomechanical data and its interpretation relating to the performance of the waste storage rooms, and to provide instructions to the panel concerning the process for resolving the technical issues.

On April 10, 1991, the panel members and observers toured the WIPP underground, specifically visiting the SPDV Test Rooms and Panel 1 of the waste storage area. The SPDV Test Rooms were constructed to provide field data on the performance of excavations having dimensions similar to those in the waste storage areas and to provide the basis for evaluating the waste storage rooms. Following the underground tour, the panel met to discuss their observations, to establish additional data needs, and to receive instructions on the preparation and submission of draft reports.

The panel members were requested to submit draft reports based on a series of prepared statements provided to them within a seven day period. These reports were summarized by project personnel to establish a draft consensus position for each statement that would be presented to the panel as a group at the second meeting.

The second meeting was held in Carlsbad on April 23 - 24, 1991. All the panel members except for Mr. Jack Parker were present. The panel members presented their technical views. On the second day of the meeting, the draft consensus position was submitted to the group, discussions were held, and group responses were revised until consensus was reached on the five statements. The final consensus position is given in Section 5.

Following the meeting, the panel members were given the opportunity to revise their reports. Their final reports are included in Appendix II.

The panel suggested that additional geotechnical instrumentation be installed to provide an even stronger monitoring program. Revised geomechanical instrumentation was proposed by project personnel at the second meeting. These plans met with the approval of the panel members and are included in Appendix III.

5.0 PANEL RESPONSES TO THE TECHNICAL STATEMENTS

5.1 GENERAL COMMENTS

The following general comments were provided by individual panel members in their reports or in conversations:

- Nobody invites me to go look at a nice mine. But this was an exception; I think that this was an unusually clean, safe operation, showing good workmanship. (J. Parker)

- The best way to assess risk in a salt/potash mine is by making measurements . . . . WIPP has a good geomechanical database on which to base predictions of future behavior. (H. Miller)

- The design of the openings in the waste storage area is satisfactory for the original purpose of emplacing waste for final disposal. The change in the intended use of the rooms in Panel 1, with the requirement that they last longer, is the reason that the support requirements for the rooms are being re-addressed (G. Griswold)

- Standard mining practice in these (evaporite) materials is to use the mine itself as a test bed. Initial mine designs are based on experience elsewhere in similar materials but during its life the mine design is constantly tailored to local conditions. (W. Thompson)

5.2 CONSENSUS PANEL RESPONSE

The panel was able to reach a consensus on the responses to the five technical statements presented at the start of the panel evaluation. The responses agreed to by the panel members (J. Parker was in absentia) are provided in Table 2.

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Table 2

CONSENSUS RESPONSES TO STATEMENTS

STATEMENT	RESPONSES
1. An estimate can be established for the period of time that Panel 1, in particular Room 1 remains accessible on a daily basis beyond July 1991.	The panel expects with confidence (and by) using engineering judgement, a life of seven to eleven years from Room 1, Panel 1 with limited maintenance as required. The panel expressed a high level of confidence for the lower bounds of room life expectancy, with confidence decreasing as room life increases. The life of rooms in Panel 1 can be extended with an enhancement of the support if routine maintenance is carried out, as required. NOTES: • The panel feels that the precise effects of rockbolting cannot at this stage be established with any degree of confidence. It is recommended that a study of the effectiveness of rockbolting at the WIPP facility be undertaken. • The stress history (sequence of mining) of the different rooms in Panel 1 should be taken into account in assessing the expected room life. • The room life can be extended indefinitely, but this would be complicated and costly and require ongoing maintenance. • Other rooms in Panel 1 which are younger also have a total life expectancy of seven years with a high level of confidence without additional support. These rooms, as is, would support a longer test period than Room 1 because they are younger.
2. The rockbolting in Panel 1, as currently configured, is sufficiently effective to ensure that the test program in Panel 1 in particular Room 1 can be completed.	The rockbolting in Panel 1, as currently configured, provides no guarantee that a test plan that may extend for a nine year period can be completed. Panel 1 rooms are expected to provide a total life of seven years (up to eleven years with decreasing confidence) without modifications. The maximum test period (nine years) requiring a total life of fourteen years may be accomplished in Panel 1 if suggested enhancements for support and maintenance work (detailed in response to Statement 4) are enacted.
3. The level of confidence that can be placed in the estimate of the life for Panel 1 provided in the response to Statement 1 is in accordance with accepted mining practices.	Formal Probability Risk Assessments are not used in evaporite mine design. Field measurements, operational experience, and modeling are routinely incorporated into designs to effect an informal probabilistic level of confidence. The success of projecting the data from SPDV Test Rooms to Panel 1 is very good due to the uniformity of geology. However, minor changes in geology can change future predictions of life. Probability estimates in the order of 1 in 10^6 of operational behavior are totally unrealistic in a geologic environment. The risk assessment in mining is based on • Operational experience • Modeling • Deformation measurement • Geologic Mapping

STATEMENT	RESPONSES
4. Modifications to the support system in Panel 1 can be implemented to ensure that access is maintained to the rooms on a daily basis until the tests are completed.	The panel proposes the following support system enhancements • A support system utilizing resin anchored bolts. • Grout anchored cables with loops, lacing and meshing covering the roof in order to contain and control roof rock failure. • Relief of the lateral stresses to prevent roof and floor failures by slotting and/or relief entries. • Yielding support. • Rely on currently installed support and upgrade when necessary based on the results of the geomechanical monitoring program. • Roof trusses. • Driving of new rooms through existing pillars in Panel 1. (This remedial action was added at the request of Mr. J. Parker who was absent from the April 23 and 24 meeting). The panel recommends that an engineering evaluation should be carried out to assess the viability of these enhancements. NOTE: • The modifications could involve a combination of these enhancements. These enhancements are proven techniques.
5. The geomechanical monitoring program and the routine observations in Panel 1, can provide sufficient warning to allow the timely retrieval of the waste from the Panel.	The panel agrees that: • The geomechanical WIPP data base is an adequate tool for giving early warning of deteriorating conditions in Panel 1. • Additional data interpretation should be performed to refine and implement the identification of deteriorating conditions. • Present geomechanical instrumentation in Panel 1, although adequate, should be upgraded. • Geotechnical criteria should be used to alert the project to changing conditions in Panel 1 and to initiate decision making courses of action.

APPENDIX I

DOCUMENTATION OF MEETINGS

Contents of Appendix I

Agenda for Meeting on April 9 and 10, 1991

Agenda for Meeting on April 23rd and 24th, 1991

List of Participants on April 9th and 10th, 1991

List of Participants on April 23rd and 24th, 1991

List of Data, Reports and Documents submitted to the Panel as the basis for their evaluation.

EXPERT PANEL - LIFE OF PANEL 1

Tuesday, April 9, 1991
Park Inn, 3706 National Parks Highway, Carlsbad, NM

AGENDA

I. Introduction.

- Introduction of participants.
- Scope of evaluation.
- Deliverables.

II. Review of WIPP Project.

III. Presentation of geotechnical data and evaluations of Panel 1.

- Overview of monitoring program.
- Ground control in Panel 1.
- Geomechanical data from rockfall in SPDV Test Room 1.
- Rockbolt performance.
- Assessment of useful life of Panel 1.

IV. Overview of tests with radioactive waste (bin-scale tests) in Panel 1.

V. Sandia National Laboratories Presentation.

VI. Rockbolting specifications.

VII. Open discussion.

Wednesday, April 10, 1991
(WIPP Site)

VIII. Safety briefing for underground visit.

IX. Underground visit to observe Site and Preliminary Design Validation Test Rooms 1, 2, 3, 4, and Panel 1, Rooms 1, 2, and 6

X. Open discussion.

Agenda for Expert Panel on
the Life of Panel 1

Park Inn, 3706 National Parks Highway, Carlsbad, NM

Tuesday, April 23, 1991
8:00 am

I. Introduction

II. Presentation by Panel Members

P. Mottahed
I. W. Farmer
T. W. Thompson
G. B. Griswold
J. R. Tillerson
S. D. McKinnon
A. T. Iannacchione
H. D. S. Miller
J. Parker (in absentia)
R. F. Cook

III. Open Discussion

Wednesday, April 24, 1991
8:00 am

IV. Presentation of Draft Summary Report

V. Discussion of Summary Report and Recommendations for Revision

VI. Presentation of Revised Summary Report

VII. Open Discussion

GEOTECHNICAL EXPERT PANEL
ATTENDANCE
PARK INN, CARLSBAD, NEW MEXICO

April 9, 1991

EXPERT PANEL MEMBERS

Dr. G. B. Griswold
Mr. T. P. Burrington
Dr. J. R. Tillerson
Dr. I. W. Farmer
Dr. T. W. Thompson
Dr. S. D. McKinnon
Dr. P. Mottahed
Dr. R. F. Cook
Mr. J. Parker
Dr. H. D. S. Miller
Mr. A. T. Iannacchione

OBSERVERS

Mr. R. C. Supka, WID
Mr. R. C. Carrasco, WID
Mr. C. T. Francke, WID
Mr. D. Galbraith, WID
Ms. J. L. Francke, WID
Dr. J. A. Mewhinney, DOE
Mr. E. K. Hunter, DOE,
Mr. S. C. Sethi, WID
Mr. H. D. Ripley, WID
Ms. R. Molgaard, WID
Dr. K. M. Chua, UNM
Mr. R. Sowers, WID
Mr. R. Batra, DOE
Mr. T. F. Brockman, WID
Dr. D. E. Munson, SNL
Mr. T. M. Schultheis, SNL
Dr. L. Chaturvedi, EEG
Mr. W. D. Greenlee, WID
Mr. J. E. Carr, DOE
Mr. J. A. Gonzalez, WID
Mr. L. B. Lilly, DOE
Mr. M. G. W. Phillips, DOE, HQ
Mr. J. E. Gilbert, DOE

GEOTECHNICAL EXPERT PANEL
ATTENDANCE
PARK INN, CARLSBAD, NEW MEXICO

April 10, 1991

EXPERT PANEL MEMBERS

Dr. G. B. Griswold
Mr. T. P. Burrington
Dr. J. R. Tillerson
Dr. I. W. Farmer
Dr. T. W. Thompson
Dr. S. D. McKinnon
Dr. P. Mottahed
Dr. R. F. Cook
Mr. J. Parker
Dr. H. D. S. Miller
Mr. A. T. Iannacchione

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Mr. D. Galbraith, WID
Mr. R. C. Carrasco, WID
Mr. M. R. Brown, WID
Mr. D. Baldwin, WID
Mr. J. A. Gonzalez, WID
Mr. G. L. Ashford, WID
Mr. R. Sowers, WID
Mr. T. F. Brockman, WID
Mr. H. D. Ripley, WID
Mr. M. A. Molecke. SNL
Mr. L. Chaturvedi, EEG
Mr. R. Batra, DOE
Dr. D. E. Munson, SNL
Mr. M. G. W. Phillips, DOE, HQ
Mr. T. W. Halverson, WID
Mr. H. L. Bibby, WID

GEOTECHNICAL EXPERT PANEL
ATTENDANCE
PARK INN, CARLSBAD, NEW MEXICO

April 23, 1991

EXPERT PANEL MEMBERS

Dr. G. B. Griswold
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Mr. H. D. Ripley, WID
Mr. J. E. Gilbert, DOE
Mr. R. C. Carrasco, WID
Mr. C. T. Francke, WID
Mr. T. F. Brockman, WID
Mr. H. L. Bibby, WID
Mr. J. A. Gonzalez, WID
Mr. R. Batra, DOE

GEOTECHNICAL EXPERT PANEL
ATTENDANCE
PARK INN, CARLSBAD, NEW MEXICO

April 24, 1991

EXPERT PANEL MEMBERS

Dr. G. B. Griswold
Mr. T. P. Burrington
Dr. J. R. Tillerson
Dr. I. W. Farmer
Dr. T. W. Thompson
Dr. P. Mottahed
Dr. R. F. Cook
Mr. S. D. McKinnon
Dr. H. D. S. Miller
Mr. A. T. Iannacchione

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Mr. J. A. Gonzalez, WID
Mr. E. K. Hunter, DOE
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Mr. R. C. Carrasco, WID
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PANEL 1 ROOM 1 EVALUATION EXPERT PANEL

Information package contents

Geotechnical Field Data and Analysis Report

July 1989 to June 1990, Volumes I and II, DOE/WIPP 91-012

Interim Geotechnical Field Data Report, Fall 1986, DOE/WIPP 86-012

Sections: Chapter 11.5 Facility Level
Chapter 12.7 Drifts
Chapter 12.8 Test Rooms

Design Validation Final Report, DOE/WIPP 86-010

Sections: Executive Summary
Chapter 1 Introduction
Chapter 2 Background of Underground Design
Chapter 11 Test Rooms
Chapter 12 Storage Area

Borns, D. J., and J. C. Stormont, 1988. An Interim Report on Excavation Effect Studies at the Waste Isolation Pilot Plant: Delineation of the Disturbed Rock Zone, Sandia National Laboratories, SAND87-1375.

Stormont, J. C., 1990, Discontinuous Behavior Near Excavations in a Bedded Salt Formation, Sandia National Laboratories, SAND89-2403.

Design Criteria Waste Isolation Pilot Plant, Revised Mission Concept - IIA, WIPP/DOE-71 Rev. 4.

Sections: Chapter 5 Underground Facilities and Systems,
Item 1 Introduction
Item 2 Ground Control

Cook, R. F., 1991. Position Paper: Life Expectancy of Room 1, Panel 1, Draft.

Breenwald, H. P. and H. C. Howarth, 1938. Compression Tests of Roof-Salt Slabs Supported by Potash Salt Pillars, R.I.-3386, U.S. Bureau of Mines.

Brockman, T. R., 1988. Panel 1 Roof Bolting, Design Calculations, EWP-51-0-0433.

APPENDIX II

FINAL REPORTS SUBMITTED BY PANEL MEMBERS.

REPORT SUBMITTED

BY

DR. I.W. FARMER

The summary report contains:

- An accurate record of the meetings of the Geotechnical Panel on Panel 1 Stability.

- A copy of the report provided to Westinghouse by this panel member.

- An accurate presentation of the consensus agreed to by the panel members at the meetings on the 23rd and 24th of April 1991.

Panel Member	5-28-91 Date

COMMENTS ON WASTE ROOM STABILITY AND RESPONSE TO STATEMENTS

by Ian Farmer

(1) BASIS OF DESIGN

The basis of design of both the SPDV Test Rooms and No 1 Waste Storage Panel appears to have been the requirement that the storage rooms be 33x 11 ft. in section and 300 ft. long and have a tolerance of -0 +1 ft. An allowance of 24 inches vertical closure during a 5 year panel life was validated by calculations based on empirical creep equations and measurements during the first 3 years life of the Test Rooms.

In both design and analysis, deformation was assumed to result mainly from creep processes. In practice, observations have shown that this is not the case and that additional mechanisms involving strain softening, fracture and movement along discontinuities - albeit time related - are involved in a complex deformation process. This may also include effective stress effects from brine and gas.

The emphasis on creep processes results from the historic emphasis on time related deformation of most laboratory test work on rock salt. This usually involves long term loading of specimens in compression at relatively high unconfined or deviatoric stresses. The results are usually expressed as power law creep equations with secondary data on modulus of elasticity, Poisson's ratio and uniaxial compressive strength. These types of test while producing useful data, sometimes have limited relevance to the performance of underground excavations - particularly of the rock near the exposed surface of the roof, sides and floor - where deformation results from stress relief after excavation (an active expansive process) rather than specimen loading (a passive compressive process).

Baar (1977) showed that under these conditions creep limits for rock salt in-situ are extremely low and that constant rate plastic flow can occur at a yield stress difference as low as 150 psi at room temperatures. At

low deviatoric stresses these creep strains are relatively low. It is only at high stresses and temperatures and particularly with high confining pressures that they are large.

Analysis of creep is complicated by the relatively high strength of rock salt measured in compression, compared with the relatively low measured tensile and shear strengths at room temperature. Baar estimates tensile strengths as 4-8% of compressive strength. Dreyer (1972), however, shows that in conventional laboratory testing of rock salt, confinement has a low effect on strength and that ϕ is low and shear strength is about 40% of compressive strength.

(2) MECHANICS OF ROOF FAILURE

The effect of these observations may be discussed in relation to the SPDV Test Room 1 excavation. Figure 1 plots contours of major and minor principal stress around a 33 ft. by 13 ft. excavation at the same depth at SPDV Test room 1. These are the stresses in an elastic material in plane strain in two dimensions. In practice, they will be modified by excavation at a finite rate (30% of relaxation will occur ahead of the face), by creep and fracture. The important thing to note, however, is that tensile fractures initiate and propagate in a direction parallel to the major principal stress, causing dilation normal to this direction.

Figure 2 plots contours of one half deviatoric stress (equivalent to peak shear stress) for the same geometry and stress. These are similar to the F and M contours plotted by Stormont (1990). Once again, the actual deviatoric stress distribution will be modified following excavation and creep, but as a general observation, shear will occur along the lines where the shear stress exceeds the shear strength of the rock salt. Shear will result in shear movement along the potential shear fracture and dilation normal to the fracture.

Assuming that the compressive strength of the rock salt is approximately 4000 psi, the tensile strength 200-300 psi and the shear strength 1600 psi around the excavation, the following general observations can be made:

(a) The high deviatoric stresses at the corners of the excavation (modified by any curvature) will be relieved at an early stage by formation of a shear fracture, following the edge of the highly stressed zone into the roof and floor and probably the sides of the excavation. The existence of this fracture in a similar size of excavation is illustrated by Stormont's (1990) permeability measurements, and by numerous observations of fractures.

(b) Most of the surface deformation around the excavation will be caused by a combination of induced tensile and shear fractures modified by creep. This is illustrated in Figure 3. The tensile fractures will tend to follow the contours of major principal stress and deviatoric stress. Dilation normal to the fracture direction will cause horizontal or vertical convergence into the sides, roof and floor and modification of stress and fracture orientation. But the overall pattern agrees very well with Borns and Stormont's (1988) permeability observations (a direct result of dilation) and with their modification of Gramberg and Roest's (1984) estimates of fracture zones in rock salt.

(c) Continuing dilation will result in bed separation at partings at the much stronger (estimated 4 times) and stiffer roof and floor anhydrite layers. This is a well known phenomenon in layered rocks and in layered rock salt and is described by Baar (1977) and others. As a result the floor and roof beams may become partly detached, the former exacerbating floor heave and the latter ultimately resulting in roof failure similar to the cutter roof failure in coal mines.

(3) SPDV TEST ROOM FAILURE

It is important to see if this postulated failure regime agrees with observations at SPDV Test Rooms 1 and 2, where the best deformation information from closure measurement, borehole anchor extensometers and inclinometers is available. The data over 6 years is plotted in Figures 4 and 5. This includes the initial nonlinear convergence at Test Room 1 in Figure 4

resulting in an additional 3 inches of lowering of the center and lower roof below the anhydrite parting; otherwise the data is essentially the same. Data during and immediately following construction when relief of construction stresses occurred is missing.

The general deformation pattern does, however, agree with the postulated pattern in Figure 3, particularly:

(a) Deformations at the corners are not extreme, indicating that high deviatoric stresses have been relieved by formations of a shear fracture.

(b) Horizontal movements in the solid rock are largely confined to the zone above the sidewall edge and close to the shear fracture and in a direction normal to the proposed tensile fractures extending into the roof.

(c) Vertical movements are largely confined to the roof and floor and are largest below the roof parting, particularly at the center, and above the floor anhydrite layer - again particularly at the center.

It can be argued, therefore, that the general pattern of movement is essentially that postulated by Stormont (1990) for the specific WIPP case and by Baar (1977) for the general case of evaporites and involves both creep and fracture, but principally fracture, albeit over a prolonged period.

It can also be argued that Waste Panel 1 Room 1, although there is less information, is deforming in a very similar manner to the SPDV Test Rooms. The basic questions, therefore, which must be asked in assessing the long term stability of Waste Panel 1 are whether the roof will behave in a similar manner to SPDV Test Room 1 and whether the current support is adequate or can be made adequate.

(4) <u>ROOF SUPPORT</u>

The roof of SPDV Test Room 1 appears to have collapsed as a single large block, probably trapezoidal in shape, breaking up on contact with the floor. It is bounded by shear planes - apparently steep on the West side and shallow on the East side and by the clay/anhydrite parting 7 ft. into

the roof. The clay/anhydrite parting may be exposed in up to 1/3 of the roof span. Calculations by Cook (1991) indicate that the North and South ends of the roof beam fractured in tension due to the weight of the detached span.

If the unit weight of rock salt is assumed 150 lbs/cu.ft., the maximum weight of the roof beam is approximately 35,000 lbs/ft. In Test Room 1, this was unsupported. In Waste Panel Room 1, it is supported by approximately 1.7 x 10 ft. long x 5/8 or 3/4 in. roof bolts per foot, with respective average pullout loads of 19,500 and 15,000 lbs. and with design loads of 13,500 and 11,900 lbs. The bolt pattern concentrates support in the center of the room.

The rockbolts have been designed to support the dead weight of the roof layer; to accommodate creep movements and to avoid fracturing of the deforming roof surface. For the latter, it was assumed that the anchorage would yield and this was tested short term. The bolts were located 2 1/2 ft. above the anhydrite layer, where vertical downward movement is approximately 1 - 1 1/2 ins./year and horizontal movement is probably relatively small.

The purpose of rockbolting in the current geology and excavation geometry is essentially to prevent movement across discontinuities/bedding lanes and particularly the anhydrite layer. It is similar to cutter roof failure in coal mines, which is also time related and difficult to control with conventional roof bolts, however long. In these circumstances roof trusses or center cribs have been successfully used and these represent an alternative, respectively long term and short term, in the present case.

The roof at WIPP is, however, different to coal mines in that only two partings are known to exist and the rock is not laminated but apparently quite massive. In this case, it may be possible to obtain a degree of medium term control with rock bolts installed in the traditional way.

(5) <u>RESPONSE TO STATEMENTS</u>

STATEMENT 1: An estimate can be established for the period of time that Panel 1, in particular Room 1, remains accessible on a daily basis beyond July 1991.

1. Available information on Waste Panel 1 appears to be limited to horizontal extensometers installed in the E and W rib at the mid point in December 1988 and convergence meters installed at the midpoint and North and South ends at various dates between 1986 and 1990. Many of these are no longer functional, but a summary of data is available indicating 19 ins. of roof to floor convergence over a 5 year period to April 1991. As far as can be seen, the deformation over this period is similar to that of SPDV Test Rooms 1-4 over a similar period. Combined with a knowledge of deformation mechanisms, this give a basis for discussion of the statement.

2. To estimate the life of Room 1, it is necessary to make some assumptions about its performance compared with SPDV Test rooms. Convergence of SPDV Test Room 1 up to 5 years reached a steady state of 3 ins/year. After 5 years, this increased as bed separations in the roof gradually led to detachment of the roof beam and ultimate collapse after about 8 years. Creep in rock salt should be a constant rate phenomenon and the constant creep rate, representing a roof or floor bay strain rate of about 0.5% per year, is moderate and almost certainly indicates a quasi-stable situation.

   Provided the deformation of the roof and floor in Room 1 can be maintained at the <u>present</u> rate and bed separations at the anhydrite/clay roof layer prevented, there is a good prospect of medium therm stability. The integrity of the roof block, based on SPDV Test Room 1 observations appears high and there is no reason why an additional 10 years life, bringing the total roof to floor convergence to about 50 ins., when the room would show considerable distortion, should not be expected.

3. A lower bound estimate of a total of 8 years (an additional 3 years) assuming the same failure mechanism as SPDV Test Room 1 is reasonable. More rapid failure is unlikely. With the proviso in paragraph 2 and good support and repair an upper limit of considerable length - say up to 20 additional years is feasible, provided the deformation can be tolerated.

4/5. Levels of uncertainty depend on the level of confidence in the assumptions made to reach an estimate. In this case, there is probably insufficient data to determine confidence levels beyond subjective terms such as high, medium or low. The most important basis for estimates is that the steady state roof and floor bay strains are moderate and in this case, in a homogeneous rock salt, it would be possible to postulate stability with a high level of confidence. The potential instability in the present case arises from the potential detachment of the roof block from the anhydrite layer and to a lesser extent buckling of the floor layer. If roof block detachment can be resisted by the support system, there will be a high level of confidence in the estimate.

6. There is limited deformation data available in the Waste Storage Panels. At the least, center line roof extensometers at the mid and quarter points are needed. These will monitor bay strains and parting separations.

7. Maintenance should be directed at maintaining roof integrity. Roof lowering at the current constant rate will lead to some extensions of shear fractures, which will require limited maintenance. The only situation which would require movement of bins would be nonlinear roof lowering. In this case either replacement of bolts or installation of cribs would be needed to maintain roof stability.

STATEMENT 2. **The rockbolt system as currently configured is sufficiently effective to ensure that the test program in Panel 1, particularly Room 1 can be completed.**

1. The rock bolt system is required to support the roof block for 10 years to 2000 A.D. in Room 1, Panel 1. As currently configured, the roof bolts are anchored in rock salt above the anhydrite layers which is deforming at an approximate rate of 0.5 ins/year vertically at the center and 0.25 ins/year vertically and 0.2 ins/year horizontally at the sides. The bolt collars are located at the surface which is deforming mainly vertically at a rate of 1.5 ins/year in the center, less at the sides.

   The resultant bolt strain of 0.8% per year may be tolerable for up to 5 years with anchor and collar deformation (3% bolt strain is usually considered a maximum). Beyond this, there can be no certainty of continuing support, without replacement or redesign.

2. The trapezoid at roof block configuration is not a conservative assumption. Typical failures of this type often have steep break lines and a better assumption would be rectangular block. This would also lead to a better distribution of support in the critical zone close to the shear fractures at the corners. There are good reasons for arguing that these shear fractures are not typically inclined at a low angle to the horizontal.

3. The current design of roof support does not appear conservative. If a unit rock weight of 150 lbs/cu.ft. is assumed then the weight of a rectangular 33 x 7 ft. roof block is 35,000 lbs/ft. and that of the design trapezoid is 23,000 lbs/ft. For 5/8 in. bolts, the design load is 11,900 lbs. and for 3/4 in. bolts (say) 13,500 lbs. These are barely adequate for the current trapezoid, which is itself a conservative assumption.

4 The salt above the anhydrite *b* layer is creeping at a rate of 0.25 ins/year - a low rate, which is unlikely to result in fracture. Horizontal deformations are equally low.

5. Slippage of anchors is not a reliable method of rock bolt roof control over an extended period of time and beyond an anchor strain of 3-4%. In the current case, beyond 5 years, anchor or collar failure would be expected.

6. Fully grouted bolts, probably with double set resins to give enhanced anchorage load are more reliable. Recent experiments by Signer and Jones (1990) have shown that high restraint can be maintained, even when part of the grouted bolt has yielded (see Figure 7). The possibility of using fully grouted 3/4 inch bolts (say) 12 ft. long with a 3 ft. quick set resin anchor tensioned to 30% of design load should be considered.

   In coal mines for similar roof configurations, where cutter roof failure is likely, truss bolts are extremely effective and these should be considered for other panels, where major redesign is possible.

STATEMENT 3. The level of confidence, which can be placed in the estimates of the life for Panel provided in the response to Statement 1 is in accordance with accepted mining practice.

1. Probability is used extensively in mining, particularly for slopes; to a lesser extent for pillars. The major requirement is that there exists an accurate and accepted analytical framework for design, and sufficient information on variability of parameters, usually expressed as variograms. In the case of Panel 1, the nature of roof failure is complex, involving several different mechanisms and geomechanical data is limited.

2. Geological information is not necessarily qualitative. Certainly at WIPP, it would be possible to build up an accurate database of rock salt mineralogy and structure which would show limited variability. Most rock discontinuities, beds, grain sizes can be expressed in terms of variograms and are often the best and "hardest" information available.

3. Probability levels of 1 in 10^6 are not feasible. The variations inherent in most geotechnical and geometric parameters means a probability of 1 in 10 is the best that can be obtained. Design in rock probably has the same type of probability levels as weather forecasting.

4. The WIPP data base is heavily orientated towards deformation measurement - since the design is based on creep. There is virtually none of the geotechnical information - particularly shear and tensile strength, which would be needed to accurately assess the performance of the openings - say by using finite element analysis with a combined creep - fracture constitutive model of the type developed by Desai and used by Stormont (1990) in his analysis.

STATEMENT 4. **Modifications to the support system in Panel 1 can be implemented to ensure that access is maintained to the rooms on a daily basis until the test are completed.**

1. The support system should be modified to perform in a roof where strain over the anchor length over a ten year period is likely to be 8%, equivalent to a differential displacement of 10 ins. Conventional mechanical anchors are likely to fail if subjected to this type of deformation. Roof to floor convergence over the same period is likely to be 30 ins. and roof lowering 15 ins. In addition, the current support system does not appear to have sufficient capacity to support the full roof block. The support system should be capable of generating a restraint of 35,000 lbs/ft. of room length and should provide better support for the edges of the block.

   Four types of support system may be suggested:

   (a) Fully grouted resin anchored bolts, 3/4 in. 12 ft. long with a 3 ft. long fast set anchorage; tensioned to 1/3 of working load. These should be set with an adjustable collar plate, and in a uniform pattern. The outer bolts should be angled towards the rib.

(b) Cable bolted trusses angled over the rib to just above the anhydrite/clay parting. The trusses should include an element of flexibility so that they can be lengthened to accommodate roof movement.

(c) Cable anchors - possibly combined with slings-installed centrally and incorporating a tensioning device which can be modified to accommodate roof lowering.

(d) Cribs installed centrally in the room including one or two elements of soft wood to allow for squeeze.

2. Some weld mesh should be installed, particularly at the pillar edges to catch loose rocks. Minimum maintenance activity should be planned - the support system should be designed to maintain roof integrity with a degree of flexibility to accommodate roof movement.

3. Once the experiment has started, installation of cribs is probably the only feasible additional support system. This should not - if planned for - require removal of cables.

STATEMENT 5. The geomechanical monitoring program and the routine observations in Panel 1 can provide sufficient warming to allow the timely retrieval of the waste from the panel.

1. The plot of rate of convergence against time from SPDV Test Room 1 provides a powerful and classic type of illustration of precursive roof movements leading to failure and also provides sufficiently early warning of deteriorating conditions to allow remedial action. Similarly, careful monitoring of SPDV Test Rooms 1 to 4 and other large span openings will provide additional ongoing precursive information - in the case of Test Room 4 for a roof including traditional rock bolts.

This is a limited data base, but the information is precise and directly relevant.

2. The geotechnical information from Room 1 is just adequate. The convergence data can be directly compared with Test Room 1.

3. Additional convergence stations and particularly roof extensometers designed to detect dilation of the parting are needed.

4. A increase in roof convergence, associated with parting dilation, which is not controlled or reduced quickly by installation of cribs or additional roof supports.

REFERENCES

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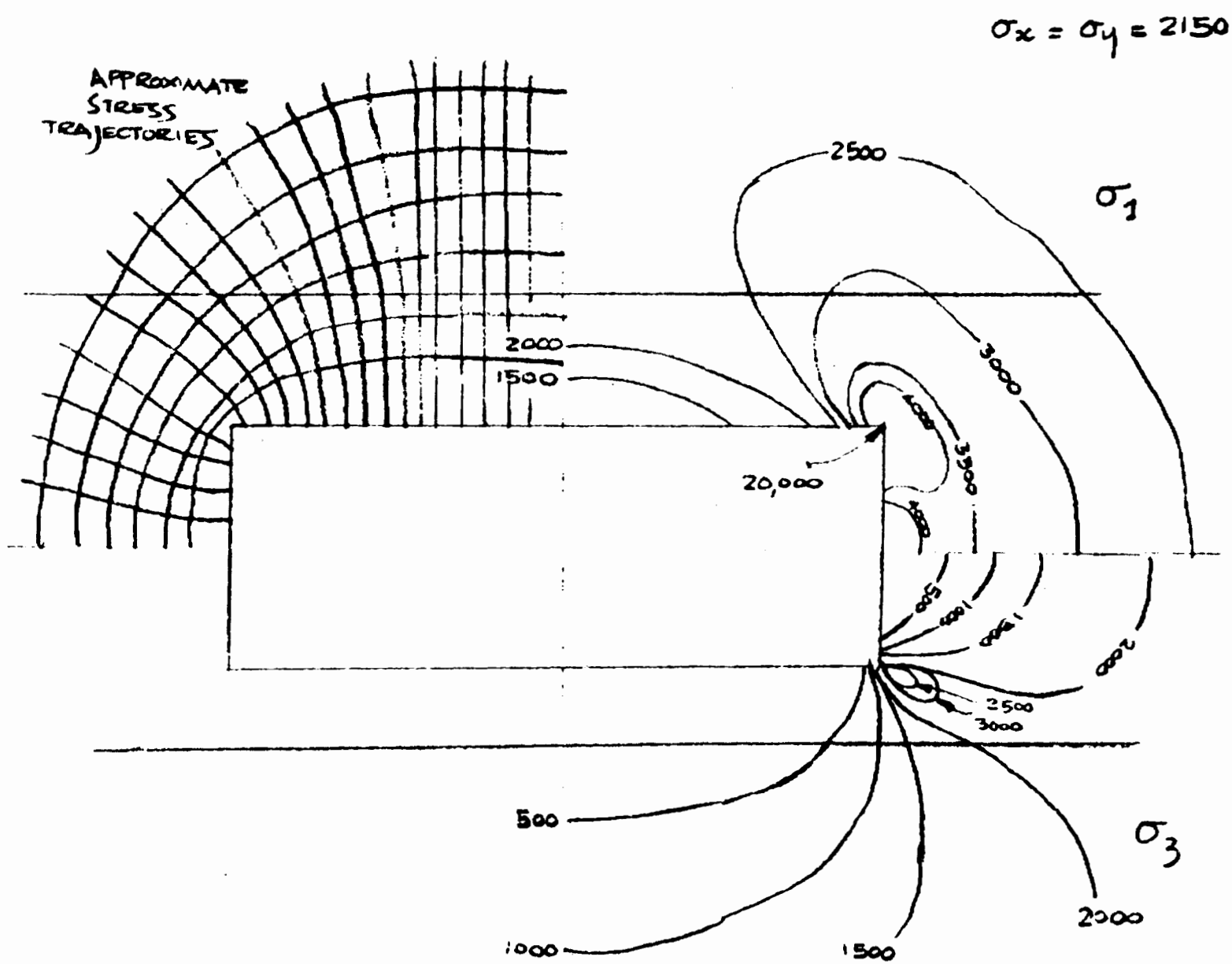

Fig 1. APPROXIMATE CONTOURS OF MAJOR & MINOR PRINCIPAL STRESS (psi)

0 1 2 3 4 5 ft.

$\sigma_x = \sigma_y$
$= 2150$ psi

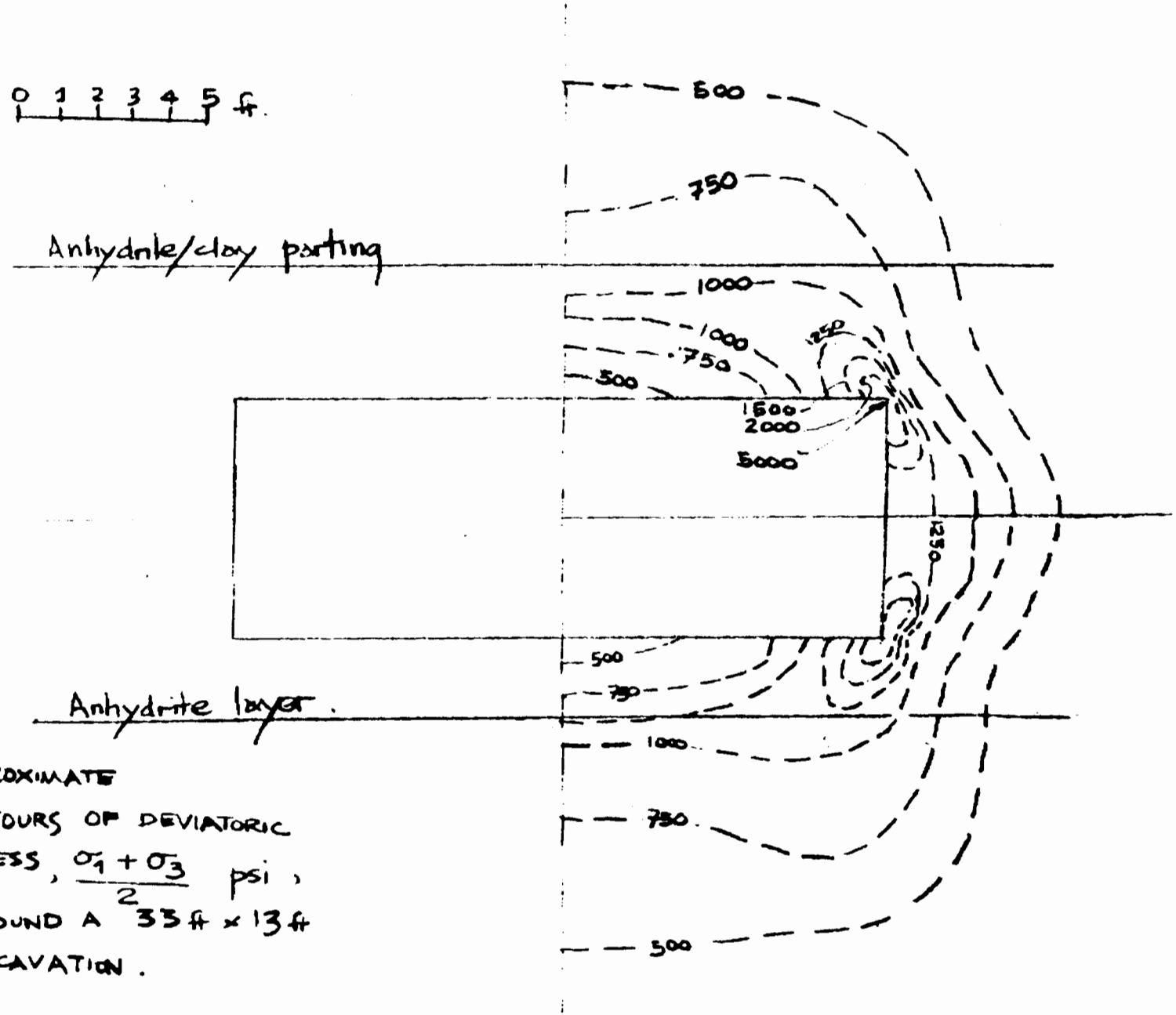

Fig 2. APPROXIMATE CONTOURS OF DEVIATORIC STRESS, $\frac{\sigma_1 + \sigma_3}{2}$ psi, AROUND A 33 ft × 13 ft EXCAVATION.

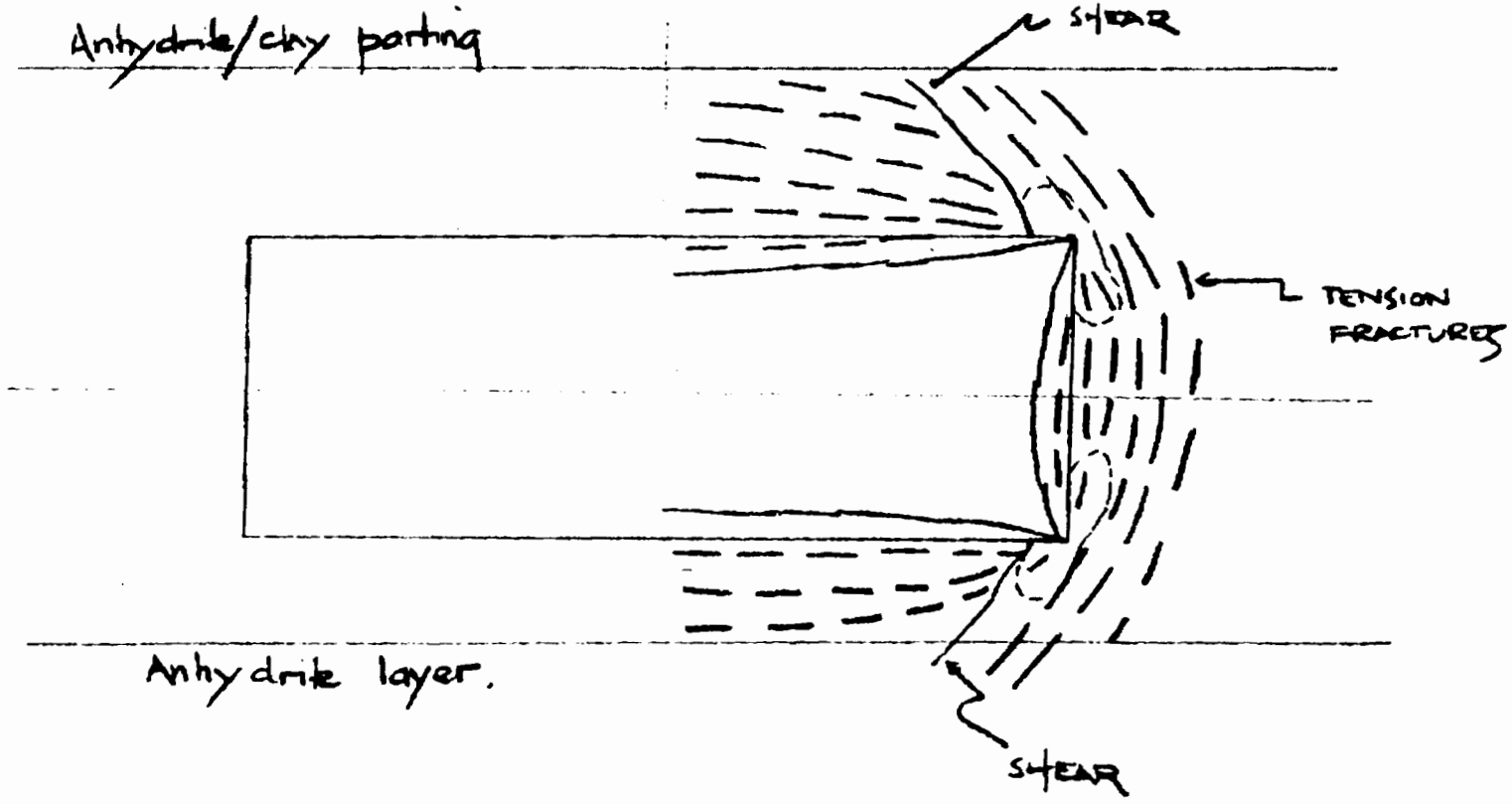

Figure 3. Proposed FRACTURE PATTERN AROUND A 33ft × 13 ft EXCAVATIO

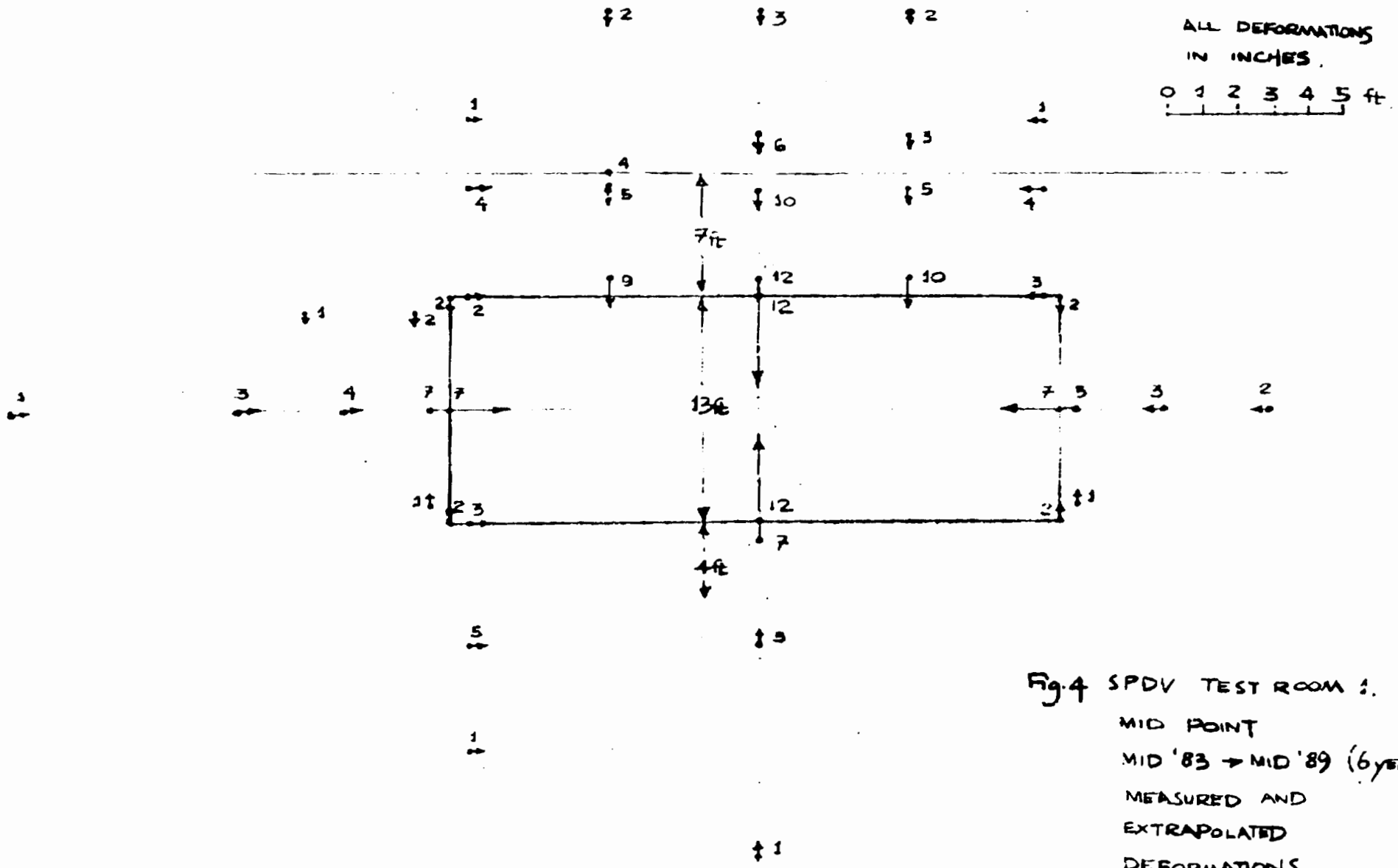

Fig.4 SPDV TEST ROOM 1.
MID POINT
MID '83 → MID '89 (6 YEARS)
MEASURED AND
EXTRAPOLATED
DEFORMATIONS
LOOKING NORTH.

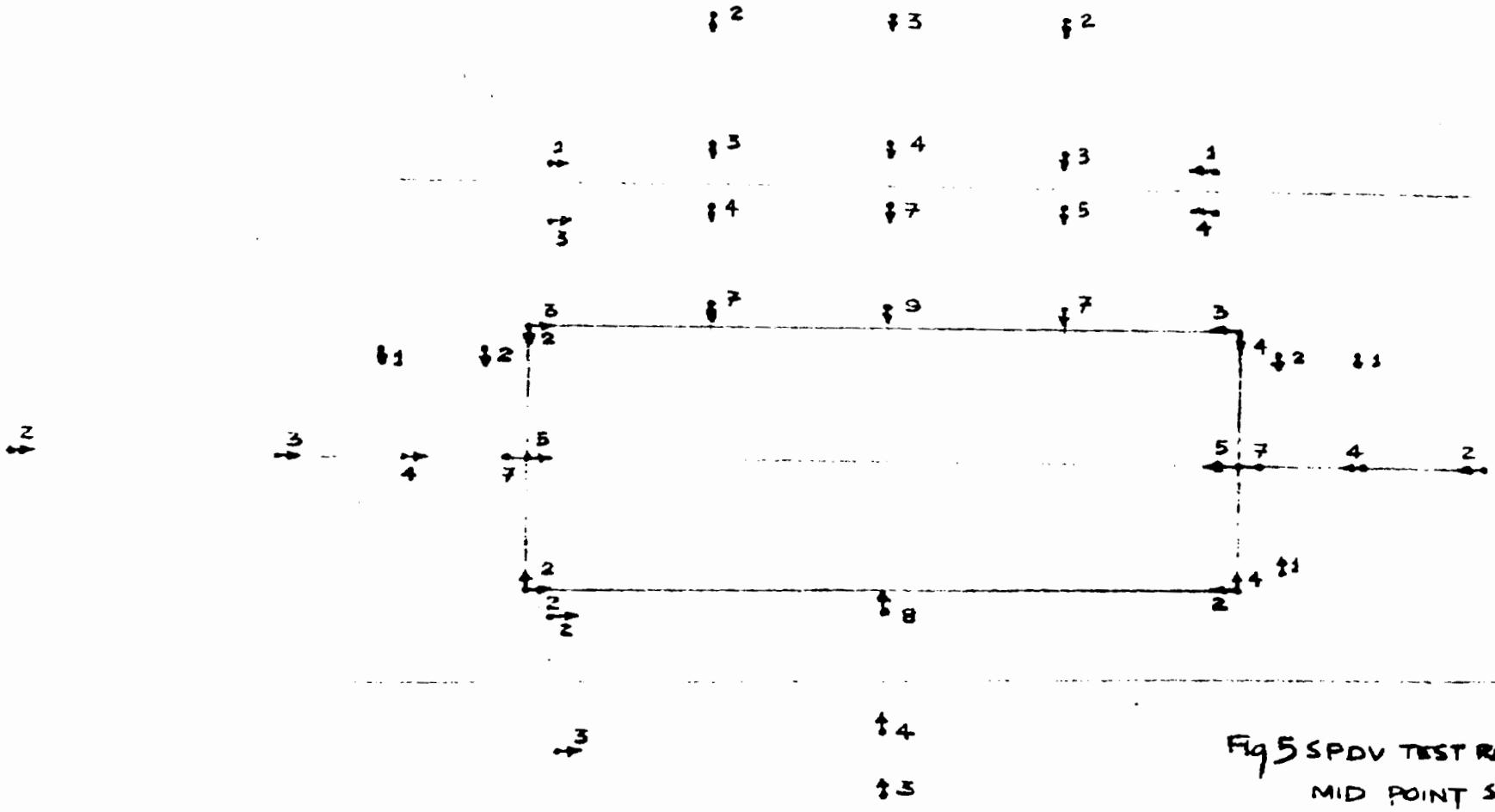

Fig 5 SPDV TEST ROOM 2
MID POINT SECTION
MID '83 → MID 89 - 6y5
MEASURED DEFORMATIONS
IN INCHES LOOKING N.

0 1 2 3 4 5 ft

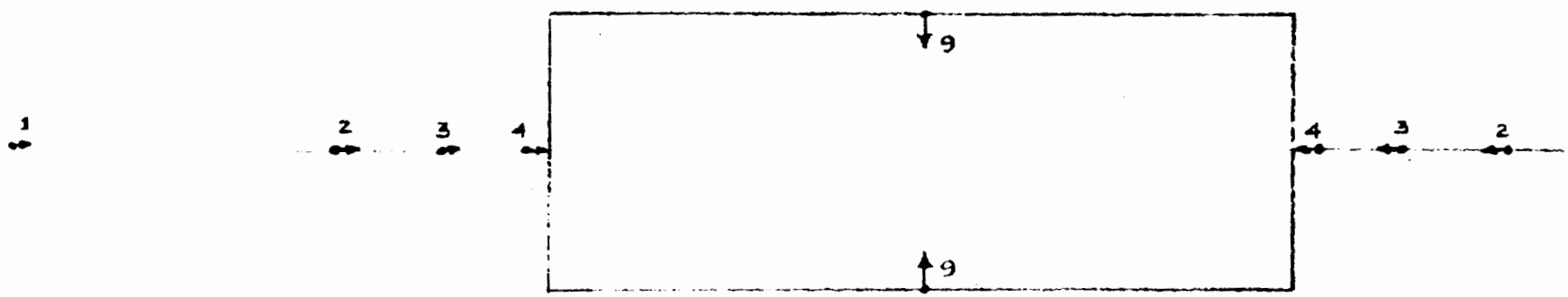

Fig. 6. WASTE PANEL 1 ROOM 1
MD POINT
JAN '86 – JAN 91 (5 YEARS)
MEASURED AND
EXTRAPOLATED DEFORMATIONS
LOOKING N.

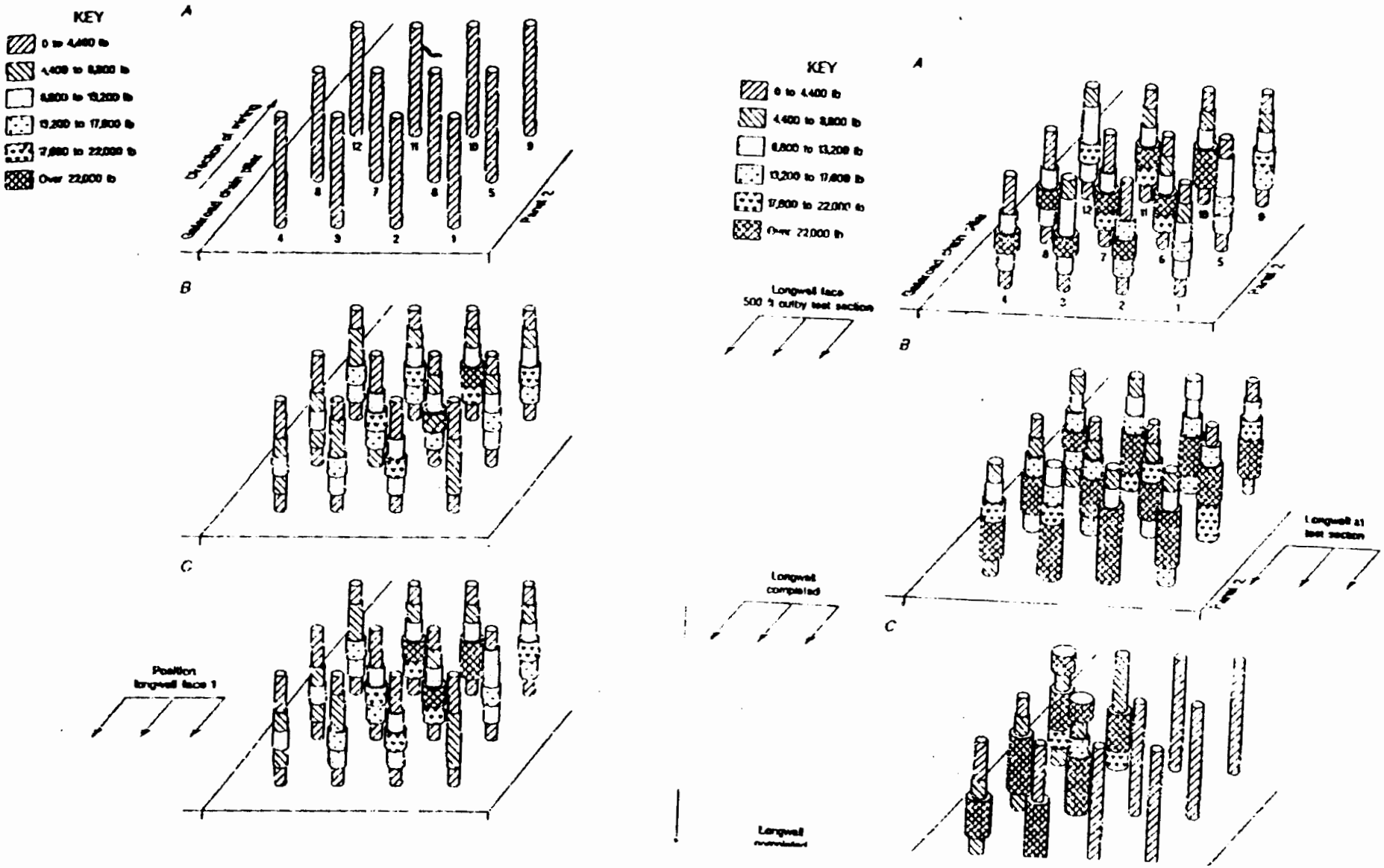

Figure 7 Circumferential load transfer and yield of resin grouted anchors (after Signer and Jones, 1990)

REPORT SUBMITTED

BY

DR. G.B. GRISWOLD

The summary report contains:

- An accurate record of the meetings of the Geotechnical Panel on Panel 1 Stability.

- A copy of the report provided to Westinghouse by this panel member.

- An accurate presentation of the consensus agreed to by the panel members at the meetings on the 23rd and 24th of April 1991.

Panel Member

5/22/91

Date

Revised 4/19/91

RESPONSE TO STATEMENT 1 Panel Member: Griswold

Factors to be Addressed

1. The ability of the Panel to address Statement 1 based on the available information.

   The geotechnical information base is excellent and far exceeds that available to normal mining operations being conducted in the nearby potash mining district. The safety record of those mines is excellent yet their extraction rates are much higher, their bolting pattern not as comprehensive, and pillar and roof loadings are higher.

2. Best estimate for the life of Room 1.

   The eight year life of SPDV Room 1 represents the minimum. Life beyond that is not quantifiable but the installation of rock bolts in Panel 1 will no doubt prolong the time when open access to Rooms 1 through 7 will be available for scientific purposes. Caution: throughout my discussions of room life I mean the time from initial mining to expected collapse. How that time is partitioned between preparation, testing and bin removal will not be discussed.

3. Lower and upper bound estimates for the life of Room 1.

   The age of Room 1 is approaching five years. Using SPDV Room 1 as the minimum model, evidence of the onset of major movement will not be detectable until year 6. Therefore, the true effectiveness of rock bolting must wait for another year. So my estimate has to be judgmental, but adding at least two additional years appear reasonable. Having stuck my neck out on the two years added life makes me conservative on the upper limit -- no longer than four years. I am comfortable with two to four year increased life because the comprehensive geotechnical monitoring that will be available for Room 1.

4. Levels of uncertainty associated with estimates.

   It is only reasonable that as expected life goes beyond the eight year life of SPDV Room 1 that uncertainty increases. If I was forced to give you an estimate I would say 90% certain for the two year increase and 60% for the four year increase.

5. Reasons for levels of uncertainty.

The roof bolts will add to stability, but quantifying it as I just have done with my answer to item No. 4 is pure speculation!

6. Additional information that would be needed to improve estimate.

The only information that I consider useful is something that you cannot provide now, and that is time. Time is required to determine what the deformation plot will look like in a bolted room. Now as to maintenance. I believe that you must be prepared for extensive maintenance on a required basis and the bins removed if necessary. You can hope for the best, but you must be prepared for the worst.

RESPONSE TO STATEMENT 2 Panel Member: Griswold

Factors to be addressed

1. The affect that the changes associated with the test program have on support requirements for Room 1, Panel 1.

   The purpose of the initial design, including the bolting, was to demonstrate drum waste disposal. No meaningful change has been done to accommodate the new mission of scientific tests.

2. The rock load to be supported is approximately the full weight of the roof bean up to the anhydrite "b" layer in the middle third of the span and half this weight over the outer two thirds.

   This is quite reasonable considering the roof failure of SPDV Room 1.

3. The adequacy of the safety factor of the bolting system used in Room 1, Panel 1 to support the design rock load.

   The 1.7 safety factor given by Dr. Cook is correct if the anchors hold and move only by long term plastic flow. If the wedges slip through the shells then the bolts are not effective. Dead weight testing of bolts can give a partial answer this question, and such tests could be done in a few months time. Of all the "would like to do" tests this is my highest priority. The forces on the bolts is a classic statically indeterminate case, but can be solved by finite element analysis. This should be done pronto.

4. The salt above anhydrite "b" will remain competent.

   Yes, it is outside the failure envelope as witnessed in SPDV Room 1.

5. Slippage of anchors provides an acceptable approach to supporting the rock load while accommodating roof closure, with daily access to the room.

   This is the key question. Experience in nearby potash mines says yes for small movements and no for large movements. Dead weight testing should quantify the phemonenology of failure.

6. <u>The mechanism by which the bolt anchors will accommodate the movement of the last while supporting the immediate roof beam.</u>

I believe that some bed separation will still occur. Therefore, the bolts only provide support by suspending the failed portion of the roof. If such will be the case then room closure rates will depart from what was witnessed in the SPDV rooms. This will place a real burden on the geotechnical staff to give an accurate analysis of closure data.

RESPONSE TO STATEMENT 3 Panel Member: Griswold

Factors to be Addressed

1. The extent to which a probabilistic basis for determining risk assessment is presently applied in mining.

   Some academics may do such analysis, but I know of no mine operators that do. You design it the best you can and then do monitoring. Roy Cook's statement No. 4 in his Summary section puts it very well.

2. The qualitative nature of geologic information.

   The advantage of WIPP is its uniformity of the bedded geologic conditions. Therefore, the SPDV geotechnical information is transferable to Panel 1 with a high degree of certainty. This eliminates the qualitative aspects of geologic information that one faces in most mining situations.

3. The extent to which database or experience is available in the mining industry from an operations point of view to provide meaningful judgments used in the nuclear industry (i.e. probabilities of less than 1 in 10^6). This is not to be applied to an assessment of the long term (10,000 year) performance of a repository.

   Impossible!

4. The adequacy of the geomechanical database developed at the WIPP and the methods currently in place to evaluate the performance of openings.

   Excellent in both aspects. The only thing missing is the design and validation of a long term stable mine opening. This was something never considered necessary until the advent of the bin scale test program.

RESPONSE TO STATEMENT 4 Panel Member: Griswold

Factors to be addressed

1. The modifications and additions to the support system needed to insure the completion of the tests.

   I would add nothing other than the monitoring system that has been scoped out by Roy Cook. I do recommend that Jack Gilbert and Harry Bibby be brought more into play concerning the design of a leveling platform for bins and providing as much structural protection as possible over the bins.

2. The maintenance activities that will be needed.

   This is the responsibility of the safety personnel and not the scientific investigators. And it will be done by "take it as it comes" methods.

3. The need to remove the cables for the bin scale tests in order to install additional support.

   I am not that knowledgeable about the test configuration. I would leave these decisions to Jack Gilbert because he will have operational responsibility for the test.

RESPONSE TO STATEMENT 5 Panel Member: Griswold

Factors to be addressed

1. The adequacy of the geomechanical database developed at the WIPP provides an adequate basis to predict and provide early warning deteriorating conditions in Room 1.

   The SPDV experience gives an excellent reference base. However, we are hoping that the roof bolts in Panel 1 rooms will alter the convergence rates. Therefore, a lot of judgment is going to be called for on making the correct decision as to when failure is apt to occur. Added to this is the problem that the deformation history of Room 1 of Panel 1 differs from those in the four SPDV rooms and rooms 2 through 7 of Panel 1. The convergence plot for Room 1, Panel 1 is quite linear versus the early curvilinear behavior exhibited elsewhere. Hopefully some of this dilemma will be answered when the instrument holes are drilled in Room 1, Panel 1. I am told that drilling will commence very soon.

2. The adequacy of the present geomechanical, installed in Room 1 is adequate to provide early warning of deteriorating conditions.

   The answer is no to the exact statement.

3. The adequacy of the proposed additional geomechanical instrumentation to be installed in Room 1 to provide early morning of deteriorating conditions.

   The answer is yes if the instrumentation scoped out by Roy Cook is implemented.

4. The criteria to determine when removal of waste becomes necessary.

   I think it consists of two parts: convergence rate and total convergence. Any rate above five inches per year or total convergence over 25 inches are trip points in my view.

REPORT SUBMITTED

BY

MR. A.T. IANNACCHIONE

The summary report contains:

- An accurate record of the meetings of the Geotechnical Panel on Panel 1 Stability.

- A copy of the report provided to Westinghouse by this panel member.

- An accurate presentation of the consensus agreed to by the panel members at the meetings on the 23rd and 24th of April 1991.

Panel Member

5/23/91
Date

REVIEW OF THE LIFE EXPECTANCY OF PANEL 1 ROOM 1 IN THE WIPP UNDERGROUND

by Anthony Iannacchione

Acknowledgement

I would like to thank the staffs of the Department of Energy, Westinghouse Electric Corporation, and Sandia National Laboratory for the opportunity to evaluate the continued stability of Panel 1, Room 1 at the Waste Isolation Pilot Plant near Carlsbad, New Mexico. I have found the staffs exceptionally well qualified and clearly focused on their mission. I hope the following comments will provide some additional insight and prove useful in any future deliberation of the expected life of Panel 1, Room 1 in the WIPP underground.

STATEMENT NO.1

An estimate can be established for the period of time that Panel 1, in particular Room 1, remains accessible on a daily basis beyond July 1991.

<u>The following cases are considered:</u>

1. No maintenance in terms of scaling of roof, milling of floor or installation of additional support.

2. Limited maintenance without moving bins.

3. Extensive maintenance on an as required basis, with bins removed from room, if necessary during maintenance activities.

<u>Assumptions</u>

1. Room height on July 1, 1991, 13.5 feet and minimum room height needed to support equipment clearances, 10.0 feet.

2. Room initially excavated in July/August 1986.

<u>Factors to be addressed</u>

1. The ability of the Panel to address Statement 1 based on the available information.

Considering the instability observed within SPDV Room 1, a worst case scenario for the expected life of Panel 1, Room 1 has been identified by the WIPP staff. This scenario establishes the potential need to support a 180 ft long triangular shaped roof member. Observations from SPDV Room 1 indicated that the immediate roof fractured after approximately 6 years at the center of a 300 ft long entry along both sides of the salt

ribs/roof intersection. These fractures propagated upward at approximately 20° to 25° from the horizontal until they intercepted a clay band approximately 7 ft above the mine roof. Failure of the detached salt wedge occurs when the shear resistance of the cross sections could no longer support the detached wedge, causing beam failure as a single unit (Cook, 1991).

A roof bolt system consisting of 10 ft vertical bolts was installed with the hopes of prolonging stable roof conditions within Room 1 for an additional 9 years. Unfortunately, the mechanism by which the bolt anchors within the salt roof is poorly understood. The WIPP staff has assumed the bolt anchors will slip downward in response to the ever present creep of the salt formation. Although this mechanism appears quite possible, there is little information confirming anchor slip in salt. If the bolt anchors do not slip, bolt yield or bolt pullout may result. The in-place bolts are capable of withstanding 10 inches of yield prior to failure. Current measurements suggest approximately 27 inches of deformation will occur within Room 1 over a the next nine years.

Horizontal deformations of 0.5 inches per year produce an additional condition not planned for in the design of the bolt system. Vertical bolts passing through roof shears may fail in shear long before they fail in the manner suggested by the WIPP staff. Until these questions are better understood, confidence in the current support plan is estimated to be 50%.

2. Best estimate for life of Room 1.

Estimation of the expected life of an entry as critical as Room 1 should be based upon worst case situations. If the bolt anchors don't slip, the bolt system will fail due to excessive elongation. Additionally, the bolt system may fail due to shearing along the salt roof wedge. Let us examine each of these cases separately.

First, if bolt anchors don't slip the bolts will stretch due to constant deformation of the roof. The deformation in the roof is estimated to be 2/3 of the total room convergence which is approximately 3 inches per year. This indicates that at least 2 inches of deformation per year will occur within the roof strata. The 3 inches per year represents a steady state creep condition, therefore, higher rates would be experienced once shear fractures occur in the roof. Unfortunately, precise knowledge of the exact height at which zero roof deformation occurs is unclear. Let use again consider the most conservative estimate. All of the movement occurs within the bolt horizon. Tension failure of the bolts would likely occur approximately 7 years after installation (assuming an extension of 10 % with a Factor of Safety of 1.5 equalling a total of 7 inches of elongation prior to failure). This would indicate a 2 1/2 year life for current bolt system. Since the bolts have been installed for approximately this long without failure, this scenario seems unlikely. Either some slip is occurring in the anchorage and/or much of the roof deformation is developing far above

the bolt horizon.

If the bolt anchors do slip and/or the roof deformations within the bolt horizon are some fraction of the total roof sag, then the projected shear surface crossing the bolt horizon at a 23° angle must still be considered. Observations within SPDV Room 1 indicate shear fractures began to develop after approximately 6 years of entry life. If we consider the worst case scenario, it should take approximately 2 years for the bolt holes to totally shear. We could then make the assumption that this process would slow the entire development of the unstable salt wedge by an additional two years. This again is an unlikely scenario since some bolt hole deformation will surely occur. Field observations have indicated that a considerable amount of lateral bending can occur prior to shear failure. Unfortunately, precise calculations of these effects are not available (see Hass et al., 1975 for more information on shear strength of roof bolts).

3. Lower and upper bound estimates for the life of Room 1.

The above discussion adequately defines the lower bound estimates for the life of Room 1. Since the roof failed approximately 2 years after roof shears developed in SPDV Room 1 and roof shears have not yet developed in Panel 1 Room 1, the lower bound estimate of roof stability would be July 1993. An upper bound estimate would follow the logic discussed by Cook (1991) where the bolt anchors would slip continuously in response to roof deformation and where the capacity of these bolts to resist shear failure is significantly increased by bolt hole deformation and bolt bending. Therefore, the upper bound estimate would be close to the completion of the test in July 2000.

4. Levels of uncertainty associated with estimates.

Because of the great deal of uncertainty involved in the performance of the intrinsic support system within Room 1, precise levels of uncertainty can not be calculated (note the above statements for a discussion of these uncertainties).

5. Reasons for the levels of uncertainty.

Please see the above statements for the reasons for the levels of uncertainty.

6. Additional information that would be needed to improve estimates.

The author recommends a research program designed to investigate the anchorage mechanism of bolts within the WIPP salt roof. It is recommended that anchor creep tests be performed on salt so that a family of load vs. deformation curves under varying confinements, bolt lengths and widths, and anchor types can be produced. These test should be compared with in situ bolt load, bolt strain, roof sag and entry convergence. In this way, the an accurate mechanism can be established for anchors in salt. Also, estimates of strength of bolts subjected to

shear forces at various angles should also be evaluated along with observations of bolt hole deformations along the shear plane. This information would help to determine what effect horizontal deformation of 0.5 inches per year may have bolt failure. (See Hass et al., 1975 and Smith and Stateham, 1987).

STATEMENT NO.2

The rockbolt system as currently configured, is sufficiently effective to ensure that the test program in Panel 1, in particular Room 1 can be completed.

Assumptions

1. The test program will start in July 1991.

2. The bins CANNOT be disconnected and moved to facilitate maintenance of the rooms.

3. The test program including retrieval will be completed by July 2000.

Factors to be Addressed

1. The effect that the changes associated with the test program have on support requirements for Room 1, Panel 1.

I do not have a high degree of confidence that the currently configured support system in Room 1 will allow for completion of the Bin Scale Test through the projected date of July 2000.

2. The rock load to be supported is approximately the full weight of the roof beam up to the anhydrite "b" layer in the middle third of the span, and half this weight over the outer two thirds.

This appears to be a reasonable estimate. Since the observed cross-sectional area of the roof wedge within SPDV Room 1 was less than that estimated for the rock bolt dead weight load, the support system has a built in Factor of Safety. It is important to note that supports with high load carrying capacities and high stiffness characteristics might produce excessive bending and tensile failure in the salt roof. Figure 1 shows an idealized load deformation for support systems within Room 1.

3. The adequacy of the factor of safety of the bolting system used in Room 1, Panel 1 to support the design rock load.

If the assumptions made in the design are true, the Factor of Safety of the bolting system would be adequate. However, some of the assumptions are in question (see comments in Statement No.1). Therefore, I do not believe an adequate Factor of Safety exists for the current support system.

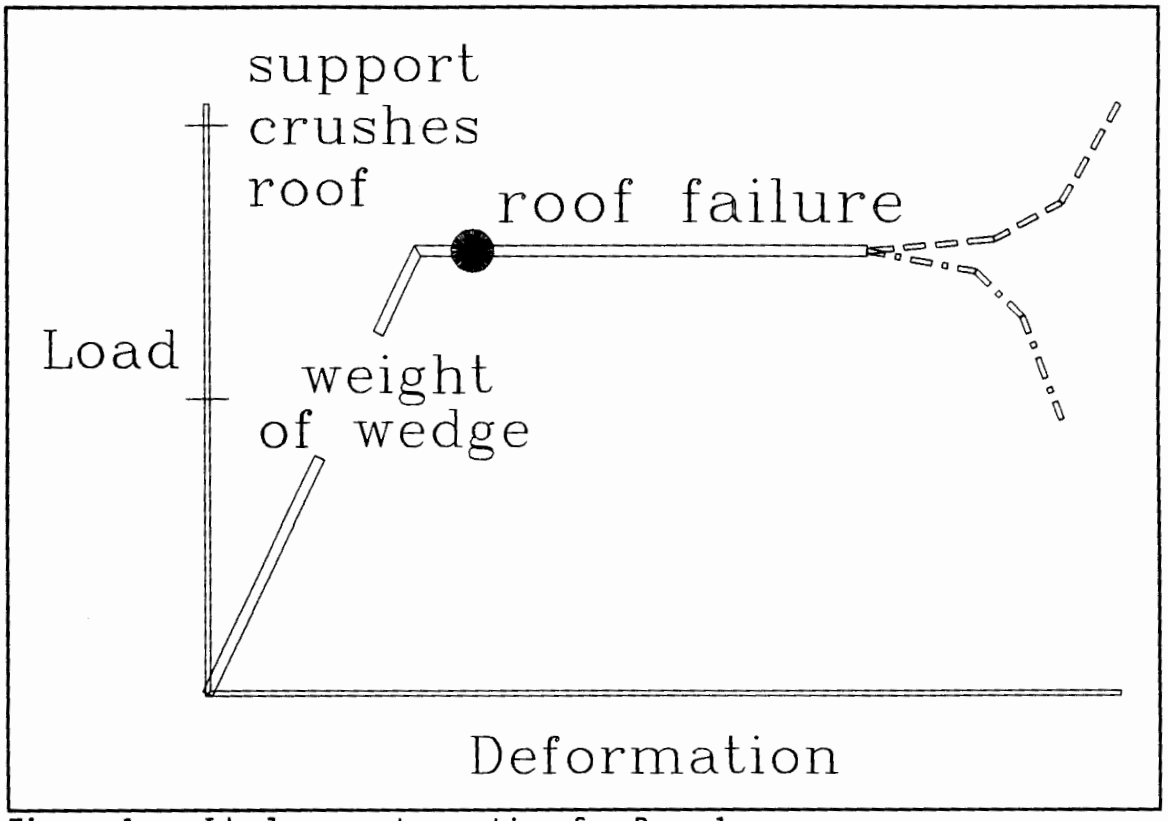

Figure 1. - Ideal support reaction for Room 1.

4. The salt above the anhydrite "b" will remain competent.

The salt above the anhydrite "b" will remain competent until separation along the "b" horizon is initiated. At this point in time accelerated deformations will begin to occur in the horizon below anhydrite "a" (approximately 15 ft above the mine roof). The unsupported span of the salt beam between anhydrite "a" and "b" will be much smaller than that of the salt roof below anhydrite "b". This should greatly reduce the size of the wedge which would eventually form between anhydrite "b" and "a".

5. Slippage of anchors provides an acceptable approach to supporting the rock load while accommodating roof closure, with daily access to the room.

Unfortunately, slippage of anchors is a suggested mechanism and has not been proven. Therefore, I would suggest an extensive research program to verify this mechanism. Also, the shear deformation characteristics of the installed support system needs to be evaluated.

6. The mechanism by which the bolt anchors will accommodate the movement of the salt while supporting the immediate roof beam.

To the best of my knowledge this has never been researched. I have

searched the literature and have been unsuccessful in finding any references which would help verify a mechanism.

STATEMENT NO. 3

The level of confidence that can be placed in the estimate of the life for Panel 1 provided in the response to Statement 1 is in accordance with accepted mining practices.

Factors to be Addressed

1. The extent to which a probabilistic basis for determining risk assessment is presently applied in mining.

The analysis used by the WIPP staff is certainly within the design procedures utilized by the mining industry. However, considering the nature of the WIPP site and necessity for safe storage of waste bins in Room 1, I don't think using risk assessments applied to conventional mines is appropriate. Commercial mines can and do take some risks. The management at WIPP must decide what risks this mine is prepared to take.

2. The qualitative nature of geologic information.

I have a very high degree of confidence in the geologic information collected at the site. Combining this data base with observational and measured strata response has already proven extremely useful.

3. The extent to which a database or experience is available in the mining industry from an operations point of view to provide meaningful judgements at the probability levels used in the nuclear industry (i.e. probabilities of less than 1 in 10^6). This is not to be applied to an assessment of the longterm (10,000 year) performance of a repository.

I refer to my comments in Factor 1.

4. The adequacy of the geomechanical database developed at the WIPP and the methods currently in place to evaluate the performance of openings.

The confidence I have in the geomechanical database developed at the WIPP is very high. The staff has done a great job. I would suggest some minor improvements. First, the ability to separate the magnitude of roof sag from floor heave was not always possible from the data collected at SPDV Room 1. I would suggest more extensometer measurements in conjunction with convergence measurements in SPDV Rooms 3 and 4 and from the various rooms in Panel 1. Some of these extensometers should extend great distances (>50 ft) into the roof. I would also recommend more remote real-time data acquisition so that extensive measurements could be made after rooms are no longer accessible.

STATEMENT NO. 4

Modifications to the support system in Panel 1 can be implemented to ensure that access is maintained to the rooms on a daily basis until the tests are completed.

Factors to be Addressed

1. The modifications and additions to the support system needed to ensure the completion of the tests.

I would highly recommend addition support systems to ensure completion of the Bin Scale Tests. Three general categories exist: destressing, additional intrinsic support, and supplemental support within the entry.

Destressing - Destressing in salt has proven to be highly successful in increasing entry stability. In particular, a destressing program in Room 1 could be designed to cut-off the excessive lateral movements which are responsible for the creation of the unstable roof wedge in SPDV Room 1. Three different

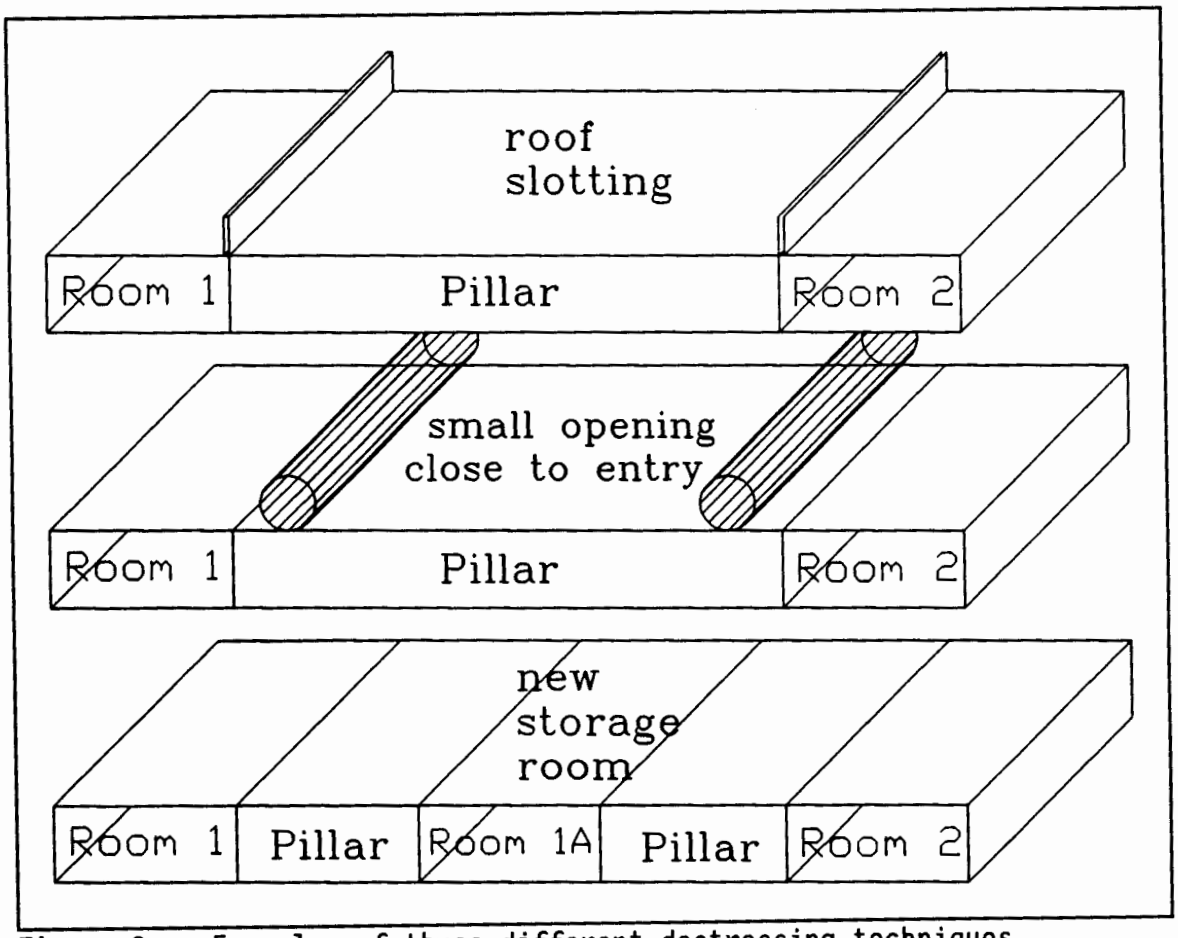

Figure 2. - Examples of three different destressing techniques.

destressing techniques could be utilized (Figure 2). The easiest destressing technique would be to slot the roof with a cutter bar along the rib-roof intersection. This would cut-off the horizontal movement of salt above the pillar from the salt roof above the entry. There are two disadvantages with this technique. Induced spotting of the roof can result in minor instabilities along the slot. Also, the dead weight loading on the bolting system would increase because the potential failure surface may take on a rectangular appearance. This would lower the previously calculated bolt system Factor of Safety.

A second destressing technique would be to drive a new entry (Room 1A) between Room 1 and Room 2, abandoning Room 1 from further use. This would provide two solutions. Room 1 would continue to deform and hopefully fail. Because Room 1A is only 33 ft away from Room 1, the lateral deformation of the roof should be slowed. Also, Room 1A would have a higher probability of remaining stable through the life of the test simply as a result of the "newness" of the entry. The disadvantages of such an approach are obvious. Driving a new room would create other operational problems in Panel 1. In addition, the effects of a 33 ft wide pillar on room deformation at the WIPP site are unknown.

A third suggestion has been made by Mr. Jack Parker. Driving a small opening close to Room 1 at a horizon equivalent to the roof salt. This idea seems most appealing to me. I will leave Mr. Parker to describe this technique in greater detail.

Additional intrinsic support - Three types of additional intrinsic support should be considered: meshing, lacing, and trussing. Clearly a wire mesh should be used in Room 1 to assist in securing small salt pieces. Lacing is a technique I believe Dr. Miller will be discussing in greater detail, therefore, I will not discuss it here.

Roof trusses have been successfully used in the mining industry to stabilize roof subjected to high horizontal movements (Mangelsdorf, 1988). Truss bolts may have the ability to support an existing wedge of salt in Room 1. Several truss bolt systems are currently available (figure 3). The Classic Birmingham truss has the capability to support high loads under considerable deformation. The Locotos truss is a more rigid system but due to the mechanics of the salt wedge this system may be able to withstand considerable deformations. The Seegmiller truss with Dywidag bolts and slip nuts theoretically has the capacity for considerable deformations. Finally, the Dywidag truss has recently been tested at the Beth Energy Mines and allowed 14 inches of vertical movement without

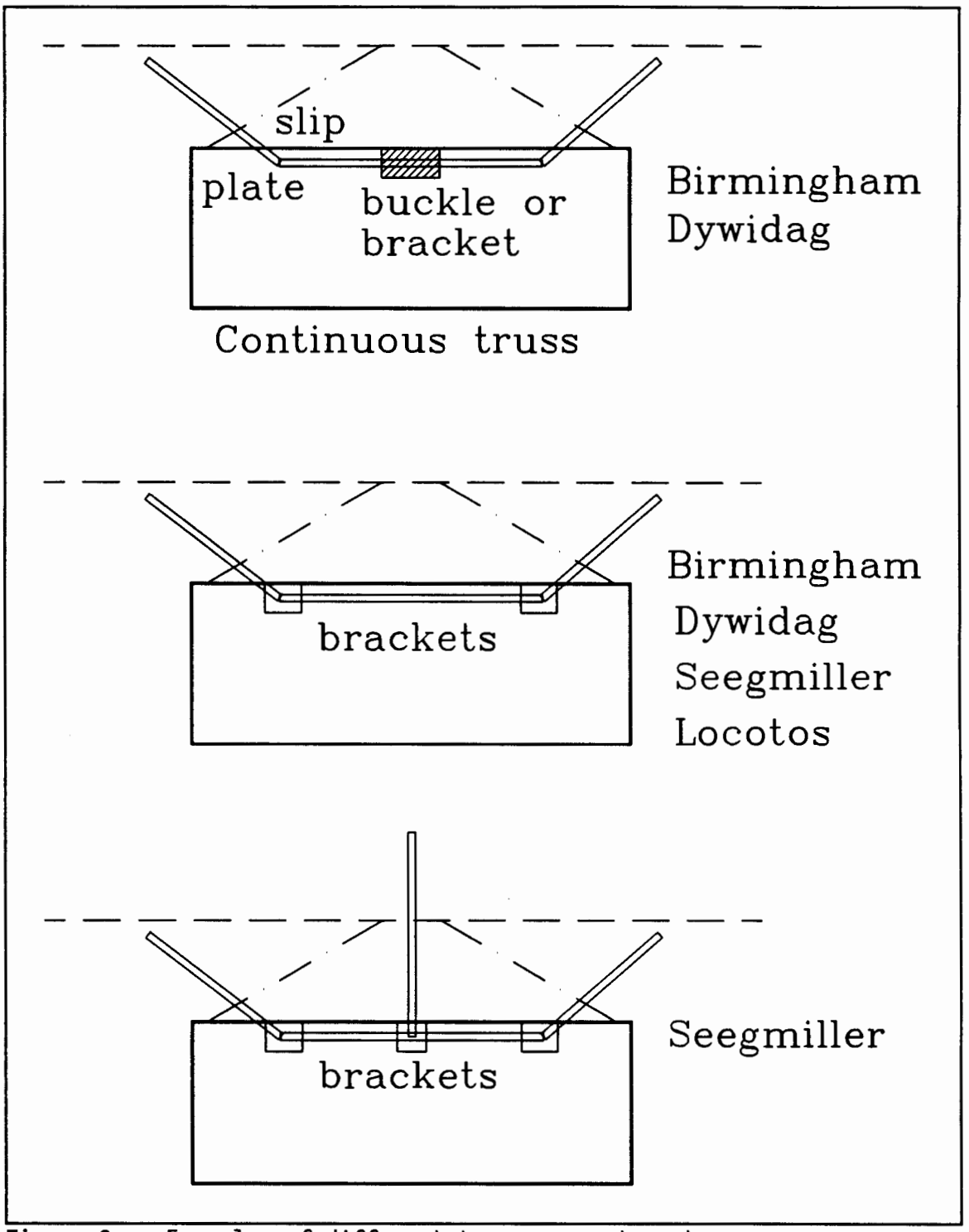

Figure 3. - Examples of different truss support systems.

failure. Five factors should be considered in designing a truss system in salt: 1) the supports should be installed with a small amount of tension; 2) the initial shearing

process should relieve tension in bolt anchors, (due to the lateral movement of the anchor and the downward movement of the roof); 3) the curvature of the roof will generate tensioning in the central rods causing the brackets to slip; 4) oversized holes would allow for more truss freedom of movement across the shearing plane; and 5) a 50% efficiency can be expected. All of the above truss systems will be discussed in some detail in my presentation with comments on the advantages and disadvantages of each technique.

Supplemental supports within the entry - Several types of supplemental support systems exist which could be designed to withstand the 20 to 30 inches of movement Panel 1 is expected to experience over a nine year period.

Wood cribs - Properly designed wood cribs can yield at loads slightly in excess of the dead weight of the salt wedge and mobilize enough deformation to withstand the total vertical movement expected over the 9 year life of the room. The stiffness of crib is dependent upon the height, width and

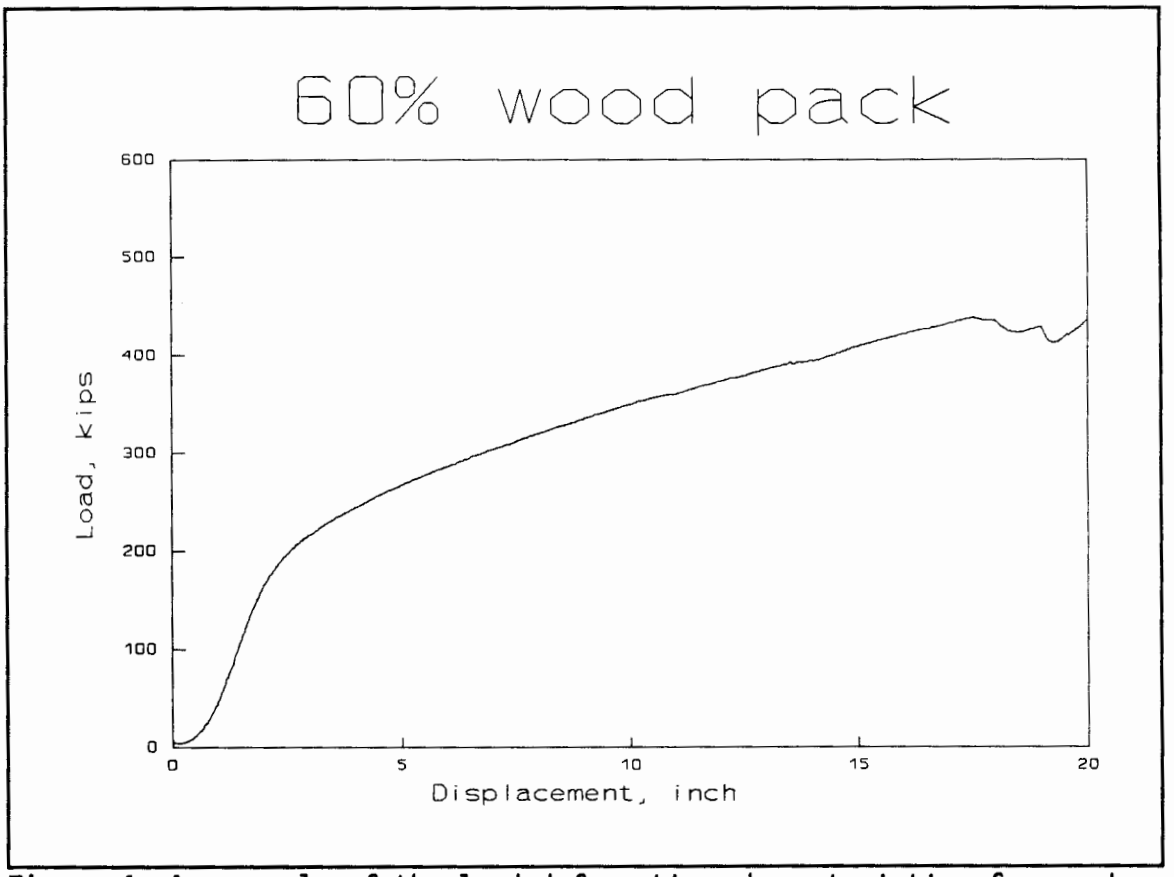

Figure 4. An example of the load-deformation characteristic of a wood crib.

contact area of the crib. It is also affected by the size and character of the individual wood pieces. Testing at the Bureau of Mines has illustrated the behavior of certain size and shape wood cribs (Barczak and Schwemmer, 1988; Barczak and Tasillo, 1988; and Barczak and Tasillo, 1991) and is illustrated in Figure 4.

Yielding jacks - Several manufacturers have yielding jacks that can hold 90000 lbs over 24 to 36 inches of displacement. Dywidag, Seegmiller, and USBM have installed these jacks under various conditions.

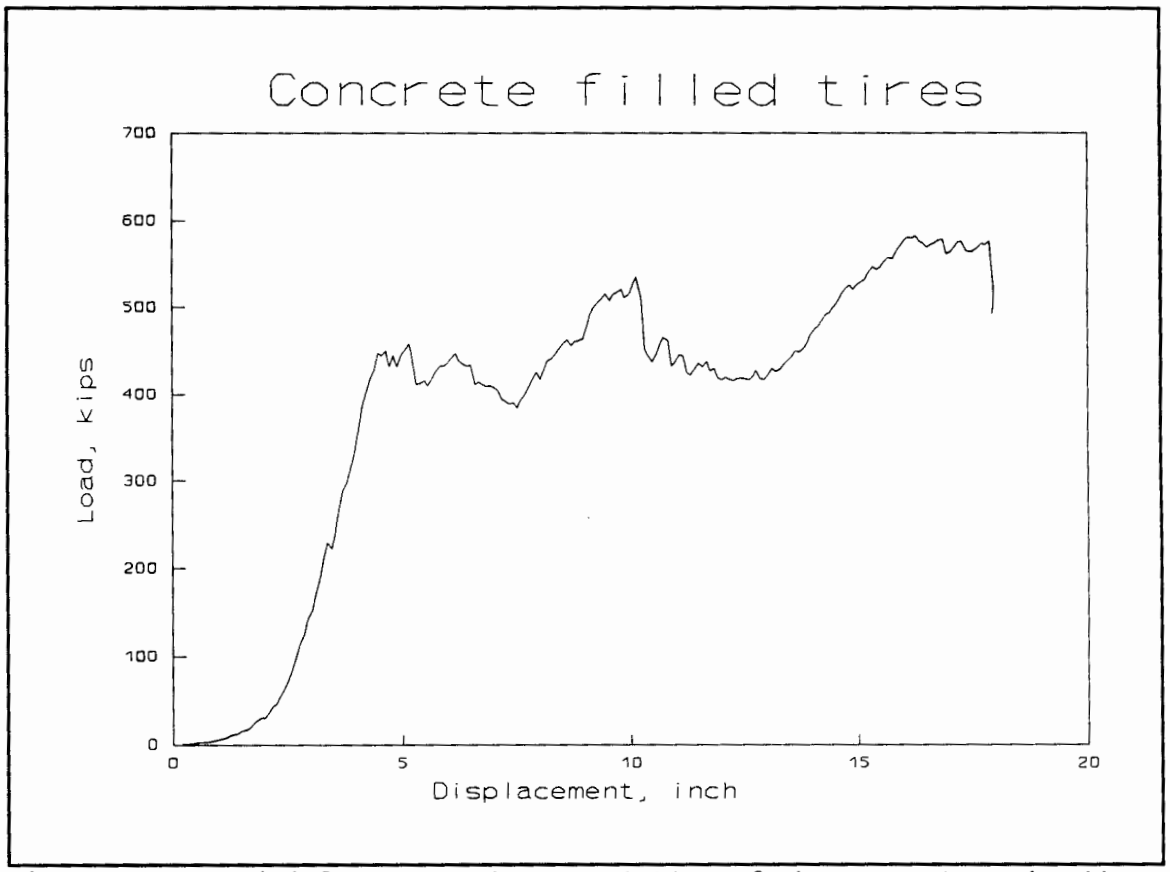

Figure 5. - Load deformation characteristics of the concrete and rubber pier.

Concrete and rubber piers - An experimental concrete and rubber pier has been tested at USBM which has the capacity to withstand large deformation under constant load. An example of the load-deformation characteristic of one of these tests is shown in figure 5.

Arch canopy - Arch supports have been extensively used in mining and civil engineering applications. The advantages of arch supports are: 1) elastic-plastic load deformation

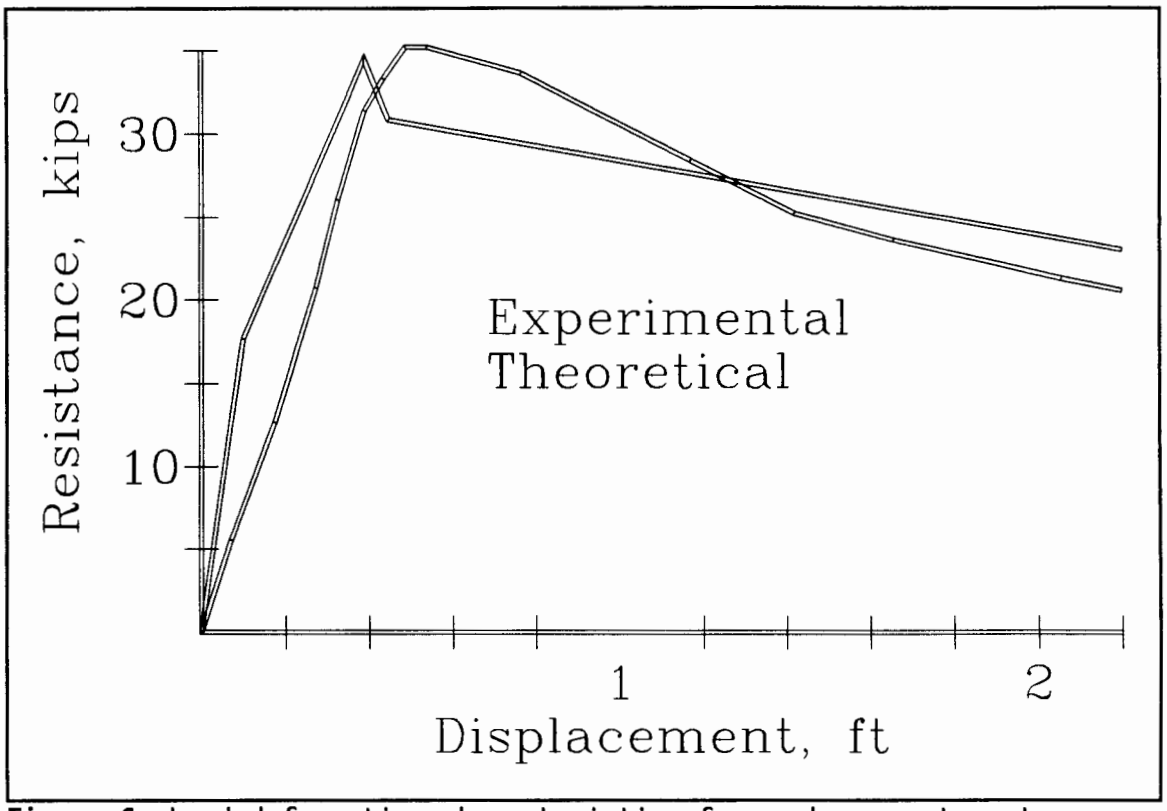

Figure 6. Load deformation characteristics for arch support system.

characteristics (Figure 6); 2) can be placed around existing equipment; 3) come in various shapes; 4) can be fitted to rectangular geometries using the preloaded roof cambered beam system; and 5) can be installed by professional construction crews. The disadvantages of arch supports are: 1) dead loads that exceed ultimate load carrying capacity of the arch could cause sudden collapse; 2) approximately 6 to 12 inches of clearance are needed; 3) the arch structures are heavy; and 4) the yield points of leg supports can be affected by torque, surface conditions and bending of the metal (See Allwes and Mangelsdorf, 1988; and Allwes and Mangelsdorf, 1990).

2. The maintenance activities that will be needed in the room.

Several temporary support systems could be used which would supply additional stability during maintenance activities.

Air bags - Air bags have been used extensively in civil engineering applications to hold unstable strata. These devices lack the ability to withstand large deformations but may prove useful in temporarily stabilizing hazardous ground.

Spring support systems - Spring support systems have also been extensively used in civil engineering applications to allow structures to deform under constant load. Placement around critical devices in Room 1 could provide adequate stability during retrieval of waste containment units.

Mobile roof support system - The USBM has developed a remotely operated Mobile Roof Support machine which can place and retrieve temporary roof support. This device would prove useful in the installation or retrieval of some above listed support techniques.

3. The need to remove the cables for the bin scale tests in order to install additional support.

I am not convinced that this would need to be considered in light of some of the techniques discussed above.

STATEMENT NO. 5

The geomechanical monitoring program and the routine observations in Panel 1, can provide sufficient warning to allow the timely retrieval of the waste from the Panel.

Assumptions

1. In an emergency, all waste can be removed from the room within a 6 month period.

Factors to be Addressed

1. The adequacy of the geomechanical database developed at the WIPP provides an adequate basis to predict and provide early warning of deteriorating conditions in Room 1.

I believe the installed geomechanical database developed at the WIPP provides an adequate bases to predict deteriorating conditions within Room 1.

2. The adequacy of the present geomechanical instrumentation, installed in Room 1 is adequate to provide early warning of deteriorating conditions.

The current geomechanical instrumentation in Room 1 should be supplemented with devices to monitor roof support behavior. The ability to provide early warning of roof falls will need this additional information.

3. The adequacy of the proposed additional geomechanical instrumentation to be installed in Room 1 to provide early warning of deteriorating conditions.

An early warning of roof failure should consist of 4 parts: 1) strata deformation measurements, 2) geophysical measurements, 3) support reaction measurements, and 4) observational data.

4. The criteria to determine when removal of waste becomes necessary.

All of the above information should be utilized by the mine management to assess the potential for impending instabilities. However, I strongly recommend that a rigid procedure for making this determination be avoided. The information should supplement the decision making process, not dictate the process. Mine management should have the flexibility to base its decisions on the opinion of its experts not the trends of its instruments.

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VENDORS

F.M. Locotos Co., Inc.
133 Camp Lane
McMurray, Pa
15317-2601
412-941-5701

Dywidag Systems International
Avon Lake, Ohio (Larry Long) 216-933-7965
Ray Barndon 303-241-0885

Ben Seegmiller
Western Support Systems
143 South 400 East
Salt Lake City, Utah 84111
801-363-1001

Gerb Vibration Control Systems
900 Oakmont Ln.
Suite 207
Westmont, Il. 60559
312-654-0790

Birmingham Bolt Co.
P.O. Box 1208
Birmingham, Al 35207
Henry Leonard, VP Marketing 205-985-0290

MatJack Air Bags, Indianapolis Industrial Products, Inc.
1428 Sadlier Circle East Dr.
Indianapolis, IN 46239
317-359-3078

REPORT SUBMITTED

BY

DR. S.D. MCKINNON

The summary report contains:

- An accurate record of the meetings of the Geotechnical Panel on Panel 1 Stability.

- A copy of the report provided to Westinghouse by this panel member.

- An accurate presentation of the consensus agreed to by the panel members at the meetings on the 23rd and 24th of April 1991.

Panel Member	Date

Signed off sheet from Dr. S.D. McKinnon was unavailable at the time of publication.

ASSESSMENT OF
PANEL 1, ROOM 1 STABILITY — WIPP SITE

Prepare for:

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April 1991

1.0 INTRODUCTION

On February 4, 1991, a substantial roof fall occurred in Room 1 of the Site and Preliminary Design Validation (SPDV) area. At the time, the Room had been open for eight years. Similar signs of deterioration, but at a less advanced stage, can currently be seen in adjacent rooms which are of approximately the same age, dimension and lithology. A second experimental area, designated as Panel 1, has been excavated to similar specifications but at a later time such that the age of rooms in Panel 1 is approximately three years less than those in the SPDV area. Room 1 of Panel 1 is designated to receive waste for experimental purposes, and therefore questions regarding its stability have been raised.

At the request of Westinghouse Electric Corporation, this report is being prepared in order to address a series of specific questions related to the stability of Room 1 Panel 1. These questions are enclosed as Appendix 1 for reference.

In order to provide background information on which to base the stability assessment, a review meeting was held in Carlsbad on April 9 and 10, 1991, which included an underground tour at the WIPP site. A period of approximately one week was then provided in order to complete the assessment and reporting. The approach and rigor of this assessment must necessarily reflect this brief allocation of time.

1.1 Methodology

The strata in which the Rooms are located are primarily comprised of halite, with nearby thin beds of clay and anhydrite. Creep is a significant factor in the deformational characteristics of these strata. More importantly, from a stability point of view, fracturing also occurs. Development of fractures in a creep susceptible material complicates considerably the ability to understand and predict rock mass behavior. Either creep or brittle failure can be modelled with available computer codes, but currently there is no constitutive model available to allow simulations to be made of a creep susceptible material that can also develop fractures over time.

Practical experience is available from coal, potash and salt mines (brittle rock under high stress also exhibits creep if sufficiently fractured) in terms of support practice, excavation geometry effects and the like, which can be applied to the WIPP site. However, in order to make use of this collected experience, it is essential to understand at least in qualitative terms, something of the fundamental mechanics of the way in which

the rock mass is behaving at the WIPP site in order that the correct experience is borrowed.

In order to address the specified questions in a meaningful manner, therefore, a conceptual model of the rock mass behavior will be proposed, based on observations from site. The model will be qualitative and incomplete, and this will be accounted for in the manner in which the questions are answered.

2.0 CONCEPTUAL MODEL OF ROCK MASS BEHAVIOR

Fracturing and deformation of the rock mass around test rooms has been documented and discussed by various authors (e.g. Stormont 1990, Cook 1991). This information provides a reasonable understanding of the mode of rock mass behavior. The next step in understanding the rock mass behavior is to develop a conceptual mechanistic model to explain why the observed mode of behavior occurred.

The data on which the model will be based has been obtained from numerous excavations at the WIPP site. Due to the high degree of geological uniformity at the site, and the observation that similar modes of behavior seem to be occurring in all rooms of similar dimension, it is reasonable to generalize the data in developing a single representative model for the rooms in Panel 1 and the SPDV area. As a starting point for the discussion, reference is made to two Figures summarizing observed modes of behavior.

Based on visual observations and instrumentation data, Stormont (1990) illustrates the typical fracture patterns observed around test rooms, Figure 1. Roof fractures are absent from his sketch, possibly because they had not developed at the time his observations were presented. Cook (1991) also shows in sketch format, Figure 2, the main aspects of deformation and fracturing observed in Room 1 of the SPDV area where the ground fall occurred. To assist in developing the conceptual model, various factors affecting the observed behavior will be discussed separately in the following sections.

2.1 Effect of Stress and Room Geometry

Based on the results of in situ stress measurements, the virgin stress field at the WIPP level is hydrostatic. This is expected in creep susceptible rocks which deform in order to minimize and dissipate shear stresses. Nominal dimensions of test rooms are 33 ft width, 13 ft high and 300 ft long. Figure 3 illustrates the main features of stress redistribution around an opening of width to height ratio 2:1. The slightly higher aspect ratio of the test rooms will show similar patterns. The most significant aspect of the resulting stress field is the development of high shear stresses near the excavation corners. Tensile stresses are induced in the roof and floor, but overall, the total stress field in these areas remains compressive.

For salt, as with most rock types, areas of high shear stress will be more susceptible to fracturing than areas subjected to more uniform compression. Creep also occurs in zones of high shear stress, but not in zones of pure compression.

A correlation has been made between drift span and the number of boreholes in which fracturing was observed. This correlation is shown in Figure 4. As span increases, there is a significant increase in degree of fracturing observed. From the underground visit, it was also noted that the smaller span access drifts did not exhibit the fracture development seen in rooms with 33 ft spans.

It is possible, therefore, that some threshold level of shear stress is exceeded in the larger span rooms which leads to fracturing in addition to creep behavior, rather than creep alone. It is also possible that the lower shear stress level in smaller span drifts results in fracturing taking place over a longer time period. The important inference is that it may be possible to limit the onset or rate of fracture growth by reducing the magnitude of shear stress to levels found around excavations of smaller span.

2.2 Effect of Geology

The stress distributions referred to in Figure 3 are for a homogeneous isotropic linearly elastic material. Figure 5 shows the stratigraphy in the vicinity of the WIPP excavations, which is far from homogeneous. Stress distributions will be affected by geology due to variations properties such as stiffness and strength. The distribution of creeping versus non-creeping materials will also affect the stress distribution with time. This is important where anhydrite is adjacent to salt, as with marker bed MB139.

Instrumentation results show that slip occurs on clay seams located near the excavations i.e. at the anhydrite "a" and "b" seams above the roof and along the lower boundary of marker bed MB139 below the floor. Shear slip is also a means of dissipating shear stress and altering the flow of stress around the excavation. Additionally, the immediate roof and floor will act as "beams" rather than as continuous portions of the rock mass. Stormont (1990) shows that shear displacements also occur on a clay seam located between approximately 32 and 38 ft above the roof of the test rooms. This observation illustrates the potential low strength of clay seams and the extent of influence of the excavation.

Fracturing of marker bed MB139 is probably related to its higher stiffness, and lack of creep behavior. Despite the salt layer in the immediate floor being of lower strength, higher stresses could be induced in the anhydrite due to its higher stiffness. Shear stresses induced in the salt layer could dissipate through creep leaving the higher strength anhydrite to bear an increasing load. Subsequent fracturing of the anhydrite would in turn result in its load carrying capacity being reduced. Load would then again be transferred to the relatively thin beam of salt in the floor. Observations indicate that fracturing of the salt beam does occurs after the anhydrite has become fractured. The role of weak parting planes, particularly that of anhydrite "b", is therefore important to account for.

2.3 Effect of Creep

The ideal excavation shape in a hydrostatic stress field is circular. This shape has the least concentration of shear stresses. Rooms in the SPDV and Panel 1 areas have a relatively high aspect ratio which results in high shear stresses near the corners of the excavations. Creep occurs most quickly where there are high shear stresses, and over time, creep will tend to reduce shear stress magnitudes and gradients.

As creep occurs, displacements, or strains, occur. From laboratory test results, the rate of strain is also greatest in areas of high shear stress, and it is reasonable to assume that if the rate of strain is high enough, the material behavior will be brittle rather than ductile. Brittle failure, or fracturing, will therefore most likely occur in those areas of highest shear stress. This is consistent with the locations of observed fracturing around the excavations.

If laboratory data on the effect of loading rate on the behavior of salt is available, it may be possible to correlate this with shear strain rates predicted from computer models of rooms of various sizes. These results could then be compared with observations.

There are many other aspects of creep which could influence the stability of the strata immediately around the experimental rooms, such as the way in which the "stress arch" may migrate away from the roof and floor strata. Horizontal compressive stresses are known to be beneficial in negating the effects of induced tensile stresses. Movement of these confining stresses away from the excavation roof may contribute to the time dependent stability problem.

These effects cannot be addressed in this study, but should certainly be considered in a more thorough analysis.

2.4 Effect of Rockbolts

Mechanically anchored rockbolts of 10 ft length were installed in Room 1 Panel 1 two years after the completion of excavation. No effect on convergence has been seen. There is concern, therefore, as to the effectiveness of rockbolts in arresting the development of potential failure surfaces in the roof, or indeed, their ability to suspend a wedge in the roof should it become detached.

The problem of rockbolt effectiveness is of great importance as rockbolts are traditionally the most common means of stabilizing potentially unstable rock conditions. Unlike passive support such as cribs or packs placed against the surface of the excavation, rockbolts are not invasive and would not interfere with the movement of personnel or machinery in the rooms.

Considering the creep behavior of salt, however, mechanically anchored bolts are not believed to be a good choice of bolt type. These bolts develop their load carrying capacity by generating high contact stresses at the anchorage. This could be relaxed with time through creep. Also, as the rock contained between the anchorage point and the face of the excavation also creeps, it is quite possible that the bolt would be kept in tension. This process would cause the anchor wedge to push the shell into the salt, which could eventually result in the wedge pulling through the shell resulting in complete failure of the bolt. For a number of reasons, therefore, it is considered that mechanically anchored bolts are a poor choice for use in salt.

Building on the concept that salt creeps in response to shear stresses, it is most likely that bolts which generate their anchorage by inducing only small shear stresses would be most effective. In mining, long bolts are normally replaced by cable bolts, which are basically long grouted cable ropes. When this type of bolt is loaded, shear stress builds up along the grout/rock interface being largest near the load and decaying along the length of the bolt. In a creep susceptible material, it is most likely that the shear stress would migrate along the length of the bolt with time to a more favorable distribution, and that the resistance of the bolt could be maintained for a long period of time relative to the desired experimental time frame.

Strategic location of rockbolts in a stress field with a high compressive stress normal to the axis of the bolt would also fa-

cilitate the maintenance of a high bond shear stress. This is an aspect which can be readily addressed by conventional numerical modelling.

With regard to the mechanical bolts currently in place, it quite likely that they are not very effective. Even if the load is being maintained, the mechanism by which the fractures in the roof develop is unlikely to be affected at all by the bolts. This mechanism will be enlarged on below at which point the effectiveness of the bolts currently in place will be discussed again.

2.4 Numerical Modelling of Experimental Rooms

As stated above, it is not possible at this time to develop a model incorporating both creep and brittle fracture behavior. Instead, a simple elastic model will be used. Since the rate at which excavations are mined is fast relative to creep time constants, the elastic state of stress will most likely provide a reasonable approximation to the state of stress existing in the short term. While it is recognized that it is not necessarily the correct model to use, it will be of use for conceptual purposes. In conjunction with some of the concepts discussed above, it is believed that a reasonable interpretation of the mechanics of the room behavior can be made. The particular code used is FLAC, developed by Itasca Consulting Group Inc.

Figure 6 shows contours of principal stress difference (actually twice the maximum shear stress) around an excavation with the same geometry and stress conditions as those in Panel 1. The model boundaries are more extensive than shown as only the area of immediate interest is shown in the figures. Also, due to the use of symmetry, only one quarter of the excavation is shown. For the immediate purpose of this discussion, absolute magnitudes of stress (in psf here) are unimportant. Rather, it is relative states of stress between the various models that will be discussed.

The figure shows that as expected, the largest shear stresses are found at the corners of the excavation. Qualitatively, the shape of the high shear stress "bulb" does in fact angle upward over the roofline. Figure 7 shows principal stress vectors for this same model. This is included for reference purposes.

A significant effect on the stress distribution shown in Figure 6 is brought about by introducing a plane with low frictional resistance at the position of the anhydrite "b" layer. As a coarse approximation to this clay layer, a plane of zero fric-

tional resistance is modelled. Figures 8 and 9 show shear stress contours and principal stress vectors, respectively, for this model. The horizontal line above the room marks the location of the slip plane. When compared to Figures 6 and 7, the effect of the slip plane is seen to relieve shear stress and to change the orientation of principal stresses.

As a result of this process, the shear stress magnitude in the corner of the roof (slightly over the excavation) is increased, and the roof beam formed is placed in a state of higher uniaxial compression. These two factors alone could aggravate the development of shear fractures in the roof due to high shear strain rates. Floor fracturing could also occur through the same mechanism, but will be complicated by the effect of marker bed MB139 as described above. Floor stability is not as important as that of the roof, therefore, attention will be focussed mainly on roof stability.

Given the uniformity of the clay layer at the anhydrite "b" location and the high magnitude of shear stresses developed, it is likely that, at least above the corners of the rooms, some amount of shear slip occurs during excavation. Instrumentation would not be able to see this slip as it would already have occurred prior to installing any instrument. In the same manner, the driving mechanism of generating the high shear stress would be unaffected by rockbolts, which would also be installed after excavation.

To account for the asymmetrical development of the shear fractures in the roof, it must be recognized that a rock mass is not uniformly strong. Spatial variations in strength will most likely lead to the initiation of fracturing at random points along the roof edges. Once a particular fracture has propagated up to the anhydrite "b" layer it would be arrested. Growth of the fracture on the opposite side of the roof would probably not be arrested however, as the driving compressive stresses could still be transmitted around the edges of the fracture surface. Completion of the fracture to the anhydrite "b" layer would therefore not necessarily result in the creation of a "stress relieving" surface. Flow of stress in the roof beam would become quite complex, certainly more than can be reasonably deduced here.

Creep is also believed to form an important part of the driving mechanism for sustained fracture growth in the roof. Referring to Figure 8, high shear stresses exist in the sidewalls of the room. Fracturing has been observed in the sidewalls (see Figure 1) which is consistent with this point, but the sustained driving mechanism for roof failure is probably rooted in the lateral creep of salt caused by high sidewall stresses . The sidewalls

of the room effectively behave as high stress pillar edges. Lateral creep can be thought of as being induced by slow foundation heave, a phenomenon observed also in non creeping rock. The most important aspect of this mechanism is that even if fractures in the roof beam have formed a complete wedge, as shown in Figure 2, continued lateral movement of the sidewall foundation would continue to push inwards, thereby driving the roof wedge downwards.

One further aspect of asymmetrical fracture growth which should be considered is the possibility of forced cantilever bending. The "intact" side of the resulting cantilever would be bent as the opposite side becomes forced down by the inward movement of the sidewall part of the roof wedge. This process could lead to induced tensile stresses on the upper side of the roof beam on the cantilevered side of the beam. Only careful observation of the fracture surface growth and displacement of the roof beam would completely resolve the mechanism.

Shear dislocation on the fracture surface would occur during downward dislocation of the roof wedge, and the effect of this on rock bolt integrity must be considered. Also, once the beam has been sheared through on both sides to form a wedge, the horizontal stresses in the roof beam could be greatly reduced. These horizontal stresses could act to stabilize the lateral deformation of the sidewall foundation, and once the restraining pressure is relieved it might be possible that lateral creep would accelerate. Convergence measurements prior to failure showed an acceleration in the rate of closure, but the complicated nature of failure processes may involve other mechanisms.

2.5 Summary of Main Aspects of Failure Mechanisms

This section highlights some of the more important aspects of the preceding discussion. The proposed mechanisms should be considered hypothetical at this stage, but there is some consistency with observed behavior and general knowledge of salt and rock behavior. It is strongly recommended that access be allowed to Room 1 of the SPDV area to inspect in detail the collapse surfaces. Valuable information on the failure process may be gained.

i) High shear stresses are induced in the corners of the rooms due to the width to height ratio in a hydrostatic stress field.

ii) The clay seams located near the rooms allow slip to take place, increasing the concentrations of shear stress and delineating beams in the roof and floor. As part of the stress redistribution caused by slip, horizontal stresses in the beams are increased.

iii) Higher horizontal stresses are induced in marker bed MB139 due to its higher stiffness and non-creeping behavior. This leads to fracturing and dilation beneath the rooms.

iv) As failure occurs in MB139, more load is transferred to the thin salt beam in the immediate floor. As a result of this, high shear stress will be induced in the ends of the floor beam near the sidewalls.

v) Fracturing of salt will occur if some critical shear stress level is exceeded, as the rate of strain will be higher than a level which can be accommodated by creep. This will lead to fracture growth and initiation in the floor beam prior to the roof beam. Fracturing will initiate preferentially on the boundary of the room and propagate inward, rather than initiating within the rock mass. This is significant in that roof instability should not be a problem until a reasonable amount of fracture growth is seen on the surface of the rooms.

vi) Fracture growth in the salt in both roof and floor will probably be asymmetrical due to variations in local strength. It is most likely, however, that even when a fracture on one side of the room has propagated completely through the beam, the flow of horizontal stress around the fracture surface in the longitudinal axis of the room will still lead to continued fracture growth. The existence of the fractures does not necessarily lead to stress relief.

vii) Creep of the sidewall foundations provides a sustainable driving mechanism to push the resulting roof wedge down. Separation of the wedge from the overlying clay seam would be expected. Along with shear displacement on the fracture surfaces, dilation related opening would be expected.

vii) Mechanically anchored rockbolts are not the most suitable type of bolt to provide long term support resistance in a creeping material. The rockbolts currently installed in Panel 1 Room 1 are probably not be contributing much in arresting fracture initiation and growth as they will not affect the magnitude of shear stresses responsible for fracturing.

viii) The length of rockbolt currently used (10 ft) is considered too short to provide good anchorage above the anhydrite "b" layer. Longer fully grouted bolts should be considered. Advantage could be taken of placing rockbolts in zones where there would be a compressive component of stress normal to the axis of the bolt.

ix) The effect of creep on fully grouted rockbolts or cable bolts needs to be examined before reliability figures can be assigned to their sustained support resistance.

xi) The effect of creep on redistribution of stress in the time frame of the required room life should be examined. It is not expected to drastically alter the picture presented above, but is required for a better understanding of the problem.

xi) Due to the qualitative nature of the conceptual model, quantitative assessment of stability cannot be addressed. Experience gained from other sites will not necessarily apply to the WIPP site unless similar mechanisms are at work. To assign confidence levels in terms of a probability to any recommendations cannot be done. It is possible, however, to make qualitative statements regarding confidence levels, which is commonly the case with engineering judgment, but the probability of the outcome must remain unquantified.

3.0 SUGGESTED REMEDIAL ACTION

Having defined at least in qualitative terms some aspects of the basic mechanisms operating at the WIPP site, a more realistic approach to developing remedial measures can be taken. The problem at hand is to carry out some action which will increase the expected life of Room 1 Panel 1. The solutions to be discussed will be limited to being applicable to the existing rooms and not to future room layouts.

From the preceding discussion, high shear stresses in the roof near the sidewalls cause fracture initiation and growth. The main factors in causing the shear stress are the aspect ratio of the rooms, the stress field, and the slip on anhydrite "b" delineating a beam. It is not possible to do anything about the anhydrite "b" layer in the existing rooms, but the stress field can be changed by further excavation or slot cutting. Support in terms of longer grouted anchors will also play an important role.

Attention will be focussed on roof stability. Floor instability is not as important, and evidence suggests that floor fracturing in Room 1 is sufficiently advanced that a wedge has already been formed. The floor component of convergence is not available at the time of writing, but it has been suggested that after approximately five years, this component reduces. Likewise, sidewall stability is not seen to be a problem and will not be considered.

3.1 Some Possibilities

Categories of remedial action are:

i) Cutting slots.

ii) Excavating nearby openings.

iii) Additional support.

Additional support will result in the least disruption to the current experimental program, but it does not eliminate or change the reason for the development of failure. However, it is recommended that additional support should be installed as soon as possible, but it should be done in conjunction with action to modify the stress field. Areal support such as mesh would also be of benefit in containing loose material, which would increase personnel safety and help to reduce maintenance such as scaling.

Due to the large size of the pillars separating rooms, the option to excavate nearby openings is viable. The purpose of these

rooms would be to result in a reduction of shear stress concentrations in the experimental room boundaries. For example, a small excavation located a short distance into the sidewall could result in the intervening pillar acting as a yield pillar. Figure 10 illustrates the concept. The yield pillar would have to be of sufficiently small width that its load carrying capacity would be limited. Foundation stresses would therefore be limited which would in turn limit the development of shear stresses. Shear fracturing would still develop in the "sacrificial" rooms, but should failure of the beam take place, it would be supported by the yield pillar, and of course, by rockbolts.

A number of variations of this layout are possible, such as increasing the height of the sacrificial rooms up to the anhydrite "b" layer and down to the clay seam below MB139. This would effectively isolate the roof and floor beams, but could lead to the formation of an additional roof beam between anhydrite "b" and "a". The consequences of this action cannot be accurately predicted at this time.

A further possibility for use of additional excavations would be to create rooms of smaller span in the middle of the existing pillars. As shown in Figure 4, a reduction in room span results in more stable conditions. The smaller span rooms could be used for experimental purposes due to their longer anticipated stable life. This solution, however, would not stabilize the existing rooms.

In general, excavation of additional openings to alter the stress field is conceptually sound, but contains numerous practical difficulties. Given the circumstances at the WIPP site, it is unlikely that these solutions could be carried out with sufficient reliability to provide the desired effect. In a mining environment this would not necessarily be a problem as some degree of experimentation with remedial measures is often carried out. This flexibility may not exist at the WIPP site.

Slots cut into the sidewalls or roof can affect the distribution of stress significantly without the need for additional excavation. Slot cutting in salt could be done using standard equipment used in coal mines. Slot depths of 8 ft to 10 ft could quite easily be mined, and with simple modifications, deeper slots could be cut.

For the rooms in Panel 1, slot cutting offers a relatively simple means of changing the stress field. In conjunction with additional rock support, this option seems to offer an effective means of extending the life of the rooms.

3.2 Location of Slots

This section considers the relative merits associated with placing slots at various positions in the rooms. Use was made of numerical modelling to carry out this comparison. As before, simple elastic models were used. In each case, a slip plane was placed at the locations of the clay seams above and below the rooms. Due to symmetry, only half of the actual geometry is modelled, with the vertical centerline of the rooms being taken as a plane of symmetry.

Figures 11 and 12 show contours of principal stress difference (twice the maximum shear stress) and principal stress vectors, respectively, around a room with the same geometry, stress and boundary conditions as the rooms in Panel 1. A window containing only the area of interest is shown, and actual model boundaries extend further away from the room. Reference will be made to these figures when examining the effects of placing slots at various locations.

3.2.1 Horizontal Slot in Sidewall at Roof Level

Figures 13 and 14 show principal stress difference contours and principal stress vectors, respectively, for the case of a horizontal slot of length 10 ft placed in the sidewall at the roof level. In comparison to Figures 11 and 12 the effect of the slot is to shift the zone of high shear stress in the roof beam into the sidewall above the slot. If a deeper slot had been cut, the shear stress would be shifted further in. Due to consideration of the thickness of the roof beam, however, a 10 ft slot is considered adequate depth.

The magnitude of the shear stress is essentially unaltered from the case where no slot is used. Similarly, horizontal stress magnitudes in the roof beam are not significantly affected. At the location where the shear stress fractures tend to develop, however, shear stress magnitude is significantly reduced. It would be expected, therefore, that further growth of shear fractures over the edge of the roofline should be arrested. New initiation and growth of shear fractures would be expected to start above and at the back of the slot.

Given the time period over which the roof fracturing take place, namely, visible initiation after approximately five years, and a growth period beyond that, overall failure would be delayed by this strategy.

In addition to the slot, a significant support effort in terms of long grouted cables should be implemented. Support of the separated roof beam could therefore be provided for, should this occur during the period in which the experiments are being carried out.

A further aspect of support which should be considered is the slot itself. The width of the slot will be less than 1 ft when cut. This dimension could be modified if desired, but it is possible that due to convergence, contact could again be re-established between the top and bottom surfaces of the slot. This is not entirely undesirable, as a reintroduction of normal stress will reduce the shear stress concentration around the tip of the slot. Provided the slot surfaces do not become locked, i.e. they would slip, high shear stresses would not be regenerated. Also, should the fracture at the back of the slot propagate completely through the roof beam on both sides of the room to delineate a roof wedge, the sides of the wedge would become supported by the lower half of the slot even if considerable slip of rockbolts would occur.

If the roof wedge were to rest on the lower surface of the slot, it would be necessary to ensure that the weight of the wedge would not result in failure of the sidewalls. This mode of failure is not likely to occur, but should be examined more carefully if this option is to be implemented.

The advantages of horizontal slots are therefore:

i) Shear stress is relieved at the roof/sidewall location and transferred into the sidewall.

ii) Shear fracturing at the edge of the roof should be arrested due to the large reduction in shear stress magnitude. Shear fracturing at the end of the slot would probably initiate, but would not be of concern for a number of years.

iii) The lower half of the slot could provide support should a wedge be formed by fracturing. Rockbolt support would also provide support resistance.

The major disadvantage of this type of slot is that fracturing would not be eliminated.

3.2.2 Vertical Slot in Roof at the Room Sidewall

Figures 15 and 16 show principal stress difference contours and principal stress vectors, respectively, for the case of a vertical slot in the roof at the sidewall. The slot is extended up to the clay seam below anhydrite "b". There is a significant reduction in shear stress magnitude all around the roof and the slot itself. The major reason for this is the slip and shear stress relief caused by the slip plane. In reality, some degree of frictional resistance would exist, and shear stress dissipation would not be as dramatic. In this sense, the clay layer acts as a pre-existing stress relief slot.

If slots are cut at both sidewalls, the roof beam would detach along anhydrite "b", requiring that rockbolts carry the full dead weight of the resulting block. Provided that long term load carrying capacity could be maintained, the weight of the block could easily be supported by rockbolts.

Note that this latter option is effectively the same as if the roof were to be taken down. This practice would be an acceptable solution in a mining environment. However, by enlarging any excavation, problems may develop in the strata exposed. shear stress failure could take place higher up. Risk of disturbing an even larger volume of rock always accompanies excavation enlargement. This possibility would need to be examined more carefully if this option were to be considered.

A slot cut along a single sidewall would provide stress relief, but over time, lateral creep in the sidewall foundation would tend to push the remaining roof beam into the slot. The effect of this shearing action on rock support is not known, and would have to be addressed if this option were to be used. To enhance the effectiveness of rockbolts in this case, it may be of benefit to incline the direction of the bolts away from the side on which the slot would be cut. This would reduce the effect of shearing somewhat. Oversized boltholes would also be of benefit.

Advantages of this type of slot are:

i) A high degree of stress relief in the roof strata.

ii) Further stress fracturing is unlikely.

Disadvantages are:

i) The full dead weight of the roof beam must be supported in the long term by rockbolts (cribs could be used but would interfere with the purpose of the rooms).

ii) With the increased effective height of the room, there could be a secondary effect on overlying strata, particularly the subsequent beam formed between anhydrite "b" and "a", or higher at the 32 to 38 ft level where slip has already been observed. It would be prudent to install rockbolts into the strata above anhydrite "a".

ii) The effect of shear dislocation of the roof beam on rockbolt integrity brought about by creep would need to be examined if only one slot were to be cut.

3.2.3 Vertical Slot in Roof at Center of Room

A single slot cut in the center of the roof up to anhydrite "b" would have a similar effect to a vertical slot cut in the roof at the sidewall. Figures 17 and 18 show principal stress difference contours and principal stress vectors, respectively, for this case. Shear stresses shown in Figure 17 in the roof beam above the abutment are most likely related to bending of the beam as a cantilever. Rock bolting would eliminate this bending and these shear stresses.

Essentially the same advantages and disadvantages apply to this option as for the vertical slots at the excavation sidewalls.

3.3 Most Favorable Slot Location

There are advantages and disadvantages to each slot location discussed. The vertical slots result in the most favorable stress distributions as the shape of the resulting "excavation" has a more favorable aspect ratio for the hydrostatic stress field. However, the requirement that rockbolt support perform well is a more important requirement for continued stability. Effective rockbolt support can most likely be provided if long grouted anchors are used, but this remains to be proven.

The horizontal slot option will most likely result in further stress fracture growth near the end of the slot, and again, the requirement for rockbolt support. However, should rockbolt support not be completely effective, the lower part of the slot could still provide additional support. While this option may not be as favorable with regard to stress distributions and fracturing, it has merit in terms of less risk in terms of what might be expected.

Further study to resolve some of these issues is clearly indicated. Once some of the uncertainties have been resolved, the most favorable choice should become apparent.

3.4 Monitoring and Further Experimentation

While there is a wealth of data concerning rock mass behavior at the WIPP site, much of the instruments were placed prior to the development of fractures. In view of the behavior now being observed, an important aspect of monitoring is to help identify and confirm the mechanics of the failure process. With this new objectives in mind, additional instrumentation should be installed.

One of the most critical aspect of the suggested remedial measures is the performance of rockbolts. It is highly recommended that the effectiveness of fully grouted rockbolts be examined experimentally, by installing instrumented bolts. The objective of these tests would be to quantify the time dependent load deformation characteristics of the bolts.

Due to the time required for such an experiment, it is also recommended that rockbolt effectiveness be examined numerically. A model of a rockbolt embedded in a creeping material could be constructed, using a creep constitutive model calibrated for the WIPP site. Figure 19(a) shows how such a model could be constructed.

The capability to carry out the latter simulation exists in the FLAC code used to perform the elastic analyses presented in this report. Once the behavior of a single rockbolt is understood in detail, the rockbolt constitutive law in FLAC could be modified to conform to the calculated creep response. A simulation of the test room with rockbolt support, such as depicted in Figure 19(b) could then be carried out. The predictions of the numerical "experiment" of the rockbolt pull test could be compared with the experimental bolt as results became available, and any corrections made.

When designing a rockbolt support pattern, efforts should be made to keep the bond shear stress as low as possible. It would be prudent, therefore to incorporate a reasonable factor of safety when computing the shear stress based on the load to be carried. The results of the pull tests would also be useful, as they would indicate whether debonding would occur, or whether the shear stress would be distributed along the length of the bolt with time.

3.5 The Role of Probability

If the probability of some event taking place can be calculated, then there is some basis for making decisions involving risk. To carry out a probabilistic stability assessment requires a good knowledge of the basic mechanics of failure and the relevant parameters, or a reasonable data base of case histories.

For rooms in Panel 1 there is insufficient data to perform any quantitative estimate of probability of collapse at a specified time period, moreover, probability is only an estimate at best. Particular geological weaknesses at a specific location under consideration may place its time to failure anywhere on the probability curve. In this sense, information on probability is only useful as general indication of stability for a large number of rooms. For assessing the stability of a single room, detailed observations would be required. Questions dealing with time estimates for failure of Room 1 cannot therefore be addressed at this time.

Failure data from other rooms can give guidance on the sequence of events leading up to failure, and also an indication of the time frame in which failure will take place, but, one zone of weakness in the rock mass in the room of interest could cause substantial differences in the failure processes to take place. Monitoring and up-to-date interpretation of the rock mass behavior is the most reliable means of predicting the development of instability.

4.0 RESPONSE TO SPECIFIED QUESTIONS

The questions to be addressed are reproduced in Appendix 1. Many of the questions are related to quantifying factors such as room stability or placing estimates of reliability on certain statements. Given that the mechanical processes at the site are not well understood, it will not always be possible to answer the questions in a meaningful manner. For this reason, the background information and suggested remedial measures were presented in the previous sections in order to provide reference material while dealing with the questions.

4.1 Statement 1

This statement relates to the stability over time of Room 1 Panel 1.

Signs of roof fracture development are visible in this room, and significant fracturing and deformation of the floor has occurred. Based on the preceding discussion, it is not likely that the room would remain stable for the required period of 11 (total) years without remedial action. Similarly, limited action such as scaling would be mainly cosmetic as it would not affect the fundamental processes related to the development of the failure process.

The "specific factors to be addressed" relate to reliability estimates which for reasons stated in the previous sections cannot be quantified.

The life of Room 1 could be increased with confidence by adopting the remedial action described in the preceding sections. These measures should also eliminate the need for maintenance of the roof for a reasonable period of time relative to the time frame of the proposed experiment. As a further means of protecting the bins from floor movements, it may be possible to mount the bins on supports that are anchored to the room sidewalls. Floor heave would therefore not be of concern or cause any disturbance.

4.2 Statement 2

This statement refers to the effectiveness of rockbolts currently installed in Room 1.

As stated, it is not likely that the rockbolts currently installed in Room 1 will be effective in supporting a wedge of the type formed in Room 1 of the SPDV area. The rockbolt pattern and the basis on which it was designed would be acceptable in frac-

tured, hard, non-creeping rock, but the mechanisms at work at the WIPP site are thought to be sufficiently different that alternate design criteria should be used. Consequently, "factor of safety" as currently calculated is most likely inapplicable, especially considering the time dependent nature of the creep loading process.

If vertical slots are cut, or any enlargement of the current room size is made, there will be an increased risk of adversely affecting strata above anhydrite "b" in the time frame of the experiments. It is not likely however that any progressive upward failure would take place as fast as with the current room geometry. In view of the known large extent of strata disturbance, however, the possibility should not be discounted. Adequate monitoring should provide warning of such behavior.

In order to determine the effectiveness of rockbolts in accommodating creep, it is recommended that instrumented long grouted rockbolts be installed. Concurrently with this, analyses of the type proposed in the previous sections could be carried out. Since performance of bolts in salt is not well documented, rockbolt behavior needs to be verified for design purposes.

4.3 Statement 3

This statement concerns the reliability of stability estimates for Room 1 in comparison to reliability estimates presently applied in mining practice.

As stated in section 3.5, probability estimates of failure as currently performed require an understanding of the mechanics of failure, the parameters involved and their numerical values, or a database of case histories on which to carry our statistical analyses. Uncertainty is normally associated with measurable quantities, and reliability estimates assume that the analytical model being used in the calculation is valid.

In the case of Room 1, it is far from clear exactly what model to use for the failure mechanisms, or the parameter values involved. In this context, therefore, there is insufficient information available to carry out meaningful probabilistic assessments of stability, particularly for prediction of stability longer than the current age of rooms for which there is no information.

Geologic structures are by their nature stochastic in behavior at various scales. Strength will vary spatially, and due to creep, it will vary over time depending on the strain rates that may occur. These factors decrease the ability to determine precisely

what will happen in a given situation. In typical mining environments, probabilistic assessments are seldom carried out, and are mostly used for comparing risk associated with the outcome of different courses of action. Experience is normally used as a substitute. Levels of risk of less than 1 in 10^6 would not be reasonable in mining situations due to lack of well documented cases on which to base such precise computations.

4.4 Statement 4

This statement is concerned with modifications to the current support system to maintain stability.

Additional support is considered essential if the existing rooms in Panel 1 are to be used for several years to come. However, support alone is unlikely to be adequate. The use of rockbolts as part of the remedial measures has been discussed previously, and as stated, it is essential to determine how they will behave over a period of several years. Additional support should also include mesh to prevent fallout of smaller pieces of salt from the roof. This will help to reduce maintenance activities.

If mesh is to be used, then it will be necessary to remove the cables currently installed in the roof of Room 1. If a slot is to be cut in the sidewall at roof level, then this will also require removal of the cables.

4.5 Statement 5

This statement concerns the effectiveness of the current monitoring program to provide early warning of failure.

Based on the experience with Room 1 of the SPDV area, up to two year's advance warning of failure was seen by examining the results of monitoring. As a forward process, however, there is always difficulty in discriminating signs of failure from other sources of noise, for example seasonal variations. Furthermore, once "failure" has started, the process will take place at different rates in different rooms due to variations in geology etc. There is insufficient data available, based on only one failure event, to know the variability of this time to failure once warning signs have started. Depending upon when it is decided that indeed failure is going to take place, six months required to remove bins may be inadequate.

Given that the mechanism of failure is not well defined, and instrumentation based on an understanding of this mechanism has not

been placed, it is considered that additional instruments should be installed. Only when the failure mechanism is reasonably well understood, and instruments have been placed to monitor the process, will there be adequate tools to provide reliable warning. Criteria to determine when waste should be removed could be developed after the preceding steps have been carried out, and only then could any meaningful estimate be made of how long a warning period could be given .

REFERENCES

Stormont, J. C., "Discontinuous Behavior Near Excavations in a Bedded Salt Formation". Sandia Report SAND89 - 2403, October 1990.

Cook, R. F., "Position Paper: Life Expectancy of Room 1, Panel 1". Draft report released to Geotechnical Panel members.

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

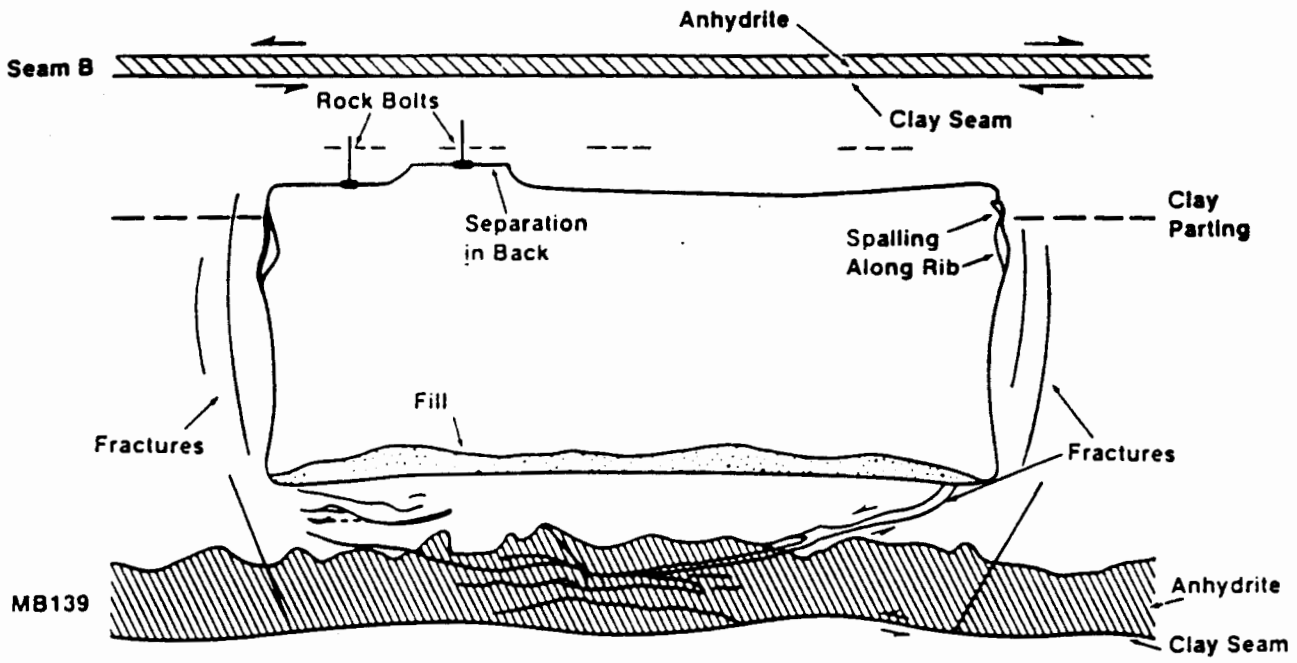
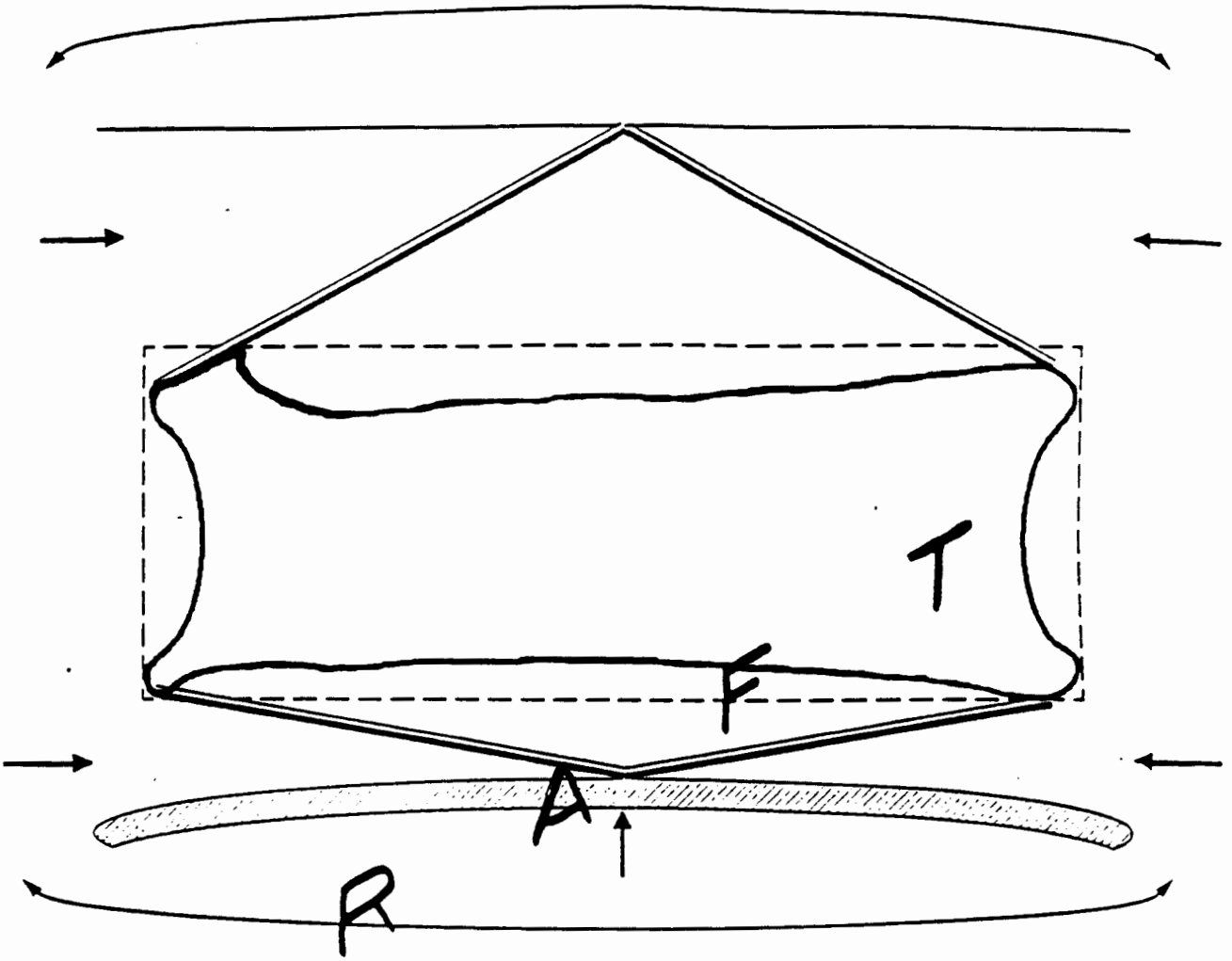


Fig. 1 Observed Discontinuous Behavior Around Test Rooms Up to 5 Years After Excavation (Stormont, 1990)



After about 8 years, the shear fractures develop along both ribs in the roof and a detached wedge with a triangular cross section develops. This wedge is first observed fully formed at mid room length and the fractures gradually migrate longitudinally along the ribs.

As the unsupported span in the longitudinal direction increases, the beam deflects with the greatest deflection occuring at mid room length. Eventually the length of the unsupported roof exceeds the strength of the roof cross section, and a fall results.

Fig. 2 Shear Fracture Development Around a Room Leading to Failure After 8 Years (Cook, 1991)

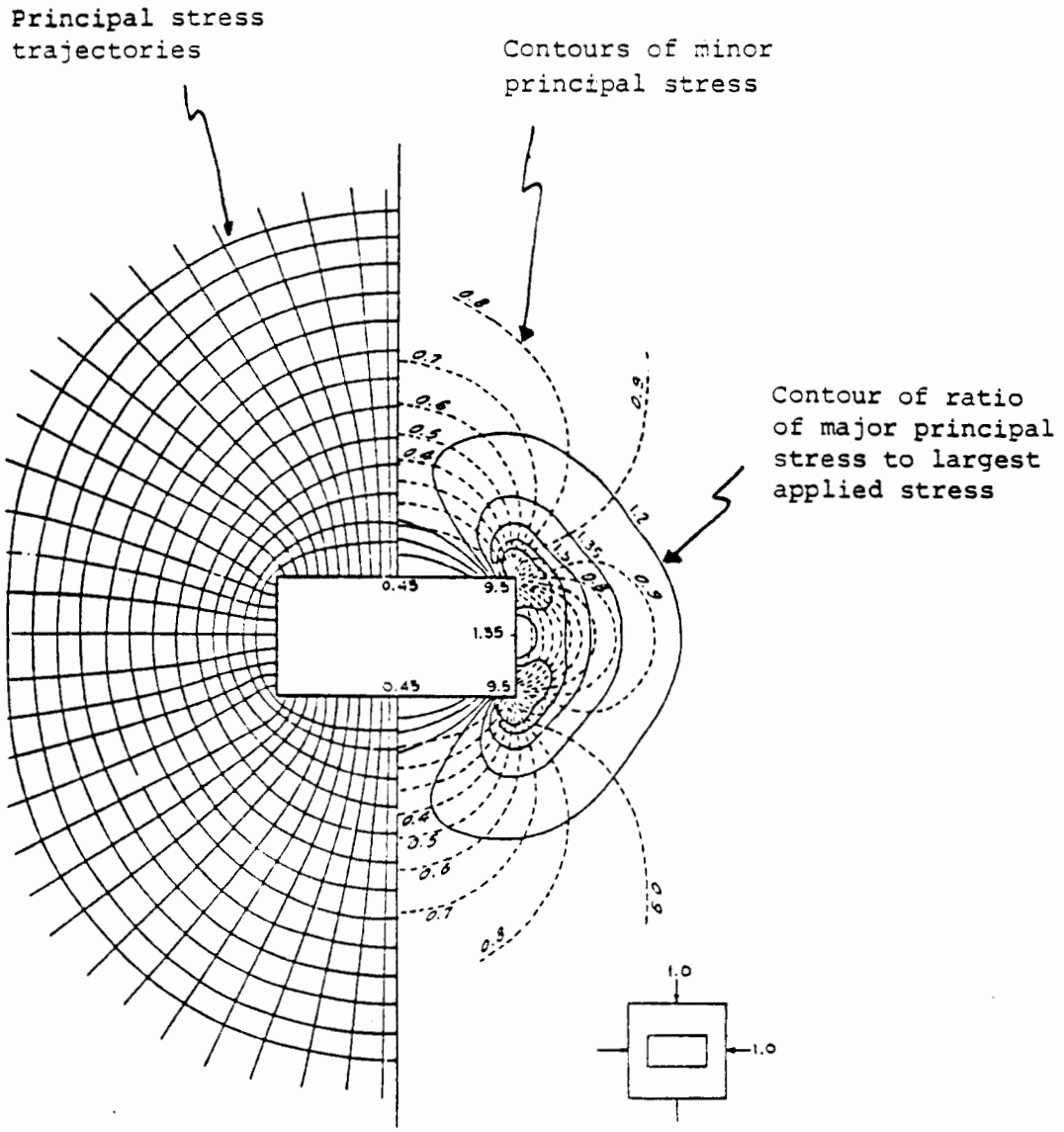


Fig. 3 Stress Distribution Around an Excavation With Width to Height Ratio of 2:1 in a Hydrostatic Stress Field (Hoek and Brown, 1980)

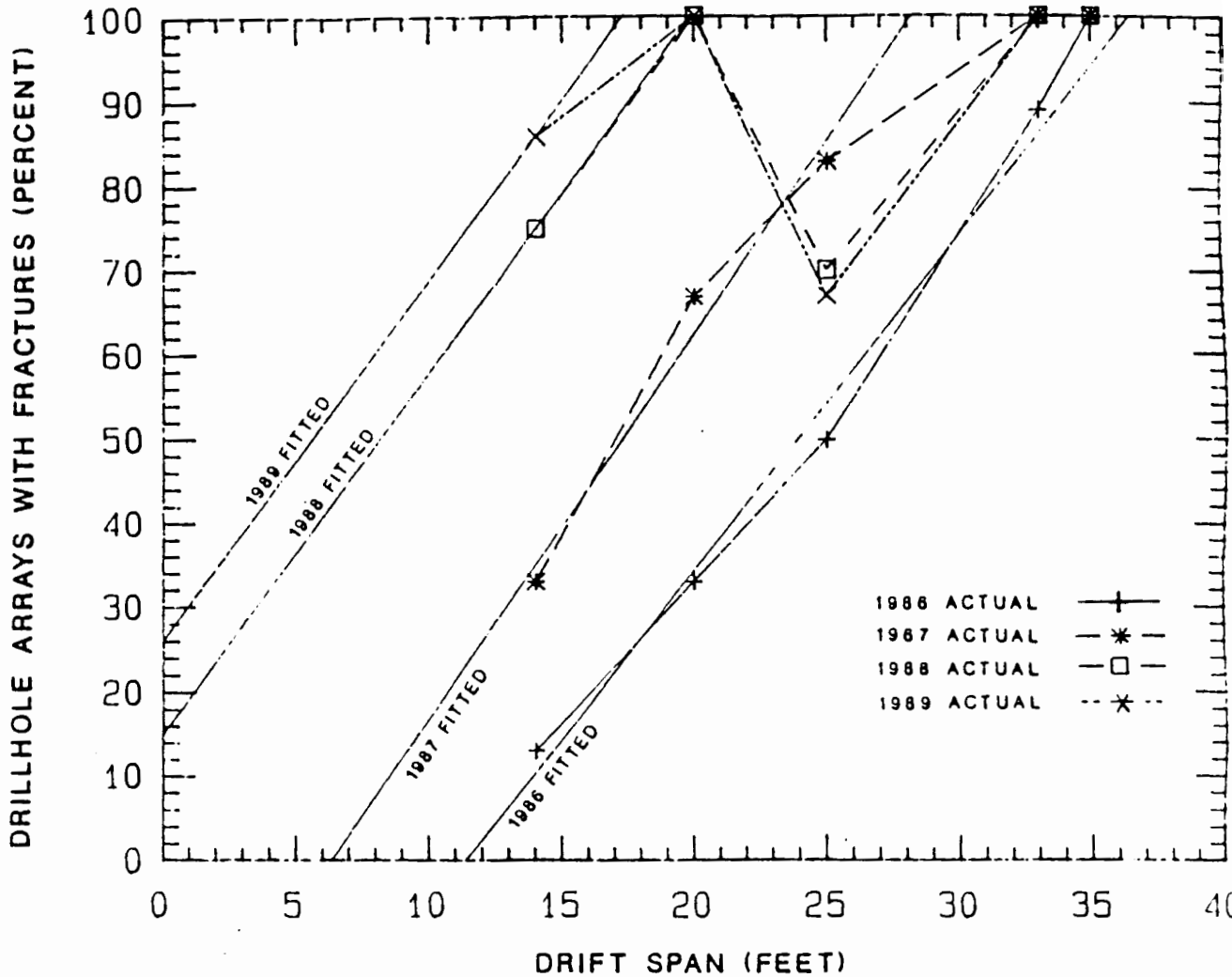


Fig. 4 Correlation of Drift Span to Degree of Fracture Development (unreferenced WIPP report)

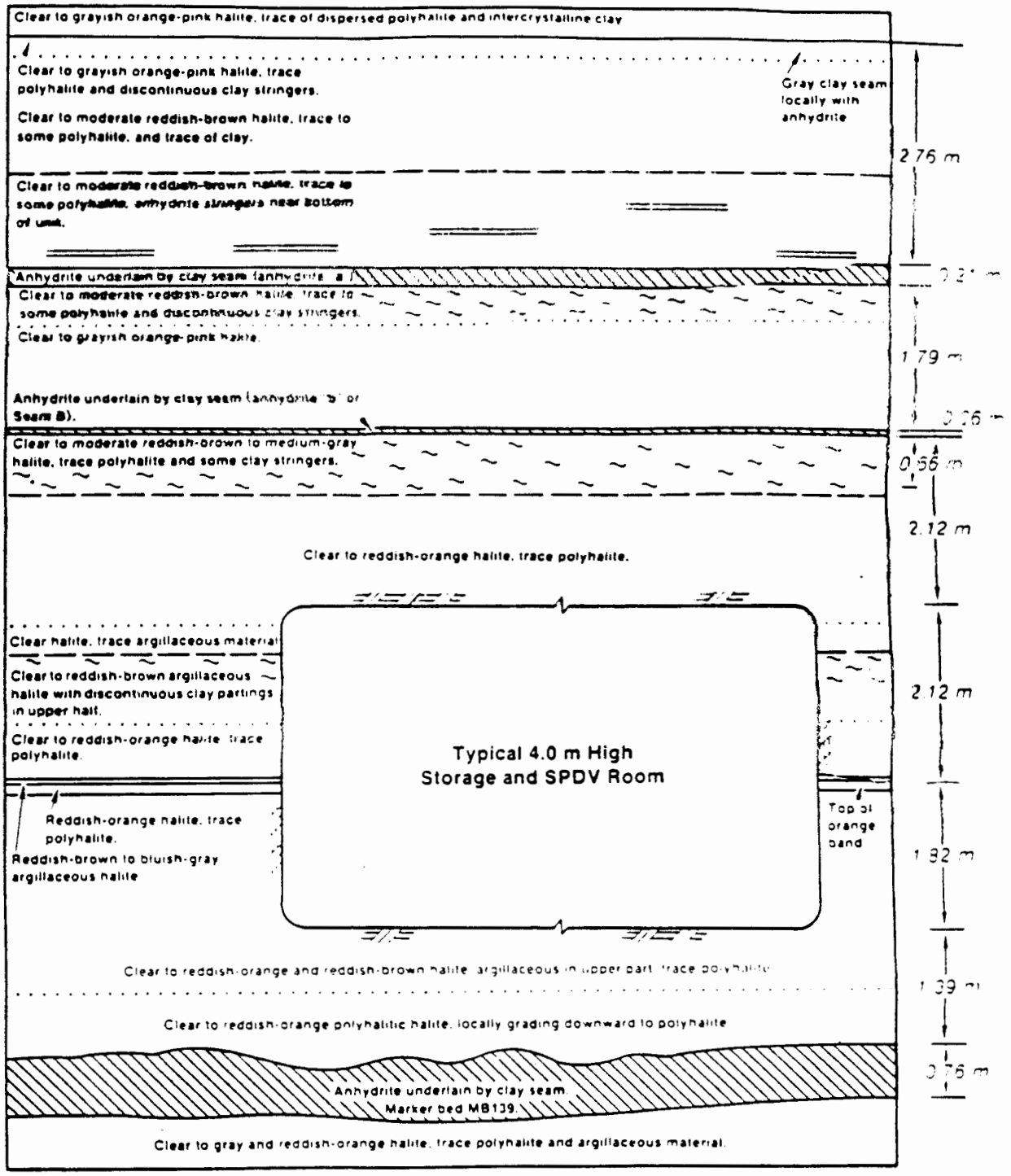


Fig. 5 Stratigraphy in the Vicinity of WIPP Excavations

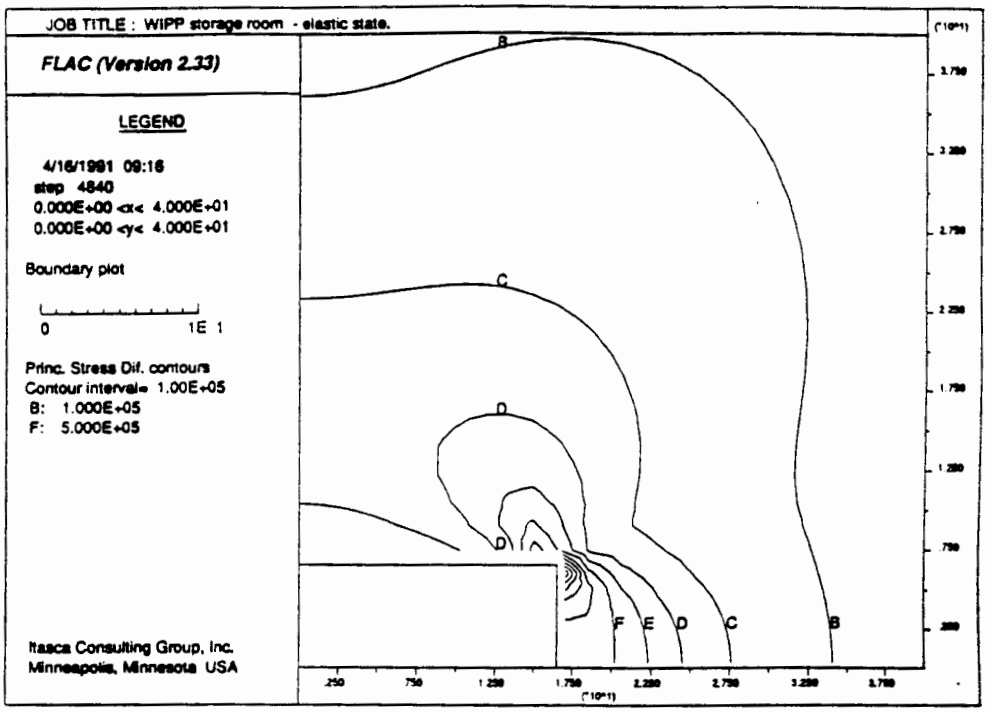


Fig. 6 Principal Stress Difference Contours, WIPP Storage Room — Elastic State

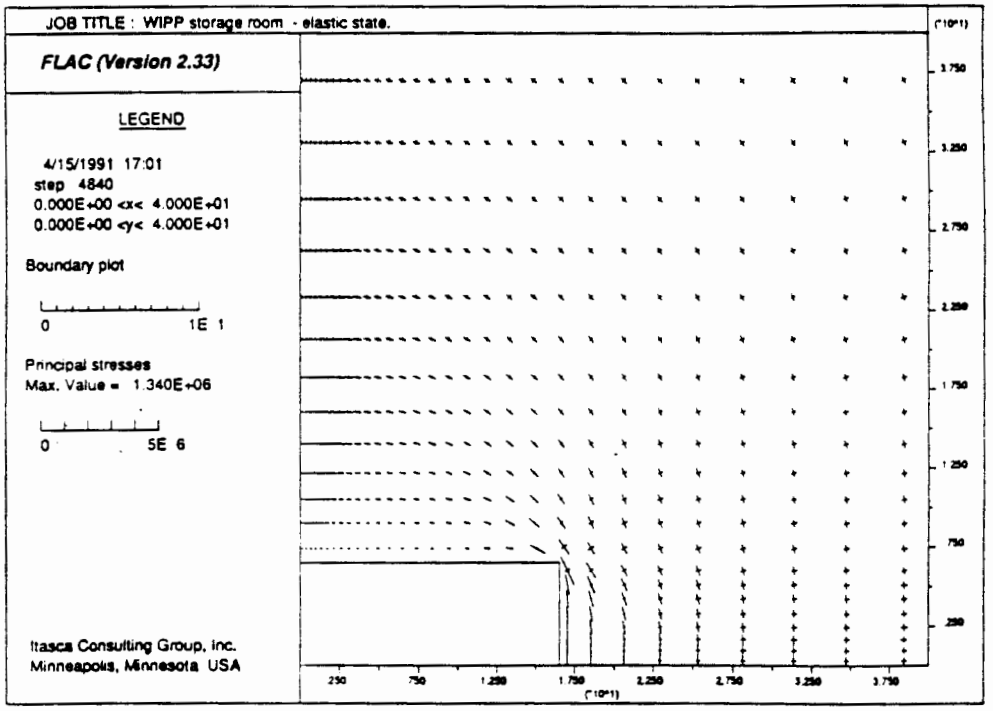


Fig. 7 Principal Stresses, WIPP Storage Room — Elastic State

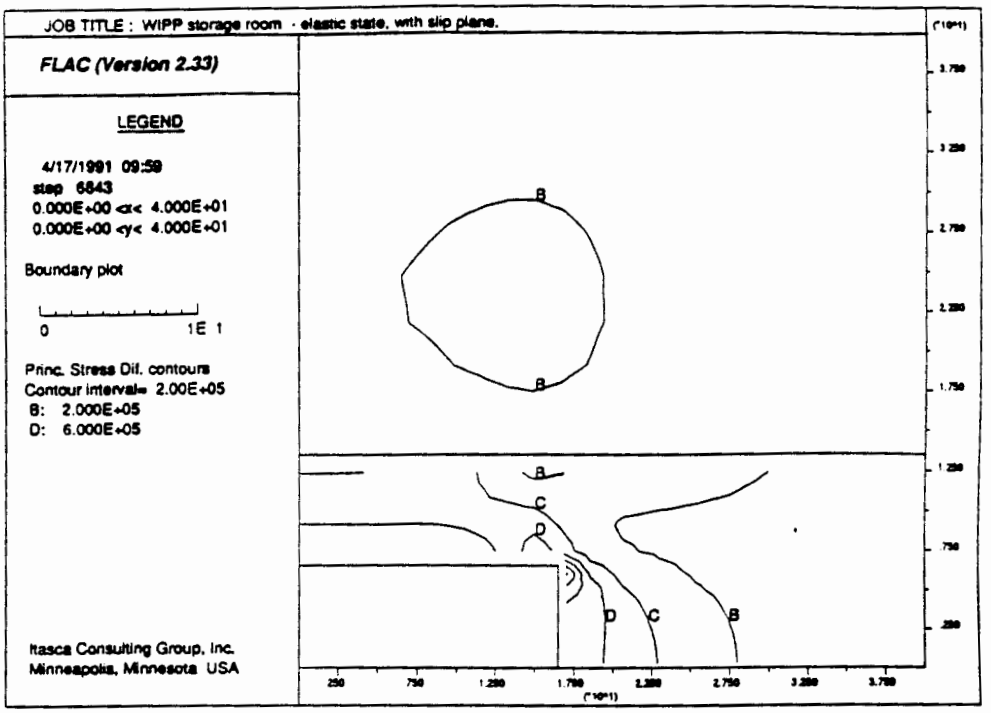


Fig. 8 Principal Stress Difference Contours, WIPP Storage Room — Elastic State With Slip Plane

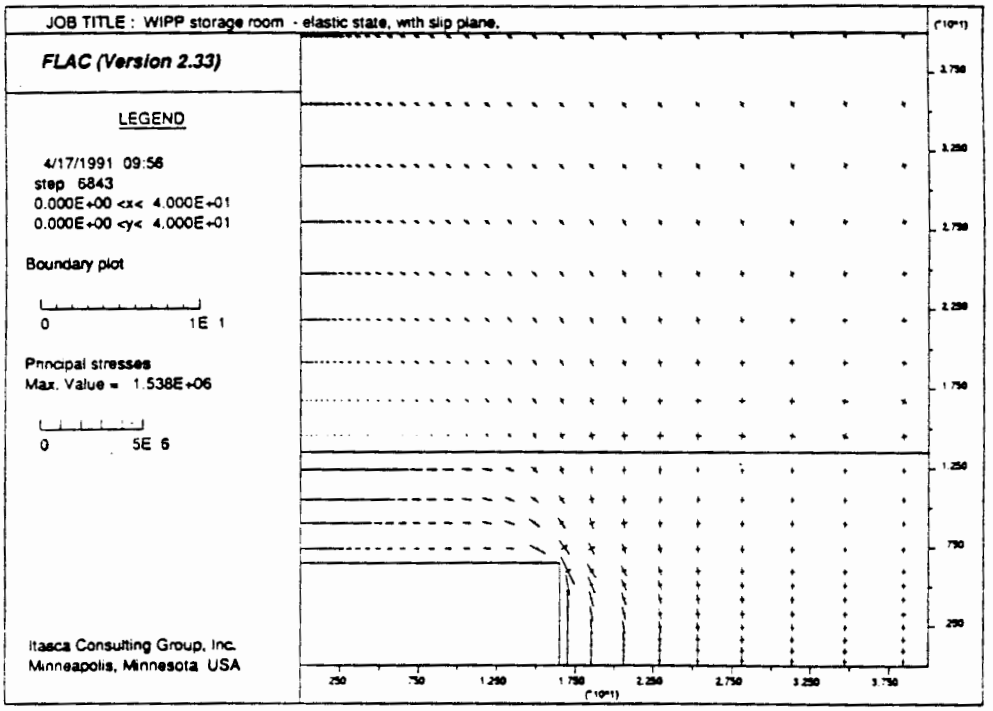


Fig. 9 Principal Stresses, WIPP Storage Room — Elastic State With Slip Plane

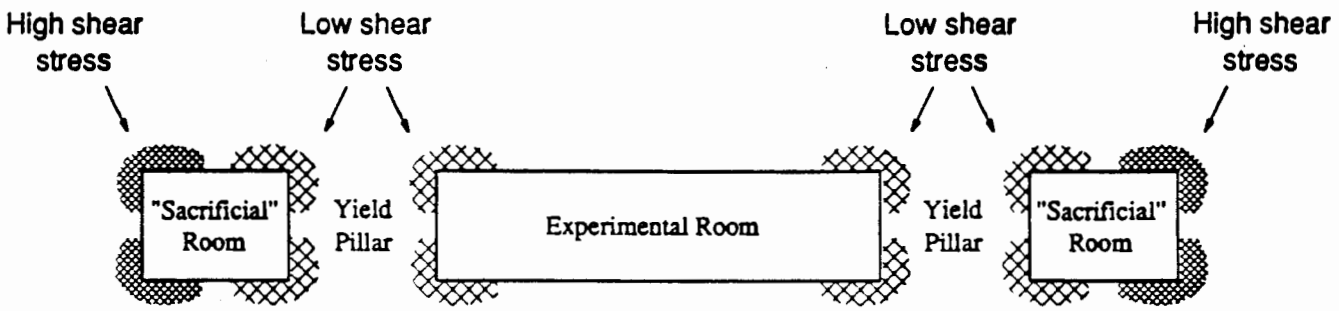


Fig. 10 Use of Nearly Sacrificial Excavation and Yield Pillars to Reduce Shear Stresses Around Experimental Rooms

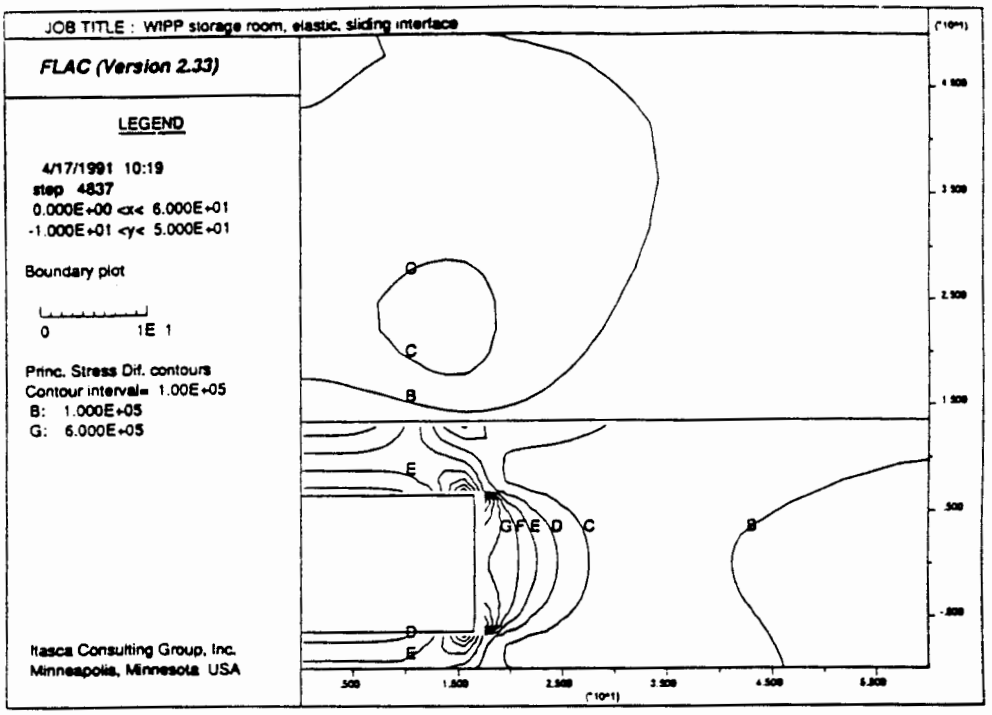


Fig. 11 Principal Stress Difference Contours, WIPP Storage Room — Elastic, Sliding Interface

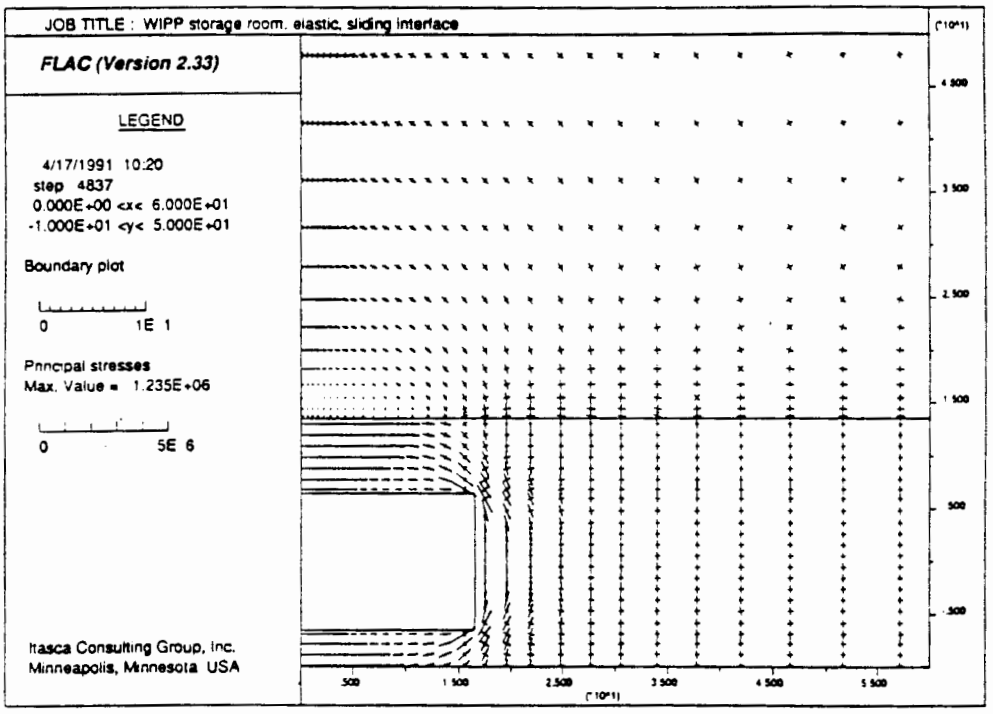


Fig. 12 Principal Stresses, WIPP Storage Room — Elastic, Sliding Interface

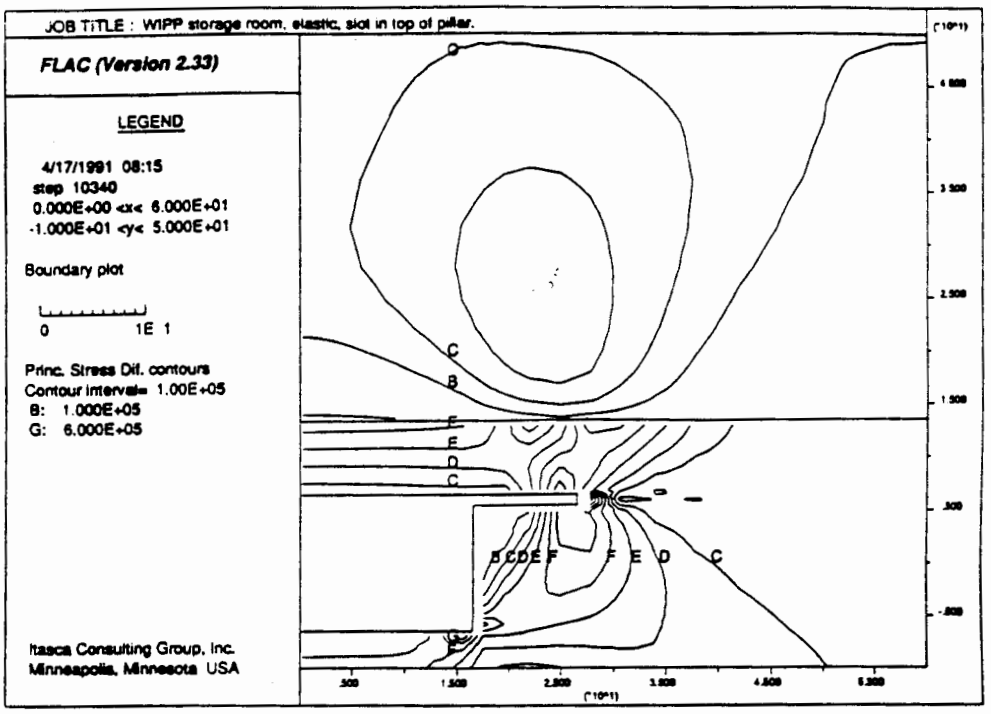


Fig. 13 Principal Stress Difference Contours, WIPP Storage Room — Elastic, Slot in Top of Pillar

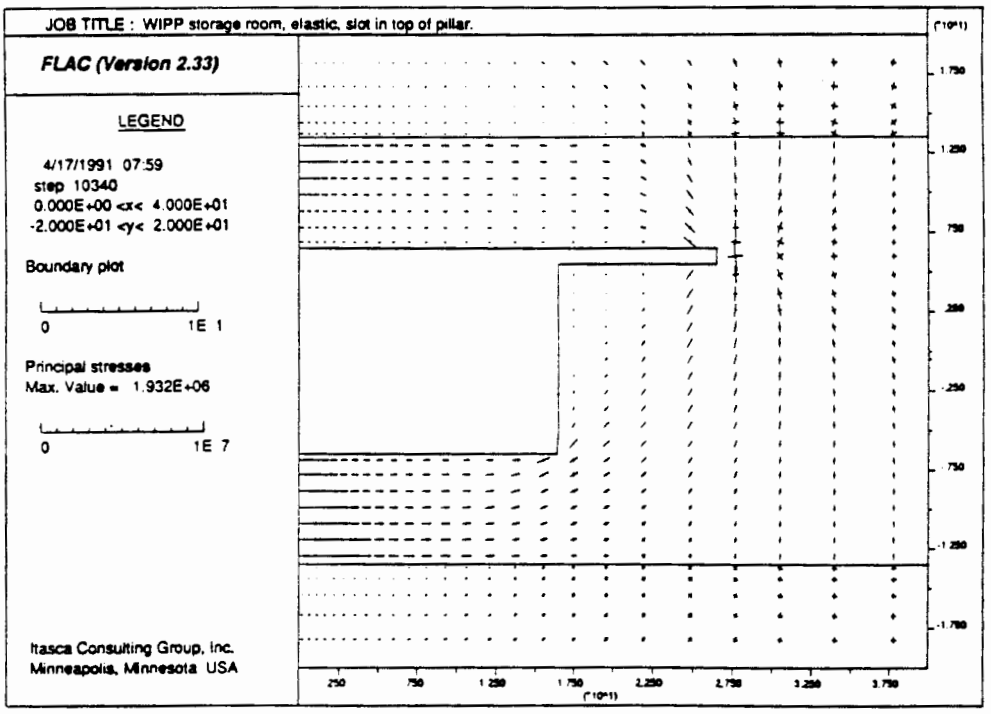


Fig. 14 Principal Stresses, WIPP Storage Room — Elastic, Slot in Top of Pillar

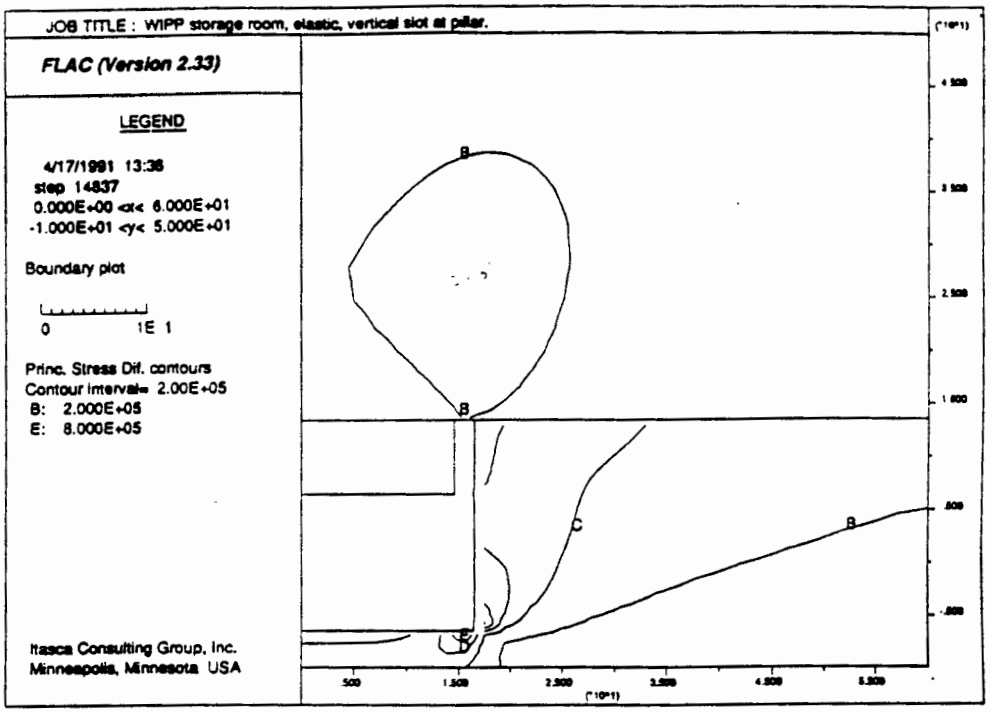


Fig. 15 Principal Stress Difference Contours, WIPP Storage Room — Elastic, Vertical Slot at Pillar

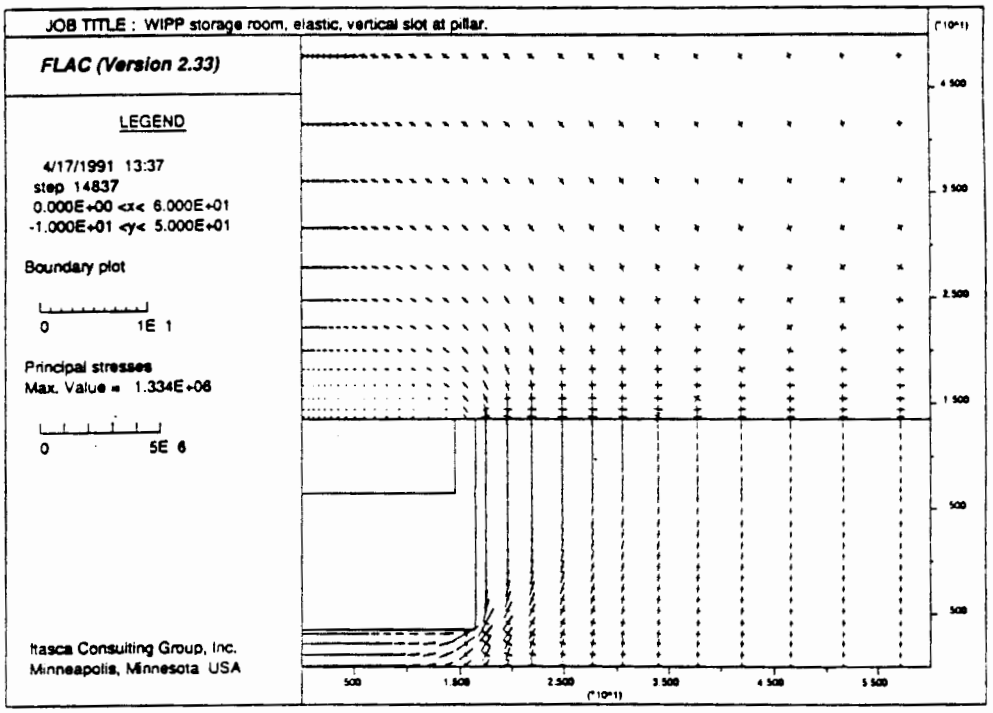


Fig. 16 Principal Stresses, WIPP Storage Room — Elastic, Vertical Slot at Pillar

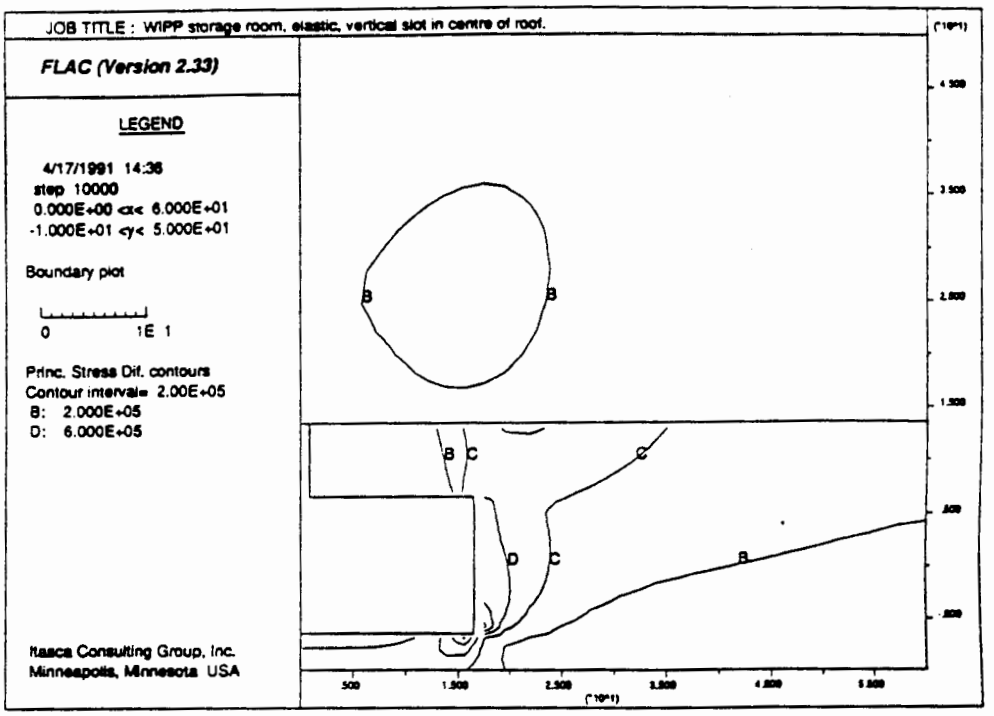


Fig. 17 Principal Stress Difference Contours, WIPP Storage Room — Elastic, Vertical Slot in Center of Roof

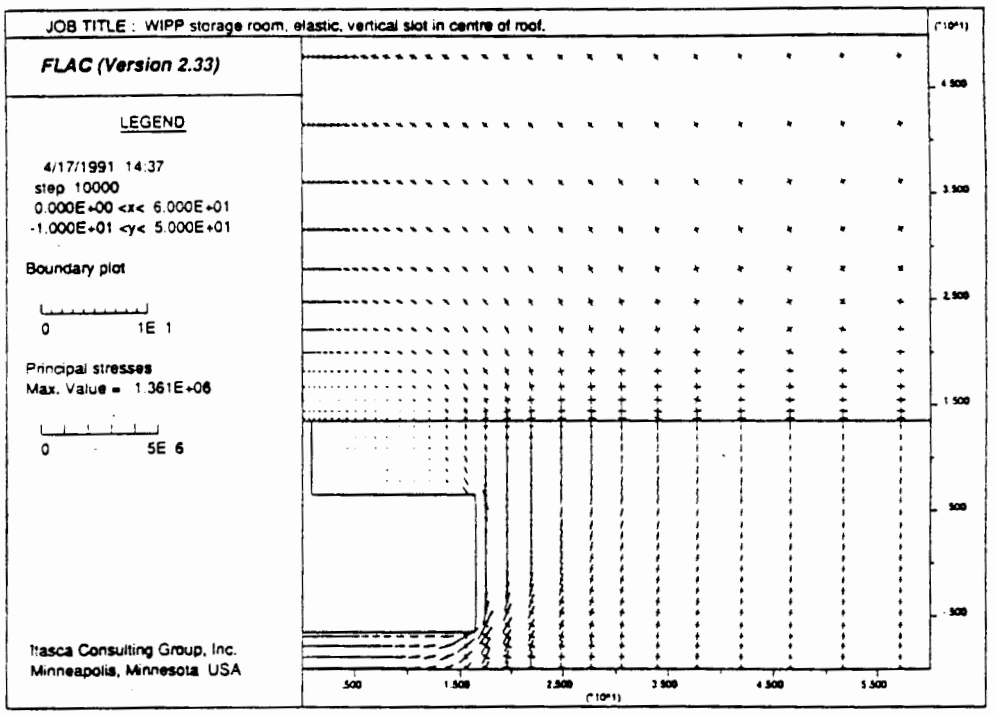


Fig. 18 Principal Stresses, WIPP Storage Room — Elastic, Vertical Slot in Center of Roof

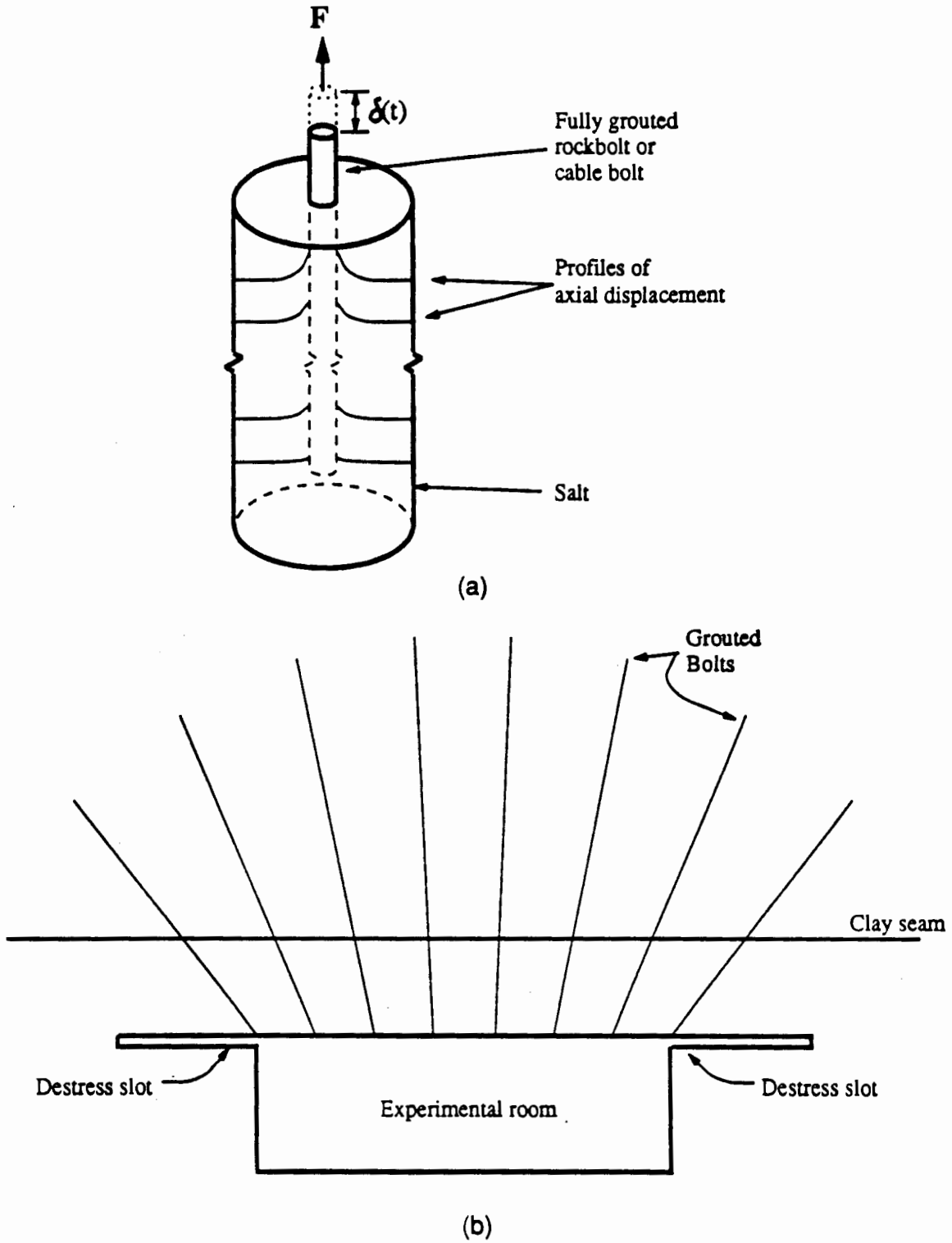


Fig. 19 Behavior of grouted bolts can be estimated by detailed modelling (a), and applying the resulting behavior to a model of the experimental room with bolts.

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APPENDIX I

Questions to be Addressed Regarding Stability of Room 1, Panel 1

STATEMENT 1 REVISION 0

An estimate can be established for the period of time that Panel 1, in particular Room 1 remains accessible on a daily basis beyond July 1991.

The following cases should be considered:

1. No maintenance in terms of scaling of roof, milling of floor or installation of additional support.

2. Limited maintenance without moving bins.

3. Extensive maintenance on an as required basis, with bins removed from room, if necessary during maintenance activities.

Assumptions

1. Room height on July 1, 1991, 13.5 feet and minimum room height needed to support equipment clearances, 10.0 feet.

2. Room initially excavated in July/August 1986.

Factors to be Addressed

1. The ability of the Panel to address Statement 1 based on the available information.

2. Best estimate for life of Room 1.

3. Lower and upper bound estimates for the life of Room 1.

4. Levels of uncertainty associated with estimates.

5. Reasons for the levels of uncertainty.

6. Additional information that would be needed to improve estimates.

STATEMENT 2 REVISION 1

The rockbolt system as currently configured, is sufficiently effective to ensure that the test program in Panel 1, in particular Room 1 can be completed.

Assumptions

1. The test program will start in July 1991.

~~2. The test program will be completed in July 1996.~~

~~3. Retrieval from Room 1 can be accomplished between July 1996 and July 1997.~~

4. The bins CANNOT be disconnected and moved to facilitate maintenance of the rooms.

Revised Assumption

(replacing Assumptions 2 & 3)

The test program including retrieval will be completed by July 2000.

Factors to be Addressed

1. The affect that the changes associated with the test program have on support requirements for Room 1, Panel 1.

2. The rock load to be supported is approximately the full weight of the roof beam upto the anhydrite "b" layer in the middle third of the span, and half this weight over the outer two thirds.

3. The adequacy of the factor of safety of the bolting system used in Room 1, Panel 1 to support the design rock load.

4. The salt above the anhydrite "b" will remain competent.

5. Slippage of anchors provides an acceptable approach to supporting the rock load while accommodating roof closure, with daily access to the room.

6. The mechanism by which the bolt anchors will accommodate the movement of the salt while supporting the immediate roof beam.

The level of confidence that can be placed in the estimate of the life for Panel 1 provided in the response to Statement 1 is in accordance with accepted mining practises.

Factors to be Addressed

1. The extent to which a probabilistic basis for determining risk assesessment is presently applied in mining.

2. The qualitative nature of geologic information.

3. The extent to which a database or experience is available in the mining industry from an operations point of view to provide meaningful judgements at the probability levels used in the nuclear industry (i.e. probabilities of less than 1 in 10^6). This is not to be applied to an assessment of the longterm (10,000 year) performance of a repository.

4. The adequacy of the geomechanical database developed at the WIPP and the methods currently in place to evaluate the performance of openings.

STATEMENT 4

Modifications to the support system in Panel 1 can be implemented to ensure that access is maintained to the rooms on a daily basis until the tests are completed.

Factors to be Addressed

1. The modifications and additions to the support system needed to ensure the completion of the tests.

2. The maintenance activities that will be needed in the room.

3. The need to remove the cables for the bin scale tests in order to install additional support.

STATEMENT 5

The geomechanical monitoring program and the routine observations in Panel 1, can provide sufficient warning to allow the timely retrieval of the waste from the Panel.

Assumptions

1. In an emergency, all waste can be removed from the room within a 6 month period.

Factors to be Addressed

1. The adequacy of the geomechanical database developed at the WIPP provides an adequate basis to predict and provide early warning of deteriorating conditions in Room 1.

2. The adequacy of the present geomechanical instrumentation, installed in Room 1 is adequate to provide early warning of deteriorating conditions.

3. The adequacy of the proposed additional geomechanical instrumentation to be installed in Room 1 to provide early warning of deteriorating conditions.

4. The criteria to determine when removal of waste becomes necessary.

REPORT SUBMITTED

BY

DR. H.D.S. MILLER

The summary report contains:

- An accurate record of the meetings of the Geotechnical Panel on Panel 1 Stability.

- A copy of the report provided to Westinghouse by this panel member.

- An accurate presentation of the consensus agreed to by the panel members at the meetings on the 23rd and 24th of April 1991.

Panel Member

Date

STATEMENT 1.

REPLY

1. Room 1, Panel 1, will remain accessible on a daily basis for a period of 2 yrs after July, 1991.

2. Limited maintenance will be required.

3. Room 1 already exhibits evidence of deterioration, with fracturing of the roof along both sides together with some scaling. The pattern of deterioration is the same as occurred in the experimental rooms, and it is felt that the eventual failure will also be the same. Sidewall slabs have also formed.
   Support will not prevent deformation and failure, as this is due to stress-induced creep in the surrounding rock. The relative stiffness of the adjacent pillars is of prime importance in creating the basic stress conditions driving the creep.
   Any support installed should be designed to control and contain the failing rock.

4. The lower bound estimate for the life of Room 1 is 1 year, while the upper bound estimate is 3 years.
   It should be borne in mind that failure is a gradual and continuing process, that begins at the time the excavation is made. "Critical failure" can be defined as when roof, sidewall or floor rock becomes detached to the extent that safe limits are exceeded. These limits can involve threats to equipment, personnel or size of opening.
   The definition itself requires a judgement call based on observation, measurement and experience.

5. Uncertainty is introduced by:
   1. Unknown variations in geology / stratigraphy / lithology.
   2. Unknown effectiveness of the rockbolt support system already installed.

6. A more detailed analysis of the measured data supplied to me could change the estimates of time to failure.

STATEMENT 3.

1. In salt and potash mining, risk is currently assessed on the following bases:
   1. Direct long-term(>5yrs) operational experience.
   2. Measurements of deformations in and around excavations, including surface subsidence.
   3. Modelling, using computer models together with associated laboratory testing to determine rock properties.
   4. Geologic mapping to determine occurrence of unusual conditions. This also includes surveying of the roof and floor elevations and variations in orebody thickness. Other unusual occurrences such as water and gas pockets are also mapped.

   Of all of these, 1,2 and 4 above have been found to be the most useful, while computer modelling is used more as a predictive tool backed up by opinions derived from the other observations.

2. WIPP is unique and different from other salt and potash mines in that the objective is not to produce a product, but to store a product. The duty and life expected from these excavations is therefore somewhat different. There is however, a similarity of life expectancy from some of the development entries in producing mines that could serve a useful basis for comparison. Development entries and shafts in producing mines are expected to have a useful life of from 5 to 50 years, and in some instances longer.

   I have analysed in great detail the rock mechanics data measured at the following mines:
   1. Boulby Potash Mine, England.
   2. Allan Potash Mine, Sask.
   3. Rocanville Potash Mine, Saak.
   4. IMC Potash Mine, Saak.
   5. Jeffersen Island Salt Mine, La.
   6. Weeks Island Mine, La.
   7. Cayuga Salt Mine, NY.
   8. Belle Isle Salt Mine, La.
   9. Cominco Potash Operation, Sask.

   The analyses were carried out in order to assess either the risk of some occurrence happening, or to determine why some occurrence took place. These could include:
   1. Shaft stability
   2. Surface subsidence

STATEMENT 2.

REPLY

1. The effectiveness of the currently installed rockbolt system to maintain accessibility to Room 1 is uncertain. This is for a number of reasons.
   1. No practical support system including the present one can prevent the deformation and failure from occurring. At some stage "critical failure" as described previously will occur despite the support system installed.

   2. The rockbolt system as designed would be adequate to support the "dead weight " load of the roof beam as described if:
      1. Continuing squeeze and deformation of the roof around the beam did not occur.
      2. Failure of the anchoring system due to creep of the salt around the anchor did not occur.

2. Slippage of the anchors does not provide an acceptable approach to supporting the rock load. Too many unknowns exist, and a number of questions are raised:
   1. Does slippage in fact occur?
   2. How does it occur?(is it continuous, stick-slip, etc.)
   3. What load conditions are required to cause it?
   4. Were the rockbolts initially installed in such a way so as to allow slipping?

3. Lateral stresses in the roof strata will result in continuing deformation and therefore loading on the rockbolts. These will in turn cause increasing point loads on the rockbolt plates. Experience at other salt and potash mines has shown that these point loads can result in break-up of the rock around the plates.

4. Another serious failure mode of rockbolts that occurs where the rockbolt anchors are installed in salt is due to the creep of salt around the highly stressed anchor. The result is that the wedge pulls down through the anchor shell. Short term pull tests on installed bolts won't show this problem.

3. Water inflows
4. Effectiveness of support
5. Roof and wall collapses
6. Life expectancy of individual entries

My opinion is that the best way to assess risk in a salt/potash mine is by making measurements, particularly of closure and extension. Computer modelling may then be done and verified using measured data.

The biggest difficulty lies in arriving at a failure criterion that would allow projections of measured or modelled data.

At this stage, experience is the only way to interpret and project the data obtained. In addition to actual room failures, WIPP has a good geomechanical database on which to base predictions of future behaviour.

It is therefore important to analyse the existing data and to compare it with other situations and experience at other salt and potash mines.

Some salt mines have been in existence for more than 100 years at similar depths and conditions at the WIPP site. Many of the original excavations are still open, while for one reason or another others have closed totally or collapsed.

STATEMENT 4.

1. No support system can prevent the deformation and consequent failure from occurring. However a support system that has been used extensively in other mining applications can be installed to contain and control the failure, so that a critical situation (in terms of the WIPP short term objectives) does not occur. This was described at the last meeting of the expert panel, and consists of grout anchored cables with lacing and meshing. Together with the existing 10 ft mechanical bolts, I have no doubts that such a support system would extend the life of a room by several years. A fuller analysis would be needed to give a firm prediction on the life that could be expected from the system.

STATEMENT 5

1. The geomechanical monitoring program and the routine observations in Panel 1 can provide an indication of impending failure. However, the type of measurements and the graphical output on which the predictions of failure are based, have a built-in problem; the confidence of the prediction only improves the closer to the actual time of failure. For instance, 18 months before failure the data shows some evidence of instablity occurring; at 12 months this evidence is confirmed, a failure mode is in progress, but no firm date of critical conditions can be given; at 6 months the closure rate seems to be accelerating, but still no firm predictions. The frequency of measurement and plotting of data is then increased. It is now that failure is virtually certain to occur, but again, the precise timing is still uncertain.

   It is felt that not enough data has been observed to date to be sure of descrbing a criterion for certain failure at a given moment in time.

2. I think that the ability to predict failure with greater precision both in time and location will improve. It is recommended that a special study be made of the data recorded to date, the objective of which should be to develop a valid and workable criterion for the prediction of "critical conditions" at the WIPP site. The term "critical conditions" should also be defined.

   Until this is done, it is impossible to say at what stage removal of waste (or human operations) would be necessary.

REPORT SUBMITTED

BY

DR. P. MOTTAHED

The summary report contains:

- An accurate record of the meetings of the Geotechnical Panel on Panel 1 Stability.

- A copy of the report provided to Westinghouse by this panel member.

- An accurate presentation of the consensus agreed to by the panel members at the meetings on the 23rd and 24th of April 1991.

Panel Member

May, 28, 1991

Date

PARVIZ MOTTAHED

A FINAL REPORT

TO

ADVANCED REPOSITORY TECHNOLOGIES
WIPP PROJECT
CARLSBAD, NEW MEXICO

ON

WIPP PROJECT
THE LIFE OF THE PANEL 1
ROCK MECHANIC CONSIDERATIONS

BY

P. MOTTAHED, PH.D., P.ENG., MIMM, C. ENG.

CANMET
ELLIOT LAKE LABORATORIES
ELLIOT LAKE - CANADA
P5A 2J6

MAY 1991

WIPP PROJECT
THE LIFE OF THE PANEL 1
ROCK MECHANIC CONSIDERATIONS

INTRODUCTION

In order to address the five statements with respect to the life of the Panel 1, Room 1, in particular and Panel 1 in general, it is pertinent to discuss some of the fundamentals of the rock mechanics applied to salt rock mining and analyze the provided data in the light of these principals. The writer would also refer to his 10 years experiences in potash mining in Saskatchewan and make a judgement based on the combination of the science and art of rock salt mechanics. In the first paragraph of the summary of the position paper by Dr. Roy Cook, where he states that "Support in an underground environment is not an exact science and therefore estimates of the period of time over which the installed support will remain effective is a matter of judgment." This statement is more pronounced with respect to salt rock mining, than the mining of hard rock, when the theory of elasticity could be confidently applied. The salt as the host of repository waste; because of its viscoelastic properties has the capability of creep and entombing the waste. On the other hand, the very same property tend to restrict the application of more predictable elastic theory for describing its behaviour in underground mining environment. Due to complexity of viscoelastic theory at time designers have to use the theory of elasticity in order to describe certain behaviourial pattern in salt rock, a procedure which has caused a great deal of

controversy in rock salt mining, and has been the focus of international salt rock community.

The use of these theories for design purposes requires explicit qualifications of the assumption and incorporation of an acceptable factor of safety for the design purposes.

The comment of Dr. Roy Cook, in summary of the position paper, in relation to use of rock bolt as a means of support in this jundure, require some comments. The comment "However rock bolts can only be considered as a temporary measure in salt and must be used in conjunction with proper maintenance of the openings and the surveillance of their Geomechanical performance." This statement is fundamental base for the use of rock bolt in salt rock mining. The rock bolt is never used as a permanent support, unlike the use of the same, in hard rock mining, alone and or in conjunction with other support system as a permanent means of support.

The item 4 in the summary of the position paper should also be addressed. The inclusion of all the parameters in the design of an underground support and difficulty in quantification of other factors, would necessitate the constant monitoring of the performance of a support system, and consequent change to the system are made when necessary and "... design evolves to meet the needs of a particular underground environment. It is (the) effective monitoring, and flexibility in design and decision making that provide the best assurance for a support system to meet its functional requirement."

The Geotechimical Surveying group of the WIPP project, have indeed, provided a comprehensive monitoring programme for detecting the geomedomical behaviour of the salt and the evaluation of the support system and performance of the pillars and the openings and the associated strata. In considering the volume of the data provided and the short period of time given for reporting on the Life of the Panel 1, it is not possible to analyze all the data. The present report is based on the pertinent data from Volume I & II & Geotechimical field data and analysis report and position report by Dr. Cook. This deduction is augmented by the site visit and presentation and discussion held at Carlsbad between April 9-10, 1991.

II Mechanism of the loading of the roof beam

The extent of roof deformation in salt rock depends upon various factors amongst which the presence of discontinuity planes, excavation of single or multiple openings, the depth of workings, pillar size and pillar behaviour and its interaction with the roof and floor rock could be enumerated.

In the position paper of Dr. Cook, figures 9a -9e, the complex nature of the load transfer, after excavation of the openings, with the surrounding strata is clearly outlined. The creation of the lateral compressive forces on the roof and the floor of the opening fig. 9 a, will result in the fracture of the roof and floor beam and eventual formation of a wedge shaped rock which in time would collapse. This collapse would reduce the magnitude of the

horizontal stress and in considering the geology of the WIPP site, it would move to the higher horizon, working on the salt below the Anhydrite a. This action in time would culminate in repetition of similar mechanism until such time a stable arch is formed. The height of fracture (failure) zone depends obviously on the width of the mine opening. This doesn't mean that the reduction in the width of the opening would automatically achieve stability of the opening. As mentioned earlier, there are many factors which are active in the present site, which in unison result in initial fracturing of the roof beam and its migration to the plane of discontinuity, and its final collapse under the gravity and the horizontal compressive stresses.

The recognition of this horizontal compressive stresses and its damaging effect on the state of roof stability was the one of the early problems associated with potash mining in Saskatchewan. Obviously, depending on the proximity of the discontinuity planes in the roof or floor of the opening, the failure of the roof and floor beam would almost quickly fail, at depths of 2000+. The Cory and Allan Potash Mines experienced these early problems before rationalizing on the present mining system which adopts the isolation of yield pillars.

It then becomes clear that the ever presence of the compressive horizontal stresses described earlier, tend to further complicate the mechanism of rock bolting, and the reduction in the magnitude of these stresses become vital in achieving a relatively

stable roof condition.

There are many different ways which could be adopted, in order to reduce magnitude of óH and achieve a stable roof condition. The choice of these methods primarily depend on at which stage of room (opening) development we are contemplating a reduction in óH.

A - In desing stages

a) The use of sacrificial roadways and use of yield pillars

The isolation of five entry system has been successfully adopted by Cominco Potash Mine in Saskatchewan, exploring the potash seam at a depth of 1100 meters without any significant roof problem. Basically four 5.5m wide by 3.5m high room and a centre room of 7m wide are isolated. These rooms are separated by a 6.7m wide yield pillars. This geometry allows the roof and floor of the two outer rooms, which are cut first to relax and separate along the discontinuity protecting the inner rooms from damaging horizontal stresses (2,3).

The Saskatchewan potash industry uses many different mining system, utilizing yield pillar techniques to allow the continuous and gradual deformation of the roof and floor rock along the clay discontinuities, which in the process demonstrates the harmful effect of the horizontal stresses.

(b) Slotting of the roof

The creation of a slot in the roof or floor of the mine working would tend to reduce the damaging horizontal stresses. This slot could be a 6" wide at a depth which will not impact

negatively on the immediate strata overlying to occur, which would accommodate the horizontal stresses by virtue of the rapid deformation of the slot. Normally the rate of closure of the slot is higher than the anticipated rate. On closure of the slot, the process has to be repeated. Normally the slots are combined with erection of 4′ x 8′ timber cribs in areas of high stress. (4)

(c) The interaction between pillars and the roof, increases with the stiffness of the pillars. As described earlier. the use of yield pillars has been adopted in solving the roof failure problem in the Saskatchewan potash mines. The undercutting of the pillars would reduce the vertical load imposed upon the pillar which consequently reduces the horizontal component of the stress field. This technique is synonymous with pillar size reduction, and has successfully been used in conjunction with roof bolting and roof slotting.

III Rock Bolting

As mentioned earlier, the rock bolts are used in salt rock mining as a temporary measure. As outlined in the position paper of Dr. Cook, section 2-5,"...even with the bolts in place the plastic nature of salt ensure that its flow can cause stress build up which can lead up to fracturing, and at strata interfaces differential movements would not stop fracturing and formation of bed separation". In the experience of the writer and after a

recent consultation with the engineers at various potash mines, the mode of failure of the rock bolts due to viscoelastic nature of the rock, and presence of the horizontal forces could be basically divided into three distinct modes:

(a) stripping of the bolt threads (wedge failure)

(b) Wedge pulled down through leaves of expansion shells (leaf failure)

(c) Entire expansion shell pulled down drill hole (anchor failure)

Out of the three above failure modes the mode (b) is the most prominent, followed by leaf failure. In a comprehensive tests in salt using D1 & D10 anchors, the ratio of wedge : leaves: anchor failures were 68.5%: 20.4%: 7.4% (5). These tests were conducted on 6'-5/8" dia and 8'-3/4" rock bolts. The torque was between 125 - 175 ft - 1b. It was also concluded that the installation torque with the experimented range appear to have very little direct effect on the type of failure, which illustrated by the fact that wedge and leaf failure occur approximately at the same frequency throughout the entire torque range.

If the rock bolts are to perform their task by suspending the weight of the roof rock, the anchorage capacity of the bolts (the ultimate failure) should be sufficient to withstand the dead weight of the rock. The presence of the horizontal stresses causing the flow of the salt beam would tend to bend the bolt, and the present assumption of the bolt slippage becomes invalid, and as mentioned

earlier the failure of the bolts would be in majority of cases in wedge failure or leaf failure mode. In the opinion of the writer, if the future rock bolting of the roof in other panels being considered, different type of anchors need be experimented upon. The present rock testing programme is too brief. A more comprehensive time dependent anchorage capacity test on the bolts should also be conducted on roof.

IV The combination of rock bolting and slotting

This option takes the advantage of both techniques by suspending the rock wedge from the bolts and reducing or momentarily eliminating the harmful horizontal stress field, would achieve the desired results. However, it must be emphasised that the vertical slotting of the back, though on one hand relieves the σH, on the other hand, would require the correct and efficient design of the rock bolts in holding the weight of a cantilever. In case of uncertainty the roof rock is supported by timber cribs as earlier stated to ensure gradual deformation of the roof. Field tests have indicated that the cribs in time, would behave as the support pillars carrying the similar load (4).

It has been argued that as the result of the lateral movement shear failure of the bolts would occur. This mode of failure though appears to be operational, in reality as the result of the overall flow of rock on mass, the bending of the bolt would occur with final leaf failure; the wedge pulling out of the leaf. No

such failure in my experience, or as the result of recent investigation has been reported in any of the Saskatchewan potash mines, where as the result of deeper depth of excavation and the presence of multiple clay seams, higher horizontal stress are being experienced, and hence more likely occurence of such mode of failure.

V Sequence of excavation and reloading of the opening

Contrary to elastic ground behaviour, the stability of salt rock openings at great depth is strongly effected by the time sequence of the excavation. This is due to the fact the stress conditions around salt rock openings change continually with time. A concept which has been used in chevron mining system in Saskatchewan potash mining.

In the course of excavation, SPDV test rooms and the subsequent mining of the seven rooms of Panel 1, the sequence of the mining rooms has been in a manner which would induce the re-loading of the openings, subjecting the roof and floor of the opening to successive high stresses.

In examining the sequence of the cutting of the SPDV rooms as shown in the fig 1, the test room no 1, was the third in the seven of the rooms cut, preceded by room 2 and 3, with room 4 being the last room in this panel to be cut. This room prior to its excavation, as the result of mining of the rooms 2 and 3 would be highly stressed. This room was subsequently subjected to a series

of reloading due to excavation of drift N1420 some 11 months later followed by the excavation of room 4, a month later. The fig 1a shows clearly this reloading of the room which would translate to a higher than normal rate of closure. The uneven distribution of the stress imposed on the roof and the floor of the workings in the Northern side of the opening would have a detrimental effect on the final failure if the roof slab towards the North of the panel. This loading and reloading pattern is seen in the closure rate graphs of the test room 4, 3 and in SPDV panel, with less drastic effects, as the excavation of rooms L3 and L4 were carried out some six years later (April 1989). The excavation of these openings have caused a reloading of all the rooms, with room 1, being the most susceptible to reloading as the result of its excavation history suffering the most. The geotechnical data from the extensometers and roof convergence depicted in the figs 2 to 7, show the sudden increase in the deformation measured by extensometer station (up to 50') floor extensometer station and room convergence. The effect of this reloading is also picked by other stations in other rooms and drifts but with less impact.

From the above analysis, it seems reasonable to assume that the roof fall in SPDV test room 1 has prematurely occured and the validation of other rooms against the geomechanical performance of this room must take into account in the stress history of this room.

Variation in Geology - Impact on stability of the room

The occurrence of the argillaceous halite near the top of the pillars, as shown in fig. 5-2 of Volume 2 of Geomechanical data, would expedite the mobilization of the horizontal stresses and the eventual shearing of the halite roof beam. The same figure depicts the variation in the floor geology changing from a thick polyhalatic halite in test room 1 to clear halite in other three rooms with variable thicknesses. The magnitude of the floor heave, being experienced in room 4, and not experienced in other rooms could be as the result of this variation.

The presence of Argillaceous Salt about 1-2′ above the floor beam in some of the rooms may also have the similar effect as its counter part above the pillar, in expediting the floor buckling and shear failure of floor.

The undulating nature of this bed, as was seen in room 6, panel 1 could have a marked effect on the magnitude of the floor heave and the floor buckling and eventual failure of floor beam.

The roof and floor slotting has already been discussed in earlier part of this report. The undesirable effect of these geological anamolies would be eliminated if in future design of the panels the mining horizon is moved up allowing the anhydrite "B" to form the immediate roof. This change in mining horizon would benefit the room stability by isolating a thicker halite floor beam eliminating or minimizing the floor heave, and at the same time eliminating the horizontal stresses along the boundary of

argillaceous halite and the halite roof beam.

Currently the pillar spalling between the upper argillaceous halite bed and what seem to be a lower argillaceous halite bed does occur. The tensile failure of the rock between these two horizons could have a detrimental effect on the stationed bins in room 1. The proposal to move the mining horizon would also eliminate this problem.

Choice of other alternatives to room 1 - Present & Future

The following discussions examine the other possibilities which could be rendering themselves for consideration if the performance of life span of the panel 1, room 1 is not acceptable.

a) **Use of other rooms 2 - 7**

The examination of Table I reveals the lower closure rate of room 2 over the same period of years as compared to room 1. This exceptionally higher rate of closure is basically due to reloading of room 1 as the result of excavation of other rooms. It has been stressed that the ventilation requirements, prohibits the use of other rooms. The choice of room 2 as the test site for waste could prove to be a compromise with minimum disruption to ventilation. In the meantime, the room 1 will be monitored for gathering of information on the performance of the bolted room providing much needed data for the future room design.

b) The use of 5 room system to minimize the effect of horizontal stress. This has been discussed in detail

c) The change in mining horizon and moving the roof height to anhydrite (b)

d) Sequential exploration of rooms to avoid reloading

e) The choice of less stiffer pillars to minimize the shear fracturing of the roof

Conclusion

This report has examined the pertinent geomechanical data related to the life of room 1 and SPDV test room, and has drawn conclusion based on the factual data and the personal experience of the writer. It is in the opinion of the writer that in this project, we are expecting the geomechanical performance of a permanent support, from a "mine opening" in a formation which is governed by a very complex behaviourial pattern. The local variation in geology, and the changes in the stress history of the model room SPDV 1 makes the engineering judgment a subjective one. Based on the best mining and rock mechanics practices, the geomechanical performance of the WIPP sites has been monitored. The factors as mentioned in Dr. Roy Cooks' position paper, some unquantifiable and some other unknown factors make the probabilistic approach to the determination of the life of the room an impossible one. There is a saying in rock mechanics community that "on shutting a mine, we will have enough knowledge to re open

the same mine"

I feel that under the circumstances, the geomechanical data has provided the early warning system for roof fall. To achieve better predictability in the range required for the proposed test could not be guaranteed in a mining environment, irrespective of expenditure.

The choice of salt for its healing properties; creep, make it a more difficult rock to predict. This is a fact that has to be accepted, maybe if such an assurance in term of room performance is required, the test should be conducted in a different environment, mining or otherwise.

The future design of the opening could ensure a more stable room but in no way reach the expectation of the risk required.

P. Mottahed, Ph.D., P. Eng., C. Eng. MIMM

Statement 1 Panel Member P. Mottahed

As described in text of this report, the additional support provided by means of rock bolt is a temporary measure. The creep of the roof beam will continue and as the result of presence of horizontal forces acting in the beam. The mechanism of roof bolting by suspension is becoming more complicated. The creep of salt will cause the bending of the bolts. The source of the problem i.e. the horizontal forces should be reduced and eliminated. This could be achieved by slotting of the roof beam and access for the maintenance of these slots need to be maintained. The same problem will be experienced by the floor. Hence the floor slotting should be performed and the slot remained open by maintenance.

The comprehensive geomechanical monitoring of the opening and associated formation has indicated the ability to predict the failure of the roof beam; SPDV Room 1. This lead time of two years could be pessimistic as the effect of rock bolts and their performance in providing additional support is not taken into consideration. On the other hand, the modelling of the performance of the SPDV room 1, to assess the life of panel 1, is not realistic as the SPVD room 1 was prematurely failed. With these two provisions in mind, with high degree of confidence could be stated that the minimum life of the room 1, Panel 1 beyond July 1991 is 2 years, (total no. of 7 years) with an upper limit of 3 years life. This life could be further extended if some remedial actions are immediately undertaken. The slotting of the roof with use of timber cribs to support the overhanng could be an early solution.

It is a proven technique and easy to monitor. The suggestion of bolt and lacing; as practiced to prevent rock burst may have some merits, but less easily quantifiable. If these additional supports are provided, the life of the room would be extended by an additional 3 years albeit at a loss of space for test programme. The above estimate is based on practical experiences in similar circumstances in salt rock. The level of confidence in the estimate would increased with evaluation of the performance of the additional support in first year and hence a more confident figure for the life of the room could be established. It must be explained that with the aging of the room maintenance of the room on a required basis is required.

Statement 2 Panel member P. Mottahed

a) The rock bolting programme could not ensure the stability of the room 1 panel 1 up to the completion of the test in July 2000 (total life of the room 14 years)

b) To minimize the effect of rock bolting immediate measures to reduce the horizontal stresses need to be carried out. This as outlined in statement (a) could increase the life of the room by a maximum factor of 2

c) The rock bolting programme with the factor of safety of 1.7 would be an effective means of support but as the complexity of horizontal stresses will diminish the effectiveness of the bolt.

d) The bolt above anhydrite b is already undergoing creep deformation. This deformation will continue causing the lateral movement of the anchors and the possibility of anchor failure, wedge or leaf failure

Statement 3 Panel Member P. Mottahed

The long term stability of the excavation in salt in a mining term, is a relative term. The haulage roads which are to remain open for the life of mine are constantly maintained. With introduction of other support provisions, eg. rock bolting in association with roof slotting, erection of wooden crib, lacing and strapping and floor and pillar rehabilitation. These measures are performed on a regular basis to ensure the long term stability requirement of the conveyance roads.

It is nice to be able to use probabilistic approaches for risk assessment, but the application of this approach is not common. Attempts in using this technique in assessment of risk associated with flooding of Potash mines was undertaken in early 80's by Potash Corporation of Saskatchewan some 2 years later Rocanville Mine was flooded.

The geological parameters as described in the text of the report, prohibits a comparison of what appear to be similar rooms together.

With regards to the data base or experience, no such information are available, or if there was, the direct application of the data base to appraise day to day performance of the openings on an operational basis would neither be practical or realistic.

As described earlier, the comprehensive rock mechanics instrumentation programme, as installed by the Geomechanical Engineering Department of the WIPP project is unique. It has incorporated every possible means of assessment of the performance

of underground openings and associated strata. There are tremendous volumes of data available which need be analyzed. The continuous analysis of the data as they become available, would further increase the level of confidence in predictability of the performance of future openings.

Statement 4 Panel Member P. Mottahed

a) This point is already addressed, however, installation of additional support, cribs and slotting (both roof and floor) or installation of additional bolts, with lacing, or combination of these supports, would guarantee the opening of the room but the required headroom of 10′ could not be achieved (statement 1).

b) Manouvering of jib cutters and rock bolting machine for future slotting operation and the rock bolting maintenance

c) The possibility of removal of cables is a fact that is to be lived with, as it is not possible to precisely predict the exact location of future rock deformation, fracture and possible slotting.

Statement 5 Panel Member P. Mottahed

This point has already been addressed throughout the text in brief. The comprehensive geomechanical instrumentation and monitoring of the rooms would provide sufficient warning well in advance of 6 months for the removal of the bins.

The installation of load cells on bolts to monitor the load transfer to the bolts would greatly assist the correct installation of additional bolts if necessary.

If cribs are installed, use of flat jacks to monitor the load sustain by the cribs and finally, in case of slotting, a gauge to indicate the closure of the slots to respond to the timely re-slotting operation.
The onset of the increase of the closure rate to 7"/year.
could be used as the criteria for removal of waste bins.

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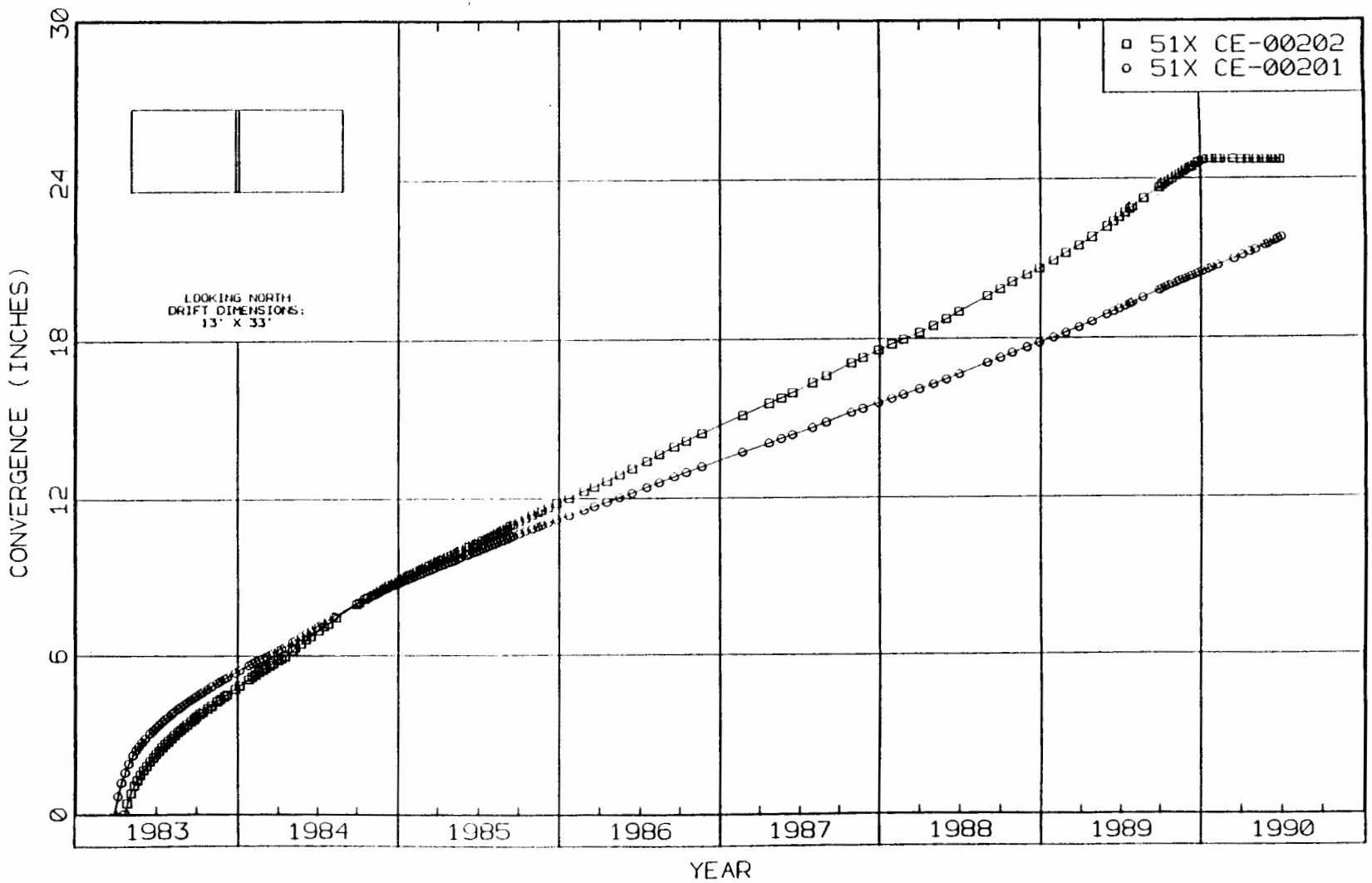
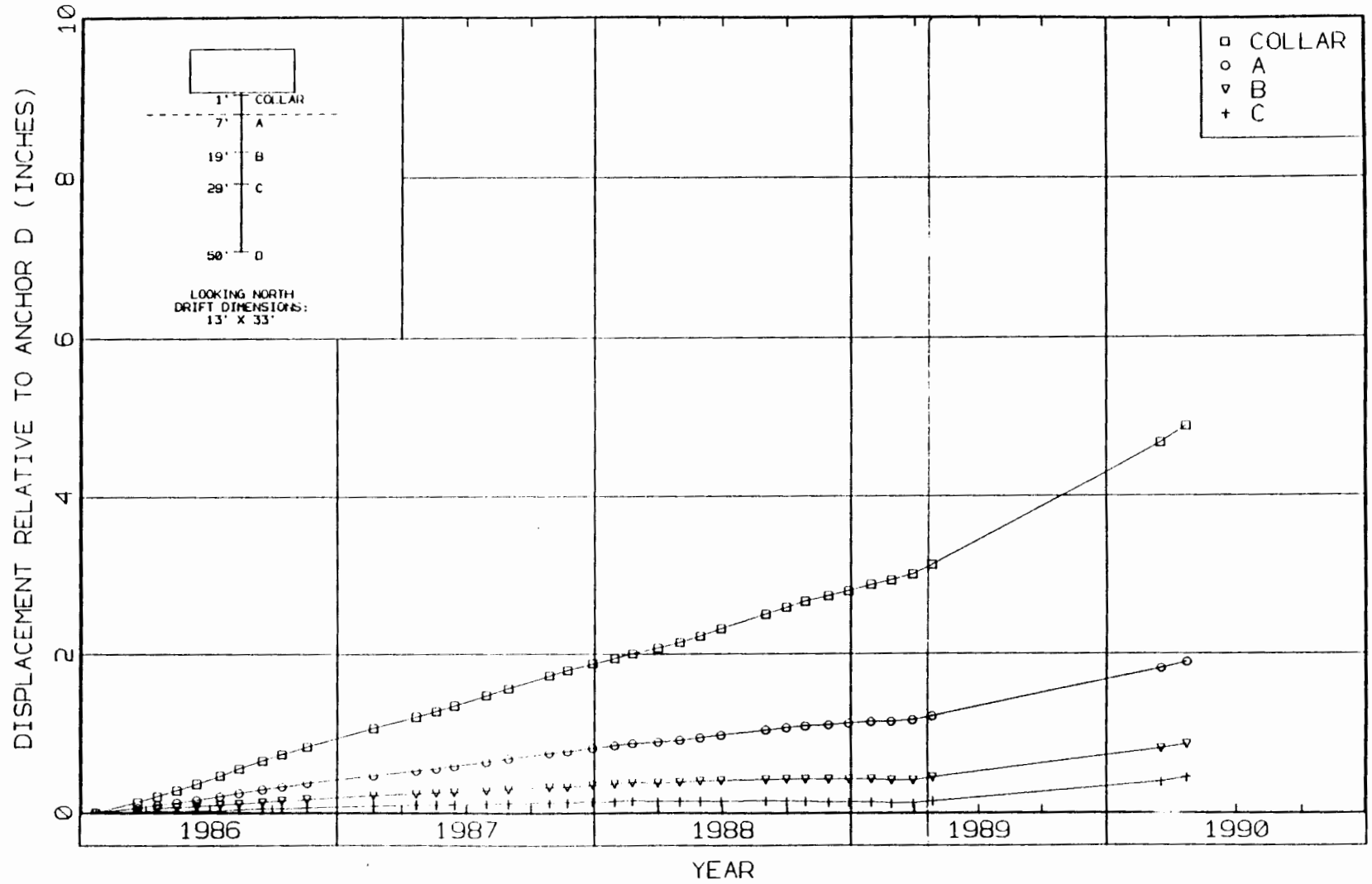


NOTES:
1. EXCAVATION DATE: TEST ROOM 1 CENTER APRIL 4,1983, TEST ROOM 2 CENTER MARCH 10,1983.

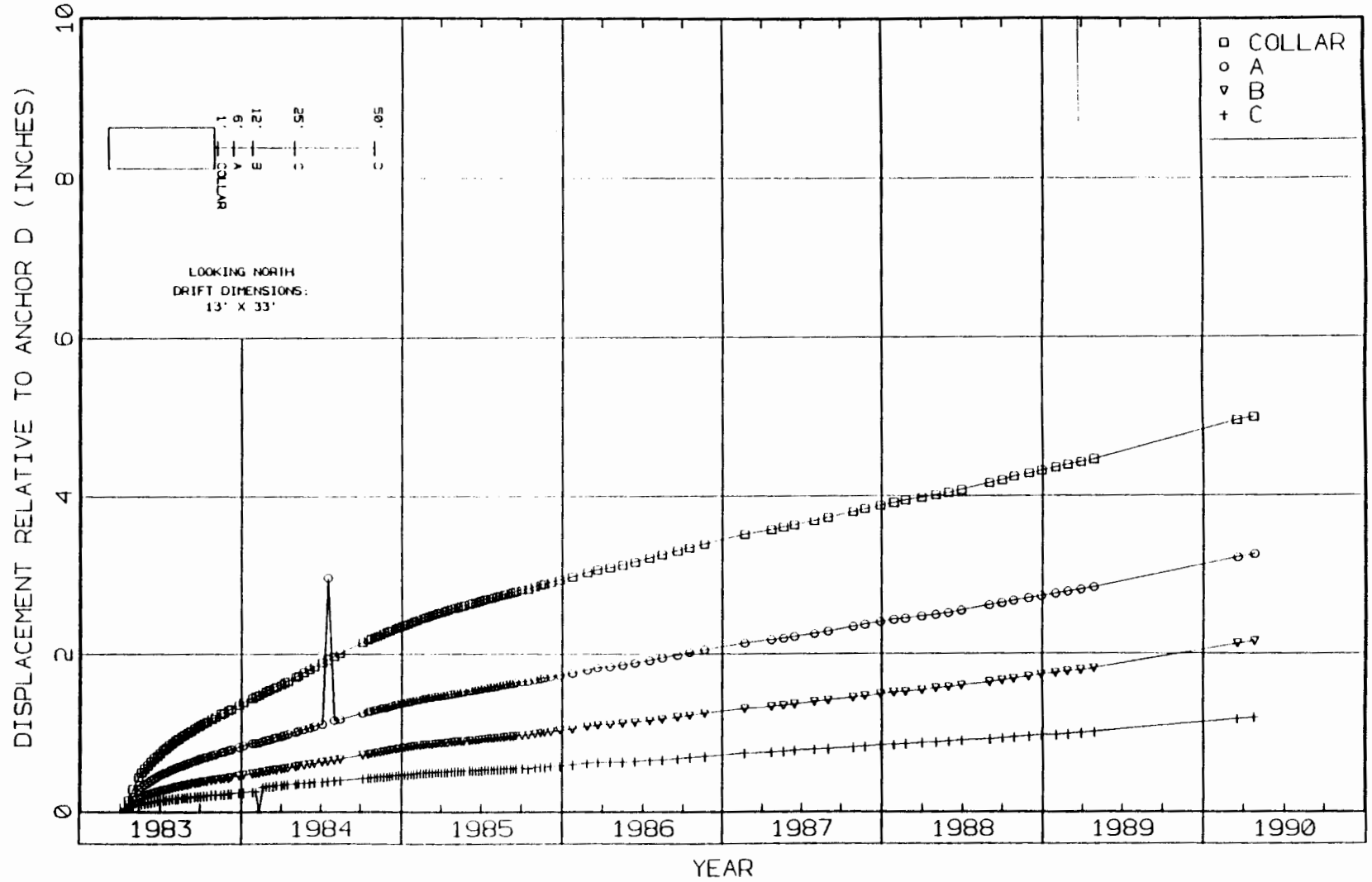
FIGURE 5-13
CONVERGENCE METERS
TEST ROOMS 1 & 2
CENTERPOINTS



NOTES:
1. EXCAVATION DATE: APRIL 4, 1983.

FIGURE 5-10
EXTENSOMETER 51X GE-00269
TEST ROOM 1-CENTER
FLOOR

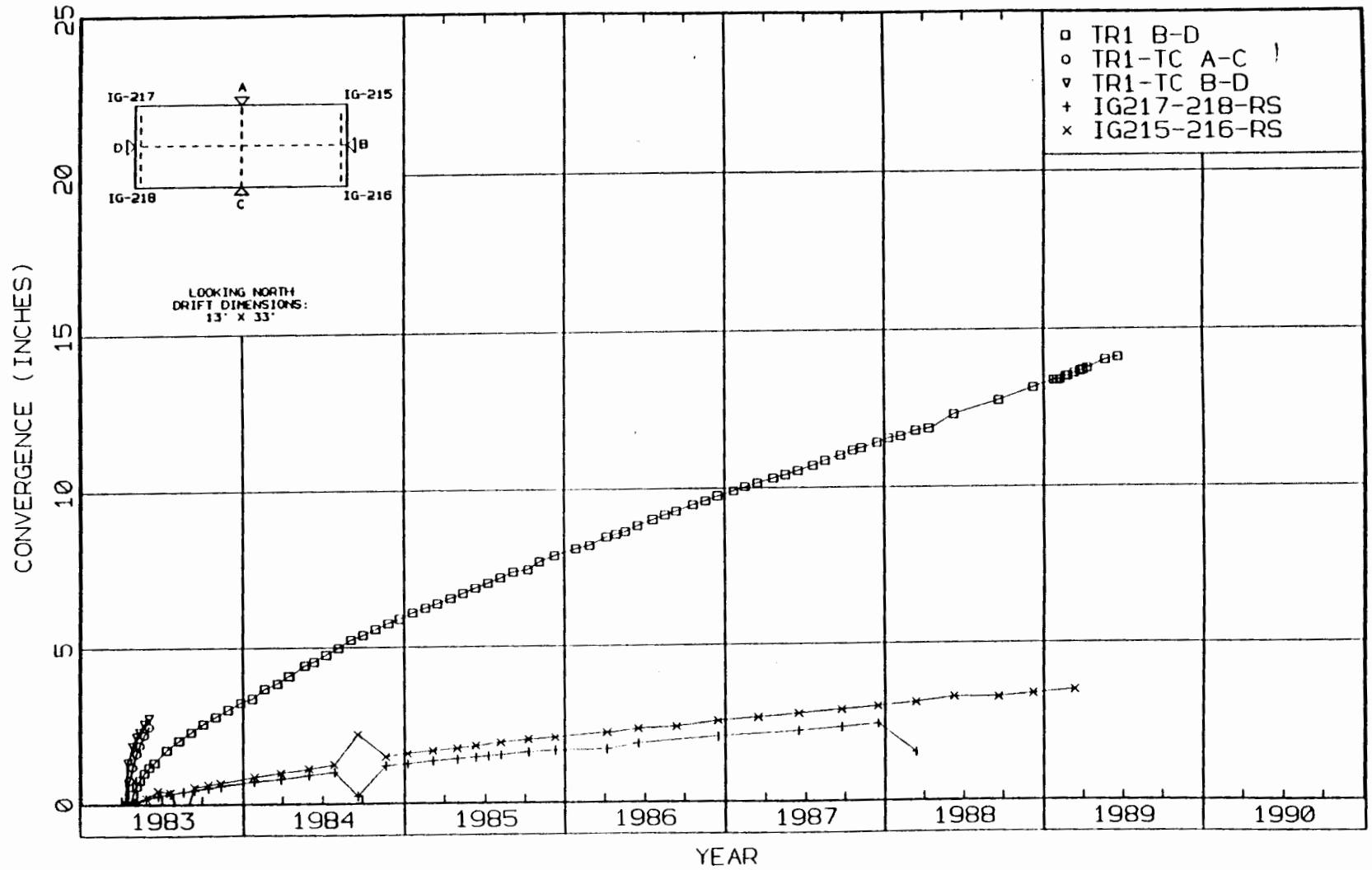
FIG 3



NOTES:
1. EXCAVATION DATE: APRIL 4, 1983.

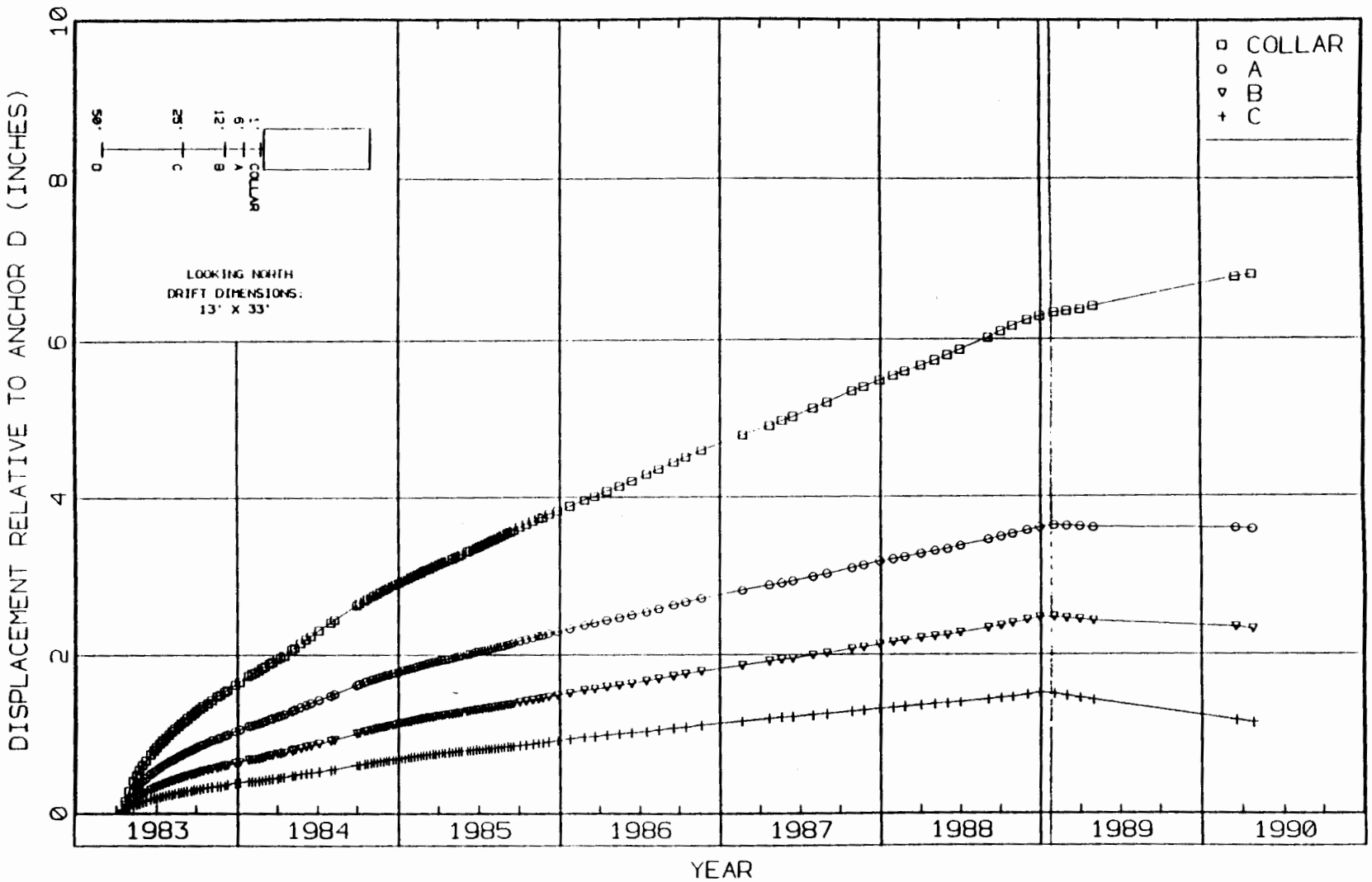
FIGURE 5-4
EXTENSOMETER 51X GE-00217
TEST ROOM 1-CENTER
EAST RIB

FIG 4



NOTES:
1. EXCAVATION DATE: APRIL 4,1983.
2. INSTRUMENT NO LONGER ACCESSIBLE.

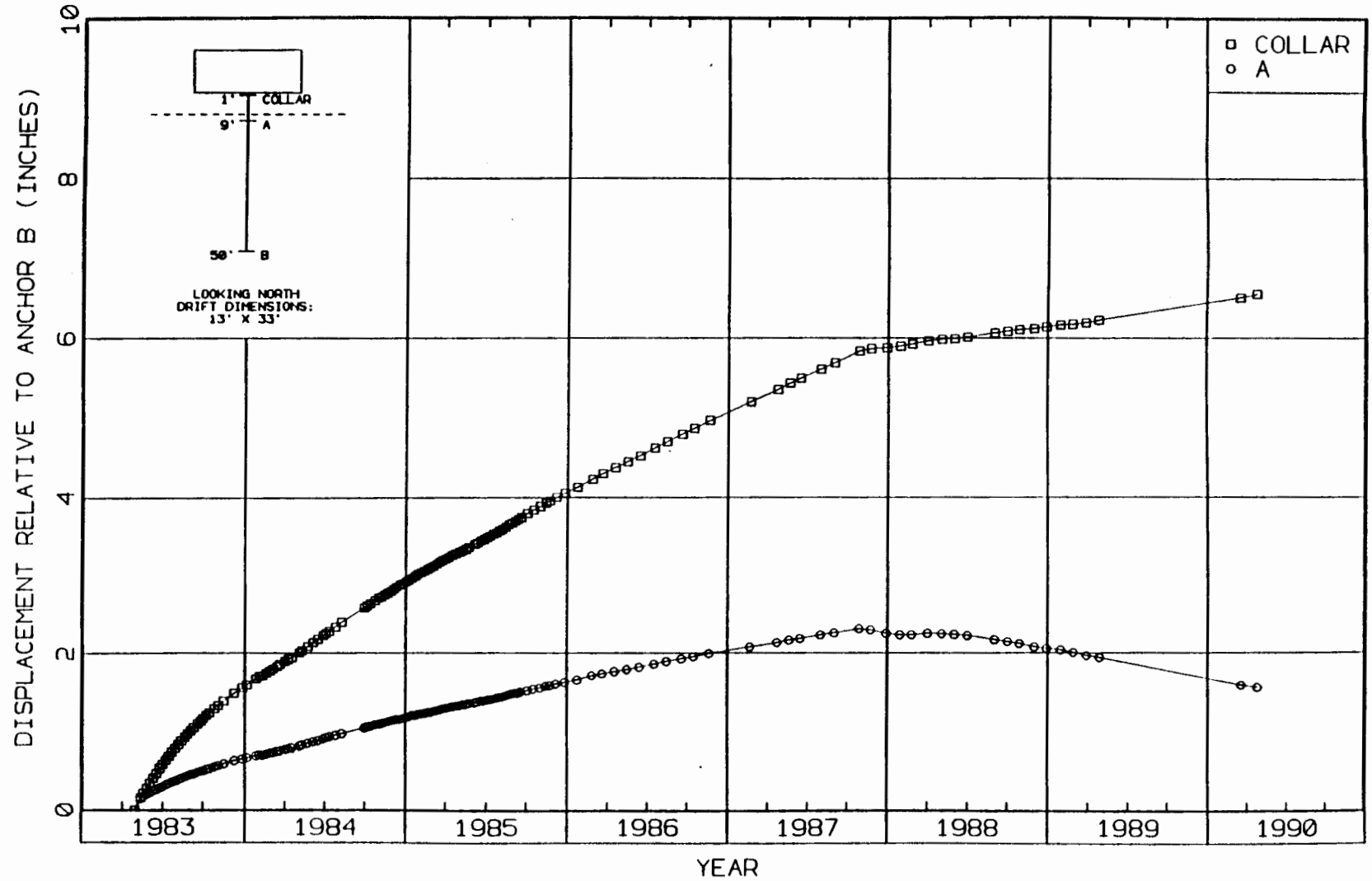
FIGURE 5-11
CONVERGENCE POINTS
TEST ROOM 1
ALL CHORDS



NOTES:
1. EXCAVATION DATE: APRIL 4,1983.

FIGURE 5-6
EXTENSOMETER 51X GE-00219
TEST ROOM 1-CENTER
WEST RIB

FIG6



NOTES:
1. EXCAVATION DATE: APRIL 16,1983.

FIGURE 5-70
EXTENSOMETER 51X GE-00209
TEST ROOM 4-CENTER
FLOOR

FIG 7.

CLOSURE RATES BY TIME SINCE EXCAVATION

Room	Year 0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8
Panel 1:								
Room 1	4.13	4.26	4.60	3.24	(3.57)			
Room 2	1.65	7.56	3.40	2.28				
Room 3	9.85	3.34	2.71					
Room 4	8.33	3.14	2.56					
Room 5	9.19	2.76	2.28					
Room 6	8.35	3.41	2.76					
Room 7	9.26	3.22	2.70					
SPDV:								
Room 1	6.05	3.71	2.93	2.88	2.85	3.41	5.27	11.11

Closure rates in (inches/year)
Room 1, Panel 1, rate for year 4-5 is only for 9 months.

TABLE 1

PRELIMINARY

REPORT SUBMITTED

BY

MR. J. PARKER

The summary report contains:

- An accurate record of the meetings of the Geotechnical Panel on Panel 1 Stability.

- A copy of the report provided to Westinghouse by this panel member.

- An accurate presentation of the consensus agreed to by the panel members at the meetings on the 23rd and 24th of April 1991.

Panel Member	Date

Signed off sheet from Mr. J. Parker was unavailable at the time of publication.

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Dr R F Cook
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April 16th 1991

EXPERT PANEL DISCUSSION - LIFE OF PANEL 1

Hello Roy:

This is a quick response to your request for comments on the Five Statements you gave us concerning the life expected for the rooms in WIPP Panel 1. It follows a review of data and reports you provided to panel members, a visit to the underground operations near Carlsbad and a discussion with the panel members and others last week (April 9 and 10, 1991). My qualifications to comment include a total of 45 years working in and around mines, with the last 20 years as a consultant working primarily on mine design and ground-control problems in a couple of hundred mines, including 11 salt mines, 2 trona mines, 3 potash mines and 3 gypsum mines. These mines in evaporites exhibit conditions much like those at the WIPP project.

First I would like to make a general comment. I sometimes complain that I lead a miserable life, dealing only with failures. "Nobody invites me to go look at a nice mine". But this was an exception; I think that this was an unusually clean, safe operation, showing good workmanship.

* * * * *

PROBABLE MODE OF FAILURE OF THE ROOMS

I want to discuss this before commenting on the Five Statements because the responses depend on the mode of failure as I see it. I will have to be brief but could expand on the topic if you wish.

I base my thinking on observations at the site, on the data you gave us, on our discussions, and on experience at the other evaporite mines.

Although measurements of convergence show that pillars 100ft wide did not prevent the mining of a new room affecting convergence rates in adjacent rooms, after that gross change in the environment the rooms seem to act independently (the fall in SPOV 1 did not affect adjacent rooms).

We were told that the stress field before mining was probably hydrostatic,

i.e. about 2000psi in all directions, because we are working in rock salt.

Under those conditions the most stable cross-section for a tunnel would be circular.

Observations and data show that that is the case, with minor modifications due to geological discontinuities. The opening is nominally 33ft wide so the radius of a circumscribed circle would be about 17ft. The opening is about 14ft high so half the height would be 7ft - so the top of the circle should be about 10 ft above the initial ceiling - which would be a couple of feet above the anhydrite/clay seam - which is just about what we see in SFDV Room 1 rooffall. A similar situation seems to exist in the other SFDV rooms, as shown by fracture mapping in observation holes.

A similar situation also seems to exist in the floor, modified by the presence of the anhydrite bed about 5ft below the initial floor.

a) FAILURE IN THE FLOOR: If we had measured the virgin stress field in the anhydrite bed I expect that we would have found conditions different from those in the salt. Because it is a stiffer material, less fluid, I would expect lateral stresses higher than 2000psi (left behind from a deeper burial) and I would not be surprised if the greatest horizontal stress had a distinctly preferred direction. I would expect the anhydrite to tend to buckle.

We were told that the thickness of the anhydrite and the top of the anhydrite are irregular, so I would expect the peaks on top of the anhydrite to express local effects on the room floors. The thickness of the salt floor would be least on top of those peaks so the salt would be stressed more there and it would be weaker there. Thus there would not be a simple geometric relationship between room orientation and room geometry. It would be interesting to check that theory by defining the top of the anhydrite in detail - although only for future planning, not for immediate value.

The stress concentrations around the opening would be highest immediately after the excavation was made, but with very high stress just inside the salt and zero at the skin of the salt something would have to give, so we should expect salt failure at the corners, and concurrent redistribution of the high stresses. That redistribution requires movement, of course, and we see it either as fracturing or as flow of the salt. As you know, we often see deterioration at the upper corners of rooms, sometimes attributable to locally clay-rich salt, but sometimes not.

With the highest stress concentrations at the corners and lower stresses further inside the salt I would expect the floor beam to want to flex DOWNWARD, and it might try to do that, and some peculiar fractures might result, perhaps a dish-shaped spall from the floor, but after a while, maybe months, the floor rocks would move in the direction of least restraint - upwards.

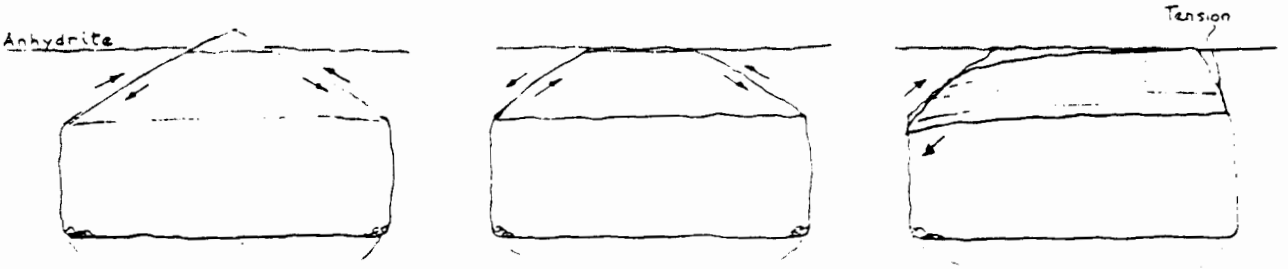
Another way to describe this activity would be to say that the stresses around the mine opening would make the opening assume the most favorable shape - the circle, with the lowest possible stress concentration factor, which is 2 x background stress, with compressive stresses around the circle, low stresses within the circle - and the dish-shaped masses popping into the opening.

b) FAILURE IN THE RIBS. For the same reasons there would be a tendency for the ribs or walls of the openings to move inward, but the amount of movement there should be less because the ribs are almost at the circumference of the circumscribed circle already.

c) FAILURE IN THE ROOF. For WIPP this is the most critical zone of failure, but the reason for failure is the same. The opening tries to assume a circular shape, which requires fractures to develop upward and inward from the juncture of roof and ribs. That is where we see low-angle fractures developing first. NOTE THAT THESE FRACTURES ARE NOT EASILY DETECTED IF WE DEPEND ON CAPLAMPS WORN ON THE CAP. IT IS BETTER TO HOLD THE LIGHT IN THE HAND AND SIDE-LIGHT THE ZONE, SO CASTING OBVIOUS SHADOWS WHERE A SHEAR HAS DEVELOPED.

We have seen that it takes 3 or 4 years for the salt to move enough to initiate these shears, time during which we measure a diminishing rate of movement, which is mainly due to gradual relief of stress in the roof salt.

It then takes more time for the fractures to propagate but, at our depth, they do it. A very thin beam should buckle and fail quickly. In a thicker roof beam the fractures would propagate fairly quickly to the top of the beam. In a beam as thick as ours, under our conditions, the evidence is that it takes several years for the shears to reach the top. Judging from our distanced inspection of the rooffall, and as seen at other mines, shears may grow from both ribs and meet at a peak, or perhaps they will level off at a discontinuity like the anhydrite/clay seam. In some places enough stress may be relieved by a shear growing from only one rib - then a cantilever situation develops and the cantilever may stand for a long time, or it may fail because of its own weight, or it may fail because it is wedged downward by lateral movement on the inclined shear. I would like to investigate why the initial shear prefers one particular side of a room.



As the shears propagate we measure significant rates of roof-to-floor convergence, rates higher than during the "stress-relief" process.

When the shears have almost cut a wedge or frustum of salt free the convergence rates increase sharply, up to complete failure of the roof.

IT IS IMPORTANT TO RECOGNIZE THE MECHANISM AND THE FACT THAT THE FAILURE MECHANISM IS DRIVEN, ESPECIALLY IN A BEAM-LIKE ROOF SUCH AS OURS, BY HORIZONTAL STRESSES. THE ENDS OF THE BEAM ARE SHOVED INWARD, AS SHOWN VERY CLEARLY BY OFFSETS IN OBSERVATION HOLES. WE CAN SEE AS MUCH AS 6 INCHES OF LATERAL MOVEMENT IN A ROOF WHICH HAS NOT YET FALLEN!

There can be little doubt that extra-high horizontal stresses come off the tops of our 100ft-wide pillars. Since those pillars are too stiff to yield a high concentration of vertical stress builds up and overstresses the roofbeam salt - which then wants to squeeze sideways into the mine opening. In the overcast we examined the cutoff ends of the roofbeam and saw that they had moved sideways into the void at a rate around 1/2" per year, at both ribs. The movement occurred at the anhydrite/clay seam, with the roofbeam apparently acting as a unit.

Prior to 1975, at the Cayuga salt mine in New York state, there were dozens of similar failures under similar conditions. The depth was 2000 to 2300ft, rooms were 32ft wide and 12ft high, and pillars 88ft square. Failures began at the juncture of roof and ribs and shears developed until heavy falls occurred, either tent-shaped, arched or as cantilevers. So many falls occurred that MSHA threatened to close the mine. The problem was solved by changing to yielding pillars, only about 20ft square instead of 88ft, designed so that they would yield rather than build up high vertical stresses in pillars - hence high horizontal stresses in the roof.

We were told that the WIPP openings were designed largely by reference to those in the local potash mines, which makes sense. The reports also state that the extraction ratio was reduced significantly, probably because the depth at WIPP is about twice the 1000ft depth of the potash mines, but in hindsight we could say that that may not have been the right move. Most of the local rooffalls I have seen in the NM potash mines HAVE BEEN ALONGSIDE PILLARS WHICH WERE UNUSUALLY WIDE AND STIFF, and the WIPP design gave, I think, an unfortunate degree of pillar stiffness which shortens the life of the storage rooms.

The problem with long-term stability in salt mines is common. I have been working on it at several mines and we recognize guidelines which may help us at WIPP.

At most mines the openings close to the shafts are stable - some have stood well for as much as 50 and 100 years - notably at the Retsof mine in NY state. They are different in that they are usually smaller than mine production openings, they are usually narrower, they usually have a lower width:height ratio (more nearly circular, or at least more nearly square), and they are further apart - isolated but in a zone of very low extraction, and often they have thicker roof and floor beams. Those factors usually contribute to long-term stability, as they have, I believe, at WIPP, in the access drifts but not in the storage rooms.

At another extreme we can design for long-term roof stability by using small pillars and high extraction ratios. The general idea is to shed the high stresses onto distant abutments. I liken it to 10 men carrying a heavy telephone pole, with eight crafty fellows in the middle bending their knees a bit...

Between those two extremes which give good long-term roof conditions there is a range of designs which contribute to long-term instability. I think that the WIPP storage rooms fall within that range but, as you have pointed out,

they would still satisfy the original requirement of a 5-year total life with eventual closure.

Now I go on to respond to The Five Statements.

1. FIRST STATEMENT: AN ESTIMATE CAN BE ESTABLISHED FOR THE PERIOD OF TIME THAT PANEL 1, IN PARTICULAR ROOM 1, WILL REMAIN ACCESSIBLE ON A DAILY BASIS BEYOND JULY 1991.

1.1. WITH NO MAINTENANCE OR ADDITIONAL SUPPORT. I believe that SPDV Room 1 which collapsed after being open 8 years gives us a clear indication of what to expect. Fracture patterns defined in roof observation holes and accelerating closure rates in the other SPDV rooms seem to confirm the 8-year life expectancy.

The rooms in Panel 1 are very much like the SPDV rooms, so I would expect them to behave similarly - with one possible exception - which is that the Panel 1 rooms have been reinforced with 10ft mechanical roofbolts. However, my personal thinking is that those bolts will not change the life expectancy of the rooms very much - because of the mode of failure which I expect. Let me explain that again:

I expect that salt failure around the rooms will be expressed as lateral movement on the planes of failure (shears), so that the mechanical bolts will not be subjected to simple tension over their full 10ft length, but to shearing, or perhaps to tension in that very short length of bolt which crosses the plane of failure. Thus I would expect bolts to fail first in the zones of greatest lateral movement, then in succession as succeeding zones were sheared sufficiently. Under these circumstances parts of the bolts might fall out of the roof - but often the broken-off lower portions of the bolts are snagged and held at the shear planes - so we do not know of the failures until the roof hits the floor. Observation of the amount of offsetting in empty holes in the roof gives us some idea of the likelihood that bolts have been sheared.

I conclude therefore that the bolts as installed will not make much difference to the life expectancy of the rooms.

Inspection of the mining progress drawings shows that Room 1 was completed in August 1986 but that the other 6 rooms were mined between January 1987 and March 1988, which should give them a year or so of additional life.

The panel of experts seemed to lean toward a slightly more optimistic forecast, as if 8 years was a minimum and additional life was a fair possibility, but I have doubts about that ...

First I remind myself that the 8-year life for SPDV 1 was TOTAL life, up to complete failure, and at present we are considering USEFUL life, which will be 6 to 12 months shorter. We would not want to be working much in the rooms while the first slabs were falling.

Second, the updated convergence graphs for Panel 1 rooms which you gave us indicate convergence rates GREATER than those measured in the SPDV rooms at a similar age; around 3.5"/year, 3", 3", 2.7", 2.2", 3" and 3"/year in rooms 1

through 7 respectively. At a similar life-stage in SPDV rooms the rates were between 2.18 and 2.85"/year.

It seems that the only significant physical differences between the SPDV and Panel 1 rooms is that there are seven of the latter vs 4 of the former, and that the Panel 1 rooms have been bolted. I suspect that the extra rooms have made the difference, noting especially that the convergence rates are highest in the outer rooms and lowest in the inner rooms (see figures above) - which suggests that the outer rooms are absorbing more of the "far-field creep", or something like that, and to some degree protecting the inner rooms ..

TO SUM UP FOR THIS CONDITION, WITH NO MAINTENANCE: You or I should ponder over the convergence and fracture data further, not so much to crunch numbers as to recognize behavior patterns. I also recommend drilling several arrays of observation holes and scratching to find fracture patterns, and the way they change with time.

MY PERSONAL THINKING IS THAT THE USEFUL LIFE FOR ROOM 1 PANEL 1 WOULD BE ABOUT 8 YEARS TOTAL, POSSIBLY LESS BECAUSE IT IS THE OUTER ROOM OF THE SEVEN AND BECAUSE IT IS MOVING FASTER THAN DID SPDV 1.

1.2. WITH LIMITED MAINTENANCE, WITHOUT MOVING BINS. I would anticipate that barring down the slabs of loose salt which can be expected to appear as the early signs of failure, mainly at the juncture of roof and ribs, would remove some of the hazards during the early stages of failure BUT WOULD NOT EXTEND THE TOTAL USEFUL LIFE OF THE ROOM SIGNIFICANTLY i.e. ONLY FOR A FEW MONTHS.

1.3. EXTENSIVE MAINTENANCE ON AN AS-REQUIRED BASIS, WITH BINS REMOVED FROM ROOM, IF NECESSARY, DURING MAINTENANCE ACTIVITIES. This approach would be much like some in salt mines where bad roof develops in critical areas, as over a main conveyor. In such a place it may not be possible to move the conveyor to another room, or even to move it temporarily, so the operator may choose to rebolt again and again. The vertical load to be suspended may not increase much with time but if roofbolts shear they have to be replaced. The roofrock usually breaks into smaller and smaller pieces so something like chain-link wire-fence material is bolted up to prevent small chunks falling.

GIVEN THE OPPORTUNITY TO GO INTO THE ROOM AND FIX AS NECESSARY, I THINK THAT THE ROOM COULD BE KEPT OPEN INDEFINITELY, i.e. FOR TENS OF YEARS. I know of one place where the roof over a room 45ft wide is now suspended by a third set of bolts; even though there is a gap 18" wide up in the roof.

2. SECOND STATEMENT: THE ROCKBOLT SYSTEM AS CURRENTLY CONFIGURED IS SUFFICIENTLY EFFECTIVE TO ENSURE THAT THE TEST PROGRAM IN PANEL 1, IN PARTICULAR IN ROOM 1, CAN BE COMPLETED.

Thankyou for this opportunity to comment on the bolting.

First I have to question the design assumptions - which are quite different from those usually encountered in mining.

Design load for the Jennmar grade 75 3/4" bolts is said to be 70% of yield strength, or 17,500lbs.

I checked with Jerry Freas today, (606-744-3600) probably Jennmar's best-informed man, and he shared this information:

Grade 75 bolts have a minimum yield strength of 75,000 psi, but batches range up to 100,000 psi, thus 3/4" bolts should not BEGIN to stretch until loaded to somewhere between 33,000 lbs and 44,000 lbs

An implication is that WIPP is designing for only 50% of yield strength, whereas most mine operators design for full ultimate strength - to get their money's worth.

Another implication is that whereas WIPP intends that the bolts will either slip or stretch at the design load in reality they are far from stretching - so they are stiff supports which might fracture the salt, especially where the bolts pass through feather-edges of salt.

It is assumed that the 10ft bolts will stretch 10%, i.e. 1 foot, before breaking. I have my doubts so I asked Jerry what he thought. He dug out his specs, which said that Grade 75 bolts should stretch 8% minimum - but that left room for debate because the tests are normally run on specimens only 8" long. As I think back over bolt failures observed I have the distinct impression that most do NOT stretch that much - instead they have a weak point at which failure occurs - in the threads or at the bolthead. Your Tom Brockman tells me, however, that he has actually stretched some 10ft bolts a full foot before failure - so I may be wrong ...

I would like to know how much stretch there has been in bolts which actually broke in the mine. I understand that at least one was recovered from Room 1, and sent to Sandia for analysis. How much did it stretch?

Another WIPP design criterion is that the bolt anchors should slip down the hole at design load instead of breaking, for controlled roof yield. A consummation devoutly to be wished, but does that really happen? Before designing that way I would want to know what really happens, and I could anticipate several types of behavior:

Given salt rock, and a short anchor with a bail on top and an open base, I would not be surprised to see the plug pull through the leaves after 3 or 4" of roof movement. That would be because the leaves pushed sideways into the walls of the hole.

Given a 4-pronged shell held together at the base I would not be surprised if the salt walls of the hole grabbed the shell well enough to hold the bolt up to its ultimate strength.

I think that most folks using mechanical anchors in salt choose a longer shell, like a D5, for greater bearing surface, but they are designing for maximum anchorage capacity. If you want to know how the Jennmar J3B's behave elsewhere in salt you may want to call Morton Salt at Weeks Island in Louisiana (318-867-4141)

I was surprised to learn that WIPP had selected anchors to match the specified design load - 17,500lbs. Normally we would select all components of the system to equal or exceed the strength of the most costly component, which would be the bolt itself. Normally then we would use either the D5

anchor or a couple of feet of resin at the top of the bolt.

When I asked why WIPP was not using resin one response was that it hadn't worked very well in early tests. Again I was surprised, and would expect new tests to show very good performance.

Most of the above discussion will not mean much if the bolts are rarely loaded in pure suspension, but I expect that WIPP will change the design criteria soon. As suggested in the panel discussion, and again in this report, I expect the bolts to be loaded largely in shear, unless we cut off the forces driving that shear - which is what I recommend. Then the bolts WILL be loaded in tension.

MY RESPONSE TO THE SECOND STATEMENT IS, THEREFORE, THAT THE CURRENT BOLTING CONFIGURATION WILL NOT ENSURE COMPLETION OF THE TEST PROGRAM. I understand that requires 8 or 9 years of stability from time present.

3. THIRD STATEMENT: THE LEVEL OF CONFIDENCE THAT CAN BE PLACED IN THE ESTIMATE OF THE LIFE FOR PANEL 1 PROVIDED IN THE RESPONSE TO STATEMENT 1 IS IN ACCORDANCE WITH ACCEPTED MINING PRACTICES

I think that Steve Mckinnon of Itasca described our position well when he said that our chances for projecting information from the SPDV rooms onto the Panel 1 rooms are exceptionally good - because rarely in the mining industry do we see conditions as closely comparable as we see them in the SPDV and Panel 1 rooms - in regional and local geology, dimensions of rooms and pillars, and probably in the stressfield too.

CONSIDERING THE SIMILARITY OF THE SITUATIONS, AND THE LESS-THAN-FORTUNATE FACT THAT ROOM 1 IS AN END ROOM, AND THE HIGHER CONVERGENCE RATES IN PANEL 1 - I WOULD ACCEPT THE PROBABILITY THAT THE TOTAL USEFUL LIFE OF THE ROOM WILL BE 8 YEARS, NO LONGER, AND WOULD NOT RISK MANY THOUSANDS OF DOLLARS BETTING ON A LONGER EXPECTED LIFE FOR ROOM 1.

4. FOURTH STATEMENT: MODIFICATIONS TO THE SUPPORT SYSTEM IN PANEL 1 CAN BE IMPLEMENTED TO ENSURE THAT ACCESS IS MAINTAINED TO THE ROOMS ON A DAILY BASIS UNTIL THE TESTS ARE COMPLETED.

I don't think that there can be any doubt that we COULD install supports capable of keeping the room accessible. In a typical saltmine with roof conditions like these the operator might choose to install exterior supports, such as wooden posts or cribs, or yielding steel supports, always considering that the salt surrounding the opening will move inward almost irresistibly. For WIPP it seems that this approach would not be acceptable - because the supports would block traffic.

Internal supports might be used by the operator instead, but again he would have to recognize the almost irresistible salt movement, which means that the supports would either have to yield or break. Many salt miners have tried putting wooden squeeze blocks between the roof and the roofbolt plate - but almost always the bolts break before the blocks have squeezed an inch. The wood becomes hard and brittle when exposed to salt, as if pickled.

WIPP has already proposed a yielding system - bolts which stretch and anchors which slide - but I would not rely on those ideas until they had been proven.

Hamish suggested that WIPP could use the "lacing" system as used in S Africa and now in Canada as protection against violent rockbursts. As you probably know, special rods (looking much like steel cotterpins) and about 6ft long, are grouted into the rock to be supported, probably on 5ft centers, then wire mesh is held against the rock by steel cables which are laced in a triangular pattern from pin to pin. There is some "give" in the system, and it really does survive serious bursts which would have broken standard roofbolts. The broken rock is held together as if in a big onion bag - and the openings are still accessible.

That might work, and it is an idea worth considering, but as with the other rockbolting systems I would be concerned that those bolts which pass through planes of shear would be sheared by the movement of the salt.

If the mode of failure as I see it is correct - and it should be checked by further study of salt movement and fracture patterns - I agree with you that THE SOLUTION TO THE STABILITY PROBLEM IS TO CUT OFF THE STRESSES WHICH ARE CAUSING THE SALT MOVEMENT.

Several approaches have been tried in the mining industry. Let me list some.

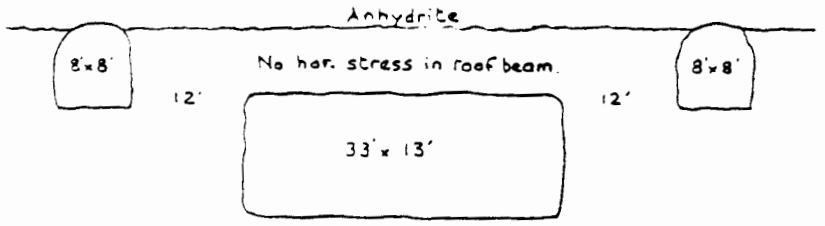
4.1. STRESS-RELIEVED ROOMS. The pillars between the storage rooms are 100ft wide. If we were to drive new rooms 33ft wide through the center of these pillars we would be leaving pillars between new and old rooms which would also be 33ft wide and about 14ft high. They would be marginally yielding pillars and almost certainly the new rooms would experience very little lateral stress, since much of the "far field" creep has been relieved by the old rooms.

An especially attractive circumstance at this site is that most of the salt mined from the new rooms could be stuffed into the existing rooms, so there would be no need to haul and hoist most of it - which means that new rooms could be mined in a couple of months from time of beginning ...

I CONSIDER THIS TO BE THE BEST WAY TO GET ROOMS WITH GUARANTEED STABILITY IN PANEL 1, QUICKLY. I RECOMMEND IT.

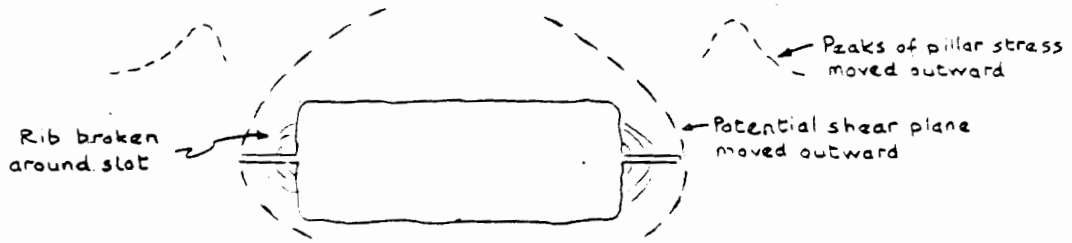
4.2. STRESS-RELIEVING TUNNELS. If we could drive small tunnels horizontally opposite the roof beam and perhaps opposite the floor beam, maybe with an Alpine-type miner, we could cut off the horizontal stresses. The new tunnels would be as small as possible, say 6x6 or 8x8ft, and separated from the rooms by narrow pillars of salt - say 10 or 12ft wide.

I did not hear much enthusiasm for this idea, probably because of restraints on time and equipment, but I think that it would work.



4.3. STRESS-RELIEVING SLOTS IN PILLARS. Some mines have used this approach successfully, using an undercutter to cut slots 6ft or more in depth, horizontally into the ribs, usually at mid-height. That seems to push the peak vertical stress further away from the room, which in turn seems to push the horizontal stress concentration higher in the roof. The low-angle shears which would normally show up at the juncture of ribs and roof originate instead at the hidden ends of the slots - and the dish-shaped mass which eventually separates from the rock above the room sits down on the rock below the slots.

This scheme would probably help the roof condition, but we would lose some additional headroom, and the ribs would probably become unstable and need bolts and mesh - because portions of the stress-relieved ellipse separate from the rockmass and sit on the slot. See sketch ...



4.4. STRESS RELIEVING SLOTS IN THE ROOF. This is a direct approach to solving the problem.

A common approach in deep Canadian potash mines is to cut a single slot along the centerline of the room, usually up to some natural plane of slippage. That leaves two cantilevered portions of roof hanging, one from each rib. If those cantilevers are short (because of a narrow room) and thick, they may need no support. The lateral stress is relieved until the slot is squeezed shut - and in our case it seems that a slot 6" wide would be closed in about 6 years.

In our case the cantilevers would each be about 16ft long, and 7ft thick, and perhaps already fractured by a shear beginning at the roof/rib juncture, so they could not be considered self-supporting - so they would have to be suspended with bolts. The problem then would be that we would expect at least 3" of lateral movement of the cantilevers - which could shear off the bolts and so leave the roof unsupported.

For those reasons I would prefer to make the slots close to the ribs, as close as conveniently possible, with one at each rib. As we discussed, the slots should be inclined slightly outward and upward, so that IF the roof slab ever did come loose it could still sit on the pillars.

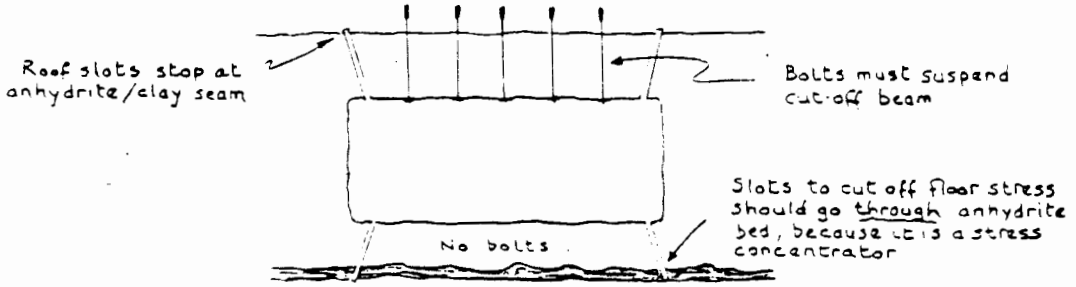
During and after the slotting operation the roofrock up to the anhydrite would have to supported entirely by roofbolts, in suspension, WITH NO LATERAL MOVEMENT. That means that we have to evaluate the bolts already installed, and perhaps install additional bolts. If we did have to I would probably recommend point-anchored resin/rebars.

The two 6" slots should provide 10 or 12 years of stress-relief.

A secondary effect, not to be forgotten, is that the stress-relief slots move the stress concentrations further away from the room, but they still exist, and they might cause failures further from the room. The comforting response to this is that we would be forcing the room to adopt a more favorable shape (circle or ellipse) around which the stress concentrations would be lower. Example: our 33 x 14 ft rectangle might have stress concentrations of x4 at the corners whereas around the circular shape they would be x2. Stress levels might thus be reduced from about 8000 to about 4000psi at the perimeters.

I understand that an undercutter could probably be procured locally. Once modified to cut the roof slots I would expect it to cut about 100 feet of slot per day, or one room per week.

THIS TECHNIQUE - ONE SLOT INCLINED UPWARD AND OUTWARD AT EACH RIB - WOULD BE MY SECOND CHOICE FOR PRODUCING STABLE CONDITIONS IN PANEL 1.



5. STATEMENT 5: THE GEOMECHANICAL MONITORING PROGRAM AND THE ROUTINE OBSERVATIONS IN PANEL 1 CAN PROVIDE SUFFICIENT WARNING TO ALLOW TIMELY RETRIEVAL OF THE WASTE FROM THE PANEL.

There is no doubt in my mind that the roof will give us warnings months before collapse, at least 6 months of advance warning.

I think that the behavior of SPDV Room 1 was typical and that a similar sequence of events will be followed in the storage rooms. I do think that the techniques could be refined a little, hence the understanding of the mode of failure, hence the interpretation of the instrumentation data.

I believe very strongly in rock mechanics as an art, not much of a science, therefore I value visual observations highly. There is a strong tendency for the science approach to be based on questionable assumptions, one of the most obvious of which has been to base WIPP design and interpretation on "creep" of rocksalt - whereas most of the movement and damage has been more like brittle behavior and fracturing.

I believe, for example, that we MUST examine the roof failure in SPDV 1, rather than speculate on it long-distance. It would take me about 10 minutes per visit, and although I don't know how to calculate the probabilities of somebody getting hurt there I feel certain that they are far lower than when I cross the street in the city, or drive to the airport, or fly commercially to US destinations. It would make sense to have only two people in the place at a time, in case one slips and falls, in case one accidentally dislodges a rock.

I would try to relate the convergence measurements to rock failure by having more arrays of observation holes drilled in roof, ribs and floor in the rooms. I would map and scratch them periodically, especially if the convergence graphs showed something unusual. As holes were closed off by salt movement I would drill new holes beside them, expecting several inches of total displacement.

Because I expect the highest stresses to be active midway along the rooms I would have one array there. To check that supposition I would have additional arrays, probably at the third or quarter points.

I would expect to define the mode and the zone of failure MUCH better in this way than by calculations in a computer.

If we do cut relief or relieved rooms or relief slots I would expect to verify the relieving behavior soon after doing the work. I would expect the salt to start moving into the slots immediately.

At the same time we should learn to interpret our convergence graphs better - and our diagnosis then should allow us to predict roof behavior more closely and with greater certainty.

* * * * *

I understand that you intend to install some USBM hydraulic load cells to measure "stresses" in the salt. I like that idea very much, recognizing that the data may not be exact (what rock data is anyway?) but I would very much like to know how close our suppositions are concerning vertical stresses, horizontal stresses, stress concentrations, changes in stress level, relief of stresses - and so on. Even crude measurements, I think, would be much better than relying on theoretical assumptions, and the cost of the instrumentation will not be great ...

One more thought on instrumentation: Could you plot your SPDV 1 convergence data on semilog paper? I have seen instances where a change in rock behavior was pinpointed better on the semilog plot, especially where the total

movements and the rates of movement were great, as in rocksalt.

Time is running out if this report is to reach you by April 19th, so I will stop now. If I can help you and your project further - just let me know

Report respectfully submitted,

Jack Parker

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JP/wp

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BOX 465

JACK PARKER AND ASSOCIATES, INC.

ROCK MECHANICS · MINING · GEOLOGY

WHITE PINE, MICHIGAN 49971

Dr R F Cook
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May 21st 1991

FOLLOW-UP ON THE SUMMARY OF EXPERT OPINIONS

LIFE EXPECTANCY OF ROOM 1 PANEL 1

Hello Roy:

Thankyou for the package of reports from experts, and your summary. I understand that we were going to talk about them by phone yesterday, but we didn't connect, and it might be difficult in the near future, so I will put my comments on paper.

First - you did a good job under difficult circumstances, with so many cooks in the kitchen ... I'm glad that I didn't have to do it.

1. GENERAL COMMENT ON THE EXEC SUMMARY. If some exec looked at the summary in haste, particularly the first section, he could get the impression that Room 1 Panel 1 has a life expectancy of 8 years from today. That should be corrected, of course.

2. THE MISSING RECOMMENDATION. My primary recommendation was and still is that the mission to provide stable storage rooms at least cost be accomplished by driving new rooms 33ft wide through the middle of the 100ft-wide pillars, ie between the existing rooms. I did not see that recommendation in the summary. Perhaps I did not state it forcefully enough. I'll try again.

The new rooms would be in stress-relieved ground.

The new rooms would be 5 years younger than the old.

The degree of certainty of life required would be MUCH higher.

Most of the freshly-mined salt could be stuffed into the existing rooms.

I estimate that the cost of cutting the salt and hauling it about 500 ft would be about $2/ton, or about $20,000/room. That would be cheaper than most of the other proposed fixes, I think.

As a check on my cost estimate I called Jim Ryan at Eddy Potash a few minutes ago. He said that I could quote these figures: Mining cost

in their thin seam, including all underground costs, for the whole of 1990, was $3.89/ton. He volunteered a guess that to cut the salt and haul it 500 feet would cost about $2/ton, and that it should be possible to cut and haul the 10,000 tons in 4 or 5 shifts.

That would be the quickest fix too.

The existing rooms would NOT be a total loss, since they provide the stress relief. We could almost claim that we planned it that way.

Backfill in the existing rooms would stabilize the system as a whole, for the long term.

Can there be any doubt that the new rooms would be the most cost-effective way of achieving the results needed?

If that approach is acceptable, most of the other statements need not be discussed, but a couple of them deserve it anyway.

3. ROOFBOLTS. First a comment on the modes of bolt failure - anchor failure vs bolt shear. I think that I understand the differences in opinion expressed. In some cases, as quoted by Parvis from the Canadian potash experience, failure has been at the mechanical anchorage. I would attribute that to choice of anchor, believing that a mechanical anchor with a larger bearing area, and/or with prongs held together at the anchor base (instead of a bail at the top) would perform much better. If failure IS at the anchor, then most operators in this country would switch to using a couple of feet of resin at the top of a rebar. Some combine a mechanical anchor with a slug of resin. Jim Scott's Dyna screws into a piece of plastic (essentially resin in a solid state) and if necessary backs it up with a slug of regular resin. If a rebar is strong enough resin will develop a fairly constant resistance to pullout, per inch of resin, so providing a yielding system.

But then, if anchorage is good enough to develop the tensile strength of a bolt - then the bolt will be subject to shear failure. Note that a fully-grouted bolt can be sheared sooner than a bolt with no grout around it, so I would normally recommend only point-anchorage, not fully-grouted.

I was somewhat dismayed by the numerous suggestions that a bolt-investigation program be set up. It seems to me that the work has already been done, and that we could get the results from manufacturers (to be taken with the proverbial grain of halite) and from many operators in salt and similar evaporites.

4. OTHER FORMS OF ROOF SUPPORT. With rooms 33ft wide I would be concerned about the design of slings which depend on anchorage of inclined bolts above the pillars - because it is hard for them to provide much vertical support of dead loads. Most of those which I have seen ended up as hammocks loaded with broken rocks, sagging as a hammock would sag. In our case they would also have to yield instead of breaking ...

I have a question concerning the use of lacing, which might end up looking like broken rock in an onion bag. I'm wondering how we would monitor the

stability of the onion bag.

5. MODES OF FAILURE OF THE TEST ROOMS. There was, of course, much discussion of the probable modes of failure of the rooms, and suggestions for further instrumentation, analysis and modelling.

Again I was somewhat taken aback, since the real answers are readily available, in the mine. As discussed earlier, if new observation holes were fanned out around the rooms, and borescoped or cored by scratching, we would quickly define the failure pattern.

Suggestions were made to separate roof and floor movement by using more borehole extensometers - but those gadgets become extinct when the holes shift too much. In some other mines we have used a precise level and rod to measure elevations of reference points on roof and floor - many from one set-up - as a technique for measuring convergence and defining the amounts contributed by roof and floor. It works well. We could, if necessary, measure to 1/1000cm, but for us, of course, that would not be necessary.

Please thank Joe for sending the reports on hot-room instrumentation. Some day we'll have to talk about them, because I was particularly interested in the load-cell data - and I did not see anything like the assumed 2000psi hydrostatic stressfield. However, that does not affect any of the comments and recommendations made above.

I hope that you find these words practical and helpful.

Respectfully submitted,

Jack Parker

Jack Parker
JP/wp

REPORT SUBMITTED

BY

DR. T.W. THOMPSON

The summary report contains:

- An accurate record of the meetings of the Geotechnical Panel on Panel 1 Stability.

- A copy of the report provided to Westinghouse by this panel member.

- An accurate presentation of the consensus agreed to by the panel members at the meetings on the 23rd and 24th of April 1991.

Panel Member	5/22/91 Date

GEOTECHNICAL EXPERT PANEL
LIFE OF PANEL 1

Response to Statement

by

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Submitted to
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May 22, 1991

RESPONSE TO STATEMENT 1

An estimate can be established for the period of time that Panel 1, in particular Room 1, remains accessible on a daily basis beyond July 1991.

Observations:

1. Data are available on the stability of the four SPDV test rooms: Room 1 (SPDV 1) failed after nearly 8 years. Failure occurred on shear fractures angled upwards at about 20° from the rib, with the apex of the fall probably coinciding with the clay seam underlying Anhydrite B. Precursors of failure included acceleration of the vertical closure, first noted in May 1988 (just under 2 years prior to the fall), detection of fractures in the roof near the rib, and indications of separations in the roof. The other rooms are still standing, and, prior to the closure acceleration, vertical closure rates were quite similar to each other and tend to be slightly less than for SPDV 1. SPDV 2 appears to show acceleration of vertical closure (starting in late 1988), though this is not as pronounced as in SPDV 1. SPDV 4 shows fractures at the rib and evidence of lateral slip in roof boreholes. This room was bolted in the 1989/1990 time period. The current life of these rooms is as below.

Room	Life to Present	Comments
SPDV 1	7.9 Yrs	Roof Fall in 2/91
SPDV 2	8.1 Yrs	Possible closure acceleration starting in late 1988
SPDV 3	8.1 Yrs	
SPDV 4	8 Yrs	Rib Fractures and Roof Slip Observed: Bolted in 89/90

2. Data on the seven Panel 1 rooms show no evidence of immediate failures (accelerating closures) at this time (3 - 5 years after mining), though incipient rib fractures are seen. Panel 1 Room 1 shows incipient fracturing in the roof, evidenced as shears developing along the rib edge. The other rooms show similar fracture development, and in some

cases this appears more severe than in Room 1 (Rooms 6 and 7 for example). The vertical closure for all of the rooms is quite similar to that for SPDV 1 up to the present, with closure rates showing a general decrease to a fairly constant current value. Closure rates for Room 1 match those for SPDV 1, and are somewhat higher than for the other SPDV Rooms. In terms of the time since mining, Panel 1 Room 1 is now at the same point as SPDV 1 was immediately prior to the acceleration of closure. Panel 1 Room 1 was bolted in 1988, two years after mining. Some local slabbing of pillars is seen in the Panel 1 rooms. The current life of these rooms is as below.

Room	Life to Present
Room 1	4.9 Yrs
Room 2	4.3 Yrs
Room 3	4.2 Yrs
Room 4	3.3 Yrs
Room 5	3.3 Yrs
Room 6	3.3 Yrs
Room 7	3.2 Yrs

3. Floor heave has been a problem in all rooms (SPDV and Panel 1). Standard practice is to recut the rooms and to backfill loose with crushed salt. The floors are apparently stable at this time.

4. An additional fall has occurred in Room A2 of the SPDV. This room had a different geometry to those of the SPDV Test Rooms and Panel 1, was at a different horizon, and was heated during its life. Failure appears to have been by a similar mode, and a precursor in the form of accelerated roof deformations was seen about two years prior to the collapse.

5. Rib fractures are evident throughout the facility, including the entries (e.g. N1100). There is no evidence as to whether these are deep shears or surface spalls.

Comments:

1. The mechanism for failure is by low angle shears in the roof. This is caused by the lateral stress due to removal of support for the horizontal stresses and by lateral movement of the pillar material into the room. The Clay underlying Anhydrite B may contribute to the severity of the effects in the roof beam due to slip along this plane and isolation of the immediate roof.

2. The SPDV Rooms and Room 1 show general similarities in their geometry and geology, though their are some differences. Thus:

   - The room geometry for Room 1 is similar to the other Panel rooms and to the SPDV test rooms.
   - The geology in and around Panel 1 appears to be similar to that around the SPDV test rooms. In particular the clay/anhydrite above the rooms appears to be similar
   - Their are no apparent anomalies associated with any of the rooms.
   - The sequence of mining was a little different with SPDV 1 mined after SPDV 2 and 3, though by only about 1 month, while Panel 1 Room 1 was mined first in the panel.
   - Panel 1 Room 1 has been bolted

Available Information:

Available information includes the Rock Mechanics instrument data from the SPDV rooms and from Panel 1, field observations by the Westinghouse geotechnical staff and by the panel members. Of particular importance are the convergence data and inclinometer data. There are no roof extensometer installations in Panel 1. No data are available of modelling results of the stress and deformation fields in Panel 1 (or in SPDV test rooms).

Factors to be addressed:

1. The ability of the Panel to address Statement 1 based on the available information.

The WIPP facility is heavily instrumented and abundant data are available. Much of these data are useful in addressing the stability of the rooms. Lacking are a) roof extensometer data to give any information of separations in the roof of Panel 1 Room 1 (or elsewhere in the panel), b) inclinometer data on horizontal movements in Panel 1, c) good data on roof bolt performance (loads, pull out tests) and a thorough analysis of modes of failure, and d) model data to give information on the stress field development.

As noted above useful data are available on the stability of the four SPDV test rooms. Based on these data some estimate of life expectancy can be made. However this estimate will have a larger uncertainty than if more rooms were available for comparison with a greater life and additional data on roof bolt performance were available.

2. Best Estimate for life of Room 1.

3. Lower and Upper bounds estimates for the life of Room 1.

4. Levels of uncertainty associated with estimates

Estimates of the life of the room should be considered in terms of the increasing uncertainty in the estimate with time. The uncertainty of the life expectancy estimate is zero at this time, increases slowly over the next two to three years, then increases more rapidly.

Estimates of life are based on a) comparison with the behavior in the longer lived rooms and b) observation of current conditions (fracturing) in Room 1. From these sources the following observations can be made:

- Of four rooms longer lived than Panel 1 Room 1, one failed after just under 8 years. This room had indications of impending failure after 5 years: this is the current life of Panel 1 Room 1. Of the others, one (SPDV 2) may be showing incipient failure (accelerated closures), the others show shear fractures at the rib but no accelerations of closure.

- Panel 1 Room 1 shows incipient roof fracturing at the ribs.

- In the other failure (A2) closure showed acceleration about 2 years before failure.

Based on these observations the lower limit of life for the room <u>in the absence of bolts</u> may be estimated as two to three years from now (seven to eight years total) with high confidence. This estimate is based on the comparison of the closure curves and the age of Room 1 and SPDV 1. A lower limit of about 10 years total life can be estimated with lower confidence based on the current life of the other SPDV rooms and an assumed two year closure precursor. The upper limit is impossible to estimate with high confidence on the basis of local data from the WIPP facility. Observations in other mines with similar conditions suggest that a life of greater than 10 years is not unreasonable to expect, but that an unmaintained life of as much as 15 to 20 years is unlikely.

The effect of the bolts on the life is unknown quantitatively. It is likely that the bolts will not delay failure of the roof, but may be able to support it: a further discussion is given in Statement 2. It should be noted here that the life of the room can be extended if careful roof monitoring is combined with an adequate support system, and if provision for maintenance of that system is provided. Failure of the roof on shear fractures can probably not be prevented, however suspension of the failed slab can be achieved.

Whether the maintenance involved in upgrading roof support during operation will require movement of the bins depends upon the final support system and the final design of the bins and associated equipment. This is an operational question and cannot be addressed further here.

5. Reasons for the levels of uncertainty

The levels of uncertainty associated with any estimate of the life of the room are the same as those inherent in any underground mine in evaporites. They arise from the natural complexity and variability of geologic materials, the additional complexity of the highly strain rate and pressure dependant properties of evaporites, and our imperfect understanding of these mechanisms, or of the detailed effect of local discontinuities (such as the overlying clay). Standard mining practice in these materials (as in many others) is to use the mine itself as a test bed. Initial mine designs are based on experience elsewhere in similar materials, but during its life the mine design is constantly tailored to local conditions. In the WIPP facility we have only eight years of experience in four rooms: this is an insufficient data base for projecting too far into the future.

An additional uncertainty comes from the lack of hard data on the efficiency of roof bolts in the current application. In most other mining applications in these materials bolts are used for local roof spalling control rather than for the suspension of large slabs. We have little site specific information on how the bolts will work, and on their life expectancy under large lateral movements.

6. Additional information needed to improve estimates.

Certain additional information would help to refine the estimates, and to reduce the uncertainties. Key data include:

a) Rock bolt failure information. A more thorough study of the current efficiency of the rock bolts, and of potential failure mechanisms (shear, anchor pull out etc) would help considerably in assessing their contribution to stability.

b) information on progress of fractures in Room 1. Data on the current state of any fracturing in the roof of Room 1 would assist us in determining where on the failure curve this room is. Data could include radar/EM surveys and exploratory boreholes. Additional data from roof extensometers and inclinometers, and microseismic activity would help in monitoring conditions.

c) modelling studies of unbolted and bolted stability would assist in estimating the progress of failure conditions.

7. Potential pillar (side wall) spalling

Pillar spalling is common in deep evaporate mines, and is seen in Panel 1. This has no impact on overall stability, but could produce operational problems in rooms used for bin tests. Provision should be made to protect the equipment from localized slabs spalling from the pillars, as well as to give access for cleanup.

RESPONSE TO STATEMENT 2

The rockbolt system as currently configured, is sufficiently effective to ensure that the test program in Panel 1, in particular Room 1, can be completed.

Without rather drastic remedial measures such as slotting, the use of sacrificial drifts, or inducing slab collapse, the "failure" of the roof on low angle shears can probably not be prevented. As noted in the remarks on Statement 1, this failure is likely to occur within the anticipated life of the bin experiments. However it is also likely that the life of the room can be extended by the use of a suitable support system to suspend the failed slab.

Comments:

1. Shear failure of the roof will occur in a similar fashion to SPDV 1 because of the lateral squeeze developed by of the high horizontal stresses and the lateral movement of salt due to the compression of the pillars.

2. This shear failure will lead to a slab separation, this slab having similar geometry to the wedge failure in SPDV 1 and A2. Current bolts will not stop the development of this shear failure, and in all probability a result of the shears will be failure of the bolts due to shear, as seen in other mines.

3. After development of the shear separations the arched roof above the slab will be stable for a reasonable period of time (several years). In developing the shear failure the material is breaking to a more stable configuration.

4. The failed slab can be suspended from the overlying salt beam, or by some other support system. If rock bolts are used they can be designed to support the required weight. Continuous monitoring of roof movements and bolt integrity (i.e. bolt loads,

deformations and anchor movement, condition) will be needed to assess the efficiency of the support system. Provisions for rebolting should exist to maintain the support system. Local protection for delicate systems may also be needed.

Factors to be Addressed

1. **The affect that the changes associated with the test program have on support requirements for Room 1, Panel 1.**

Changes in the test program include the need to extend the life of Room 1 from approximately five years (for a five year test program starting in 1986/1987) to 14 years (through July 2000). We already have evidence of the ability of the rooms to stand for at least 5 years (the current life of Room 1) and have no evidence of failure before nearly eight years (the life of SPDV 1). Several rooms are still stable after eight years. On the other hand based on current knowledge a life of 14 years without supplementary supporting systems is very unlikely. The changed test program and life requirements have clearly added the need for support, and put quite stringent requirements on that system.

2. **The rock load to be supported is approximately the full weight of the roof beam up to the anhydrite "b" layer in the middle third of the span, and half this weight over the outer two thirds.**

Based on the evidence from SPDV 1 and A2 this assumption is reasonable.

3. The adequacy of the factor of safety of the bolting system used in Room 1, Panel 1 to support the design rock load.

A factor of safety of 1.7 for suspension of the roof is adequate provided that:

- The mechanism for bolt failure is better understood
- The roof and bolts are monitored for excessive movement and failure of the bolts/anchors.
- Provisions are made for maintenance of the bolting system during the tests.

Without these items (especially b and c) the safety factor is not adequate: indeed without these no safety factor may be adequate.

4. The salt above the anhydrite "b" will remain competent.

There is no reason to believe that this salt will not remain competent for a reasonable period under the current conditions. Allowing the failure of the lower unit will aid in maintaining stability since it will force the room to a more stable configuration. Care should be taken if one of the more drastic remedial actions is taken (e.g slotting) to ensure that failure due to lateral squeeze is not transmitted to this higher horizon.

5. Slippage of anchors provides an acceptable approach to supporting the rock load while accommodating roof closure, with daily access to the room.

6. The mechanism by which the bolt anchors will accommodate the movement of the salt while supporting the immediate roof beam.

It is extremely doubtful that anchor slippage will occur after the bolts have been set for a long time period. The anchors are set by applying a torque which expands the anchor shell: this leads to a lateral stress which, given the creep properties of the salt, will tend to embed the anchors. It is likely that the current bolts are stretching to accommodate creep rather than the anchors slipping. The estimated vertical roof movement of 3" - 4" since bolt emplacement will have given about 3% strain. If tensile failure occurs at 10% strain this would occur in about 1993 at current closure rates. Further information, including bolt loads and strains) are needed to evaluate this.

Bolt failure is more likely to happen due to:

- Shear of the bolts due to differential lateral movements.
- Stripping of anchor threads
- Wedge pull-out due to excessive creep expansion of the shells.

These potential failures should be analyzed by calculation, field proving of bolts and, possibly, laboratory studies.

RESPONSE TO STATEMENT 3

The level of confidence that can be placed in the estimate of the life for Panel 1 provided in the response to Statement 1 is in the accordance with accepted mining practices.

The levels of uncertainty associated with any estimate of the life of the Panel are the same as those inherent in any underground mine in evaporites. They arise from the natural complexity and variability of geologic materials, the additional complexity of the highly strain rate and pressure dependant properties of evaporites, and our imperfect understanding of these mechanisms, or of the detailed effect of local discontinuities (such as the overlying clay). Standard mining practice in these materials (as in many others) is to use the mine itself as a test bed. Initial mine designs are based on experience elsewhere in similar materials, but during its life the mine design is constantly tailored to local conditions. In the WIPP facility we have only eight years of experience in four rooms: this is an insufficient data base for projecting too far into the future.

Factors to be Addressed

1. **The extent to which a probabilistic basis for determining risk assessment is presently applied in mining.**

Formal probabilistic risk assessment analyses are not typically used in the operational side of mining, although they do have application in the marketing and strategic planning aspects of the industry. The only cases of which I am aware of the application of these techniques was in the development of coal mine pillar design formulae in South Africa in the 1960's (Salamon, personal communication) where a large data base on failed pillars was available and in the design of open pit slopes (Ross-Brown, personal communication).

Informal risk assessment is the basis for mine development, that is an understanding of "what works" in a particular mine is used in further developments, together with a basic understanding of the inherent uncertainties. This is coupled with a constant monitoring and inspection program. Reasons for not applying PRA in a formal sense are the inherent complexity and variability of geologic conditions an inadequate date base and our poor understanding of how to quantify the behavior of these materials.

2. The qualitative nature of geologic information

Geologic information, as currently available and used, is basically qualitative in nature, although attempts are made to quantify these data (by, for example, rock mechanics). The overriding reason for this is the inherent complexity and variability of the materials. In the current case of WIPP which is developed in a fairly uniform geologic environment this complexity still tends to overwhelm attempts to quantify behavior. Data taken in one room, or one location in one room, for example, can vary in another room or location due to subtle differences in geology, nearby mining or geometry. Moreover we have only an imperfect understanding of how to quantify mechanisms for such apparently simple phenomena as creep closure and shear failure.

3. The extent to which a database or experience is available in the mining industry from an operations point of view to provide meaningful judgments at the probability levels used in the nuclear industry(i.e. probabilities of less than 1 in 10^6).

A wealth of data exists from other mines which can be applied to the WIPP facility. However much of this data is qualitative (see #2 above), and differences in its application can occur because of site specific conditions. It is totally unreasonable, and well outside of normal practice, to provide probability levels used in the nuclear industry in this situation.

4. The adequacy of the geomechanical database developed at the WIPP and the methods currently in place to evaluate the performance of openings.

In general the geomechanical database at WIPP is excellent - it is certainly much better developed than at almost any other underground facility, and is far and away better than available in the typical mining environment.

With a few exceptions the current monitoring is adequate. The exceptions are :

- Vertical extensometers and inclinometers in the roof of Panel 1 are needed to assess/monitor roof movement and separations.
- Pressure cells in and around the rooms would help to monitor stress fields.
- Rock bolt load cells, and methods to assess rock bolt strains, are needed to evaluate performance of the support system.
- The addition of microseismic monitoring of the roof in Panel 1 would assist in monitoring impending fracturing and failure.
- Additional roof integrity investigations (radar, EM or borehole) would also help to monitor roof stability.

RESPONSE TO STATEMENT 4

Modifications to the support system in Panel 1 can be implemented to ensure that access is maintained to the rooms on a daily basis until the tests are completed.

Without rather drastic remedial measures such as slotting, the use of sacrificial drifts, or inducing slab collapse, the "failure" of the roof on low angle shears can probably not be prevented. As noted in the remarks on Statement 1, this failure is likely to occur within the anticipated life of the bin experiments. However it is likely that the life of the room can be extended by the use of a suitable support system to suspend the failed slab.

Shear failure of the roof can only be prevented by the use of some method to relieve the lateral squeeze. This relief can be achieved by a) slotting of the roof, or b) the use of sacrificial drifts either in the large pillars or above the pillars. These methods are normal in other deep evaporite mines. These are not discussed in further detail here since they are probably unacceptable in the current facility at this time. However they may require consideration for future developments.

If shear failure is allowed to develop this will lead to a slab separation, this slab having similar geometry to the wedge failure in SPDV 1 and A2. Maintaining access then depends upon supporting the failed roof by bolts, rope cradles or massive steel sets and/or timber. Any of these systems could be designed to provide the required support, but all will require the ability to monitor and maintain, which will require access to the roof.

Factors to be Addressed

1. The modifications and additions to the support system needed to ensure the completion of the tests.

As noted above several additional support systems could be used to maintain access. These are briefly summarized below:

- Bolts. As discussed in the response to Statement 2, bolts could be used to suspend the roof provided that they are continually monitored and provision exists to maintain the system by rebolting as required.

- Cradles. The use of a wire rope cradle keyed into the overlying salt beam has been suggested by Dr. Miller. This system relies on supporting the broken roof on a laced rope and mesh support. This should be successful provided that the roof breaks satisfactorily, or that the system is engineered to support the unbroken slab. Keying the ropes into the overlying salt relies on adequate adhesion to this member: keying into the areas over the pillars (on 45° angles) might be considered.

- Cribbing. The use of cribs along the room length (centerline) with local side support by bolts would support the wedge failure, but would complicate access. Nevertheless this is probably the most positive and easily maintained system. Steel sets could be used to the same end, but with similar access problems.

2. The maintenance activities that will be needed in the room.

Whatever support method is used monitoring of roof and support behavior and the ability to maintain the system are mandatory. The details will vary with the system:

- o Bolts. Bolt load and strain must be monitored. Further investigations of failure modes, including field pull-tests are needed to properly design the system. Maintenance activities will include rebolting as needed and possible local scaling.

- o Cradles. The performance must be monitored by regular inspection, monitoring of roof movement before and after failure. Pre testing to ensure the adequacy of keying of the support ropes should be conducted. Maintenance will be minimal. In the event of loss of support due to rope pull out or failure a secondary system (such as cribbing) may be needed.

- o Cribbing. Crib monitoring would include the use of pressure cells to monitor loads on the cribs, and convergence meters and extensometers to monitor roof movement. Visual inspection of cribs and for local slabs will be required. Access will be needed to inspect the cribs and roof, and for bolting of local slabs.

3. The need to remove the cables for the bin scale tests in order to install additional support.

Given the likelihood of roof failure with any support system, and the need for access to bolt/scale any local spalls, removal of the cables from the roof is needed. Cables should be slung in trays supported by long bolts into the pillars.

RESPONSE TO STATEMENT 5

The geomechanical monitoring program and the routine observations in Panel 1 can provide sufficient warning to allow the timely retrieval of the waste from the Panel.

Based on the evidence from SPDV 1 and A2 acceleration of the convergence data gives about 2 years of warning of impending failure. In practice this will probably be closer to 18 months due to the criticality of conditions immediately prior to failure. Given the assumption of 6 months to remove the waste this should be adequate warning. Note that on the one hand this time does not account for the delays possible due to the current bolting, or the use of additional remedial support. On the other hand the two years is based on only two data points and could be shorter in other cases. Continuous monitoring after a critical acceleration is recognized, and the ability to use short term remedial support are necessary.

Factors to be Considered

1. **The adequacy of the geomechanical data base developed at the WIPP provides an adequate basis to predict and provide early warning of deteriorating conditions in Room 1.**

   As noted above the current data base is adequate to give the necessary early warming.

2. **The adequacy of the present geomechanical instrumentation installed in Room 1 to provide early warning of deteriorating conditions.**

   The present instrumentation is adequate, but minimal, for early warning.

3. **The adequacy of the proposed additional geomechanical instrumentation to be installed in Room 1 to provide early warning of deteriorating conditions.**

The proposed new instruments will greatly enhance the early warning capability. Key here are the additional convergence stations (which cover a larger roof area) and the roof extensometers (which should extend well into the roof: i.e. well beyond anhydrite b).

Further instrumentation which should be added include:

a) Roof inclinometer holes to detect lateral movements

b) Rock Bolt load cells, and strain gaged rock bolts, to monitor bolt load and deformation.

c) Microseismic monitors to monitor rock noise.

4. The criteria to determine when removal of waste becomes necessary.

Based on previous experience impending failure is signalled by accelerating closure. This will continue to be the best pre-cursor if additional support is not planned. In these conditions it is likely that acceleration of closure will occur about two years prior to failure, while six months are required to remove the waste. On this basis the following criteria are proposed:

a) Acceleration of closure and/or accelerated separation from convergence data and MPBX results. Given the natural variation observed due to thermal and other sources these accelerations should be continuous for a period of six months. This time lag will allow confirmation of the trend as well as a period to attempt remedial measures.

b) If the acceleration does proceed for six months, and if remedial actions do not stabilize the roof, then waste removal should be started. This would be complete

one year after first detection of the accelerating trend. This time frame completes removal one year before projected failure, or six months before critical roof conditions are developed, giving a six month margin of error for earlier failure development or for delays in the removal of waste.

REPORT SUBMITTED

BY

DR. J.R. TILLERSON

The summary report contains:

- An accurate record of the meetings of the Geotechnical Panel on Panel 1 Stability.

- A copy of the report provided to Westinghouse by this panel member.

- An accurate presentation of the consensus agreed to by the panel members at the meetings on the 23rd and 24th of April 1991.

Joel Tillerson
Panel Member

May 28, 1991
Date

Rep. Corp.

Sandia National Laboratories

Albuquerque, New Mexico 87185

date: May 28, 1991

to: Tod Burrington, Westinghouse

from: J. R. Tillerson, 6346

subject: Transmittal of Documents

Attached are my final responses to the statements provided to the Expert Panel. These are provided as input to the final report.

Enclosures: As stated

Copy to:

6340	W. D. Weart
6340A	A. R. Lappin
6346	File 294
6346	Day file
6346	J. R. Tillerson

RESPONSE TO STATEMENT 1 Panel Member: Joe R. Tillerson

Very limited data exist for judging the longevity of even unbolted rooms at the WIPP. The data that do exist have significant scatter related to the 4 most direct areas of comparison (ie the SPDV rooms). As regards the performance of bolted rooms at the WIPP site, almost no data exist at this time on the effectiveness of the bolting system. This lack of data makes it very difficult to predict longevity with any degree of certainty. However, it is my opinion that none of the three cases considered as part of this statement will provide sufficient longevity at a high level of confidence to assure satisfactory completion of the testing program. Detailed estimates of the longevity are therefore of little value for the current support system and are not included in my response except to say that high confidence cannot be achieved for the desired 14 year lifetime needed (5 years old now plus up to 9 years possible for the experiments).

STATEMENT 2 REVISION 1

The rockbolt system as currently configured, is sufficiently effective to ensure that the test program in Panel 1, in particular Room 1 can be completed.

Assumptions

1. The test program will start in July 1991.

~~2. The test program will be completed in July 1996.~~

~~3. Retrieval from Room 1 can be accomplished between July 1996 and July 1997.~~

4. The bins CANNOT be disconnected and moved to facilitate maintenance of the rooms.

Revised Assumption

(replacing Assumptions 2 & 3)

The test program including retrieval will be completed by July 2000.

Factors to be Addressed

1. The affect that the changes associated with the test program have on support requirements for Room 1, Panel 1.

2. The rock load to be supported is approximately the full weight of the roof beam upto the anhydrite "b" layer in the middle third of the span, and half this weight over the outer two thirds.

3. The adequacy of the factor of safety of the bolting system used in Room 1, Panel 1 to support the design rock load.

4. The salt above the anhydrite "b" will remain competent.

5. Slippage of anchors provides an acceptable approach to supporting the rock load while accommodating roof closure, with daily access to the room.

6. The mechanism by which the bolt anchors will accommodate the movement of the salt while supporting the immediate roof beam.

RESPONSE TO STATEMENT 2, REV. 1 Panel Member: Joe R. Tillerson

Three cases were identified in Statement 1: I will answer statement 2 by considering each of those three cases. The responses to each individual case are given below:

1. No maintenance in terms of scaling of roof, milling of floor or installation of additional support.

Without maintenance, the data from the unbolted rooms, the age of the rooms in Panel 1, and the questions related to the potential for shearing of the existing bolts clearly indicates it is doubtful that high confidence can be achieved in the performance of the current support system for the entire duration of the experiments. However, the same data indicate there would be sufficient warning of impending large roof falls to allow starting experiments in such rooms provided bins could be moved, if necessary, during testing to a more suitable area.

2. Limited maintenance without moving bins.

While "limited maintenance" would certainly require further definition, it is doubtful in my opinion that high confidence in the performance of the support system could be achieved for the entire duration of the tests. This is based on the fact that, with only limited maintenance, this option does not relieve the concerns related to bolt shearing effects and would not allow replacement of bolts that have become ineffective.

3. Extensive maintenance on an as required basis, with bins removed from room, if necessary during maintenance activities.

This option would allow bolt replacement and even installation of additional bolts, possibly longer, stronger ones, between the currently installed bolts. I cannot recommend this approach for Room 1 because of the large amount of interference that would exist with the instrumentation and "plumbing" already installed within the room.

Factors considered in the above response:

Some of the factors considered in the response given above are the age of the current openings (about 5 years for room 1), the behavior of the unbolted (or minimally bolted) SPDV rooms, the lack of data at the WIPP site on the multi-year performance of bolts, WIPP fracture data that clearly indicate significant rates of lateral deformation, the lack of ability of the bolts to retard motion (hence fracturing) within the roof, the potential for the bolts to shear as a result of the lateral deformation of the roof, and the promises made related to assuring retrieval of the bins after the completion of the experimental program. These items lead me to believe that the bolts will certainly extend the useful life of the rooms in panel 1. However, none of the approaches listed above leave me with high confidence that the rooms can be used for the duration of the testing without significant modification or enhancement of the support systems so I will not attempt to give a useful life for these rooms without modification.

The level of confidence that can be placed in the estimate of the life for Panel 1 provided in the response to Statement 1 is in accordance with accepted mining practises.

Factors to be Addressed

1. The extent to which a probabilistic basis for determining risk assesessment is presently applied in mining.

2. The qualitative nature of geologic information.

3. The extent to which a database or experience is available in the mining industry from an operations point of view to provide meaningful judgements at the probability levels used in the nuclear industry (i.e. probabilities of less than 1 in 10^6). This is not to be applied to an assessment of the longterm (10,000 year) performance of a repository.

4. The adequacy of the geomechanical database developed at the WIPP and the methods currently in place to evaluate the performance of openings.

STATEMENT 3, REV. 0 Panel member: Joe R. Tillerson

Probablistic approaches to judging the lifetime for usable access to openings in underground operations are, at best, in their infancy and, hence, are not likely to provide significant credibility if applied to the current questions surrounding the stability of rooms in Panel 1. Underground safety for facilities that require a significant lifetime is generally approached with conservative, but reasonable designs for support systems and a very strong and unwavering commitment to monitoring and prompt maintenance. The data gathering activities at the WIPP site have provided much valuable information for use in making decisions related to underground operations but do not provide, as yet, sufficient basis for the extensive application of probabilistic methods for failure predictions. Some applications of probabilistic methods are probably appropriate for evaluating some concerns that arise in evaluating the current data; one example of this would be probabilistic-based evaluations of how long it would take to determine if the rate of room closure were accelerating if the uncertainties in individual measurements is considered. The current geotechnical database provides some very good information related to the performance of openings but, in my opinion, should be expanded in the rooms in which the bin tests will be conducted. The current measurements rely very heavily on closure information; difficulties in determining whether the predominant motion is occuring in the floor or the back could be overcome by the addition of a few multipoint extensometers, predominately in the back, in each room and in the accessways. The extensometers would provide excellent indications of the extent and principal location of roof motion. Some extensometers placed in the floor could also provide excellent insights into the extent and timing of the behavior of the floor. In addition, observation boreholes should be added to the rooms and accessways in panel 1 to assess potential shearing motion as fractures form in the roof.

STATEMENT 4

Modifications to the support system in Panel 1 can be implemented to ensure that access is maintained to the rooms on a daily basis until the tests are completed.

Factors to be Addressed

1. The modifications and additions to the support system needed to ensure the completion of the tests.

2. The maintenance activities that will be needed in the room.

3. The need to remove the cables for the bin scale tests in order to install additional support.

RESPONSE TO STATEMENT 4 Panel member: Joe R. Tillerson

It is certainly conceivable that the current support system with timely maintenance could allow the rooms to be usable for the entire duration of the bin experiments. However, without additional enhancement of the support systems in Panel 1, it is my belief that we cannot have high confidence in the usability of the current rooms as is for the intended duration of the experiments.

Numerous options exist that have been effectively used in other underground applications and could be used to further enhance the usable lifetime of the rooms in which the experiments will be conducted. These enhancements could provide the required high confidence level. This is especially true since the data from the SPDV rooms and other underground areas have established the expected displacement patterns and failure mode of the rooms.

For the behavior observed in the WIPP, proposed enhancements of the support systems generally fall into two categories:

1. Enhancements that relieve the stresses on the roof beam that could fail (eg. slot cutting in the roof or mining of adjacent openings) and

2. Enhancements that prevent large blocks of the roof from falling on the bins (eg. installation of longer, stronger bolts between the current bolts, cribbing, cable systems that are combined with wire mesh, yielding trusses)

Since my experience and expertise lie more in the modeling of the behavior of the salt and the support systems, please rely on other panel members with support system design experience for detailed definition of the enhancements. My principal comment is that the mechanics of the proposed enhancements are sound and with proper installation should be capable of being implemented effectively to assure with high confidence the stability of the openings for the duration of the experiments. As regards the mechanics of the potential behavior of concern, the following items are noted:

1. Lateral movement of the salt in the "roof beam" is the predominant mechanism of concern.

2. Sliding occurs along the clay seams since shear stresses are not effectively transfered from one side of a seam to the other.

3. Fracturing occurs progressively with time in the roof area as a result of the strains that build up with time in the salt. The degree of fracturing is a function principally of the size of the opening, age of the opening, distance to interbeds, and specific location in the opening.

RESPONSE TO STATEMENT 4, CONT. Panel Member: Joe R. Tillerson

4. Bolts are unlikely to affect the rate of deformation occuring in the rooms prior to the point at which the separation in the roof begins to accelerate. This has been shown in numerous published analyses completed in the last 15 years.

4. Slippage of the anchors is not the likely mechanism for long-term degradation of the bolt performance.

5. Little measured data are available on the mechanics of the performance of support systems in evaporite deposits. Observational data are often available that clearly confirm the acceptable performance of such systems or the need for modifications.

Engineering and associated implementation of proposed enhancements should be able to be completed in most rooms in Panel 1 within 6-9 months. If Room 1 were substantially modified, it would probably take longer since extensive bin-related cables are already installed.

RECOMMENDATIONS:

Assure that contingency planning and procedure development is complete related to such activities as where, when and how bins can be moved, geotechnical conditions under which bin removal would be initiated, and support system maintenance. Such planning must also establish how the experiments will be terminated (eg.cable or hardware removal requirements and should allow sufficient time for backfilling the rooms prior to conditions becoming unsafe.

Initiate bin testing in Room 1 after only limited enhancements are added if the current schedule is maintained.

As soon as practical after recommendations are received from the expert panel, initiate the engineering and implementation of both categories of support system enhancements in other rooms in Panel 1 or, if preferable, in other freshly-mined rooms. Support system enhanvements should be evaluated in both design studies and in detailed numerical modeling. Also, site-specific data on the performance of support system enhancements should be obtained. Strong consideration should be given to installing the enhancements in the most recently constructed rooms in the panel and in the 33′ wide portions of the accessways. This would provide in a timely manner the needed space for safely conducting all the bin experiments for the entire potential duration. If necessary, bins initially emplaced in Room 1 could be moved to this area. Enhancements and associated data monitoring may also be desired in areas outside of Panel 1 to assure timely availability of data on the support system performance.

RESPONSE TO STATEMENT 4, CONT. Panel Member: Joe R. Tillerson

Initiate contingency planning related to conducting the alcove experiments. This planning should consider advantages and disadvantages of conducting the experiments in an alternate location outside of Panel 1. This planning is needed since those experiments are likely to be delayed for several years and since those experiments would require use of the 33′ wide accessways around the Panel 1 rooms for a significant period of time beyond that currently being considered by the expert panel. The planning should also define the maintenance required to keep the wide accessways open if the alcove experiments are to be conducted in Panel 1.

STATEMENT 5

The geomechanical monitoring program and the routine observations in Panel 1, can provide sufficient warning to allow the timely retrieval of the waste from the Panel.

Assumptions

1. In an emergency, all waste can be removed from the room within a 6 month period.

Factors to be Addressed

1. The adequacy of the geomechanical database developed at the WIPP provides an adequate basis to predict and provide early warning of deteriorating conditions in Room 1.

2. The adequacy of the present geomechanical instrumentation, installed in Room 1 is adequate to provide early warning of deteriorating conditions.

3. The adequacy of the proposed additional geomechanical instrumentation to be installed in Room 1 to provide early warning of deteriorating conditions.

4. The criteria to determine when removal of waste becomes necessary.

RESPONSE TO STATEMENT 5 Panel Member: Joe R. Tillerson

Excellent data exist that document the behavior of the unbolted SPDV rooms that are the same size and spacing as those in Panel 1. A portion of the roof in one of the SPDV rooms has failed about 8 years after construction. Data obtained in this room provided advanced warning of the roof stability concerns and clearly indicate that a "beam" of material failed in the roof after substantial vertical and lateral movement. This advanced warning of impending failure of a slab of rock had also been monitored in other underground measurements made in a heater experiment at WIPP. Because of the many similarities in size and spacing, the data from the SPDV rooms are the best source of information available upon which to estimate performance of the rooms in Panel 1.

There is also little doubt that substantial advanced warning of impending roof stability concerns can be provided by an effective monitoring program. This warning should be sufficient to allow safe removal of bins from Room 1, if necessary. Some expansion of the current measurement program is necessary to assure confidence in the monitoring program.

Additional regions of separation and fracturing could be anticipated to occur in "beams" above the one seen in the SPDV rooms. Data from the rock monitoring activities indicate that such fracturing would likely occur much later and slower than that observed in the immediate vicinity of the roof. Continued monitoring of the SPDV rooms, rooms in Panel 1, and other areas of the WIPP should determine the extent and rate of such phenomena.

RECOMMENDATIONS:

1. Add additional instrumentation and observations to the current monitoring program for Panel 1. This would include multipoint extensometers in the roof, rock bolt load evaluations made periodically in the panel, observation holes in the roof to evaluation the potential amount of lateral movement, and monitoring of the wall areas to determine the maintenance necessary for the hardware bolted to the ribs.

2. Commit to long-term monitoring of the behavior of the SPDV rooms, particularly Room 4 that was bolted.

3. In addition to expanded geotechnical evaluations made by site personnel, consortium usage should be considered relative to a program that seeks to understand and improve how support systems behave in evaporites. Potential areas of university contribution relate to statistical evaluations of existing data to assess confidence levels and accuracies implied for individual readings, assessments of how quickly accelerating behavior can be developed, data on various bolt and anchor system performance, and evaluations of load monitoring systems.

RESPONSE TO STATEMENT 5, CONT. Panel Member: Joe R. Tillerson

4. Review the design of the cable systems and "hardware" attached to the ribs and wall to determine if significant changes are needed to facilitate access for support system maintenance in rooms where such hardware have not yet been installed.

5. As previously mentioned, complete contingency planning and procedure development related to bin movement and support system maintenance.

REPORT SUBMITTED

BY

DR. R.F. COOK

The summary report contains:

- An accurate record of the meetings of the Geotechnical Panel on Panel 1 Stability.

- A copy of the report provided to Westinghouse by this panel member.

- An accurate presentation of the consensus agreed to by the panel members at the meetings on the 23rd and 24th of April 1991.

Panel Member

5/31/91
Date

Statement 1

An estimate can be established for the period of time that Panel 1, in particular Room 1 remains accessible on a daily basis beyond July 1991. (Revision 1)

The following cases should be considered:

- Limited maintenance without moving bins.
- Extensive maintenance on an as required basis, with bins removed from room, if necessary during maintenance activities.

RESPONSE

Factor 1.

The geomechanical database for the WIPP underground is extensive. It includes 8 years of instrumentation and observation data from the Site and Preliminary Design Validation Test Rooms that is directly relevant for establishing the performance of Panel 1.

Factor 2.

The data indicates that the life of a room in a panel depends on its position within the panel. In both the SPDV Test Room Panel and Panel 1, the rooms closest to the pillar protecting the access roadways have undergone the greatest deformation. In the SPDV Panel, this is Test Room 1 and in Panel 1, it is Room 1. A rock fall occurred in SPDV Test Room 1 after 8 years but the other SPDV Test Rooms are still standing. It is anticipated that a range of performance can also be expected from the rooms in Panel 1.

Since location within the panel is an important determining factor for room stability, it should be taken into account in deciding the best location of the bin scale tests. The following range of conditions are estimated for the panel:

ESTIMATE OF ROOM LIFE (YEARS)

	Room 1	Room 4
No maintenance	8	9
Limited maintenance	9	11
Extensive maintenance	10	12

Factor 3.

Lower Bound	7	8
Upper Bound	11	indefinite

NOTE: Extensive remedial actions will be needed to ensure an indefinite life. These may include a combination of repeated bolting of the roof, removal of the rock in the roof, or the installation of a support system within the room in the form of steel sets or timber cribs.

Factor 4.

Level of uncertainty associated with estimates cannot be determined quantitatively. However, the uniformity of the geologic conditions across the site, and the similarities in the geomechanical properties, give a high level of confidence that the SPDV Test Rooms do reflect the behavior that can be expected in the panel.

Factor 5.

No response provided for this factor.

Factor 6.

Other geotechnical data is needed to understand more fully how the fractures behave and how the bolts are working. Improvement to our estimates of room life will come as more data on actual performance becomes available.

STATEMENT 2

The rockbolt system as currently configured, is sufficiently effective to ensure that the test program in Panel 1, in particular Room 1 can be completed. (Revision 1)

RESPONSE

Factor 1.

The requirement for daily access into the rooms in Panel 1 ensures that the support system must be fully effective at all times. Since remedial measures inside the room probably should be minimized during the bin scale tests, it is suggested that the support requirements in the rooms be re-examined prior to the start of tests.

Factor 2.

The assumption for the rock load appears reasonable. However, since questions have been raised regarding the thickened of the rock fall in SPDV Test Room 1, accurate dimensions of the roof fall should be obtained and used as the basis for the design rock load.

Factor 3.

The factor of safety for the bolting based on a triangular rock wedge with a maximum height of 7.5 feet is about 1.7. The unknowns with respect to the mechanism of support provided by the anchorage (fixed or slipping), the dimensions of the rock wedge to be supported, and the possible effects of lateral rock shifts on the bolts indicate that a conservative approach to design should be adopted.

Factor 4.

The salt above Anhydrite "b" will remain competent. The geomechanical data, particularly the inclinometer and extensometer data indicate that the large movements are primarily taking place within the immediate roof beam up to the Anhydrite "b" layer.

Factor 5.

If anchor slippage is to be used as a design approach, then more technical data is needed to evaluate this performance. Discussions with Dr. J. Scott indicate that the other rockbolt anchorage systems may provide more controlled anchorage slip.

Factor 6.

The bolts will support the roof by suspension. Bolts will be subject to anchor slip, and bolt elongation. Mr. J. Parker has suggested that bolt shear should be considered.

STATEMENT 3

The level of confidence that can be placed in the estimate of the life for Panel 1 provided in the response to Statement 1 is in accordance with accepted mining practises. (Revision 0)

RESPONSE

Factor 1.

A probabilistic basis for determining risk assessment is not routinely applied to underground mining due to the lack of an appropriate database. Information is often confidential to the mining companies and not readily shared, and in addition, geologic information is not always readily quantified.

Factor 2.

Geologic information is often of a qualitative nature and not readily quantified.

Factor 3.

The database for establishing a probabilistic approach to mine design is not available.

Factor 4.

No response provided for this factor.

STATEMENT 4

Modifications to the support system in Panel 1 can be implemented to ensure that access is maintained to the rooms on a daily basis until the tests are completed. (Revision 0)

RESPONSE

Factor 1.

Tests may be started in Room 1, Panel 1; however, modifications to the support system or the room will be required in order to obtain a further 5 years of life. The room is currently five years old, and its position within the panel as well as its age indicate that it will be the first to show deterioration. The modifications for extending its life include:

a. slotting
b. cable lacing
c. rebolting within 2 years with the provision to carry out additional bolting, if necessary.

If Rooms 4 and 5 of Panel 1 are used for tests, less extensive modifications to the support system or the rooms may be required. These rooms are now only 3 years old. The data from SPDV Test Panel indicates that they have a life span of at least 5 years from March 1991, but without extensive remedial activities with routine maintenance.

In addition, other measures may be appropriate including:

a. a redundant support system in the room (roof trusses, cribs, yielding steel support, additional roof bolting)
b. relief of the lateral stresses that are causing the fracture development.

Factor 2.

Maintenance activities will be required in the rooms in which the bin scale tests are carried out. Access to scale the roof and install additional bolts will be needed as a minimum.

Factor 3.

If Rooms 4 and 5 are used for the bin scale tests, additional support can be installed before the instrumentation cables are attached to the roof.

STATEMENT 5

The geomechanical monitoring program and the routine observations in Panel 1, can provide sufficient warning to allow the timely retrieval of the waste from the Panel. (Revision 0)

RESPONSE

Factor 1.

The geomechanical database at the WIPP has proven to be effective. It provided early detection of deteriorating conditions in the SPDV Test Panel. This deterioration was first reported in May 1988 and the roof fall did not occur in the room until February 1991.

Factor 2.

The geomechanical instrumentation presently installed in the rooms of Panel 1 would provide early warning of deteriorating conditions. However, a more comprehensive instrumentation should be implemented to ensure that no conditions are overlooked.

Factor 3.

The proposed geomechanical instrumentation for the rooms in which the bin scale tests will be carried out is shown in Figure 1.

Factor 4.

Criteria are currently in place to evaluate routinely (i.e. every 2 months) the performance of the drifts in the underground. The criteria used to assess when additional surveillance becomes necessary are as follows:

- Measured convergence rates that exceed predicted rates. The predictions are based on an equation that is derived from a nonlinear regression analysis of selected convergence data from the underground. This approach has established a relationship between convergence rate, room geometry and excavation age.
- Convergence rates that accelerate.
- Bed separation.
- Development of rib fractures.

The criteria used by Geotechnical Engineering for the SPDV Test Rooms was to recommend that access to the rooms be restricted once the rate along the center line of the drift reached 4.5 inches per year and to recommend the prohibition of all access once the convergence rate reached 6 inches per year.

The following criteria are proposed to determine when removal of waste becomes necessary:

a. a roof/floor closure rate along the center line of ther room of 6 inches/year.

b. a fracture that extends for a length of 80 feet continuously along a rib/roof/interface.

Factor 5.

There are difficulties in predicting in a geologic environment. However, at the WIPP conditions are very similar across the site, and the SPDV Test Room data will very likely provide an acceptable prediction of panel performance.

APPENDIX III

INSTRUMENTATION FOR ROOM 1, PANEL 1

INSTRUMENTATION FOR ROOM 1, PANEL 1

Existing Instrumentation

Borehole extensometer Two borehole extensometers are installed in each rib of the room. The extensometers are installed horizontally at wall mid-height in the pillar near the center of the room. The extensometers measure movements within the salt.

Convergence points Room closure is currently measured at room midspan at three locations along the room center line.

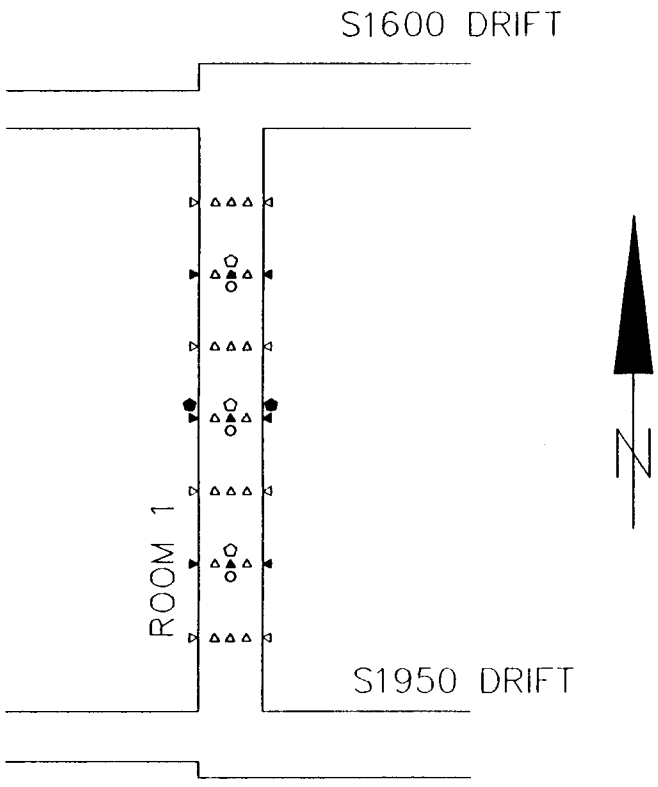
Proposed Instrumentation

Borehole extensometer Roof extensometers will be installed at three locations along the center line of the room. The purpose of these extensometers will be to monitor the possible development of bed separations at the clay seams below the anhydrite "a" and "b" layers.

Convergence points Additional convergence points will be installed to provide room convergence at a total of seven cross sections along the length of the room.

Observation boreholes Observation boreholes will provide visual observation of fracture development within the immediate roof beam. These boreholes will be approximately 12 feet deep and will be inspected on a regular basis.

PANEL 1, ROOM 1
INSTRUMENTATION LAYOUT



Key to Instrumentation
Existing:
▲ Vertical RC Chord
◀ ▶ Horizontal RC Chord
⬟ Roof or Rib MPBX
● Observation Borehole
Planned:
△ Vertical RC Chord
◁ ▷ Horizontal RC Chord
⬠ Roof or Rib MPBX
○ Observation Borehole

100'