U.S. Department of Justice

Washington, D.C. 20530

July 1, 1991

FACTORS IN DECISIONS ON CRIMINAL PROSECUTIONS FOR ENVIRONMENTAL VIOLATIONS IN THE CONTEXT OF SIGNIFICANT VOLUNTARY COMPLIANCE OR DISCLOSURE EFFORTS BY THE VIOLATOR

I. Introduction

It is the policy of the Department of Justice to encourage self-auditing, self-policing and voluntary disclosure of environmental violations by the regulated community by indicating that these activities are viewed as mitigating factors in the Department's exercise of criminal environmental enforcement discretion. This document is intended to describe the factors that the Department of Justice considers in deciding whether to bring a criminal prosecution for a violation of an environmental statute, so that such prosecutions do not create a disincentive to or undermine the goal of encouraging critical self-auditing, self-policing, and voluntary disclosure. It is designed to gi federal prosecutors direction concerning the exercise of prosecutorial discretion in environmental criminal cases and t ensure that such discretion is exercised consistently nationw It is also intended to give the regulated community a sense c how the federal government exercises its criminal prosecutor discretion with respect to such factors as the defendant's voluntary disclosure of violations, cooperation with the

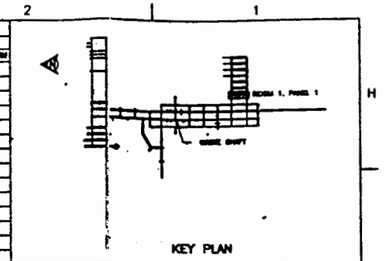
910702

LIST OF MATERIAL

QTY REQ GR.3	QTY REQ GR.2	QTY REQ GR.1	ITEM NO.	ORDERING INFORMATION	NOTE	DRAWING NO. OR PART NO.	MATERIAL SPECIFICATION MAT.	STD	NUMBER	COMP	FORM
		-	1	ENHANCED ROOF SUPPORT ASSEMBLY		54-D-003-D-001					
	-	AR	2	STEEL SET ASSEMBLY	"	54-D-003-D-002					
-	1		3	STEEL SET CORNER PIECE ASSEMBLY		54-D-003-D-003					
4			4	BAR, FLAT .38 THK STK	A		CS	ASTM	A36		
1	2		5	CHANNEL, C15X40	A		CS	ASTM	A36		
	4		6	BOLT, .83-11 UNC-2A X 2.50 LG. HEX HEAD	B	912478298	CS	ASTM	A325		
	4		7	NUT, .83-11 UNC-2B, HEX HEAD	B	904734035	CS	ASTM	A563		
		AR	8	PLYWOOD, .75 THK STK 4'X10' SHT	F		CDX				
		AR	9	METAL, EXPANDED 16 GA. 4'X10' SHT	G		CS				
		AR	10	WIRE, MESH .25 W/4X4 OPENINGS 5'X14' SHT	C		CS				
		AR	11	FENCING, CHAINLINK 9 GA.	H		GAL.	ASTM	A392		
		AR	12	CABLE, 0.83 6X25 IWRC	B	3455715					
		AR	13	CLAMP, CABLE	B	3465718					
		AR	14	ROCKBOLT, 13' LONG	D						
		AR	15	ROCKBOLT, 4' LONG	D						
		AR	16	PLATE, STEEL 6X6X.50 THK STK	D		CS				
		AR	17	PLATE, TRUSS	C						
		AR	18	LOADCELL							
		AR	19	RESIN, EPOXY	E	3212H80					
	8		20	WASHER, FLAT .83	B	918804035	CS	ASTM	F436		

SUPPLIERS:

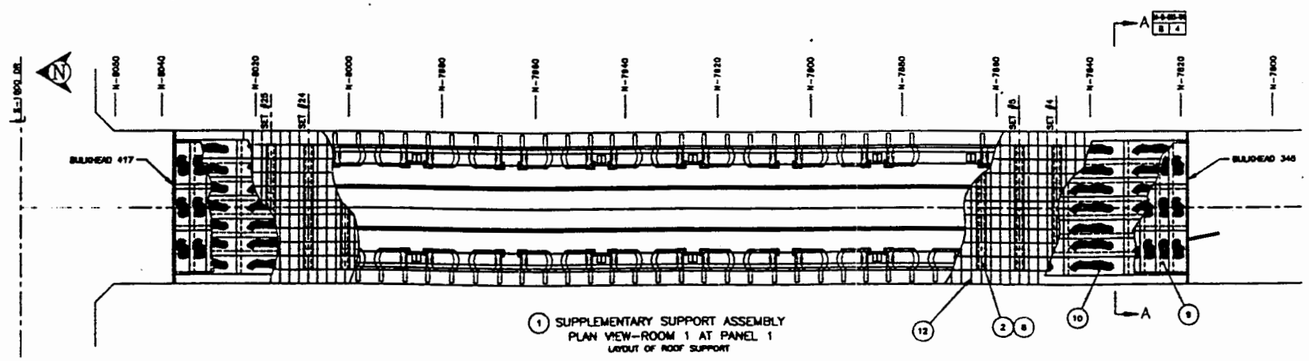
A. RYERSON OR APPROVED EQUAL
B. McMASTER-CARR OR APPROVED EQUAL
C. JOHNMAR CORP. OR APPROVED EQUAL
D. DYNIDAG OR APPROVED EQUAL
E. CELTITE OR APPROVED EQUAL
F. TEXAS LUMBER OR APPROVED EQUAL
G. RAYNELL OR APPROVED EQUAL
H. ACME FENCE OR APPROVED EQUAL



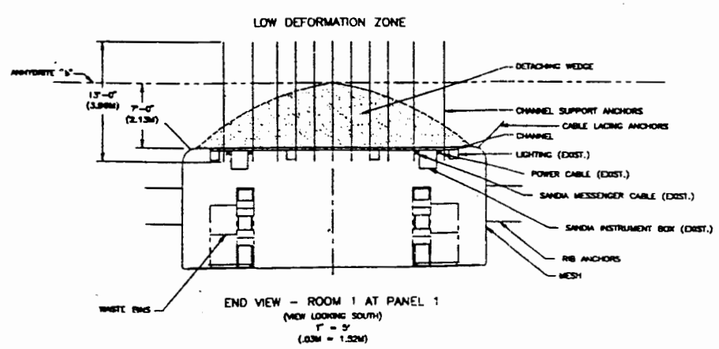
KEY PLAN

REFERENCE DRAWINGS

BIN-SCALE TESTING PANEL 1 ROOM 1 POWER AND LIGHTING	63-J-053-W
UNDERGROUND WIRING PANEL 1-PROFILES ALONG ROOM 1 AND S1850 DR.	51-W-088-W
BIN SCALE TESTS VOC MONITORING AND HELIUM STATION, DETAILS	52-L-002-W
BIN-SCALE TESTS ROOM 1 LAYOUT	52-W-001-W



1 SUPPLEMENTARY SUPPORT ASSEMBLY
PLAN VIEW-ROOM 1 AT PANEL 1
LAYOUT OF ROOF SUPPORT



END VIEW - ROOM 1 AT PANEL 1
(VIEW LOOKING SOUTH)
1" = 5'
(.03M = 1.52M)

REV	ISSUE DESCRIPTION	DFTR DATE	CHK DATE	CDS DATE	APRD MGR DATE	ECO	PRB

U.S. DEPARTMENT OF ENERGY
Westinghouse Waste Isolation Division
CARLSBAD NEW MEXICO

SUPPLEMENTARY ROOF SUPPORT
ROOM 1, PANEL 1
GENERAL ARRANGEMENT

SCALE 1" = 10' U.N.O. | STYLE AU06 | DWG. NO. 54-D-003-W1 | REV. NOW | SHEET 1 OF 5

APPENDIX A

DRAFT 09/10/91

WASTE ISOLATION PILOT PLANT
SUPPLEMENTARY ROOF SUPPORT SYSTEM

APPENDIX A

Geology and Rock Mechanics

Westinghouse Electric Corporation
Waste Isolation Division
Carlsbad, New Mexico
August 1991

GEOTECHNICAL DESIGN SUMMARY REPORT

FOR ROOF SUPPORT SYSTEM IN ROOM 1, PANEL 1

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GEOTECHNICAL DESIGN SUMMARY REPORT

1.0 INTRODUCTION

The purpose of this report is to provide the geotechnical basis for the design of a system to support the rock in the roof of Room 1, Panel 1. The system must ensure that the room meets the functional requirements necessary to support its use as an underground laboratory for the study of gas generation from CH TRU radioactive waste (Molecke, 1990). This research program, called the Bin Scale Testing Program, is under development at the present time and the experiments are expected to start in the second half of 1991. The Geotechnical Design Summary Report interprets the geologic and rock mechanics data presented in the annual Geotechnical Field Data and Analysis Reports (US DOE, 1991a; US DOE, 1990) and other occasional reports (US DOE, 1991b) and presents the geotechnical assumptions that have been made for the design.

The geotechnical investigations at the WIPP are comprehensive and provide detailed information on the site conditions that is not typically available for an engineering design. This has enabled the Geotechnical Engineering Section, Westinghouse, WID to establish a phenomenological model that explains the performance of openings. This model establishes the mechanisms that must be addressed by the design of the support system in order to control the roof conditions in Room 1, Panel 1.

1.1 BACKGROUND

Room 1 of Panel 1 is currently 5 years old and must remain accessible for a further 7 years in order to support the bin scale testing program. Following the collapse on February 4th, 1991 of the roof in the Site and Preliminary Design Validation (SPDV) Test Room 1 that confirmed the concerns raised by the Geotechnical Engineering Section concerning the capability to maintain the Panel 1 room for the period of the bin scale tests, a panel of Geotechnical experts was formed to evaluate the life expectancy of the underground room in which the tests will take place.

The panel concluded that if no additional remedial measures were taken, the rooms in Panel 1 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 (US DOE, 1991b). The most current geotechnical field data from this room (US DOE,1991, in preparation) does not indicate that its geomechanical performance differs significantly from that observed in SPDV Test Room 1. On this basis, the remaining life for Room 1 as currently supported is about two to three years.

The panel members agreed that measures could be taken that would provide reasonable assurance that the bin scale tests could be carried out to completion in Panel 1. They suggested a number of alternative actions that could be taken and recommended that the WIPP project evaluate the alternatives and select one, or a combination, of 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.

The WIPP project has evaluated the support systems suggested by the Geotechnical Expert Panel. The initial evaluations looked at support systems that could be installed within the rooms and would provide a passive support as the rock moved into the excavation. These systems were eventually abandoned because they interfered with the functional use of rooms as a location for the bin scale tests. Problems were associated not only with the physical size of the supports which limited the number of bins that could be placed in a room but more importantly, the support could not be placed where it was needed (i.e. midspan, where the largest loads develop) without eliminating access to the bin locations.

The project has then assessed the installation of additional rock reinforcement in the roof of Room 1, Panel 1 as a means of extending the life of the room. Rock bolts as normally installed do not provide the capability to establish with any level of confidence either a support system with a specific working life or a measure of performance on an ongoing basis. Therefore, a composite system of support has been designed that incorporates beams at nominal nine feet centers along the length of the room, supported by a system of tendons anchored in competent salt in the roof with lacing and meshing. The rock reinforcement system (i.e. anchored steel tendons) has been designed as rock anchors, where appropriate, because rational design approaches are available for their design and extensive field testing programs are typically used to confirm the design.

1.2 DESIGN METHODOLOGY

The primary emphasis of the anchorage system is to guard against the most probable modes of movement that may lead to collapse. The design requires detailed site specific geologic information, the study of information from relevant case studies, design calculations based on available data for the rock and the anchorage systems and field tests. Field proving tests of the anchorage system and the monitoring of support performance during its operational life are essential considered to the success of the system.

The bases for the design approach are the recommendations prepared by the Post-Tensioning Institute (Post-Tensioning Institute, 1986) that provide guidance in the design installation and testing of rock anchors. In addition, information from other publications that relate to the design of rock anchors and their field performance have been used, where appropriate (BS 8081, 1989; Corps of Engineers, 1980;) Littlejohn and Bruce, 1976). These publications provide guidelines to rationalize procedures for the design of rock anchors. As far as possible, the guidelines given in these documents have been followed but where the recommendations have not been, the reasons are discussed in this document.

Although the support requirements for underground excavations at the WIPP are not as great as that typically needing rock anchor support, the rigor of the design approach for ground anchors and the extent of the proof testing that accompanies the installation of every anchor justify this approach for the design of the roof support system in Room 1, Panel 1.These anchor systems are designed to be effective for extended periods of time and consequently require special design and quality control in installation. The design approach requires the performance of each anchor to be establish quality control in installation so that the performance of the overall support system is effective. This is in contrast with the design for rock bolting which does not generally attempt to determine the performance of every bolt or to determine system performance.

2.0 STRATIGRAPHY OF REPOSITORY HORIZON

The proposed underground storage facility is located 655m below the surface in bedded salt of the Permian Salado Formation. A generalized stratigraphy showing the facility level is given in Figure 2.1. Over 365m of impermeable evaporitic deposits separate the facility horizon from the overlying sedimentary rocks and 620m of evaporites lie below the facility horizon and provide a barrier to Permian limestones and sandstones.

Halite is the most abundant mineral in the Salado and occurs in thick beds intercalated with thinner beds of polyhalite and anhydrite. Salado halite is rarely pure and usually contains trace and minor amounts of foreign material including clay, anhydrite and polyhalite. Halite crystal size and morphology vary considerably, and various large and small scale sedimentary features are abundant throughout out all of the Salado Salt. A detailed discussion of the Geology of the Salado formation can be found in the Geologic Mapping of the Air Intake Shaft at the Waste Isolation Pilot Plant (U. S. DOE, 1991c).

The facility horizon lies within a 12m thick unit consisting of halite, argillaceous halite, and polyhalitic halite. Figure 2.2 identifies the typical geology within this unit. Observations indicate that these geologic conditions are consistent across the site at the repository horizon. Figure 2.3 a, b, and c provide the stratigraphy exposed in Room 1, Panel 1.

A 0.3m to 0.5m thick layer persistent bed of sulfate (anhydrite and polyhalite), identified as Marker Bed 139 lies about 1.5m below the floor level. Considerable lateral variability in composition and thickness exists within this sulfate bed at both the regional and repository scale. The variability in thickness is associated with the top of the deposit and undulations of the order of 6 inches have been observed in 4 inch diameter bore (Holt, 1991). The bottom of the Marker Bed is sub horizontal and is underlain by Clay "E".

Anhydrite beds (less than 10mm thick), called anhydrites "a" and "b" occur about 4m and 2m above the roof. Thin clay seams called Clay G and Clay H are associated with the bottom of these beds. In addition, a thin clay layer identified as Clay F is found intermittently in the immediate roof of excavations.

The Marker Bed 139 and the clay layers can have a significant impact on the mechanical performance of excavations. The clay layers provide surfaces along which slip can occur whereas the Marker Bed acts as a unit that does not deform plastically with time. In addition, the undulating nature of the top of the Marker bed will resist shear movements along the interface with the overlying salt.

3.0 PROPERTIES OF ROCKS AT REPOSITORY HORIZON

The reference material properties for the repository horizon rocks are provided in Table 3.1. These properties are based on laboratory tests carried out during the site characterization phase of the WIPP Project.(Kreig, 1983). The tests have demonstrated the range of mechanical properties associated with the WIPP strata, and in particular have defined the time dependent behavior of the salt.

Salt is a material that flows when subject to deviatoric stress conditions. This behavior has long been recognized in the mining industry (Baar, 1977; Dreyer, 1981) and considerable efforts have been made to characterize this response from laboratory creep tests. However,

extensive experience within the mining industry has demonstrated the difficulties involved in establishing in situ performance based on the laboratory characterization of salt creep (Baar, 1977).

The mechanical response of the salt at the WIPP has been characterized by a steady state creep law that was developed in the early 1980's by laboratory creep tests on the Salado salt (Hansen, 1979; Hansen and Mellegard, 1979; Heitmann et al, 1980). This constitutive law relates creep strains to stress and temperature. The relationship ignores transient creep effects that will influence early time deformations and does not include dilation of the salt that will occur when the rock is subject to deviatoric stresses and low confining pressures. The steady state creep law has been used in the model studies to establish predictions of the structural performance of the openings.

Field observations at the WIPP (Cook and Roggenthen, 1991) and in the Carlsbad Potash Basin (Greenwald and Howarth, 1938) have shown that the brittle behavior of salt under deviatoric stresses with low confinement can be a significant factor contributing to the mechanical performance close to excavations. However, the brittle behavior of salt has not been characterized by laboratory studies and constitutive laws for salt do not include fracture development or rock dilation. The evaluation of performance based on the steady state stress law must take into consideration the limitations of the constitutive law applied to the salt. Provided that these limitations are understood, and the structural responses of other stratigraphic zones are properly modelled, it will be possible to establish useful models with which to predict the structural performance of excavations at the WIPP.

4.0 IN SITU STRESS REGIME

The initial stress state at the repository horizon is established from Heims Rule for weak rocks (Hoek and Brown, 1980). This rule establishes the vertical stress as dependent on the depth of overburden and its average density, and the horizontal stresses to be equal to the vertical stress. Taking the average density for the overburden at the WIPP site as 2130 kg/m^3, the initial stresses at the repository horizon are about 2000 psi.

Measurements of virgin in situ stress in salt are difficult to achieve since the measuring techniques assume that rock behaves in an elastic manner (Hoek and Brown, 1980) whereas salt deforms plastically. However, hydraulic fracturing tests have been carried out in boreholes in salt at the WIPP in order to estimate the stresses (Wawersik and Stone, 1986). Although data interpretation was difficult, it was concluded that the virgin in situ stress state at the WIPP is approximately uniform in all directions and that the stress magnitudes correspond to the weight of the overburden. This conclusion confirms, the assumptions normally made for the far field stress distribution in salt, and with theassumptions used on for design the WIPP Project (DOE, 1986).

5.0 STRUCTURAL RESPONSE OF EXCAVATED ROOM

Field observations form the basis for a phenomenological model of the structural performance of the underground excavations. This performance is best characterized by data from the SPDV Test Room Panel. These test rooms are among the oldest excavations underground, having been constructed in March and April, 1983. The rooms have the same size and shape as those

in the proposed waste storage panels and are located at the same geologic horizon. They are relatively large excavations with each room having a nominal height of 3.95m, width of 10m, and a length of 91m. The rooms are separated by 30m wide pillars. This configuration results in an extraction ratio in the Test Room panel of about 25 percent. The rooms were excavated in order to confirm the geology, validate design assumptions for the underground, and provide data, where necessary for revision of the design.

Observations of the performance of these rooms have been routinely made over the past eight years. They have established room performance in terms of room closure, rock movements and the development of fractures in the immediate vicinity of openings (US DOE, 1989; US DOE, 1990; Cook and Roggenthen, 1991). SPDV Test Room 1 has provided the most complete picture of the structural performance of an excavation. Measurements were taken in this room over a period of almost eight years, from immediately following its excavation until a major roof fall occurred. Different stages in the performance of the room can be related to its roof/floor closure history (see Figure 5.1). Other rooms are showing the same general behavior but none others (outside of the SNL experimental area) have yet failed. SPDV Test Room 1 provides the most detailed example of the performance that can be expected from other rooms having similar geometries.

In addition, numerical analyses have been carried out to evaluate the structural performance in terms of stress and strain redistribution taking place about excavations with time. The analyses have used the near repository horizon stratigraphy described in Section 2.0 and the mechanical properties of the rock types provided in Section 3.0. Of particular significance to the interpretation of the model, are:

- the time dependent relationship governing the mechanical response of the salt
- the properties that control bed separations at the strata interfaces

The field observations and the numerical analyses have been used to develop a model of the mechanisms that occur in the roof of an excavation with time. The model is primarily based on the performance monitored in SPDV Test Room 1. The various stages through which the room passes according to the model are shown in Figure 5.2. The field and analytical data supporting this phenomenological model are given in the following sections of the report. It is expected that rooms having a geometry similar to the SPDV Test Rooms and the waste storage rooms will eventually pass through all the stages identified in Figure 5.2 unless remedial actions are taken to control roof deterioration and roof movements. The roof support system for Room 1, Panel is based on the need to control the conditions identified by the model.

5.1 ROOM DEFORMATION

Rooms with similar geometries have shown relatively consistent deformation characteristics. Although actual magnitudes of the room closures show a range of values. The variability in the closure rates is demonstrated in Table 5 which lists rates of closure at mid room, mid span for the SPDV Test Rooms and the rooms in Panel 1 all of which have similar geometries. The highest closure rates appear to be related to the room closest to the barrier pillar and are lower in the middle of a panel and at the solid abutment. At present, no correlation has been established between closure rate and variables such as mining and variations in stratigraphy that might explain these differences. The variability in composition and thickness of the Marker Bed 139 may provide another explanation.

Table 5.1

CLOSURE RATES BY TIME SINCE EXCAVATION

Room	Date of Excavation at Instrument	Excavation Completed To Final Dimensions	Year 0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	10-11	11-12
Panel 1:														
Room 1	June 1986	August 1986	4.13	4.26	4.60	3.24	(3.57)							
Room 2	March 1987	March 1988	3.02	6.71	3.33	2.26								
Room 3	February 1987	March 1988	1.60	9.85	3.34	2.71								
Room 4	February 1988	March 1988	8.33	3.14	2.56									
Room 5	February 1988	March 1988	9.19	2.76	2.28									
Room 6	February 1988	May 1988	8.36	3.41	2.76									
Room 7	March 1988	March 1988	9.26	3.22	2.70									
SPDV:														
Room 1	April 1983	April 1983	6.79	3.75	2.93	2.88	2.85	3.36	4.70	12.77				
Room 2	March 1983	March 1983	7.69	3.16	2.39	2.19	2.14	2.33	2.74	2.87				
Room 3	March 1983	March 1983	7.59	3.65	2.64									
Room 4	April 1983	April 1983	5.70	3.18	2.65	2.33	2.13	2.33	2.42	2.31				
Working Assumptions for Room Closure:														
Case 1			7.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
Case 1 Revision 1			7.0	3.0	3.0	3.0	3.0	3.0	5.0	5.0	5.0	5.0	5.0	5.0
Case 2			7.0	3.5	3.0	3.0	3.0	3.5	4.5	5.5	6.5	7.5	8.5	9.5

Closure rates in (inches/year)

"Year" is determined from date of excavation at instrument

Room 1, Panel 1, rate for year 4-5 is based on 9 months' data

Data for Rooms 1 through 7 in Panel 1 and SPDV Test Rooms is from data published up to June 30th, 1990 (DOE/WIPP 91-012). Data for period from July 1, 1990, through March 1991 is unpublished

Design cases and estimates are unpublished

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The changes in roof profile with time are shown in Figure 5.3 based on data from SPDV Test Room 1. The room deformation, initially symmetrical about the room center but after about five years, the roof/floor closure, become assymmetric with one side closing faster than the other side. This behavior identified from SPDV Test Room 1 has been measured in other locations within the facility, and may be considered typical of the performance in the wider span excavations at the WIPP.

5.2 ROCK DEFORMATIONS ABOUT ROOM

FIELD DATA

Deformation measurements show that the rock mass deforms with time and that rock movements generally reduce with distance from an excavation surface although this behavior is modified at strata interfaces. Movements occur both normal and parallel to excavation surfaces. In particular, the Anhydrite 'b' in the roof and the Marker Bed 139 in the floor are associated with relatively large vertical and lateral deformations. Typical inclinometer data from a room cross section are shown in Figure 5.4 for inclinometers, and in Figure 5.5 for extensometers.

Bed separation has been identified at the Anhydrite 'b' in the roof after approximately three years. Separation at the clay/salt interface beneath the anhydrite 'b' appears to increase at a rate of about 25mm per year once the bond across the interface is broken.

The geotechnical data show that the roof and floor of an excavation act as a series of flexing beams separated by zones or planes across which differential movement occurs. This is largest at the anhydrite 'b' but does occur at other horizons above and below the excavations in association with strata interfaces, generally clay/salt interfaces. Lateral shifts in the roof indicate that beam flexure was still occurring at a depth of 15m. In the floor, deflections were not as pronounced, and have largely disappeared at a depth of 15m. Differential lateral movements of about 12 mm per year have been were measured in the immediate roof beam, five years after excavation. These rates of movement have been confirmed by the monitoring of lateral displacements in old excavations (Francke, 1991). Rock deformations about a room also is governed by fracture development. The typical fracture development observed in the wide excavations is shown in Figure 5.6. The most significant fractures are low angled shear fractures develop at the rib/roof interface of excavations. In SPDV Test Room 1, these fractures became sufficiently persistent and continuous that a detached wedge formed the roof in SPDV Test Room 1 fell in eight years after the excavation of the room. Precise surveys of the roof in SPDV Test Room 1 following the rock fall are shown in Figure 5.7 and indicate the geometry of the roof cross-sections to be arch shaped. The condition of the roof in Room 1, Panel 1 is shown in Figure 5.8 as of the summer 1991.

MODEL DATA

Roof/floor closure for the model is compared with the field data in Figure 5.9. Roof/floor closure rate is 0.1 m per year (4 inches per year) and does not vary significantly with time.

Bed separation provided by the model at the anhydrite 'b' level indicate that the separation is at a maximum above the mid span of the excavation and increases at about 12mm per year. The model is compared with the field data in Figure 5.9b. The model also shows that bed separation is occurring at the clay beneath the anhydrite 'a' layer.
This is not consistent with the field observations which do not indicate bed separation at this level. The model has therefore shown good agreement in the vicinity of the opening but may not be accurate with regard to performance at the anhydrite 'a' layer.

The shear strains are shown in Figure 5.10. They build up with time as creep occurs under relatively constant compressive stress in the immediate roof beam and the contours of effective strains indicate potential failure planes. These planes are consistent with the fracture development that occurs in the roof of excavations with time.

5.3 STRESSES AROUND ROOM

Prior to excavation, the strata at the repository horizon are subject to an in situ stress field that is uniform in all directions and has a value of about 2000 psi which is equivalent to the overburden loading. Immediately the excavation is made, the stresses adjust to an elastic distribution. Of particular importance are the high shear stress concentrations that develop in the corners of the excavation (Miller, 1991). These may provide incipient fracturing that later develop into discrete fracture planes. With time, the stresses in the immediate vicinity of the excavation reduce due to stress relief as the salt moves into the opening. The excavation disturbs the stresses and the redistribution continues over time dependent properties of the salt. The principal maximum and minimum stresses induced at 0, 3 and 5 years following excavation are shown in Figures 5.11 & 5.12. These plots indicate the changes in stress that take place with time.

The influence of the stratigraphy on the stress distributions is evident immediately that the excavation is formed. The Marker Bed 139 modifies the structural performance of the floor and the anhydrite 'b' effects the stress distribution in the roof.

In the floor, the Marker Bed 139 acts as a stiff unit which does not exhibit time dependent behavior. The variable elevation in the upper boundary of the Marker Bed indicates that a high resistance to shear movements at this boundary develops. These immediate effects were also observed in numerical analyses presented by Mckinnon (WIPP/DOE 91-023) at the Geotechnical Expert Panel Meeting The salt above and below the Marker Bed will deform with time and depending on the slippage between the salt and the anhydrite will maintain high compressive stress into the Marker Bed. These high stresses may cause brittle fracture of the Marker Bed 139 and its failure which would result in floor heave.

The clay beneath the anhydrite 'b' introduces a plane of low frictional resistance into the strata sequence. The plane will not support shear stresses, and this isolates the immediate roof beam. The plastic flow of the clay ensures that high shear stresses cannot develop at the interface and the low bond strength between the clay and the salt leads to separation at the interface. Once the immediate roof beam becomes isolated, the lateral movements of the pillars maintain induce lateral stresses into the beam. With time, the shear strain build up in the beam eventually results in the development of failure.

Failure relieves the lateral stresses in the beam. However, due to the continued lateral creep of the salt the roof beam continues to be subject to residual compressive stress. As the shear failures propagate in the longitudinal direction, the weight of the unsupported section of the roof spanning increases. At some critical length, the strength of the beam cross section is exceeded and the roof fails as a unit.

6.0 PERFORMANCE REQUIREMENTS FOR SUPPORT SYSTEM

The performance requirements can be divided into two sets. These relate to the functional requirements imposed by the experiments and the geotechnical considerations that emanate from the functional requirements.

6.1 PERFORMANCE

The functional requirements in order to support the bin scale experiments in Room 1 of Panel 1 are as follows:

1. Room 1, Panel 1 must remain accessible for a total life of 12 years based on the current five year age of the room and the requirement for a further effective life of seven years to support the bin scale experiments (FSAR Addendum).

2. The bin locations are established along each rib of the room and a central access way 17 feet wide is maintained down the center of the room.

These functional requirements mean that not only must room stability be maintained but also that creep closure of the room should not impinge upon the envelopes for the access ways and for maintenance of the bins.

6.2 GEOTECHNICAL DESIGN BASIS FOR THE ROOF SUPPORT SYSTEM FOR ROOM 1, PANEL 1

A geotechnical design basis has been developed from the discussions presented in Section 5. The design provides a system support of the roof in Room 1, Panel 1 to meet the functional requirements described in Section 6.1. The geotechnical design basis is as follows:

- The support system shall support the weight of the detached rock wedge that forms in the roof as a result of the development of low angled fractures from the ribs.

- The support system shall accommodate vertical movements of the roof that include both differential and total displacements.

- The support system shall accommodate lateral shifts

These requirements are discussed in the following sections in terms of the bounding values that encompass the conditions expected to be encountered in the field.

6.2.1 ROCKLOAD TO BE SUPPORTED

The weight of the detached rock wedge that forms in the roof depends on the orientation of the fractures that develop. An estimate for the geometry of the cross-section of the wedge that must be supported is given in Figure 6.1a. The estimate is based on the observations of the rock fall geometry seen in SPDV Test Room 1 and the interpretation contours of shear strain in the roof (Miller, 1991). The wedge geometry consists of an arched shape that forms from low angled fractures starting at the the ribs whose propagation are bounded and controlled by the bed separation occurring at the Anhydrite 'b'.

The design load for each rock anchor has been based on the maximum weight of rock that is predicted from the rock fall data. This only develop at midspan and it can be expected that the loads requiring support will reduce towards the ribs. A representation of the expected loads that will develop in each row of anchors across a cross-section is given in Figure 6.1b. Based on these estimates, the design load for each anchor has been taken as 20,000 lb.

The rock load will be supported in suspension by the rock anchors from overlying competent rock. The concept of transfer of part of the weight of weaker or thinner beds to flexures more rigid strata by rock bolts was originally described by Panek (1962, RI 6138). The mechanism is relatively simple and calculations based on it cover the problem os stabilization completely (Habenicht, 1983) However, despite the simple nature of the suspension process and the relatively simple analysis associated with the design approach, the calculations have a limited role in the design and must be supplemented by performance tests on the anchors. This is due to the uncertainties associated with rock mechanics data, such as rock properties their heterogeneity, stress fields and rock mass composition. Design calculations are made to provide a theoretical background, while the practical details of the design rely on practical site specific tests, field experience and field observations (Habenicht, 1983). For instance, it is possible that the irregularity in the shape of the planes of separation that outline a rock body and the unpredictability of potential fracture planes can significantly alter the volumes and therefore the weights of the rock load that is being designed to be held.

6.2.2 VERTICAL MOVEMENTS

The vertical movements that the roof must accommodate are a combination of salt creep, dilation of the salt due to fracture development, and gravity effects once fractures have formed.

The total displacement has been taken as 38mm per year vertical lowering of the roof at midspan. The differential movement is taken as the difference between displacement of the roof at mid span and that close to the ribs. The differential movement to be accommodated by the design has been taken as a maximum of 25mm per year. Roof displacements are not always a maximum at mid span. Once fracturing becomes visible in the roof along one rib, then the field data shows that the roof deforms asymmetrically with one side lowering more rapidly than mid span or the other side.

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Fig 6.1 goes away

Fig 6.1a goes away completely

Fig 6.1b is replaced by Fig 4.1 of the Appendix D Geotechnical Monitoring Plan.

6.2.3 LATERAL MOVEMENTS

The lateral displacements occur at strata interfaces and within the immediate roof beam where discrete fractures have formed. These lateral shifts may be associated with the widening of fracture apertures and bed separation. The lateral differential displacements have been observed up to 15m into the roof at strata changes particularly the clay/salt contacts. The largest shifts are found at the clay/salt contact below the Anhydrite 'b' layer. Lateral shifts can also be expected within the immediate roof beam where fractures form. The support system should be designed to accommodate a lateral shift of 12mm per year and bed separation of 25mm per year at the clay/salt contact below the Anhydrite 'b' layer once the bond at the interface is disrupted.

6.3 DESIGN CONSIDERATIONS

Despite the simple nature of the suspension process by which provide support to an excavation and the relatively simple mechanical analysis associated with the design approaches, based on this process, calculations play a limited role in the design anchors and must be supplemented by performance tests on the anchors. This is due to the uncertainties associated with rock mechanics data, such as rock properties their heterogeneity, stress fields and rock mass composition. Design calculations are made to provide a theoretical background, while the practical details of the design rely on practical site specific tests, field experience and field observations (Habenicht, 1983). For instance, it is possible that the irregularity in the shape of the planes of separation and the potential fracture planes that outline a rock body can significantly alter the volumes and therefore the load that must be supported..

A primary consideration in the design of the anchors must be to ensure that they are anchored in stable ground that will not be subject to fracturing. The proposed support system will support the rock load due to the detached rock wedge from competent ground located between anhydrites "a" and "b". The load transfer is accomplished by means of ground anchors anchored from 9 to 12 feet into the roof. The rock mechanics field data (Section 3.2) indicates that fracturing can be expected in the immediate roof and up to the anhydrite "b" layer. Although slips do occur above this level, there is no indication that large separations have developed above the anhydrite "b" level. Therefore, the location for the length of the fixed anchor has been above the anhydrite "b" level with sufficient allowance to ensure that the anchor capacity is not affected by the separations that develop at the anhydrite "b" level or by the location of the mechanical anchors previously installed.

A feature of the support system is the capability to reduce the tension on the steel tendon once the load exceeds the design load. Based on the evaluation of the rock mechanics field data (US DOE, 1987; US DOE, 1988; US DOE, 1989; US DOE, 1990), the support must permit roof lowering of upto 50mm per year. The overall roof lowering over a nine year period will therefore, be 450mm. Since the roof will not develop load uniformly, the design is based on the capability to adjust the loads on each anchor independently.

The rock between the anchors may break up as a result of the constraint being applied by the rock anchors. The broken rock will be supported by the system of lacing and mesh emplaced on the roof between the rows of rock anchors. In reality, it is expected that the roof rock will retain much of its inherent strength and the rock will bridge between the ground anchor supports located at nominal nine centers along the length of the room. This expectation is substantiated by the performance in SPDV Test Room 1 where the detached wedge remained a single unit and fell essentially intact.

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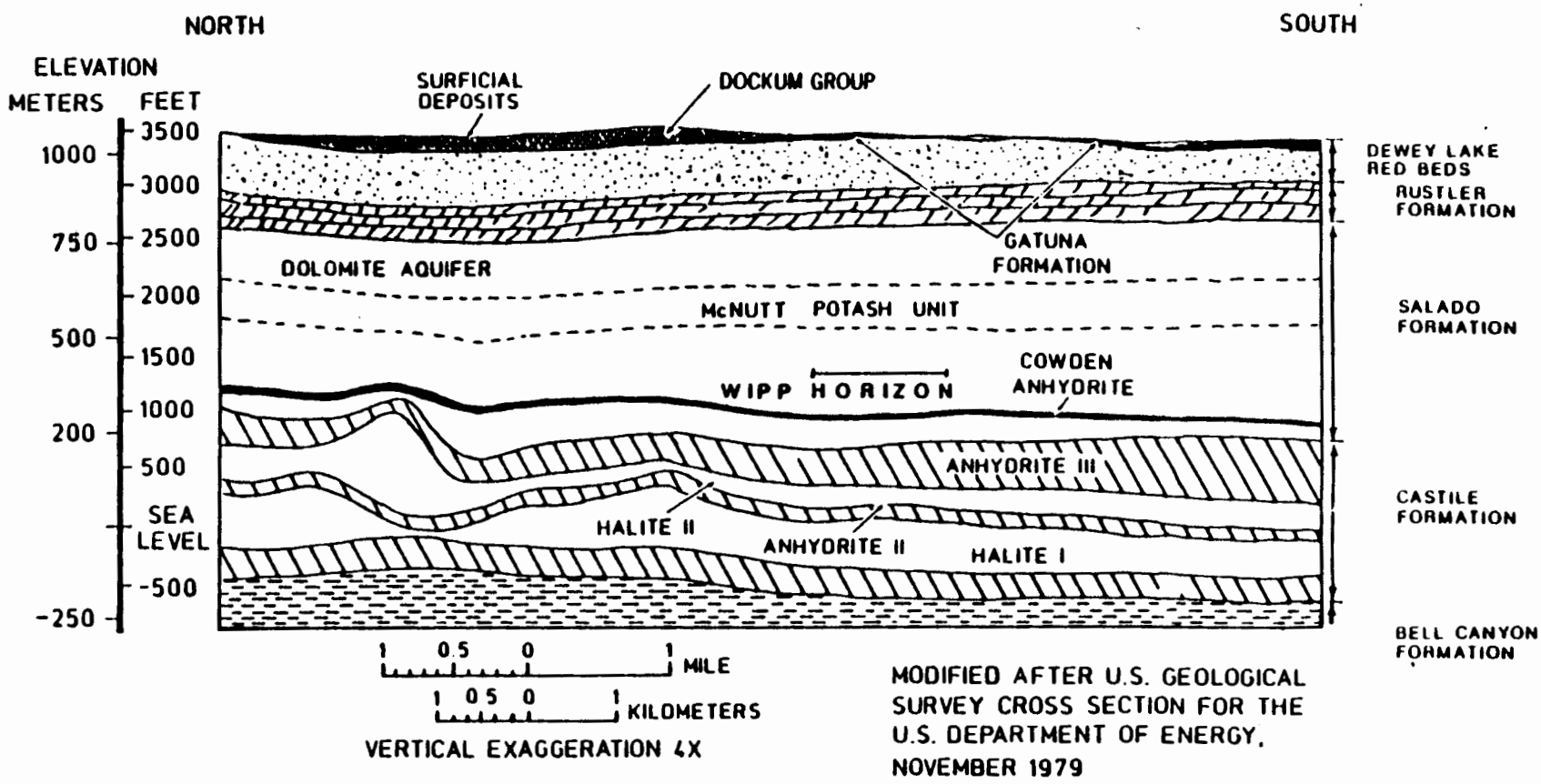

Figure 2.1 WIPP Site Stratigraphy

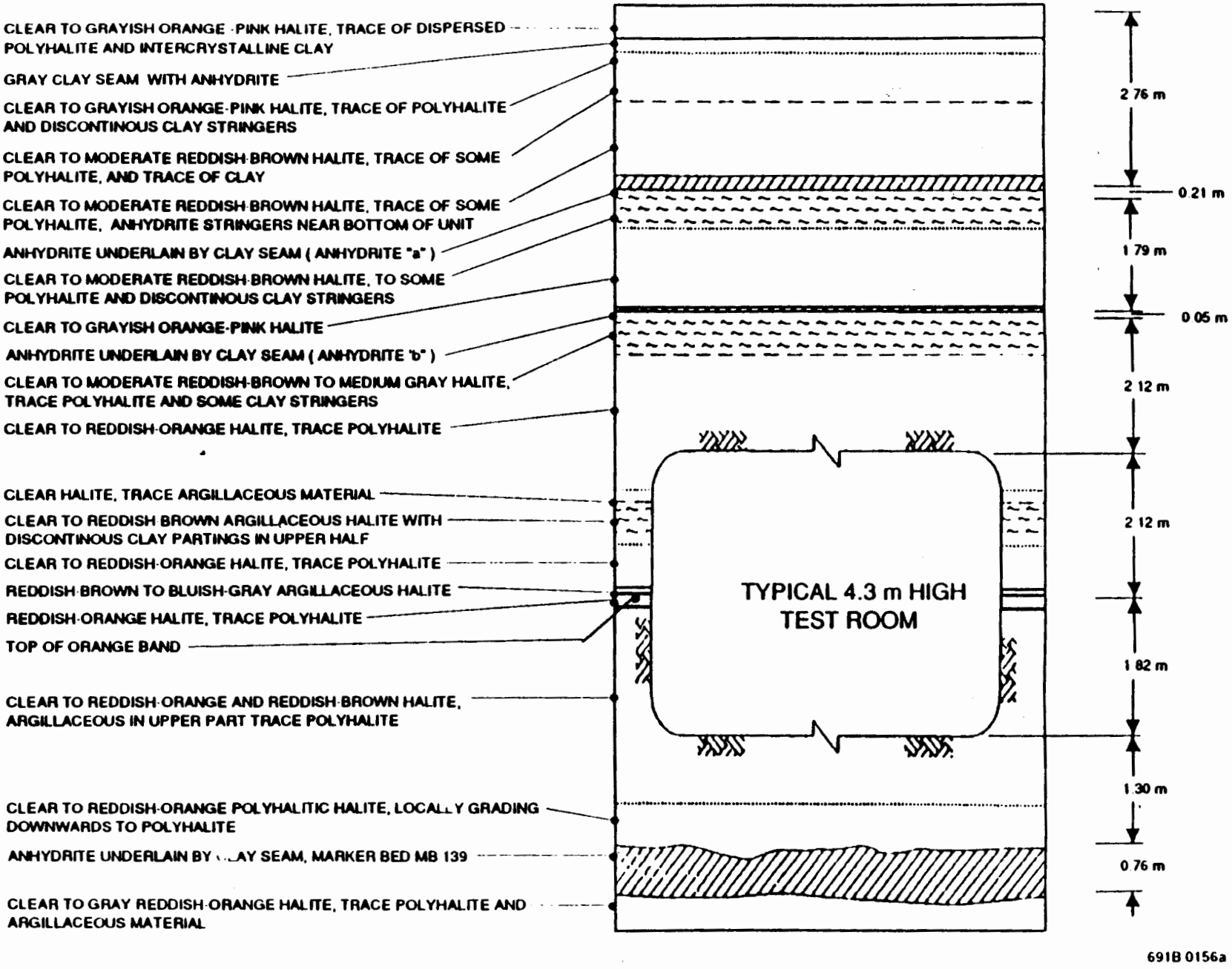

Figure 2.2 Stratigraphy of the WIPP Repository Horizon

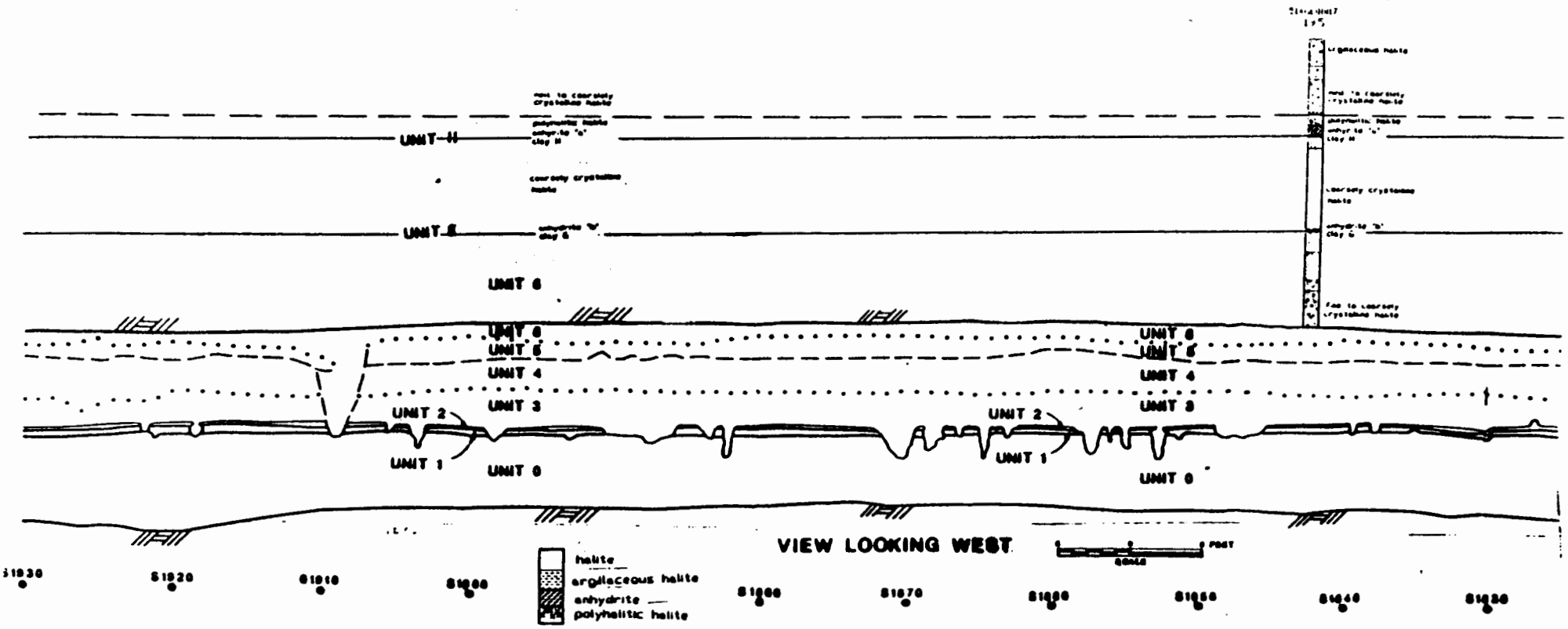

Figure 2.3a PANEL 1 - ROOM 1

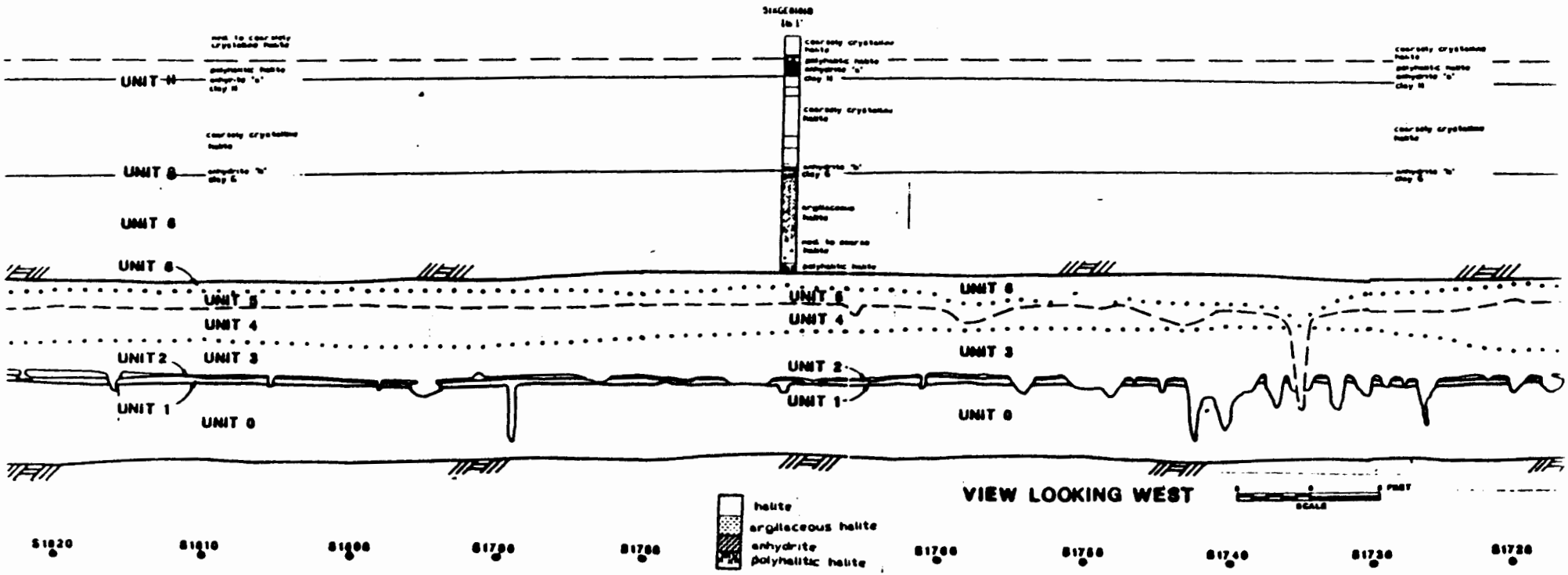

Figure 2.3b PANEL 1 - ROOM 1

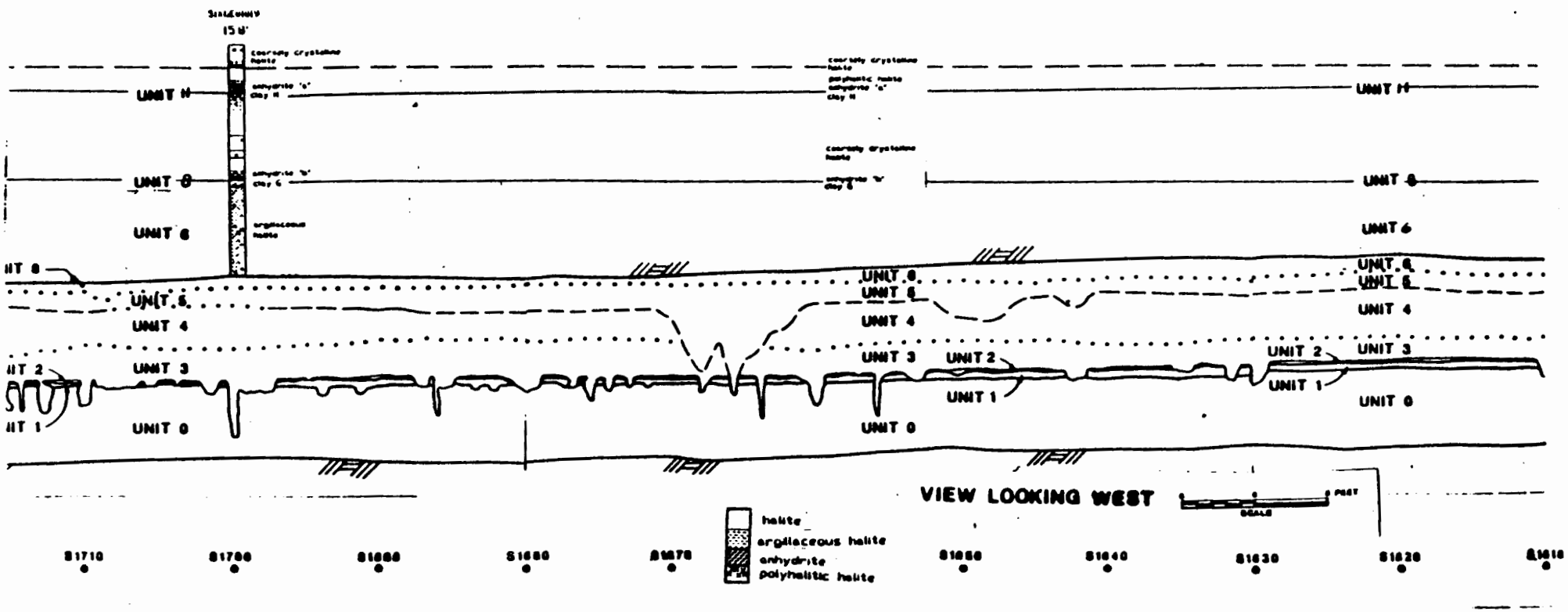

Figure 2.3c PANEL 1 - ROOM 1

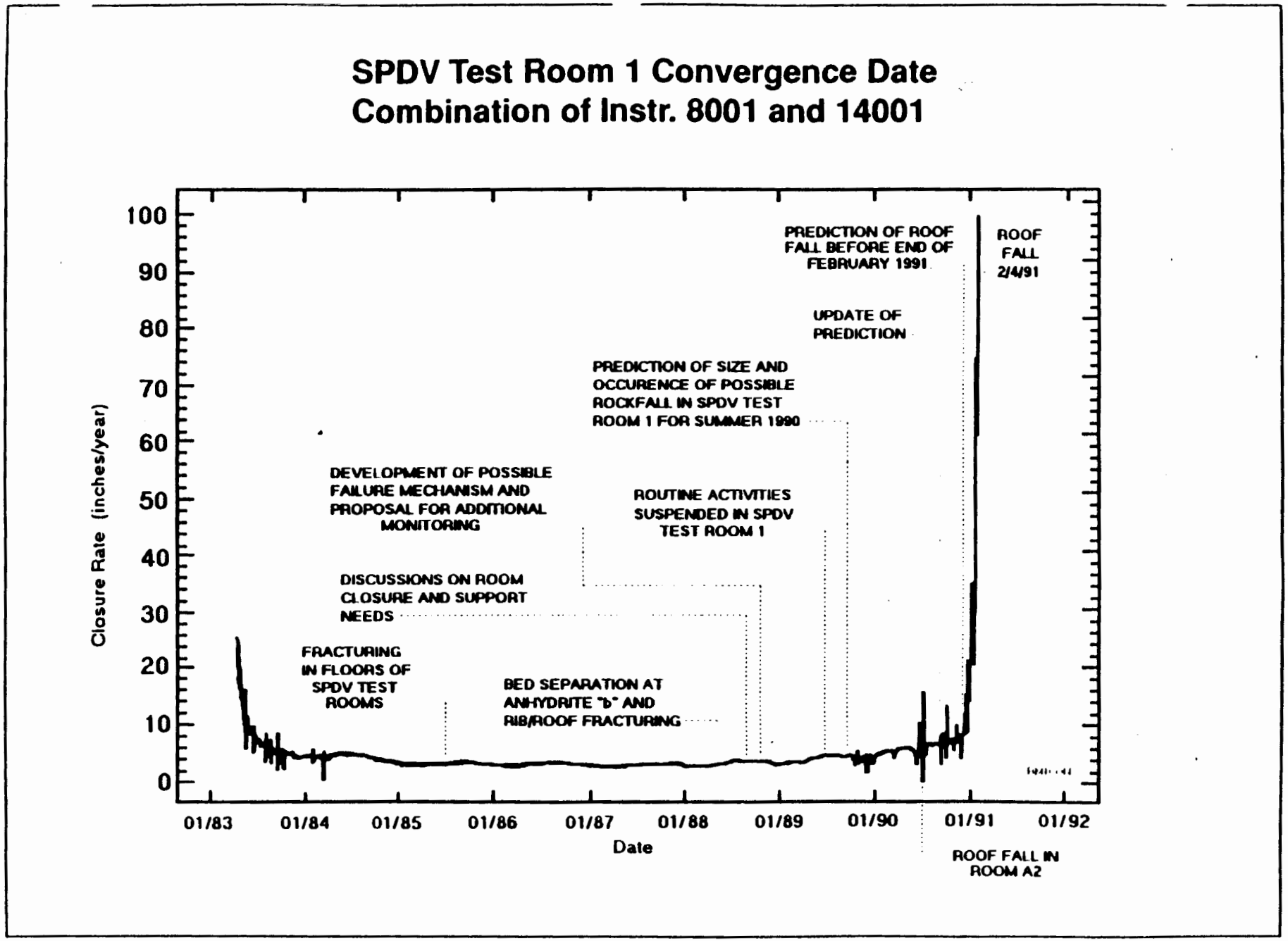

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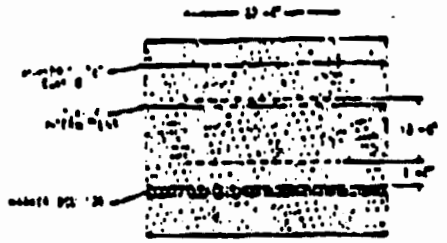
Figure 5.1



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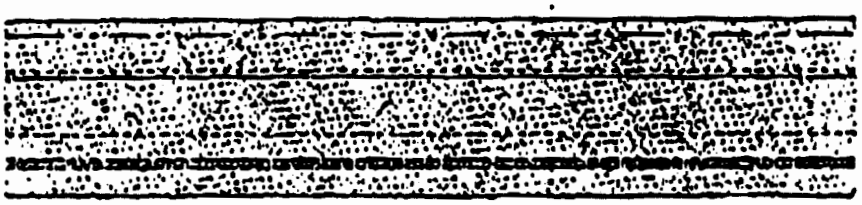

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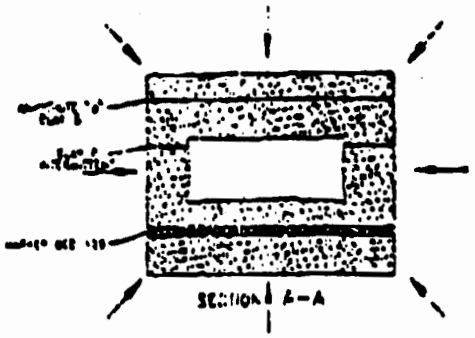

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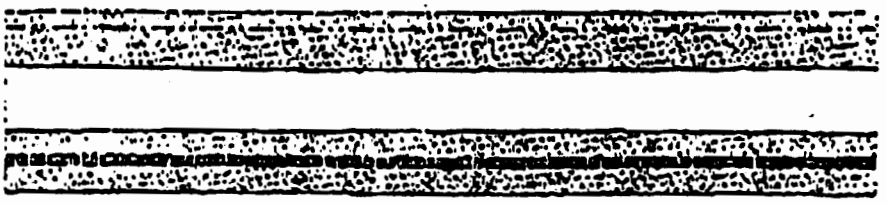

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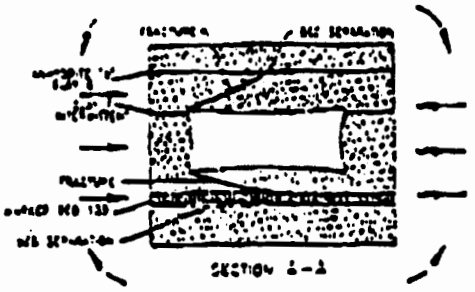

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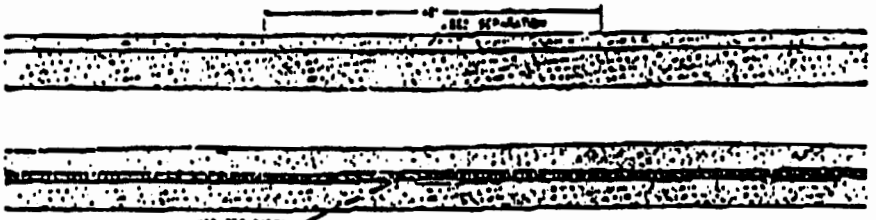

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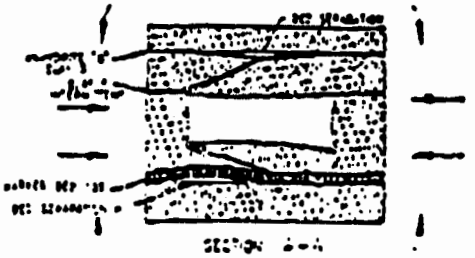

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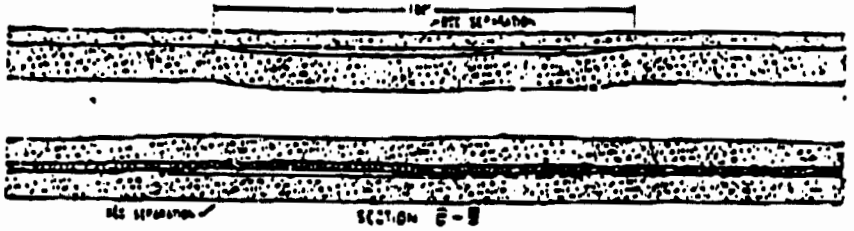

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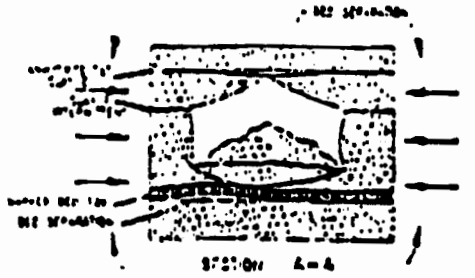

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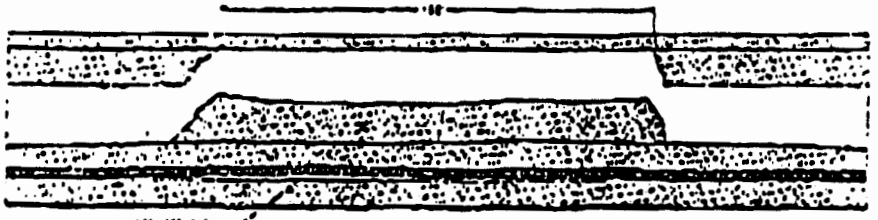
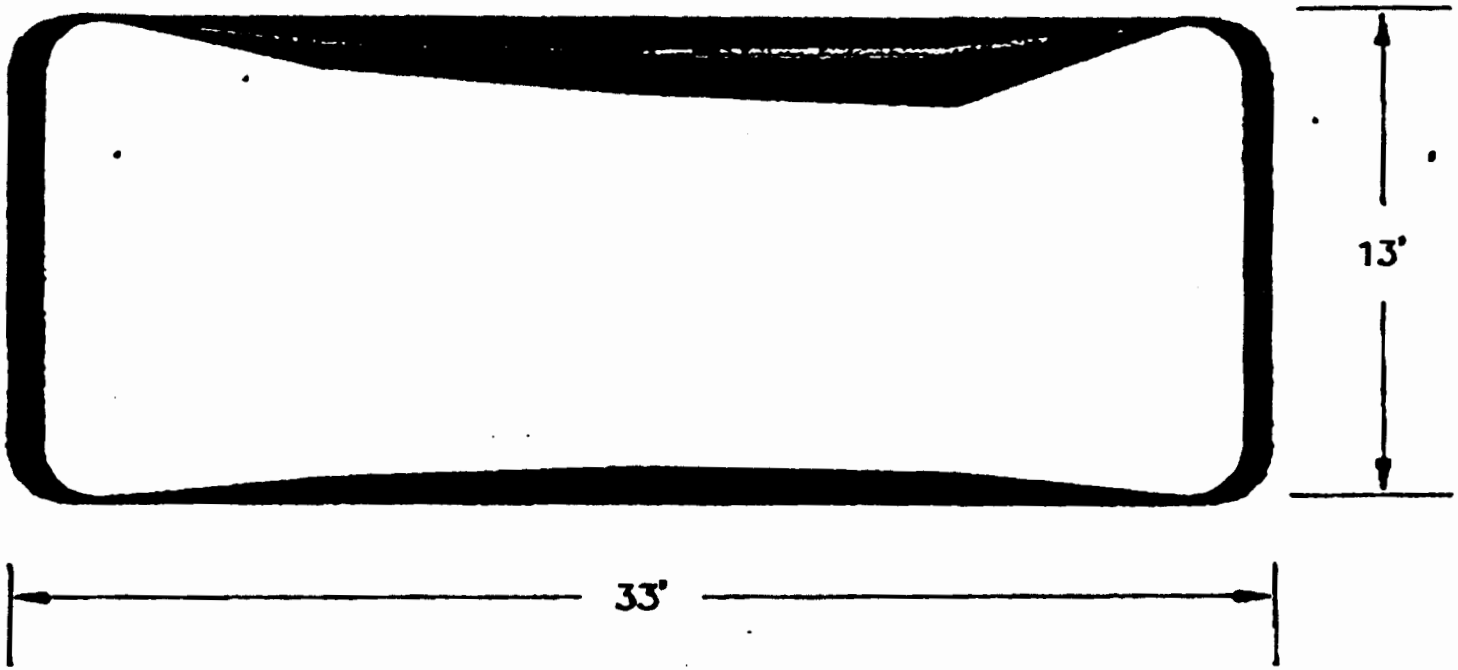


SECTION B-B

SPDV TEST ROOM 1 DEFORMATION

Section view looking north



YEAR 1 YEAR 5
YEAR 2
YEAR 3 YEAR 7
YEAR 4 YEAR 8

Deformations are shown at actual size.

Figure 5.3

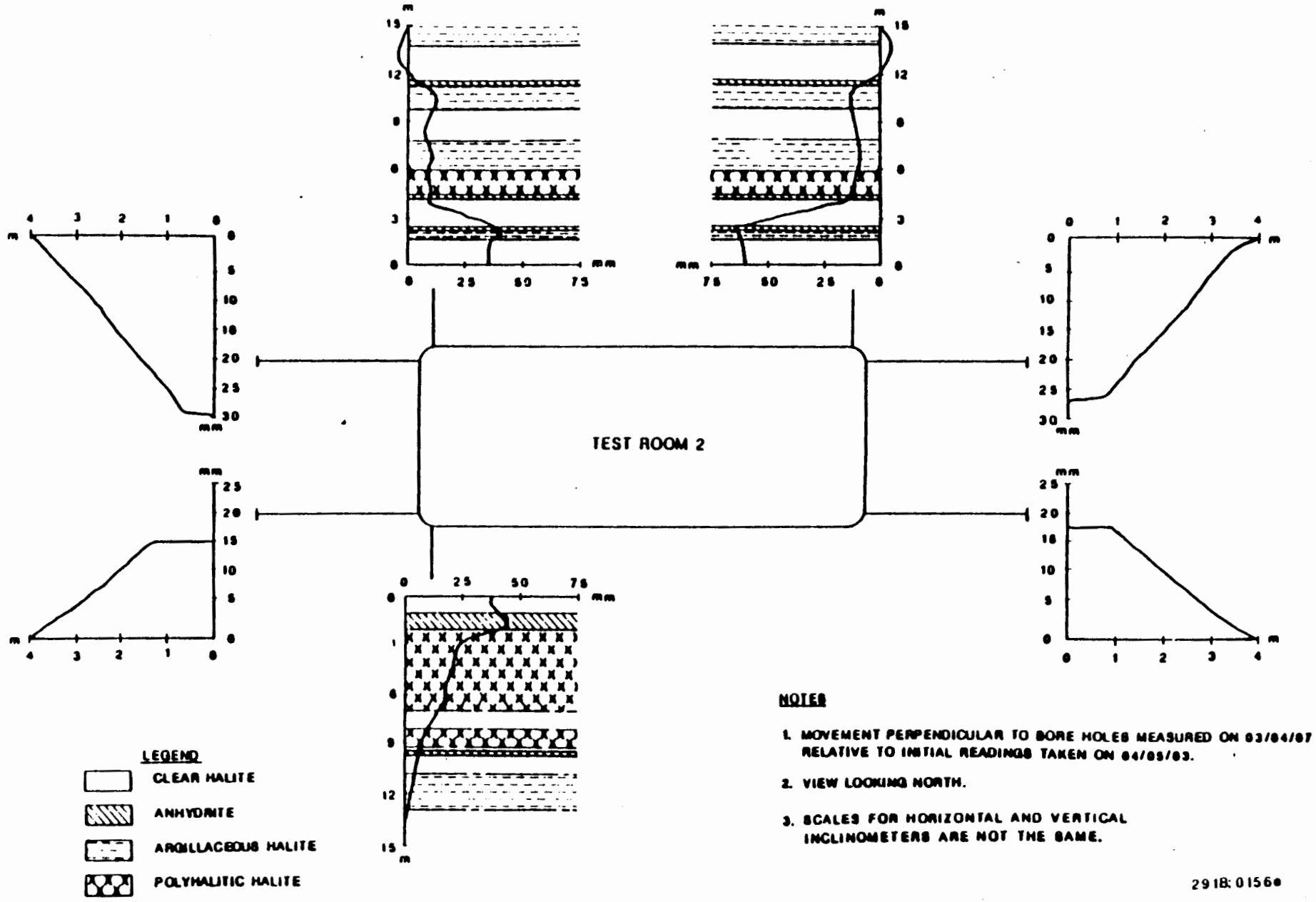


Figure 5.4 Inclinometer Data from SPDV Test Room 2

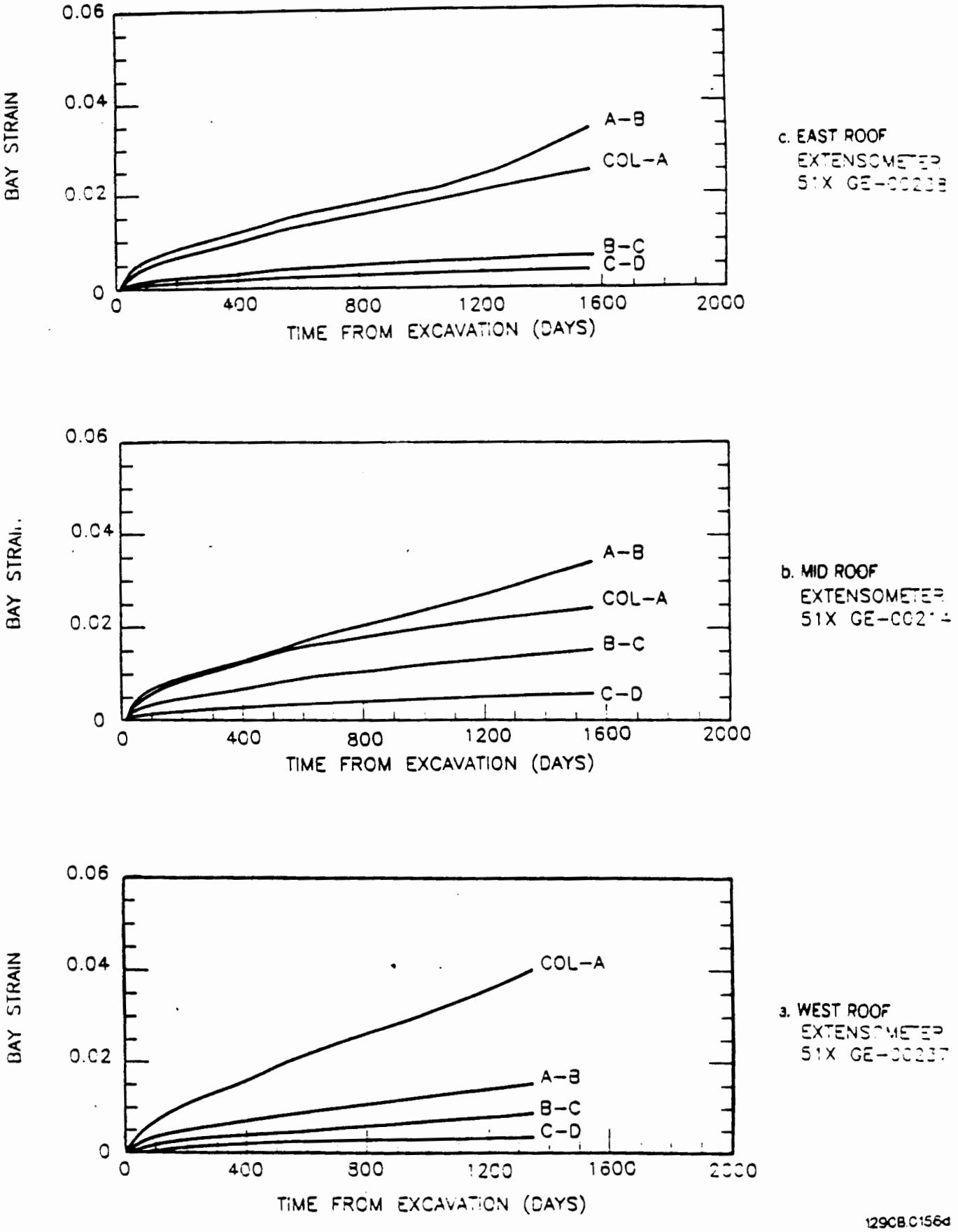


Figure 5.5 Extensometer Data from SPDV Test Rooms

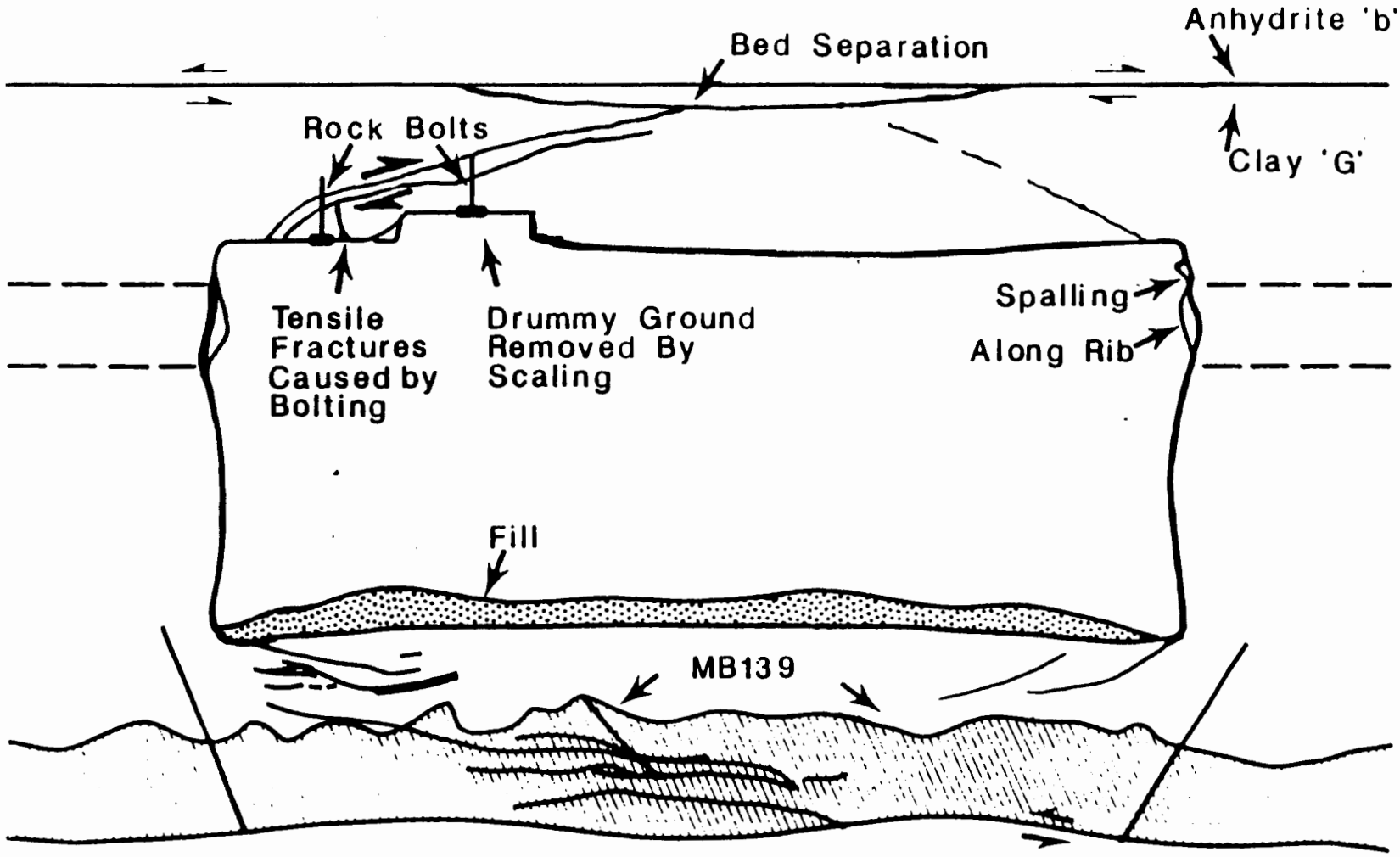


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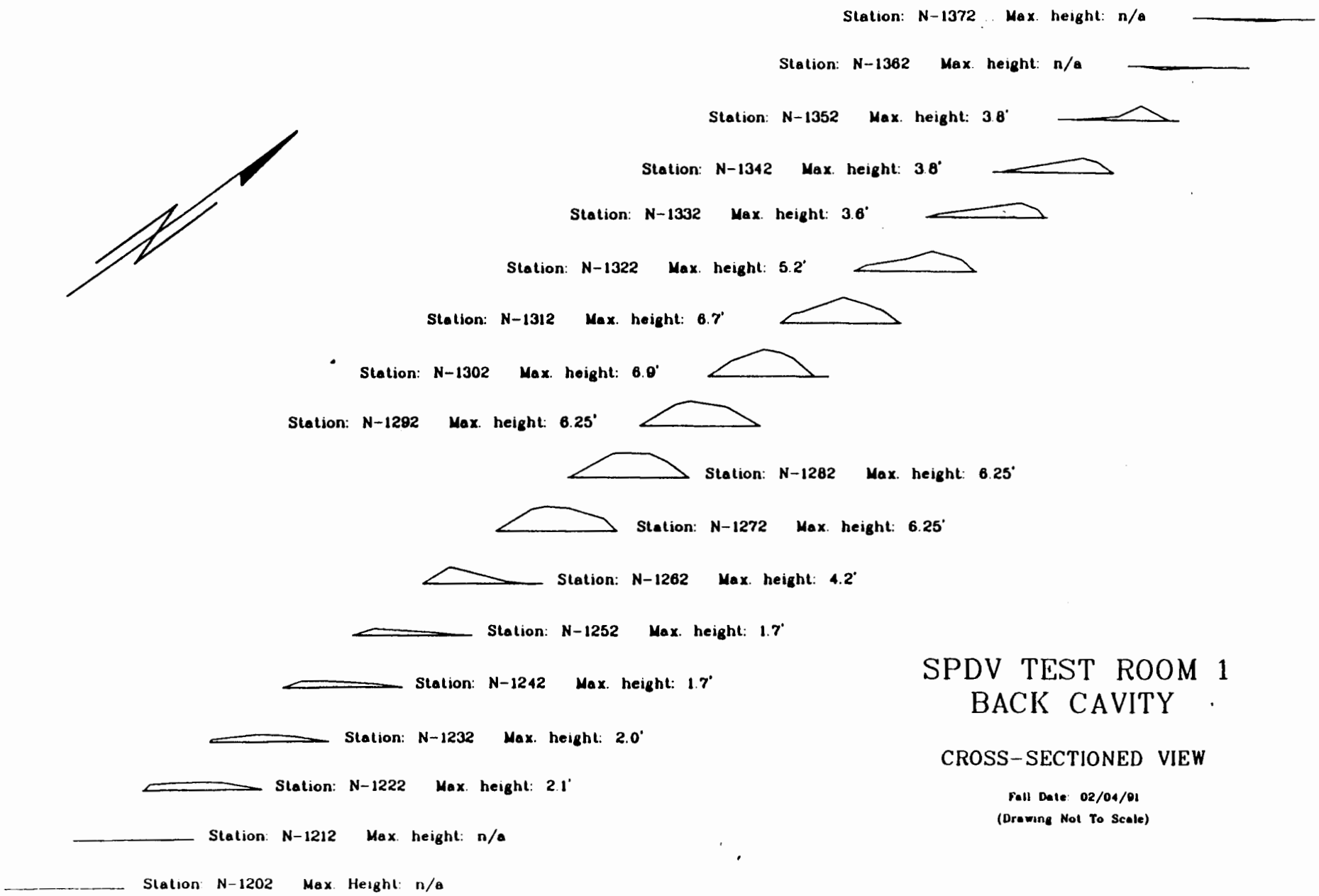
Figure 5.6 Patterns of Fracturing (Idealized excavation effects modified from Borns & Stormont, 1987)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Station: N-1362 Max. height: n/a
Station: N-1352 Max. height: 3.8'
Station: N-1342 Max. height: 3.8'
Station: N-1332 Max. height: 3.6'
Station: N-1322 Max. height: 5.2'
Station: N-1312 Max. height: 6.7'
Station: N-1302 Max. height: 6.9'
Station: N-1292 Max. height: 6.25'
Station: N-1282 Max. height: 6.25'
Station: N-1272 Max. height: 6.25'
Station: N-1262 Max. height: 4.2'
Station: N-1252 Max. height: 1.7'
Station: N-1242 Max. height: 1.7'
Station: N-1232 Max. height: 2.0'
Station: N-1222 Max. height: 2.1'
Station: N-1212 Max. height: n/a
Station: N-1202 Max. Height: n/a
SPDV TEST ROOM 1
BACK CAVITY
CROSS-SECTIONED VIEW
Fall Date: 02/04/91
(Drawing Not To Scale)

Figure 5.7

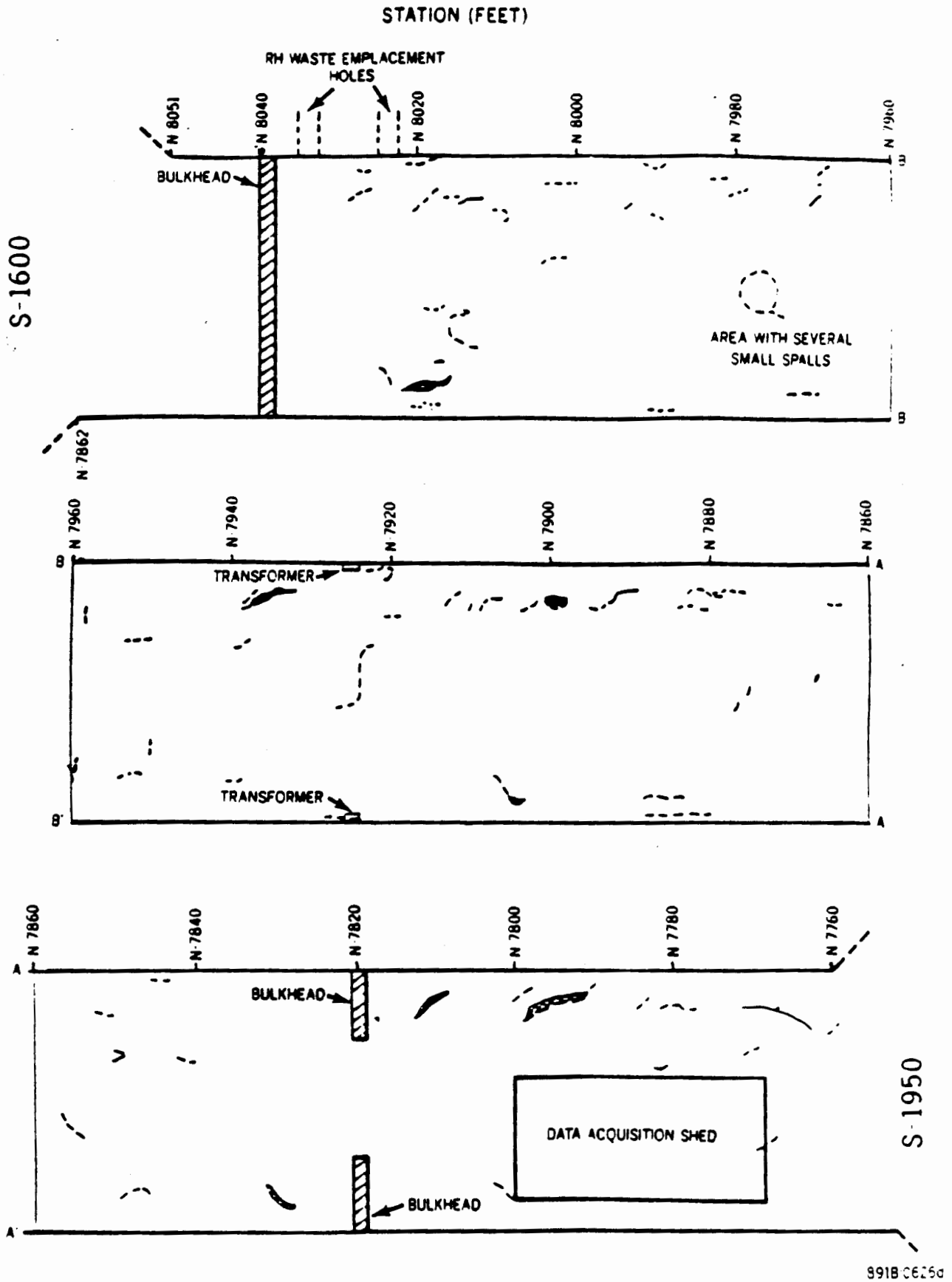
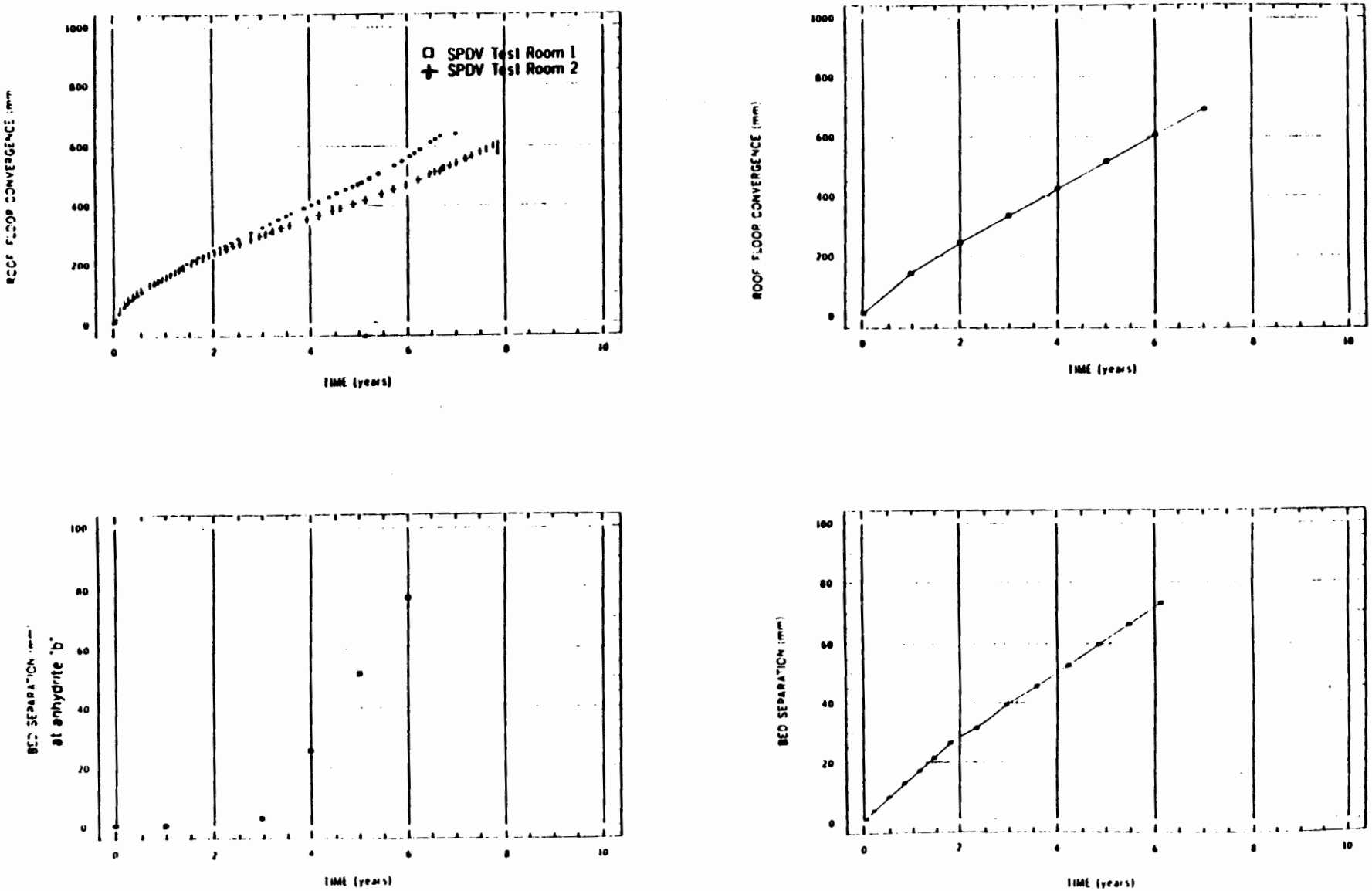


Figure 5.8 Back Fractures, Panel 1, Room 1 mapped on 2/12/91

Figure 5.9

FIELD MEASUREMENTS

MODEL RESULTS



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Figu. 5.10

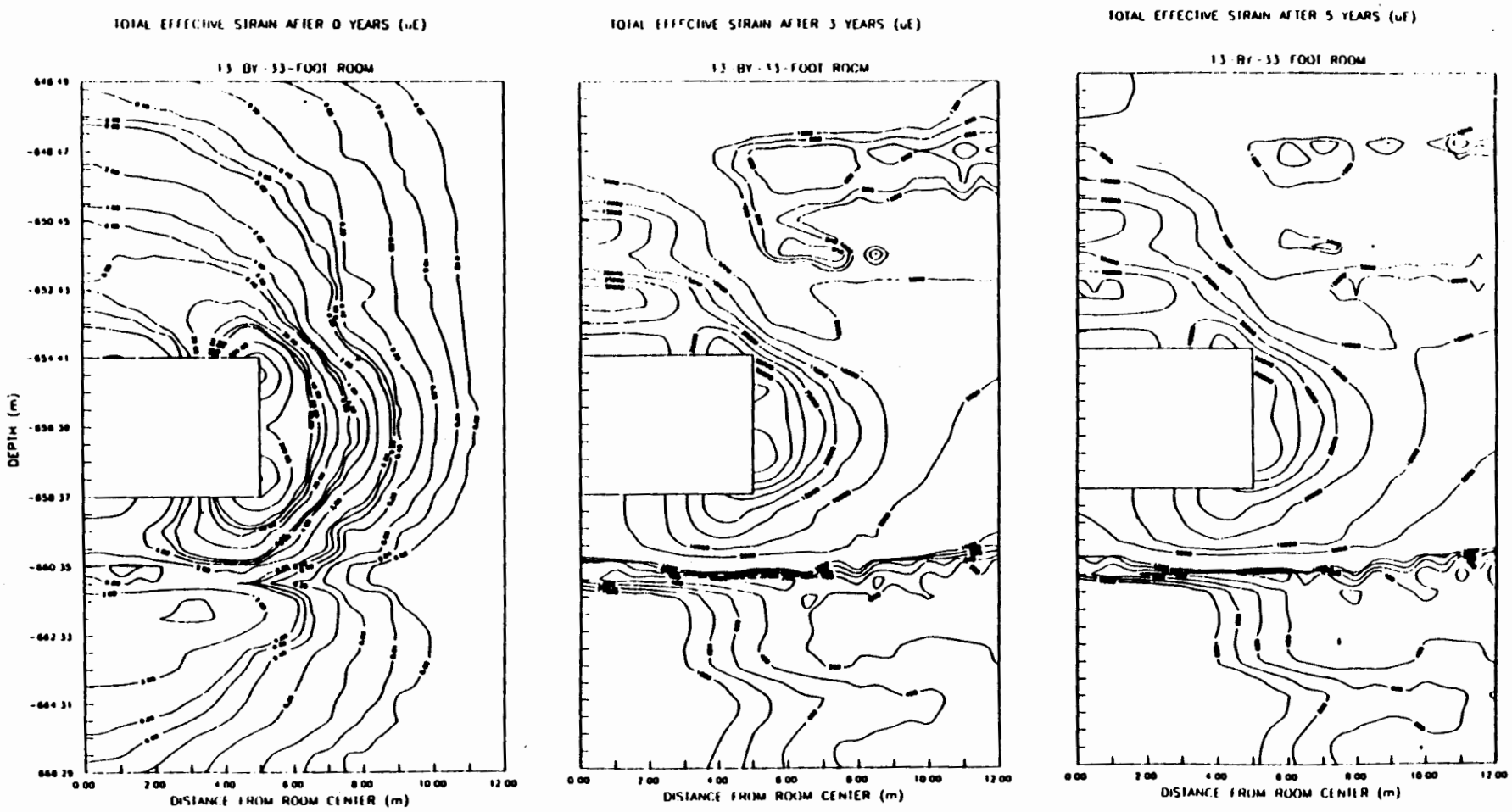

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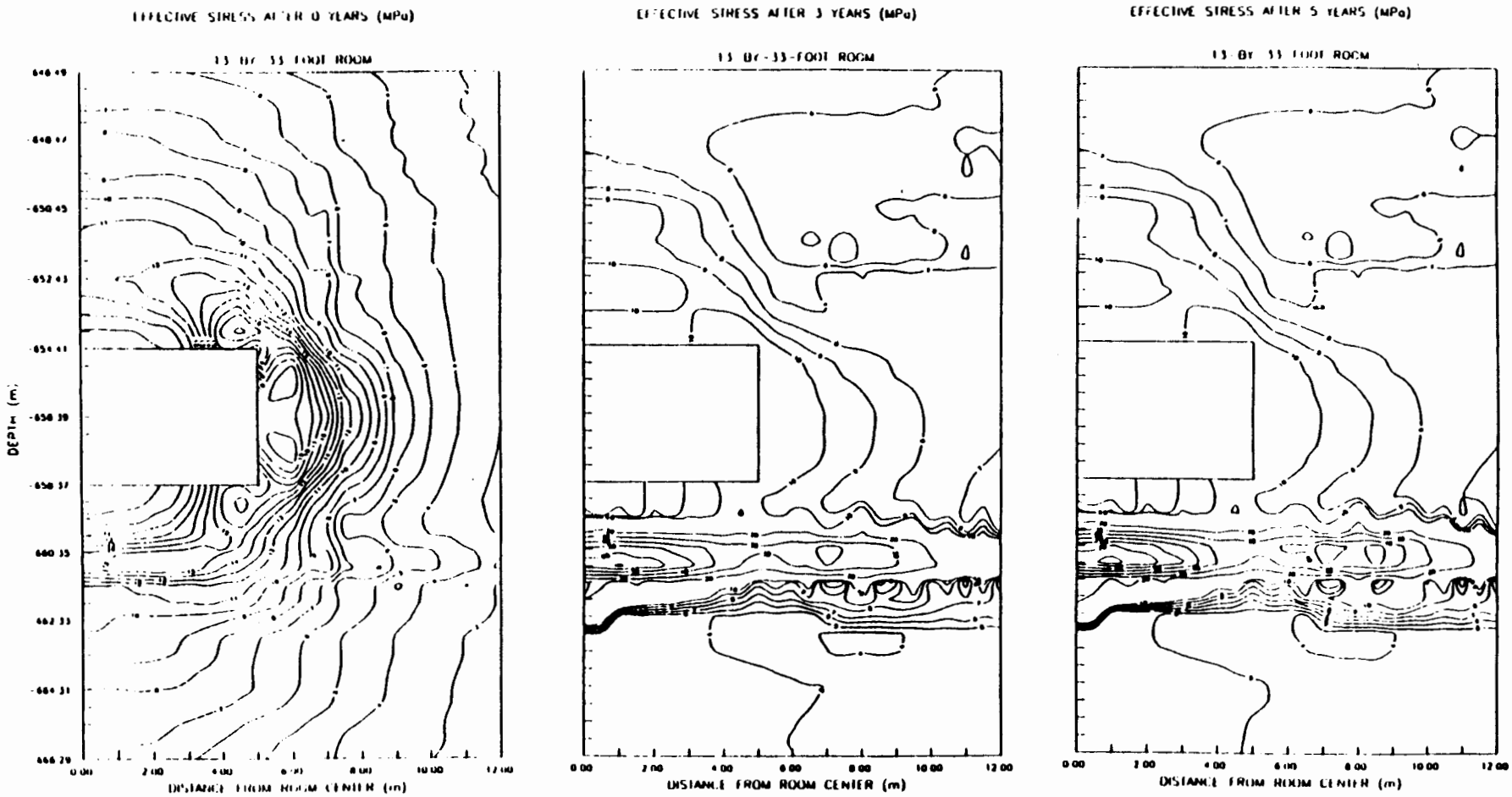
Figure



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APPENDIX B

WASTE ISOLATION PILOT PLANT
SUPPLEMENTARY ROOF SUPPORT SYSTEM

APPENDIX B

Destructive Tests

Westinghouse Electric Corporation
Waste Isolation Division
Carlsbad, New Mexico
August 1991

INTERNATIONAL MINING SERVICES INC.
RETYPED FAX RECEIVED FROM HAMISH MILLER

CONCLUSIONS FROM DESTRUCTIVE ROCKBOLT TESTS

The curves of load vs. extension were plotted for the 10 rockbolts that were tested to "failure". The points beyond which non-linear behavior occurred, were noted and these values are given in Table 1 below.

TABLE 1

TEST NO.	"YIELD" LOAD X 1,000 LB	BOLT STRESS AT "YIELD" PSI X 1,000
1	58.045	73.475
2	58.045	73.475
3	56.81	71.911
4	56.81	71.911
5	60.52	76.608
6	58.045	73.475
7	58.045	73.475
8	58.045	73.475
9	58.045	73.475
10	58.045	73.475

NOTE: "Yield" represents the point on the LOAD-EXTENSION curves where the curve departed from linearity. (see attached test results). The manufacturers of the Dywidag bolts give the "Yield Load" as 47.4 KIPS, which with a cross-sectional area of 0.79 sq. ins., gives a yield stress of 60,000 psi.

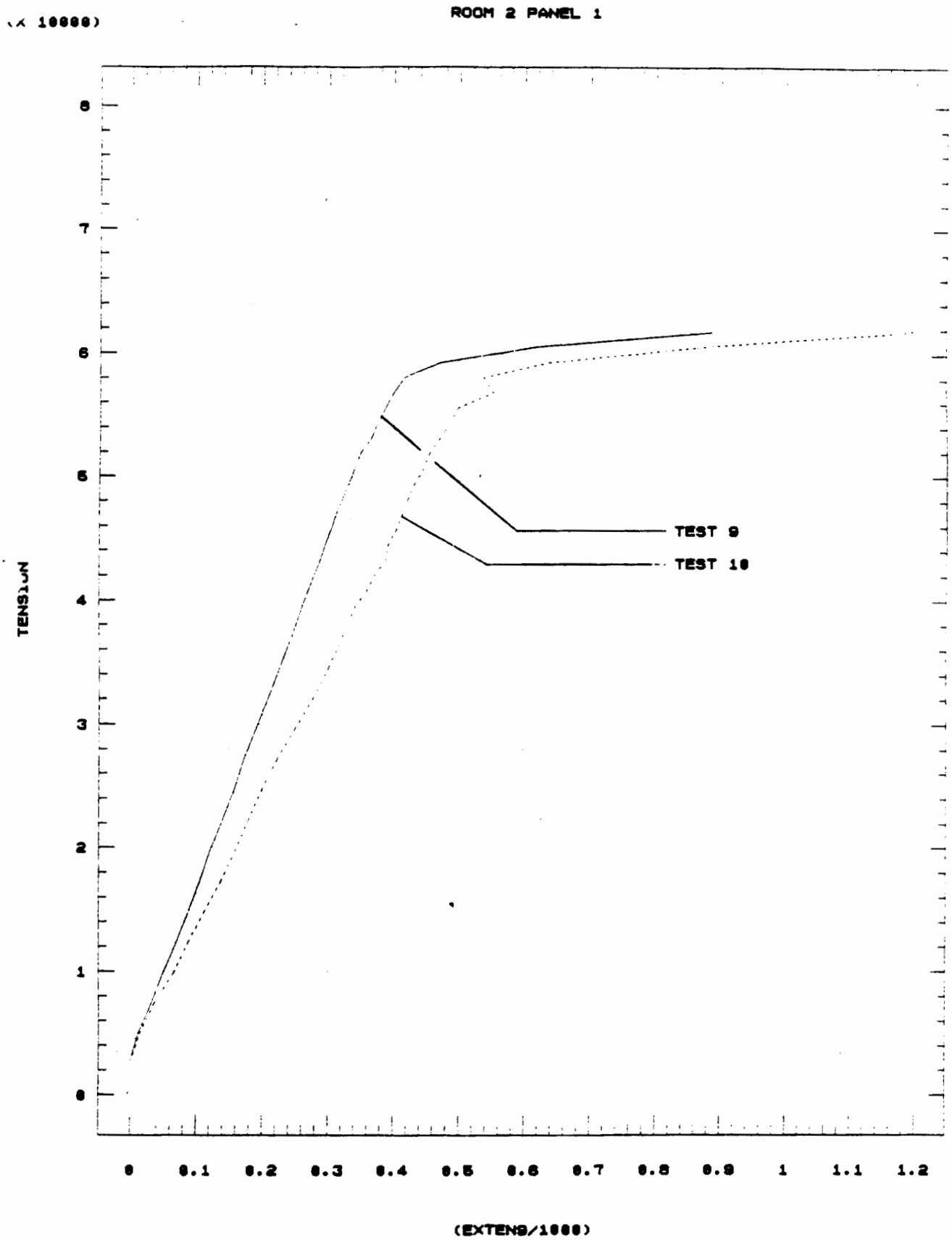
The results thus indicate that the mode of failure is that of bolt yield and not failure of the resin anchor bond.

There are slight indications of non-linearity in some of the test results, but it is felt that these are due to deformations of the plate and the "bedding-in" of the plate on the salt.

The test "yield" stresses are between 22% and 28% higher than the ASTM yield stress of 60,000 psi.

BOLT TENSION VERSUS EXTENSION

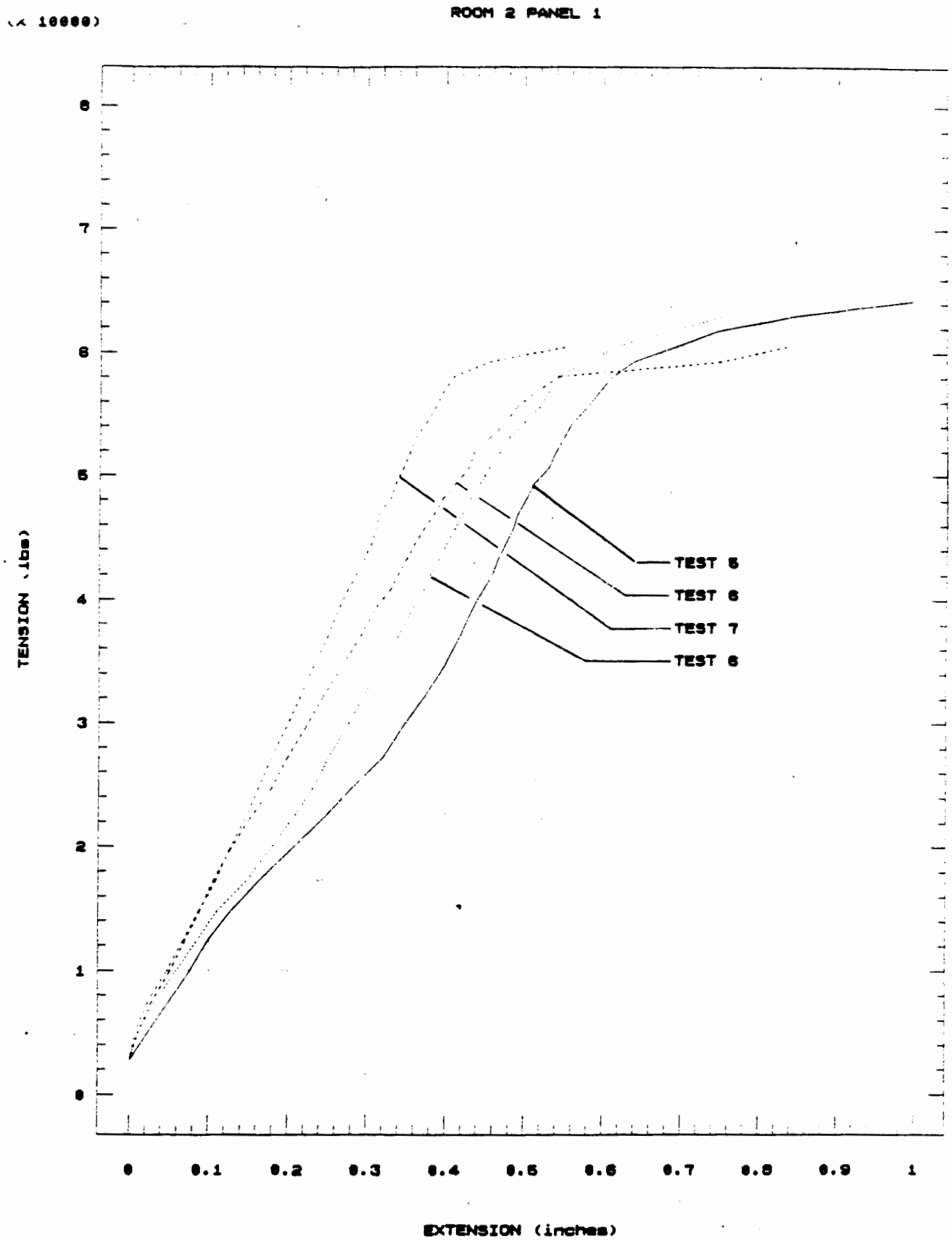
ROOM 2 PANEL 1



2 AND 3 DAYS CURE

BOLT TENSION VERSUS EXTENSION

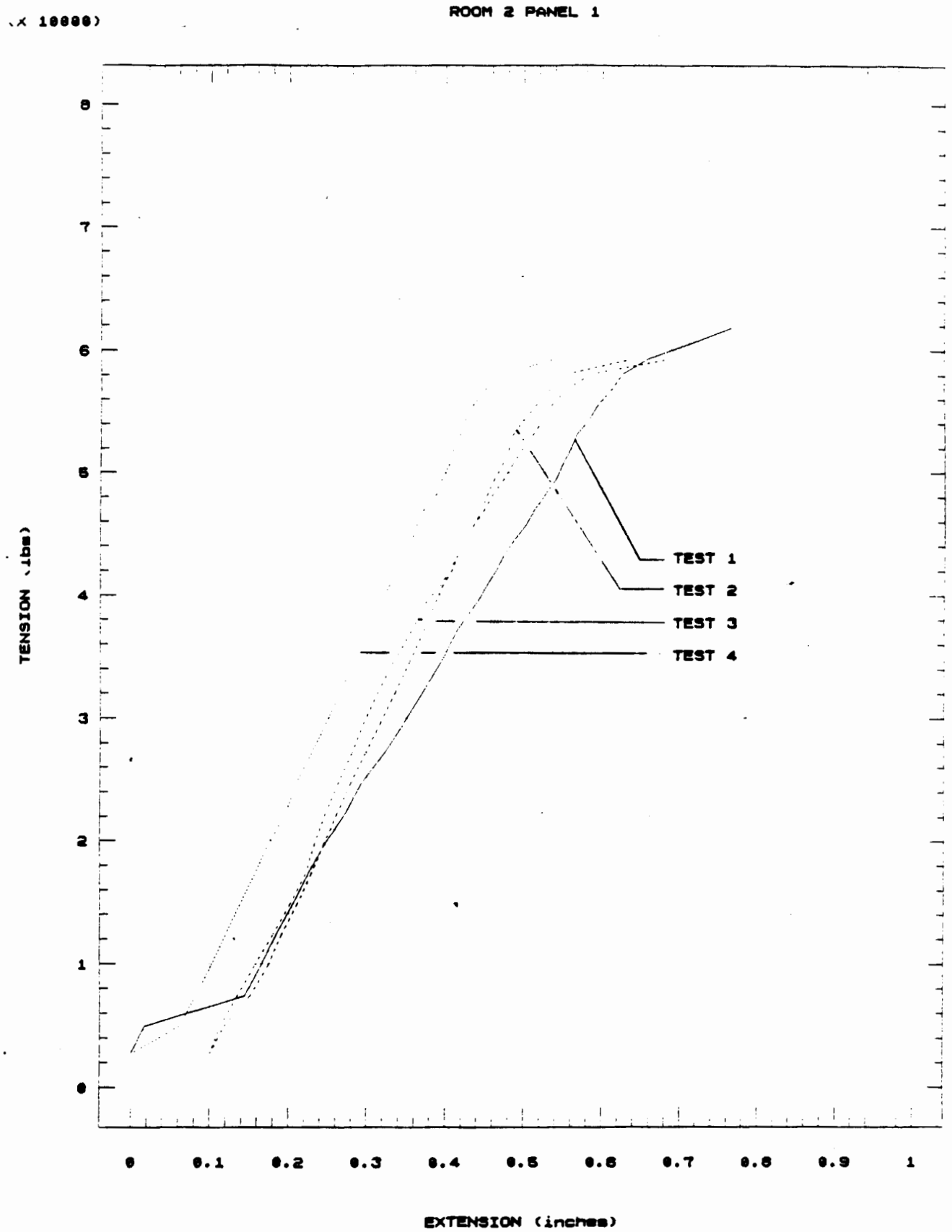
ROOM 2 PANEL 1



25 HOURS CURE

BOLT TENSION VERSUS EXTENSION

ROOM 2 PANEL 1



26 HOURS CURE

Test 10

ROCK BOLT CERTIFICATION TEST
DATA SHEET

Page 1 of 2

Mine WIPP Rock HALITE Rock Bolt No. PT081391-7
Hole Location PANEL 1-ROOM 2

Hole Length - 11'-7" Dia. - @ Anchor GAUGED /@ midpoint N/A /@ collar N/A
Orientation - VERT. (5° TOLERANCE) SMOOTH

Bolt Grade - GR60 Length - 13'-0" Diameter - #9

Shell Manufacturer - N/A Model - N/A Diameter - N/A
Length - N/A

Resin Manufacturer - CELTITE Model - H90.0204 Diameter - 1¼"
Length - 12" No. Cartridges - 3

Installation Date - 08/13/91 Time - 11:16 AM Torque - N/A ft/lb

Test Date - 08/16/91 Time - 1:05 PM Torque - N/A ft/lb
Test Jack - RC-60-CH95A. Test Pump - RC-60-CH95 SN 23886
Dial Indicator - AMES CD-0040 Calibrated - DUE 10-4-91
Pressure Gauge - U.S. GAGE DSI # 001 Calibrated - 08/06/91

~~Pump~~ PSI X 100 Pressure	~~Bolt~~ LBS Tension	Extensometer Reading	Displacement	Remarks
2	2470	0.000		
4	4940	0.015		
6	7410	0.036		
8	9880	0.066		
10	12350	0.088		
12	14820	0.112		
14	~~17290~~	0.138		17290 lbs
16	19760	0.157		
18	22,230	0.175		
20	24,700	0.195		
22	27,170	0.223		
24	29,640	0.250		
26	32,110	0.278		
28	34,580	0.301		
30	37,050	0.322		
32	39,520	0.341		
33	40,755	0.353		
34	41,990	0.370		
35	43,225	0.380		
36	44,460	0.391		
37	45,695	0.403		
38	46,930	0.412		
39	48,165	0.421		
40	49,400	0.431		
41	50,635	0.445		
42	51,870	0.454		
43	53,105	0.470		
44	54,340	0.485		
45	55,575	0.497		
46	56,810	0.511		Rig Load To 4800 PSI
47	58,045	0.536		REMOVE DIAL INDICATOR
48	XXXXX	0.636		ATTempt. 4800 Hold To 4600
49		0.862		ATT. 4900 Holding 4800 59,304
50		1.171		ATT 5000 Holding 4900 60,515
51		1.333		ATT 5100 Holding 5100 62,985
52		1.714		ATT 5200 Holding 5300 65,455
53			RDM 23/14	ATT 5600 Holding 5500 67,925

Test Results - ACCEPTED Maximum Pull Force - 69,160#
Displacement at Maximum Pull Force - APPROX. 2.65 in.
Nature of Failure/Yield - BOLT YIELD @ 4700 PSI
Other Remarks - 73.5 HOURS AFTER INSTALLATION
Tested By - D. Dallmann Approved By - P.R. Brechner

TEMPORARY TEST 1 of 10

Test 10

ROCK BOLT CERTIFICATION TEST
DATA SHEET

Page 2 of 2

Rock Bolt No. PT081391-7

Mine _____ Rock _____ Hole Location PANEL - ROOM 7

Hole Length - ____ Dia. - @ Anchor _____/@ midpoint _____/@ collar ____
Orientation - _____

Bolt Grade - _____ Length - _____ Diameter - _____

Shell Manufacturer - _____ Model - _____ Diameter - _____
Length - _____

Resin Manufacturer - _____ Model - _____ Diameter - _____
Length - _____ No. Cartridges - _____

Installation Date - _____ Time - _____ Torque - _____ ft/lb

Test Date - _____ Time - _____ Torque - _____ ft/lb
Test Jack - _____ Test Pump - _____
Dial Indicator - _____ Calibrated - _____
Pressure Gauge - _____ Calibrated - _____

Pump Pressure	Bolt Tension	Extensometer Reading	Displacement	Remarks
~~58~~			Ram 2 ft	AT 5800 Holding AT 5800 / 8100 P / 69,100 #
~~58~~				
~~56~~				
~~57~~				

Test ~~Results~~ _____ Maximum Pull Force - _____
Displacement at Maximum Pull Force -
Nature of Failure/Yield -
Other Remarks
Tested By - D. Bellmont Approved By - R.E. Brown

UNDERGROUND DRILLING HOLE FIELD DATA SHEET

Date: 8/13/91

Room/Drift: ROOM 2 - PANEL 1

Purpose of Hole: PULL TEST: DESTRUCTIVE (PWR 12017T)
P+081391-7

Location: N. END 4+/ W Rand 45-50 FT. FROM N. BULKHEAD

Depth at Completion: 11'-7"

Diameter: 1 3/8" ϕ

Approximate Collar Coordinates

Vertical Angle: VERT. (5° TOLERANCE)

Direction/Azimuth:

Remarks: RESIN/ROCK ANCHOR TEST AT 1:05 pm (
AND COMPLETED AT 2:00 pm FOR
CERTIFICATION.

Signature/Department: D. Ballinger 8/16/91

ROCK BOLT CERTIFICATION TEST
DATA SHEET

Mine WIPP Rock HALITE Rock Bolt No. PT081391-6
Hole Location PANEL 1 - ROOM 2

Hole Length - 11'-7" Dia. - @ Anchor GAUGED /@ midpoint N/A /@ collar N/A
Orientation - VERT. (5° TOLERANCE) SMOOTH

Bolt Grade - GR60 Length - 12'-11 7/8" Diameter - #8

Shell Manufacturer - N/A Model - N/A Diameter - N/A
Length - N/A

Resin Manufacturer - CELTITE Model - HRO-0204 DS/8/13/91 Diameter - 1 1/4"
Length - 12" No. Cartridges - 3

Installation Date - 08/13/91 Time - 11:04 am Torque - N/A ft/lb

Test Date - 8/15/91 Time - 12:50 PM Torque - N/A ft/lb
Test Jack - RC-60-CH 9.5A Test Pump - RC-60-CH 9.5 SN 23886
Dial Indicator - AMES CD-0040 Calibrated - DUE 10-4-91
Pressure Gauge - U.S. GAUGE DSI # 001 Calibrated - 08/06/91

PSI x 100 ~~Pump~~ Pressure	Bolt LBS Tension	Extensometer Reading	Displacement	Remarks
2	2470	0.000		
4	4940	0.012		
6	7410	0.032		
8	9880	0.050		
10	12,350	0.070		
12	14,820	0.088		
14	17,290	0.105		
16	19,760	0.121		
18	22,230	0.140		
20	24,700	0.158		
22	27,170	0.172		
24	29,640	0.190		
26	32,110	0.207		
28	34,580	0.226		
30	37,050	0.244		
32	39,520	0.260		
33	40,155	0.269		
34	41,790	0.278		
35	43,235	0.288		
36	44,460	0.298		
37	45,695	0.309		
38	46,930	0.317		
39	48,165	0.321		
40	49,400	0.331		
41	50,635	0.340		
42	51,870	0.347		
43	53,105	0.365		
44	54,340	0.373		
45	55,575	0.387		
46	56,810	0.399		Block to 4600 PSI
47	58,045	0.416		~~REMOVE DIAL INDICATOR~~ DS/8/15/91
~~48~~	59,280	0.470		Block to 4700 PSI
49	60,515	0.616		Approx 5 min. Block to 4800 PSI
50		0.886 *		Remove Dial Indicator *
51				Bolt Yield @ 24800
52				
53				Terminate Test @ 1:20 PM

Test Results - ACCEPTED Maximum Pull Force ~~59,280#~~ 64,220#
Displacement at Maximum Pull Force - Approx. .886 in
Nature of Failure/Yield - Bolt Yield @ 24800 PSI DS/8.16.91
Other Remarks - 49.8 HRS AFTER INSTALLATION. HELD AT
Tested By - D. Hallmark Approved By - P.E. Bruce Simons

UNDERGROUND DRILLING HOLE FIELD DATA SHEET

Date: 8/13/91

Room/Drift: PANEL 1 - ROOM 2

Purpose of Hole: PULL TEST: DESTRUCTIVE (PWR12017T)
DT081391-6

Location: N END 4+' W ROOM & 36 - 40 FT. FROM N. BULKHEAD

Depth at Completion: 11'-7"

Diameter: 1 3/8" ϕ COMPLETION

Approximate Collar Coordinates

Vertical Angle:

Direction/Azimuth: VERT. (5° TOLERANCE)

Remarks: RESIN / ROCK ANCHOR TEST AT 12:50 PM
AND COMPLETED AT 2:45 PM FOR
CERTIFICATION.

Signature/Department: D. Galbraith 8/15/91

ROCK BOLT CERTIFICATION TEST
DATA SHEET

Mine _____ Rock _____ Rock Bolt No. PT081391-6 Hole Location _____

Hole Length - ____ Dia. - @ Anchor _____/@ midpoint_____/@ collar ____
Orientation - _____

Bolt Grade - _____ Length - _____ Diameter - _____

Shell Manufacturer - _____ Model - _____ Diameter - _____
Length - _____

Resin Manufacturer - _____ Model - _____ Diameter - _____
Length - _____ No. Cartridges - _____

Installation Date - _____ Time - _____ Torque - _____ ft/lb

Test Date - _____ Time - _____ Torque - _____ ft/lb
Test Jack - _____ Test Pump - _____
Dial Indicator - _____ Calibrated - _____
Pressure Gauge - _____ Calibrated - _____

Pump Pressure	Bolt Tension	Extensometer Reading	Displacement	Remarks
~~54~~				COMMENTS — 1:30 PM
~~55~~			ATT 4900	Holding AT 4900 PSI 1 1/2" RAM 59280# 1:35 PM
~~56~~			ATT 5000	Holding AT 4700 PSI 1 5/8" RAM 60315# 1:40
~~57~~			ATT 5100	Holding AT 5000 PSI 1 3/4" RAM 61350# 1:46 PM
			ATT 5200	Holding AT 5000 PSI 1 7/8" RAM 61760# 2:00 PM
			ATTAIN 5000	Holding AT 5100 PSI 1 7/8" RAM 62985 2:07 PM
			ATT. 5300	Holding AT 5200 PSI 2" RAM 64[illegible] 2:13 PM
			ATT. 5500	Holding AT 5300 2 1/8" RAM 65455# 2:25 PM
			ATT 5600	Holding AT 5400 PSI 2 1/4" RAM 66690# 2:33
			ATT 5700	Holding AT 5500 PSI 2 3/16" RAM 67925# 2:40
			ATT 5800	Holding AT 5600 PSI 2 1/2" RAM 69160# 2:45
			ATT 5900	Holding AT 5700 PSI 2 5/8" RAM 70395
			08/16/91	BLED TO 5200 PSI 12:40 PM 64230# MAX PULL FORCE

Test Results - _____ Maximum Pull Force - _____
Displacement at Maximum Pull Force - _____
Nature of Failure/Yield - _____
Other Remarks - _____
Tested By - D. Pallmaier Approved By - [illegible]

ROCK BOLT CERTIFICATION TEST
DATA SHEET

Mine WIPP Rock HALITE Rock Bolt No. PT081391-1
Hole Location PANEL - ROOM 2

Hole Length - 11.50" Dia. - @ Anchor GAUGED /@ midpoint N/A /@ collar N/A
Orientation - VERTICAL (5° TOLERANCE) SMOOTH

Bolt Grade - GR60 Length - 13'-0" Diameter - #8

Shell Manufacturer - N/A Model - N/A Diameter - N/A
Length - N/A

Resin Manufacturer - CELTITE Model - H90 - 0204 Diameter - 1 1/4"
Length - 12" No. Cartridges - 3

Installation Date - 08/13/91 Time - 10:16 Torque - N/A ft/lb

Test Date - 8/14/91 Time - 11:30 Torque - N/A ft/lb
Test Jack - RC-60-CH 95A Test Pump - RC-60-H95 SN# 23886
Dial Indicator - AMES 4822 CD-0040 Calibrated - DUE 10-14-91
Pressure Gauge - US GAGE PSI #001 Calibrated - 08-06-91

Pump Pressure (PSI)	Bolt Tension (LBS)	Extensometer Reading	Displacement	Remarks
200	2470	0.000		
400	4940	0.062		
600	7410	0.082		
800	9880	0.102		
1000	12350	0.120		
1200	14820	0.138		
1400	17290	0.157		
1600	19760	0.174		
1800	22230	0.194		
2000	24700	0.210		
2200	27170	0.229		
2400	29640	0.248		
2600	32110	0.266		
2800	34580	0.282		
3000	37050	0.300		
3200	39,520	0.319		
3300	40755	0.328		
3400	41990	0.338		
3500	43225	0.347		
3600	44460	0.356		
3700	45695	0.363		
3800	46930	0.372		
3900	48165	0.388		HOLD 1 MIN - BLEED SLIGHTLY TO 3800
4000	49400	0.349 .394 DB 8.14.91		
4100	50635	0.404		
4200	51870	0.409		
4300	53105	0.418		
4400	54340	0.425		
4500	55575	0.435		
4600	56810	0.451		HELD - BLED BACK TO 4600 B.J.
4700	58045	0.472		REMOVED DIAL INDICATOR DB 8/14/91
4800	59280	0.539		ATTEMPTED 4900 BLED TO 4700
4900				REMOVED DIAL INDICATOR
5000				HELD FOR 5 MIN. BLED TO 4900
5100				PUMPED 4800 PSI HELD AT 4800
5200				WOULD NOT GO TO 4900
5300				TEST TERMINATED @ 12:05 PM

39,520#

Test Results - ACCEPTED Maximum Pull Force - 59280 #
Displacement at Maximum Pull Force - .539 in.
Nature of Failure/Yield - BOLT YIELD 4600 PSI to 4700 PSI
Other Remarks - @ 25.3 HRS AFTER INSTALLATION
Tested By - D. Ballinger Approved By - R.E. Bunn

UNDERGROUND DRILLING HOLE FIELD DATA SHEET

Date: 8/13/91

Room/Drift: PANEL 1 - ROOM 2

Purpose of Hole: PULL TEST: DESTRUCTIVE (PWR 120177)
PT081391-1

Location: N. END APPROX 4'+ W/E ROOM ℄ APPROX. 4-6 FT FROM BULKHEAD
DX/S.1541

Depth at Completion: 11'-7"

Diameter: 1-3/8" ϕ COMPLETION

Approximate Collar Coordinates

Vertical Angle: VERT. (5° TOLERANCE)

Direction/Azimuth:

Remarks: RESIN/ROCK ANCHOR TESTED AT 11:30 am COMPLETED AT 12:05 PM
FOR CERTIFICATION.

Signature/Department: D. Ballmann 8/14/91

ROCK BOLT CERTIFICATION TEST
DATA SHEET

Mine WIPP Rock HALITE Rock Bolt No. PT081591-2
Hole Location PANEL 1 - ROOM 2

Hole Length - 11.58' Dia. - @ Anchor GAUGED /@ midpoint N/A /@ collar N/A
Orientation - VERTICAL (5° TOLERANCE) SMOOTH

Bolt Grade - GR60 Length - 13'-0" Diameter - #8

Shell Manufacturer - N/A Model - N/A Diameter - N/A
Length - N/A

Resin Manufacturer - CELTITE Model - H90-0204 Diameter - 1 1/4"
Length - 12" No. Cartridges - 3

Installation Date - 08/13/91 Time - 10:31 AM Torque - N/A ft/lb

Test Date - 8/14/91 Time - 1:37 PM Torque - N/A
Test Jack - RC-60-CH 9.5A Test Pump - RC-60-CH 9.5 SN 28886 2398
Dial Indicator - AMES 4822 CD-0040 Calibrated - DUE 10.4.91
Pressure Gauge - USG 486 PSI #001 Calibrated - 08-06-91

Pump Pressure (PSI x100)	Bolt Tension (LBS)	Extensometer Reading	Displacement	Remarks
2	2470	0.000		
4	4940	0.023		
6	7410	0.049		
8	9880	0.073		
10	12350	0.098		
12	14820	0.128		
14	17290	0.164		
16	19760	0.204		
18	22230	0.244		
20	24700	0.281		
22	29640	0.320		
24	29640	0.345		
26	32110	0.373		
28	34580	0.398		
30	37050	0.418		
32	39,520	0.435		
33	40755	0.442		
34	41990	0.458		
35	43225	0.466		
36	44460	0.475		
37	45695	0.485		
38	46930	0.491		
39	48165	0.502		
40	49400	0.512		
41	50635	0.520		
42	51870	0.535		
43	53105	0.550		
44	54340	0.560		
45	55575	0.575		
46	56810	0.595		
47	58045	0.611		REMOVE DIAL INDICATOR
48	59280	0.638		BLEED TO 4700 PSI
49		0.652		BLEED TO 4700 PSI 2 min
~~50~~	08/13/91	0.746		
51		0.846		
52		0.996		BLEED TO 4800 PSI
53				TERMINATED TEST 1:55 PM

Test Results - ACCEPTED Maximum Pull Force - 59280 #
Displacement at Maximum Pull Force - 0.996 in.
Nature of Failure/Yield - BOLT YIELD AT 4800 PSI
Other Remarks - ~~22#~~ 27.1 HCS AFTER INSTALLATION
Tested By - D. Ballmann Approved By - [illegible]

UNDERGROUND DRILLING HOLE FIELD DATA SHEET

Date: 8/13/91

Room/Drift: PANEL 1 - ROOM 2

Purpose of Hole: PULL TEST: DESTRUCTIVE (PWR 12017T)
PT081391-2

Location: N. END APPROX 4" W OF ROOM ℄ APPROX 12 FEET FROM N. BULKHEAD

Depth at Completion: 11'-7"

Diameter: 1 3/8" ϕ COMPLETION

Approximate Collar Coordinates

Vertical Angle: VERT. (5° TOLERANCE)

Direction/Azimuth:

Remarks: RESIN/ROCK ANCHOR TESTED AT 1:37 PM
AND COMPLETED AT 1:55 PM FOR
CERTIFICATION

Signature/Department: D Ballweiler 8/14/91

ROCK BOLT CERTIFICATION TEST
DATA SHEET

Mine WIPP Rock HALITE Rock Bolt No. PT081391-3
Hole Location PANEL 1 - ROOM 2

Hole Length - 11'-7" Dia. - @ Anchor GAUGED /@ midpoint N/A /@ collar N/A
Orientation - VERTICAL (5° TOLERANCE) SMOOTH

Bolt Grade - GR 60 Length - 13'-0" Diameter - #8

Shell Manufacturer - N/A Model - N/A Diameter - N/A
Length - N/A

Resin Manufacturer - CELTITE Model - H90-020L Diameter - 1¼"
Length - 12" No. Cartridges - 3

Installation Date - 08/13/91 Time - 10:41 Torque - N/A ft/lb

Test Date - 08/13/91 Time - 2:00 PM Torque - N/A ft/lb
Test Jack - RC-60-CH 9.5A Test Pump - RC-60 CH95 SN 23836
Dial Indicator - AMES CD-0040 Calibrated - DUE 10-4-91
Pressure Gauge - US GAGE DSI ±.001 Calibrated - 08-06-91

DSI x100 ~~Pump~~ Pressure	LBS ~~Bolt~~ Tension	Extensometer Reading	Displacement	Remarks
2	2470	0.000		
4	4940	0.011		
6	7410	0.028		
8	9880	0.050		
10	12350	0.069		
12	14820	0.088		
14	17290	0.106		
16	19760	0.127		
18	22230	0.151		
20	24700	0.173		
22	27170	0.199		
24	29640	0.222		
26	32110	0.244		
28	34580	0.268		
30	37050	0.292		
32	39520	0.313		
33	40755	0.329		
34	41990	0.345		
35	43225	0.358		
36	44460	0.360		
37	45695	0.372		
38	46930	0.385		
39	48166	0.396		Bled TO 3800 PSI
40	49400	0.413		
41	50635	0.429		
42	51870	0.433		
43	53105	0.456		
44	54340	0.475		
45	55575	0.499		
46	56810	0.519		
47	58046	0.544		Bled TO 4600 PSI ~~Remove Dial Indicator~~
48		0.749		Bled TO 4600 PSI
49		0.834		Accepted 4900 Bled TO 4700 PSI
50				
51				
52				
53				TERMINATE TEST 2:15 PM

Test Results - ACCEPTED Maximum Pull Force - 58045#
Displacement at Maximum Pull Force - 0.834 in.
Nature of Failure/Yield - Bar Yield 4700 PSIg
Other Remarks - 27.3 HRS AFTER INSTALLATION
Tested By - D. Ballrauch Approved By - [illegible]

UNDERGROUND DRILLING HOLE FIELD DATA SHEET

Date: 8/13/91

Room/Drift: PANEL 1 - ROOM 2

Purpose of Hole: PULL TEST: DESTRUCTIVE (PWR 120177)
PT081391-3

Location: N. END APPROX 4+' W ROOM ℄ APPROX. 15 FT FROM BULKHEAD

Depth at Completion: 11' - 7"

Diameter: 1-3/8" ⌀ COMPLETION

Approximate Collar Coordinates

Vertical Angle: VERT. (5° TOLERANCE)

Direction/Azimuth:

Remarks: RESIN/ROCK ANCHOR TEST AT 2:00 pm AND COMPLETED AT 2:15 pm
FOR CERTIFICATION

Signature/Department: D. Dallmann 8/14/91

ROCK BOLT CERTIFICATION TEST
DATA SHEET

Mine WIPP Rock HALITE Rock Bolt No. PT081391-4 Hole Location PANEL 1 - ROOM 2

Hole Length - 11'-7" Dia. - @ Anchor GAUGED /@ midpoint N/A /@ collar N/A
Orientation - VERT. (5° TOLERANCE) SMOOTH

Bolt Grade - GR 60 Length - 13'-0" Diameter - #8

Shell Manufacturer - N/A Model - N/A Diameter - N/A
Length - N/A

Resin Manufacturer - CELLITE Model - H90-0204 Diameter - 1 1/4"
Length - _____ No. Cartridges - _____

Installation Date - 08/13/91 Time - 10:51 am Torque - N/A ft/lb

Test Date - 08/14/91 Time - 2:25 PM Torque - N/A 08/13/91 ft/lb
Test Jack - RC-60-CH 9.5A Test Pump - RC-60-9.5 SN ~~23886~~ 23886
Dial Indicator - ANTES CD-0040 Calibrated - DUE 10-4-91
Pressure Gauge - LG GAGE DSI #001 Calibrated - 08/06/91 08/13/91

PSI X100 Pump Pressure	LBS Bolt Tension	Extensometer Reading	Displacement	Remarks
2	2470	0.000		
4	4940	0.007		
6	7410	0.025		
8	9880	0.046		
10	12350	0.067		
12	14820	0.087		
14	17290	0.109		
16	19760	0.126		
18	22230	0.146		
20	24700	0.162		
22	27170	0.179		
24	29640	0.197		
26	32110	0.215		
28	34580	0.232		
30	37050	0.250		
32	39520	0.266		
33	40755	0.275		
34	41990	0.286		
35	43225	0.295		
36	44460	0.305		
37	45695	0.311		
38	46930	0.315		
39	48165	0.330		
40	49400	0.336		
41	50635	0.346		
42	51870	0.354		
43	53105	0.362		
44	54340	0.375		
45	55575	0.386		
46	56810	0.396		Bleed TO 4600 PSI
47	58045	0.410		~~REMOVE DIAL INDICATOR~~
48		0.456		Bleed TO 4700 PSI
49		0.556		
50				
51				
52				
53				TERMINATE TEST 2:35

Test Results - ACCEPTED Maximum Pull Force - 58045 #
Displacement at Maximum Pull Force - 0.556
Nature of Failure/Yield - BOLT YIELD @ 4700 PSIG
Other Remarks N/A
Tested By - D. Pattison Approved By - [signature]

UNDERGROUND DRILLING HOLE FIELD DATA SHEET

Date: 8/13/91

Room/Drift: PANEL 1 - ROOM 2

Purpose of Hole: PULL TEST : DESTRUCTIVE (PWR 120MT)
PT 081391-4

Location: N. END 4th W Room & APPROX. 28-30 FT FROM W. BULKHEAD

Depth at Completion: 11'-7"

Diameter: 1 3/8" ϕ COMPLETION

Approximate Collar Coordinates

Vertical Angle: VERT. (5° TOLERANCE)

Direction/Azimuth:

Remarks: RESIN/ROCK ANCHOR TEST AT 2:25 PM
AND COMPLETED AT 2:35 PM FOR CERTIFICATION.

Signature/Department: D. Balhiane 8/14/91

ROCK BOLT CERTIFICATION TEST
DATA SHEET

Mine WIPP Rock NALITE Rock Bolt No. PT081391-5 Hole Location PANEL 1-ROOM 2

Hole Length - 11'-7" Dia. - @ Anchor GAUGED /@ midpoint N/A /@ collar N/A
Orientation - VERT. (5° TOLERANCE) SMOOTH

Bolt Grade - GR60 Length - 13'-0" Diameter - #8

Shell Manufacturer - N/A Model - N/A Diameter - N/A
Length - N/A

Resin Manufacturer - CELTITE Model - H90 .020d Diameter - 1¼"
Length - 12" No. Cartridges - 3

Installation Date - 08/13/91 Time - 10:54 am Torque - N/A ft/lb

Test Date - 08/14/91 Time - 2:38 pm Torque - N/A ft/lb
Test Jack - RC-60-CH 9.5A Test Pump - RC-60-CH 9.5 SN 23886
Dial Indicator - AMES CD-0040 Calibrated - DUE 10-4-91
Pressure Gauge - US GAGE DS1 #001 Calibrated - 08/06/91

PSI x 100 Pump Pressure	LBS Bolt Tension	Extensometer Reading	Displacement	Remarks
2	2470	0.000		
4	4940	0.010		
6	7410	0.038		
8	9880	0.058		
10	12350	0.084		
12	14820	0.111		
14	17290	0.148		
16	19760	0.177		
18	22230	0.206		
20	24700	0.231		
22	27170	0.251		
24	29640	0.274		
26	32110	0.255		
28	34580	0.316		
30	37050	0.340		
32	39520	0.360		
33	40755	0.372		
34	41990	0.380		
35	43225	0.391		
36	44460	0.400		
37	45695	0.411		
38	46930	0.421		
39	48165	0.432		
40	49400	0.443		
41	50635	0.455		
42	51870	0.468		
43	53105	0.480		
44	54340	0.492		
45	55575	0.518		
46	56810	0.530		
47	58045	0.542		
48	59280	0.580		
49	60515	0.615		Bled TO 4800 PSI
~~50~~ 8.14.91		0.685		Bled TO 4800 PSI
51		0.755		
52				
53				TERMINATE TEST 2:55 PM

Test Results - ACCEPTED Maximum Pull Force - 60515#
Displacement at Maximum Pull Force - 0.755
Nature of Failure/Yield - BOLT YIELD @ 4700 PSIg
Other Remarks - N/A
Tested By - D. Hallmark Approved By - T.R. Bruce Jones

UNDERGROUND DRILLING HOLE FIELD DATA SHEET

Date: 8/13/91

Room/Drift: PANEL 1 - ROOM 2

Purpose of Hole: PULL TEST: DESTRUCTIVE (PWR 12017T)
PT081391-5

Location: N. END 4th W ROOM & 30-35 FT. FROM N. BULKHEAD

Depth at Completion: 11'-7"

Diameter: 1 3/8" ϕ COMPLETION

Approximate Collar Coordinates

Vertical Angle: VERT. (5° TOLERANCE)

Direction/Azimuth:

Remarks: RESIN/ROCK ANCHOR TEST AT 2:38 PM
AND COMPLETED AT 2:55 PM FOR
CERTIFICATION.

Signature/Department: D. Sullivan 8/14/91

10/15/9

ROCK BOLT CERTIFICATION TEST
DATA SHEET

page 1 of 2

Mine WIPP Rock HALITE Rock Bolt No. PT080891-1 Hole Location PANEL 1 - ROOM 2

Hole Length - 11.50 Dia. - @ Anchor GAUGED/@ midpoint N/A /@ collar N/A
Orientation - VERTICAL (5° TOLERANCE) SMOOTH

Bolt Grade - GR 60 Length - 13'-0 1/4" Diameter - #8

Shell Manufacturer - N/A Model - N/A Diameter - N/A
Length - N/A

Resin Manufacturer - CELTITE Model - H90-0204 Diameter - 1 1/4" (32mm)
Length - 12" No. Cartridges - 3

Installation Date - 08/08/91 Time - 12:46 pm Torque - N/A ft/lb

Test Date - 08/09/91 DS 8/9/91 Time - 1:45 pm Torque - N/A ft/lb
Test Jack - RC-60-CH9.5A Test Pump - RC-60-CH9.5A 23886
Dial Indicator - AMES 4622 CD-0060 Calibrated - 10-4-91 DUE
Pressure Gauge - 42 GAGE DS1 DD1 Calibrated - 8-6-91

Pump Psi Pressure	LBS Bolt Tension	Extensometer Reading DS 8-9-91	Displacement	Remarks
200	2470	0.000 DS 8-9-91		
400	4940	0.017		
600	7410	0.144		
800	9880	0.165		
1000	12350	0.184		
1200	14820	0.205		
1400	17290	0.225		
1600	19760	0.246		
1800	22230	0.272		
2000	24700	0.293		
2200	27170	0.322	DS	BLED TO APPROX. 2100
2400	29640	0.~~348~~ 347	DS	NC IG - PG
2600	32110	0.371		
2800	34580	0.394		
3000	37050	0.414		
39,520 • 3200	39520	0.441		BLED BACK TO 3100 AFTER 2 MIN.
~~34~~ ~~39~~	~~4 4~~	0.464		* MISSED READING AT 3300 PSI
~~38~~				
~~40~~ DS/8-9-91				
~~40~~ DS/8-9-91				
~~42~~ DS/8-9-91				
3500	43225	0.474		HELD AT 2 MIN.
3600	44460	0.487	•	BLED BACK 3500 PSI AFTER 2 MIN
3700	45695	0.502		
3800	46930	0.513		• PLATE AT BACK DEFORMING
3900	48165	0.527		BLED TO 3800 AFTER 2 MIN.
4000	49400	0.540		HOLDING 4000 AFTER 2 MIN BLED 3900 AFTER 3 MIN.
4100	50635	0.549		BLED TO 4000 AFTER 2 MIN
4200	51870	0.559		HOLDING AFTER 2 MIN
4300	53105	0.567		BLED TO 4200 HELD AT 4200 AFTER 3 MIN
4400	54340	0.583		HOLDING AFTER 2 MIN
4500	55575	0.594		HOLDING AFTER 2 MIN
4600	56810	0.612		HOLDING BLED SLIGHTLY
• 4700	58045	0.626		HOLDING AFTER 2 MIN
* 4800	59280	0.657		HOLD BLED TO 4700 AFTER 2 MIN
~~4900~~		0.712		ATTEMPT TO GO TO 5000
		0.765		BLED BACK TO 4800

Test Results - ACCEPTED Maximum Pull Force DS 8-9-91 59280#
Displacement at Maximum Pull Force - ~~0.765 in.~~ 0.842 in.
Nature of Failure/Yield - BOLT YIELD ~~4800 psig~~ 4800 psig
Other Remarks - CONTINUED PULL ON 8/12/91 TO 6700 psi with approx 2.3" elongation (apparent elongation)
Tested By - D. Ballmann Approved By - [illegible]

ROCK BOLT CERTIFICATION TEST
DATA SHEET

Mine WIPP Rock HALITE Rock Bolt No. PT080891-1
Hole Location

Hole Length - ____ Dia. - @ Anchor ____/@ midpoint ____/@ collar ____
Orientation -

Bolt Grade - ____ Length - ____ Diameter - ____

Shell Manufacturer - ____ Model - ____ Diameter - ____
Length - ____

Resin Manufacturer - ____ Model - ____ Diameter - ____
Length - ____ No. Cartridges -

Installation Date - ____ Time - ____ Torque - ____ ft/lb

Test Date - ____ Time - ____ Torque - ____ ft/lb
Test Jack - ____ Test Pump - ____
Dial Indicator - ____ Calibrated - ____
Pressure Gauge - ____ Calibrated - ____

Pump psi Pressure	Bolt Tension	Extensometer Reading	Displacement	Remarks
~~5000~~		0.842		ATTEMPTED 5000 psi BLEEDS BACK TO 4800
				BOLT YIELD - ~~NATURE OF FAILURE~~ 12/26/91
				TEST TERMINATED 2:30 pm

Test Results ____ Maximum Pull Force - ____
Displacement at Maximum Pull Force -
Nature of Failure/Yield -
Other Remarks -
Tested By - D. Bellmann Approved By - C.E. Brush

UNDERGROUND DRILLING HOLE FIELD DATA SHEET

Date: 08/08/91

Room/Drift: PANEL 1 - ROOM 2

Purpose of Hole: PULL TEST : DESTRUCTIVE (PWR 12017T)
PT080891-1

Location: NORTH END: APPROX. 4'1" E OF ROOM & APPROX. 10- FROM NORTH BULKHEAD

Depth at Completion: 11'-6.5" (+1" -0) 11'-6.5"

Diameter: 1 3/8 COMPLETION

Approximate Collar Coordinates

Vertical Angle: VERTICAL (5° TOLERANCE

Direction/Azimuth:

Remarks: BOLT ROCK/RESIN ANCHORS TESTED ON 8-9-91 AT 1:45pm
TEST COMPLETED 2:30pm FOR CERTIFICATION.

Signature/Department: D. Ballenaut / Mine Engineering

ROCK BOLT CERTIFICATION TEST
DATA SHEET

Mine WIPP Rock HALITE Rock Bolt No. PT080891-2
Hole Location PANEL 1 - ROOM 2

Hole Length - 11.55' Dia. - @ Anchor GAUGED/@ midpoint N/A /@ collar N/A
Orientation - VERTICAL (5° TOLERANCE) SMOOTH

Bolt Grade - GR60 Length - 13'-0¼" Diameter - #8

Shell Manufacturer - N/A Model - N/A Diameter - N/A
Length - N/A

Resin Manufacturer - CELTITE Model - H90-0204 Diameter - 1¼" (32mm)
Length - 12" No. Cartridges - 3

Installation Date - 08/08/91 Time - 1:13 pm Torque - N/A ft/lb

Test Date - 08/09/91 Time - 2:55 pm 08/8-9-91 Torque - N/A ft/lb
Test Jack - RC-60-CH-25 Test Pump - RC-60-CH9.5 23886
Dial Indicator - AMES 4322 CD-0040 Calibrated - 10-4-91 DUE
Pressure Gauge - U.S. GAGE DS1#001 Calibrated - 8-6-91

Pump Pressure psi	Ex. Rod ~~Bolt Tension~~	Δ Ex Rod ~~Extensometer Reading~~	Tension ~~Displacement LBS~~	Remarks
200	0.100	~~0.100~~ DS 8-9-91	2470	
400	0.118	0.100 DS 8-9-91	4940	
600	0.151	~~0.100~~ DS 8-9-91	7410	
800	0.174	0.100 DS 8-9-91	9880	
1000	0.191	~~0.100~~ DS 8-9-91	12350	
1200	0.211		14820	
1400	0.229		17290	
1600	0.245		19760	
1800	0.262		22230	
2000	0.279		24700	
2200	0.293		27170	
2400	0.317		29640	
2600	0.336		32110	
2800	0.354		34580	
3000	0.372		37050	
3200 (39,520#)	0.388		39520	HOLD FOR 2 MIN - BLED TO 3000 psi
3300	0.396		40755	HELD AT 3400
3400	0.408		41990	HOLDING AFTER 2 MIN.
3500	0.415		43225	DS/8/09/91
* 3600	0.425	0.326 *	44460	*ADJUST TO ~~0.326~~ 0.326
3700		0.337	45695	GAUGE CAME OFF BOLT
3800		0.347	46930	• PLATE AT BACK DEFORMED
3900		0.354	48165	
4000		0.362	49400	HELD FOR 2 MIN. BLED TO 3900
4100		0.373	50635	
4200		0.381	51870	
4300		0.387	53105	
4400		0.400	54340	
4500		0.417	55575	
4600		0.434	56810	
4700		0.455	58045	
~~4800~~		0.530 DS 8-9-91		BOLT YIELD - PUMP BLEEDS TO 4700: BACK 4600 0.577 AT ~~4600~~ 4700 psi
				TEST TERMINATED 3:05 pm DS/8/9/91

Test Results - ACCEPTED Maximum Pull Force - ~~56810~~ / 58045#
Displacement at Maximum Pull Force - 0.530 in.
Nature of Failure/Yield - BOLT YIELD ~~(APPARENT)~~ DS/8-9-91 4700 psig
Other Remarks 24+ HOURS AFTER INSTALLATION
Tested By - D. Bathrait Approved By - R.E. [illegible]

*BACK "CLOCKED" - ALIGNMENT OF PULL EQUIP. ADJUSTED

UNDERGROUND DRILLING HOLE FIELD DATA SHEET

Date: 08/08/91

Room/Drift: PANEL 1 - ROOM 2

Purpose of Hole: PULL TEST : DESTRUCTIVE (PWR 12017T)
PT080891-2

Location: NORTH END: APPROX. 4+' E OF ROOM ℄ APPROX. 15' FROM NORTH BULKHEAD

Depth at Completion: 11'-6" (+1" -0") 11'-7"

Diameter: 1-3/8" COMPLETION

Approximate Collar Coordinates

Vertical Angle: VERTICAL (5° TOLERANCE)

Direction/Azimuth:

Remarks: ROCK/RESIN ANCHOR TESTED ON 8.9.91 AT ~~3:05 pm~~ 2:55 pm.
COMPLETED AT ≈ 3:05 pm FOR CERTIFICATION.

Signature/Department: D. Galbraith / Mine Engineering

ROCK BOLT CERTIFICATION TEST
DATA SHEET

Mine WIPP Rock HALITE Rock Bolt No. PT080091-3
Hole Location PANEL 1 - BOOM 2

Hole Length - 11.58 Dia. - @ Anchor GAUGED /@ midpoint N/A /@ collar N/A
Orientation - VERTICAL (5° TOLERANCE) SMOOTH

Bolt Grade - GR60 Length - 13'-0 1/4" Diameter - #8

Shell Manufacturer - N/A Model - N/A Diameter - N/A
Length - N/A

Resin Manufacturer - CELTITE Model - H90-0204 Diameter - 1 1/4" (32 mm)
Length - 12" No. Cartridges - 3

Installation Date - 08/08/91 Time - 1:46 pm Torque - N/A ft/lb

Test Date - 08/09/91 Time - 3:30 pm Torque - N/A ft/lb
Test Jack - RC-60-CH95A Test Pump - RC-60.H95 23886
Dial Indicator - ~~AMES~~ 4822 CD-0040 Calibrated - 10-4-91 DUE
Pressure Gauge - USGAGE DS1#001 Calibrated - 8-6-91

~~Pump~~ PSI Pressure	~~Bolt~~ LBS Tension	Extensometer Reading	Displacement	Remarks
200	2470	0.100		
400	4940	0.113		
600	7410	0.135		
800	9880	0.157		
1000	12350	0.180		
1200	14820	0.202		
1400	17290	0.222		
1600	19760	0.233		
1800	22230	0.247		
2000	24700	0.263		
2200	27170	0.281		
2400	29640	0.296		
2600	32110	0.314		
2800	34580	0.334		
3000	37050	0.356		
3200	39520	0.376		HELD FOR 2 MIN. 3000
3300	40755	0.393		HOLDING
3400	41990	0.405		
3500	43225	0.415		HOLDING AFTER 1 MIN
3600	44460	0.425		HOLDING
3700	45695	0.436		HOLDING AFTER 1 MIN.
3800	46930	0.449		
3900	48165	0.460		
4000	49400	0.473		
4100	50635	0.485		
4200	51870	0.497		HOLDING AFTER 1 MIN
4300	53105	0.511		
4400	54340	0.524		HOLDING AFTER 1 MIN.
4500	55575	0.536		HOLDING
4600	56810	0.555		HELD FOR 2 MIN BLED TO 4500 #
4700	58045	0.585		HELD 1 MIN. BB 4600 X PLATE DEFORMATIO
~~4800~~	~~59280~~	0.686		BOLT YIELD - FAILURE APPARENT
4900 ATTEMPTED				CONTINUE ~~LOADING~~ PUMPING
				DEF. .686 AT 4600 GAUGE
				.702 AT 4600 READING
				TEST TERMINATED @ 3:45 PM

39,520 #

Test Results - ACCEPTED Maximum Pull Force - 58045 #
Displacement at Maximum Pull Force - ~~0.686 in~~ 0.702 in
Nature of Failure/Yield - BOLT YIELD @ 4700 psi (58045 lbs) ~~APPARENT~~
Other Remarks - 24 HOURS AFTER INSTALLATION
Tested By - D. Bathurst Approved By - R.E. Brown

UNDERGROUND DRILLING HOLE FIELD DATA SHEET

Date: 08/08/91

Room/Drift: PANEL 1 - ROOM 2

Purpose of Hole: PULL TEST: DESTRUCTIVE (PWR 12017T)
PT080891-3

Location: NORTH END: APPROX. 4+' E OF ROOM ℄ APPROX. 20-25 FT FROM NORTH BULKHEAD

Depth at Completion: 11'-6" (+1" -0) 11'-7"

Diameter: 1-3/8" COMPLETION

Approximate Collar Coordinates

Vertical Angle: VERTICAL (5° TOLERANCE)

Direction/Azimuth:

Remarks: ROCK/RESIN ANCHOR TESTED ON 8.9.91 AT 3:30 pm.
COMPLETED AT 3:45 pm FOR CERTIFICATION

Signature/Department: L. Galbraith / Mine Engineering

WORK CHANGE NOTICE

1. WCN No.: 1		2. Critical System Yes ☐ No ☑
3. Work Request: 12017-T	4. Originated By / Phone No.: T.E. BROCKMAN 8304 Date: 8/7/91	5. Incorporated By / Phone No.: T.E. BROCKMAN 8304 Date: 8/7/91

6. **DESCRIPTION OF CHANGE** (Print or Type Proposed Change)

Page	Step or Paragraph	Instructions: Rewrite Paragraph Changes, Draw Illustration Changes
ATTACHMENT 6		CHANGE LARGE REAMED HOLE FROM 3" TO 3-4"
5	7.2.4b	CHANGE LOAD FROM 1,000# TO 200 P.S.I.
5	7.2.6	CHANGE ALL REFERENCE TO 1,000# TO READ 200 P.S.I.
5	7.2.7b	CHANGE 40,000# TO READ 3,200 P.S.I.
5	7.2.7	CHANGE 500# TO READ APPROXIMATELY 100 P.S.I.

7. Reason for Change:

TO ACCOMODATE FOR AVAILABLE EQUIPMENT. CHANGE DOES NOT AFFECT SCOPE NOR INTENT OR DESIGN

8. **DESIGN CLASSIFICATION**

☑ Class II or IIIA ☑ Class IIIB or Non-Facility

9. Required Approval

	Signature	Date
PIC All Classes	M. Campbell	8/8/91
Originator All Classes	T.E. Brockman	8/7/91
Safety As Required	[illegible]	8/8/91
Quality Assurance II or IIIA	[illegible]	8/8/91
Cognizant Engineer II or IIIA	for J.A. GONZALES [illegible]	8/7/91
Cognizant Operations Mgr. All Classes	T.E. Brockman	8/7/91
Other		
Other		

290B:3919

DESTRUCTIVE TESTING OF RESIN ANCHORS

1.0 SCOPE

A representative sample of resin anchored Dywidag tendons will be pull tested to destruction. This will be achieved by installing the rock anchors in Room 2, Panel 1 in conditions as similar to those in Room 1 as possible. The results will be used as basic input data in the design of the supplemental support system proposed for Room 1, Panel 1.

2.0 OBJECTIVES

The objectives of the destructive testing are expected to yield results that:

- o provide data on both the mode and load at failure of the halite/resin/tendon bond.
- o establish the allowable design load for the rock anchor system.

In addition, other in situ tests are expected to provide results that:

- o check for interaction effects between rock anchors.
- o provide short term creep response of the rock anchor system.

3.0 SAFETY REQUIREMENTS

3.1 Personnel Safety - All personnel participating in or observing destructive testing shall be properly equipped.

3.2 For the safety of observers, personnel not actively participating in the destructive testing are requested to stay outside the designated work area. The boundary for a designated work area is to be established at least fifty (50) feet away.

3.3 During the performance of the destructive tests, safety chain or cables shall be attached to each item of test equipment weighing more than 10 lbs. that may be violently released or fall as a result of testing.

4.0 TEST MATERIALS

4.1 Resin: Celtite Lockset Polyester Resin Cartridge
High Viscosity, Code (H)
Gel Time, Two-to-Four Minutes Code (90)
Cartridge, 3212

0204 DG 8.7.91

QA Verification: KA Lacy

For additional information refer to Attachments 1, 2, and 3 for Material Safety Data Sheets, installation instructions, and product information.

4.2 Tendons: Dywidag Post-Tensioning System
#8 THREADBAR (LHT), ASTM A615 GR 60
FULL LOAD ANCHOR NUTS FOR GRADE 60 THREADBAR

QA Verification: KA Lacy

For additional information refer to Attachments 4 and 5.

5.0 TEST EQUIPMENT AND TEST REQUIREMENTS

5.1 A minimum of ten (10) rock anchors shall be installed, and loaded to failure. Initially, only one set (of five) rock anchor installations shall be tested in any given twenty-four (24) hours. The load will be recorded all the way to failure.

5.1.1 The failure mode is to be determined by pulling tendons completely of the hole.

5.1.2 For the purpose of destructive testing, failure is defined as an increasing or continuous deformation with no increase of the applied load.

5.2 There is to be a waiting period of at least twenty-four (24) hours between resin/anchor activation and the commencement of destructive testing. The waiting period assures that the resin has cured and is approaching ultimate strength. Fully cured resin develops compressive strength of 14,000 psi and tensile strength of 5,000 psi or more.

QA Verification: Mike Rogers

5.3 The testing equipment includes a hydraulic ram with a 60 ton capacity, a pressure gauge readable in 200 psi increments, and a dial indicator gauge for measuring deformation in increments of 0.001 inch. Rock/resin anchor deformation will be measured by means of the dial indicator.

QA Verification: Mike Rogers

5.4 Installation of properly calibrated Geotechnical instrumentation may be used where required.

5.5 All instruments and devices used for measuring or recording loads or deformation during the test shall have been properly calibrated with tags affixed indicating calibration due date.

QA Verification: Mike Rogers

6.0 DRILLING AND ANCHOR/RESIN INSTALLATION INSTRUCTIONS

6.1 Drill each test hole as per Attachment 6 in accordance with applicable sections as prescribed in WP 04-220 and provided by factory representative's instructions.

6.1.1 Test hole locations shall have not less than four (4) feet spacings.

6.1.2 The 1-3/8 inch diameter bits shall be gauged prior to drilling above Anhydrite "b". Gauging assures the annular tolerance needed for a minimum bond length of three (3) feet.

6.1.3 Holes shall be drilled to a depth of 11 feet 6 inches with 1 inch tolerance, for test purposes only.

6.1.4 The perpendicular hole tolerance shall be 5 degrees as shown on Attachment 6.

6.2 Initially, only one set (of five) rock anchor installations shall be tested in any given twenty-four (24) hours. This may be changed subject to improved installation performance gained by experience and signed by cognizant engineer or his designee.

6.3 Resin and threadbar will be installed in accordance with the manufacturers' recommendations. A manufacturer's representative will be present during resin/anchor installation and destructive testing.

6.4 Insert required number of resin cartridges (3 minimum) through plastic or steel pipe and fed into the end of the hole. The resin is then followed by the threadbar with spin adapter.

Care must be taken to avoid rupturing the resin cartridges.

6.5 Start rotation at about 50 rpm or more and gradually insert the threadbar bolt into the resin cartridge to rupture the cartridge. *

6.5.1 Advance threadbar bolt at an approximate rate of 2 to 4 inches per second or as recommended by the factory representative.

* DUE TO AVAILABLE EQUIPMENT AND PER CONTACT INSTRUCTIONS. THE BAR IS TO BE INSERTED TO FULL DEPTH THEN ...

6.5.2 Hold bolt in the hole and hold until the resin sets, approximately for two (2) to four (4) minutes.

6.5.3 Threadbar should have approximately eighteen (18) inches of thread protruding from the hole in its final position.

6.5.4 Thorough mixing of the resin ingredients is essential.

6.6 There is to be a waiting period of at least twenty-four (24) hours between resin/anchor activation and destructive testing.

A waiting period assures that the resin has cured and is approaching its ultimate strength. When fully cured, the resin develops a compressive strength of 14,000 psi and tensile strength of 5,000 psi or more.

QA Verification: Mike Bogue

7.0 FULL TEST INSTRUCTIONS

7.1 Preparation for tests

7.1.1 All instruments and devices used for measuring or recording loads or deformation during the testing shall have been properly calibrated with tags affixed indicating calibration due date.

7.1.2 Prepare data sheets. Applicable information and test data shall be recorded on data sheets, Attachment 7. See Section 8 for QA verification.

7.1.2.1 Record at least the following: rock bolt number (i. e., PTmmddyy-#); mine; rock type; hole location; orientation; hole length and diameter; bolt grade, length, and diameter; installation date and time; test date and time; resin type, manufacturer, number of resin cartridges; and, identify test equipment by serial number and indicate calibration due date.

7.1.2.2 Measure the hole length and record.

7.1.2.3 When testing is complete, submit the data sheet to Mine Engineering.

7.1.2.4 When testing is complete, submit WP Form 2014 to Mine Engineering, Attachment 8.

7.1.2.5 Mark the final depth of the threadbar at 11 feet 6 inches with 1 inch tolerance from the anchor end and record.

7.1.2.6 Estimate hole orientation and roughness (i.e., vertical and smooth) and record.

7.2 FULL TEST

7.2.1 Install a bearing plate against the rock followed by placing the ram over the threadbar followed by another bearing plate and load cell and bearing plate, and finally torque the anchor nut.

The anchor nut shall be tightened to the manufacturer's recommended torque values (150 ft-lbs to 300 ft-lbs).

7.2.1.1 Fasten safety chain or cable to all equipment weighing more than 10 lbs and anchor to adjacent rock bolt plates.

7.2.2 The ram alignment should be near parallel to the axis of the tested bolt and within the limits allowed by the installation.

7.2.3 Make hose connections from the hydraulic pump to the ram.

7.2.4 a Make load cell and other instrumentation connections to data logger.

7.2.4 b Apply 1,000 lb load on the ram to eliminate apparatus slack.

7.2.5 Attach a magnet mounted dial indicator gauge on the ram with the point set in place on the bolt or to a fixed reference point in the salt. Adjust the dial indicator gauge to zero.

7.2.6 Test loading shall continue to be applied starting with 1,000 lbs and increased by 1,000 lb increments up to 40,000 lbs. Loads are to be held for approximately one (1) minute before recording the load and deformation data. Record the test data, Attachment 7.

7.2.7 From 40,000 lbs, the load shall be applied in 500 lb increments. While approaching failure, data readings may be taken more often.

7.2.8 Testing the resin/rock anchor to failure is the acceptance criteria. The test is accepted as complete at failure.

7.2.8.1 For the purpose of destructive testing, failure is defined as an increasing or continuous deformation with no increase of the applied load.

7.2.9 Terminate testing.

7.2.10 Remove equipment and pull bolt from hole, if possible

7.2.11 Continue testing of the next installation.

8.0 TEST DATA, ILLUSTRATIONS AND REPORTS

8.1 The data sheet form (Attachment 7) shall be filled out for each test bolt installation. The data sheet shall be keyed to match the rock anchor pull test number (PTmmddyy-#).

QA Verification: Mike Rogers

8.2 Each test shall be identified by rock anchor number and plotted on an appropriate scale. Each data point shall be plotted with the horizontal (x) axis showing displacement while the vertical (y) axis indicates load.

8.3 Photographs of a typical rock anchor installation shall be referenced to the rock anchor test number.

8.4 A report shall be prepared by Mine Engineering summarizing the test results and certifying the installation procedures and resin/rock anchors that have been tested and approved for installation at the WIPP.

8.5 The results will be used as basic input data in the design of the supplemental support system proposed for Room 1, Panel 1.

CONCURRENCE BY:

D. Galbraith, Senior Engineer
Mine Engineering

J. A. Gonzalez, Cognizant Engineer
Mine Engineering

8/7/91

8/7/91

S. C Sethi, Manager
Mine Engineering

MATERIAL SAFETY DATA SHEET

SECTION 1 - PRODUCT IDENTIFICATION AND USE

MANUFACTURER: CELTITE TECHNIK USA
150 CARLEY COURT
GEORGETOWN, KY 40324
(502) 863 - 6800

CANADIAN CONTACT: FOSECO CANADA INC.
GUELPH, ONTARIO

MANUFACTURERS IDENTIFICATION CODE:
20,35,37,40,45,50,70,90,0204,0510,1530
PRODUCT NAME: LOKSET POLYESTER RESIN CARTRIDGE
CHEMICAL FAMILY: Polyester Resin & Catalyst
PRODUCT USE: Anchoring Compound
WHMIS CLASSIFICATION: Class D, Division 2A; Class B, Division 3

SECTION 2 - HAZARDOUS INGREDIENTS

INGREDIENT	%	ACGIH/OSHA TLV TWA	STEL	CARC	NTP CARC	IARC CARC
Polyester resin CAS# N.R.	10-30	N.E.	N.E.	NO	NO	NO
Styrene monomer CAS# 100-42-5	5-10	50P	100P	NO	NO	2B
Benzoyl Peroxide CAS# 94-36-0	.5-1.5	5M	N.E.	NO	NO	NO
Calcium Carbonate CAS# 471-34-1	60-100	N.A.	N.E.	NO	NO	NO
Propylene Glycol CAS# 57-55-6	.5-1.5	N.E.	N.E.	NO	NO	NO

units - P suffix denotes PPM and M suffix denotes mg/m3
m suffix denotes mppcf and c suffix denotes ceiling value
N.A.- Not Applicable N.E.- Not Established N.D.- Not Determined
CARC- Carcinogen N.D.F.- No Data Found N.R.- Not Reported
NOTE: See SUPPLEMENT for SARA Section 313 reporting information.

This document is prepared pursuant to the OSHA Hazard Communication Standard (29 CFR 1910.1200) and the Canadian Workplace Hazardous Materials System (WHMIS). In addition, other ingredients not 'Hazardous' per these standards may be listed.

SECTION 3 - PHYSICAL DATA

APPEARANCE: Tan or black resin mortar and white paste catalyst in plastic package.
ODOR: N.A.
ODOR THRESHOLD: 0.1 ppm as Styrene
PERCENT VOLATILE: 11
SOLUBILITY IN WATER: N.A.
VAPOR PRESSURE(mmHg): 4.5 @ 20C(Styrene)
SPECIFIC GRAVITY: 1.75-1.85
VAPOR DENSITY: 3.6 as Styrene
EVAPORATION RATE: N.D.
BOILING POINT: 293F
FREEZING POINT: N.D.
pH: Acidic
COEF. OF WATER/OIL DISTRIBUTION: N.D.
PHYSICAL HAZARDS: Organic peroxide; Combustible Liquid

ATTACHMENT 1

SEC ON 7 - PREVENTIVE MEASURE

SPECIAL PROTECTION INFORMATION:
RESPIRATORY: Use MSHA (NIOSH) approved respirator if application produces vapors, mists, or fumes above the TLV.
VENTILATION: Adequate to prevent vapor build-up above the TLV.
PROTECTIVE GLOVES: Chemical resistant polyethylene or equivalent.
EYE PROTECTION: Safety glasses or goggles as required to prevent eye contamination.
OTHER: Use protective clothing to minimize contact with skin. Wash contaminated clothing before reuse.

SPILL OR LEAK PROCEDURES:
SPILL: Ventilate area. Remove all sources of ignition. Absorb with inert material and collect.
WASTE DISPOSAL METHOD: Dispose of in accordance with Federal, State, and local regulations.
STORAGE: For maximum shelf-life avoid storage in direct sunlight, elevated temperatures or near sources of heat such as steam pipes and radiators. Store in a cool, dry well-ventilated area.
OTHER: Since the product is a sealed cartridge, handling hazards are minimal unless product is damaged or misused.
SHIPPING INFORMATION: Not Applicable

SECTION 8 - FIRST AID MEASURES

EYES - Flush with water for at least 15 min. Consult a physician.
SKIN - Wash thoroughly with soap and water.
INGESTION - Consult a physician immediately.
INHALATION - Remove to fresh air if effects occur. Call a physician if effects persist.

SECTION 9 - PREPARATION DATE OF MSDS

Revision date: 12/11/89
Previous revision date: 01/01/89
For further information contact: Leo Hickam
Phone number: (502) 863 - 6800

SUPPLEMENT:

HAZARDOUS INGREDIENTS Continued...

The following chemicals are subject to the reporting requirements of Section 313 of Title III of the Superfund Amendments and Reauthorization Act of 1986 (SARA) and 40 CFR Part 372.

Chemical Name	CAS#	Maximum % by Weight
Benzoyl Peroxide	94-36-0	1.5%
Styrene	100-42-5	10.0%

Dy / G. Sealy & G. Greenfield Celtite
GEORGETOWN, KY : 150 CARLEY CT. 40324
502/863-6800
PRINCETON, WV Rogers St & Caperton Ave 24740 304-425-7501
GRAND JUNCTION, CO 1440 Winters Ave 81501 303-245-4007

CELTITE, INC.

INSTALLATION INSTRUCTIONS FOR CELTITE RESIN CARTRIDGES

1. Measure and mark drill steel 1" longer than bolt length. Note: The bore hole should be shortened accordingly when headers or thick plates are used.

2. Insert required number of cartridges and place bolt into the base of the hole behind the resin.

3. With the head of the bolt placed firmly in the chuck of the bolter or in a spin-in wrench, push the bolt into the hole. Slow rotation is recommended but not required for adequate mixing.

4. When the bolt reaches just below the roof, stop upward movement and spin the bolt rapidly for five to ten seconds. (See note for stoper.)

5. Stop rotation and push the bolt upward with the full thrust from the machine and hold until the resin sets.

6. If the bolt tends to drop out of the hole after the chuck or adapter is removed, simply push it back up into the hole and hold until the resin sets.

7. NEVER re-rotate the bolt after the final spin, as damage to the partially-set resin may occur.

8. Always wear safety glasses or eye shields when installing resin bolts.

9. To achieve maximum performance, selection of bolt diameter, cartridge type and related hole diameter are very important. Celtite® technical representatives are available to assist in the selection of the correct combination of components based on specific applications.

10. Note for stoper: Contact Celtite® technical representative.

11. For applications other than fully grouted bolts, Celtite® technical representatives will instruct mine personnel in correct installation procedure.

12. Cartridges should be stored in a cool, well-ventilated and dry area away from direct sunlight. High temperature conditions can reduce shelf life of cartridges. Stock rotation is recommended so that oldest stock is used first.

CAUTION: Do not open or puncture cartridges. Contents of cartridges may cause mild irritation and should be avoided. Eye protection should always be used when bolting. If resin contacts the eyes, flush immediately with water for at least 15 minutes. Call a physician.

We believe that the information contained herein (which supersedes all previous information on this subject) is true and reliable. We cannot be held responsible for any loss, injury, or damage resulting from its use, as, of necessity, the information given is of a general nature, so that users are advised to consult us about their specific problems.

Celtite® warrants that its products, at the time of shipment, conform to the applicable descriptions herein and are free from defects in materials and workmanship. NO OTHER WARRANTY, WHETHER EXPRESS, IMPLIED OR STATUTORY, INCLUDING ANY WARRANT OF MERCHANTABILITY OR FITNESS FOR A PARTICULAR PURPOSE, SHALL EXIST IN CONNECTION WITH THE SALE OR USE OF ANY CELTITE® PRODUCT, AND ALL SUCH WARRANTIES ARE HEREBY EXPRESSLY EXCLUDED.

IN NO EVENT SHALL CELTITE® BE LIABLE FOR INJURY TO PERSON OR PROPERTY, LOSS OF BUSINESS OR PROFIT ON ANY OTHER DIRECT, INDIRECT, INCIDENTAL, SPECIAL OR CONSEQUENTIAL DAMAGES.

U.S. Patent Nos. 3731791, 3915297, Canada No. 873075, Other World Patents

03-00-03
04-22-86

ATTACHMENT 2

The Lokset® Polyester Resin Cartridge Anchor System

This system's simplicity of application enables bolts of various lengths to be anchored and grouted in one easy operation, with no need for injection equipment.

The Celtite Technik system consists of an easily handled "cartridge," containing a highly reinforced polyester resin component, together with its catalyst, in accurately measured quantities. The components are isolated from each other by a *physical-chemical barrier* which prevents reaction between the components until required. The cartridges are sausage-shaped and designed for rapid insertion into a range of bore hole sizes, where they may be readily pushed to the extremity at any angle above or below the horizon. Therefore, optimum bolt anchorage in a wide range of rock or concrete strengths is easily achieved simply by adjusting the length of the resin anchor zone.

No reaction takes place until the roof or rockbolt is rotated through the cartridge, mixing the components and initiating the curing action. The chemical nature (thixotropy) of the Lockset® cartridge allows the contents to be easily mixed yet minimizes resin displacement after mixing is complete.

The mixed resin totally fills the area (annulus) around the bolt, which for standard "point" anchorages will be firmly bonded to the substrate and bolt within minutes.

The setting time of the resin components can be controlled. A combination of fast and slow-setting cartridges makes possible the simultaneous operation of anchoring, grouting and tensioning a rockbolt. The simplicity of this method of anchoring grouting eliminates the need for cumbersome injection equipment.

APPLICATIONS

- **Rock bolting in mines and tunnels.**
- **Permanent rock reinforcement on highway rock cuts, dams and underground rock structures (power-houses and machinery galleries).**
- **Integral ties between reinforced concrete and rock faces above or below water.**
- **Vibration resistant anchorages for attachment of "critical" equipment to concrete or rock.**
- **Anchorage for electrical transmission towers.**
- **Uplift anchorages for near surface structures.**
- **Immediate post-tensioning of steel reinforcements in rock or concrete structures.**

Celtite Technik's Lokset® Polyester Resin Cartridge.

ADVANTAGES

Accuracy. All Anchorages can now be accurately designed with Celtite Technik resins having reproducible strength characteristics.

Speed. The fast-gelling Lokset® resins enable rapid installation to be carried out, a significant advantage in the area of tunnel bolting and rock slope stabilization. Application of load can be completed within minutes.

Permanence. The resins protect the embedded bolt from corrosion due to acid-bearing water, sea or ground water. Atmosphere is precluded from the bore hole, preventing further deterioration of the strata.

Safety. Millions of Lokset® resin anchors are used every year for critical jobs such as roof support or permanent rock reinforcement in mines, tunnels and foundations.

Vibration. Celtite Technik anchors are not affected by vibration and require *no* retensioning even after close proximity blasting.

Stress-free. No internal stresses are set up in the rock or concrete by resin anchors.

TECHNICAL SERVICE

Celtite Technik is available to discuss with you both installation techniques and the selection of proper Lokset® cartridges for your project. Our field representatives are ready to review your applications and help you to develop successful, economical anchoring systems.

ATTACHMENT 3

Anchoring

INTRODUCTION

Installation of Celtite Technik polyester resin cartridges in rock and concrete have ranged from application of short 3/4-inch rebar "starter bars" into concrete to 1 3/8" diameter bolts for rock reinforcement, measuring 55' in length, weighing over 300 pounds and fully embedded in resin!

ANCHORAGE STRENGTHS

To achieve maximum anchorage strength between bolt and concrete or rock, the difference in diameter between hole and bolt should be kept to a minimum. This also insures better mixing of both catalyst and resin.

Standard Rebar or Threadbar bolts (conforming to ASTM A-615 specifications) are used without further refinement. Deformations on the bar serve to effectively mix the cartridge components during the "spin-in" or insertion cycle. (SEE BOLT SELECTION ON PAGE 6).

Spin adaptors are available from several manufacturers. *Consult CELTITE® TECHNIK for source information.*

UNDERWATER ANCHORAGES

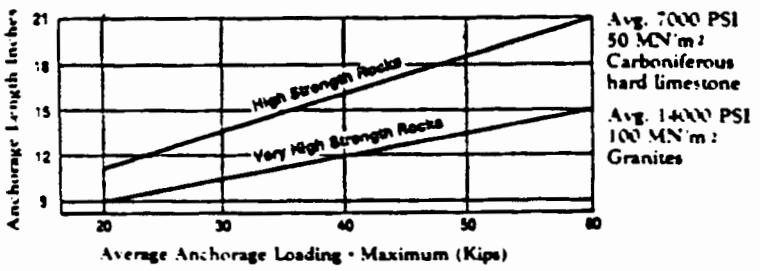
Lokset® cartridges can be applied in underwater anchorage applications over 24 inches in length, in both concrete and rock. *Consult CELTITE® TECHNIK for your specific job requirements.*

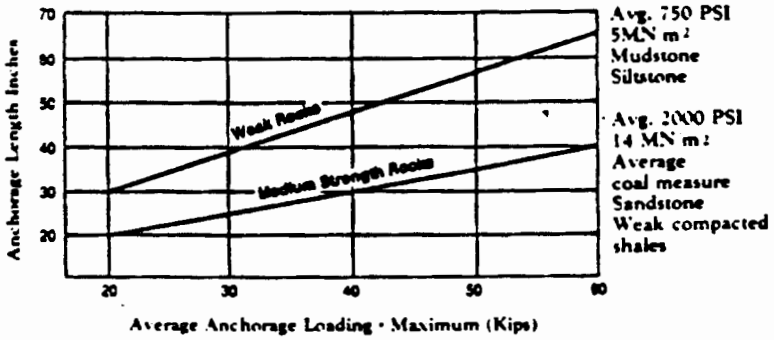
APPLICATION EQUIPMENT

The hole-drilling equipment mentioned in this brochure is generally suitable for spinning in bolts. This equipment should be rotary percussive and have provision for independent rotation to maintain 100 rpm under load. Under no circumstances should the bolt be simply pushed through the resin cartridges as improper mixing can result, providing for possible anchorage failure.

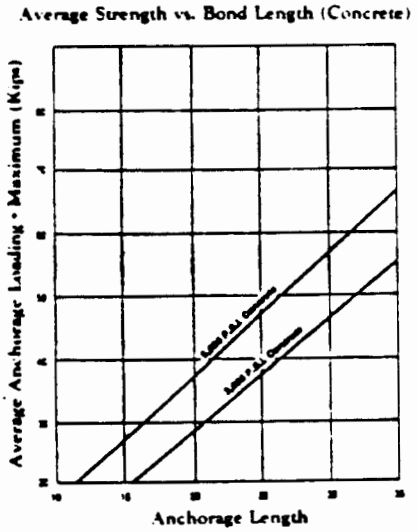
RESIN ANCHORAGE CHARTS

Typical anchorage loadings* in rock for point anchored rockbolts used in accordance with the manufacturer's recommendations. Intended as a guide for site trials, which will establish the working specifications in the actual ground conditions. ***Shown in Kips (1000 lbs. = 1 Kip).***





Rock strengths are in uniaxial compression terms. Information based on trials conducted by Imperial College, London (1970), J. A. Franklin, B.S.C., M.Sc., Ph.D, D.I.C.



Packaging.

Order Code (1)	Cartridge Diameter (inches)	Pieces Per Carton	Weight Per Carton
2312	15/16	50	27.5 lbs.
2812	1 1/8	35	29.5 lbs.
3212	1 1/4	25	26.5 lbs.
3512	1 3/8	20	26.0 lbs.
4012	1 9/16	16	27.0 lbs.
4512	1 3/4	12	25.0 lbs.

Minimum shipment; single full cartons.
A full pallet contains 75 cartons of standard 12-inch cartridges.
(1) Do not forget to specify gel time (see description section on page 5).

STORAGE PROCEDURES

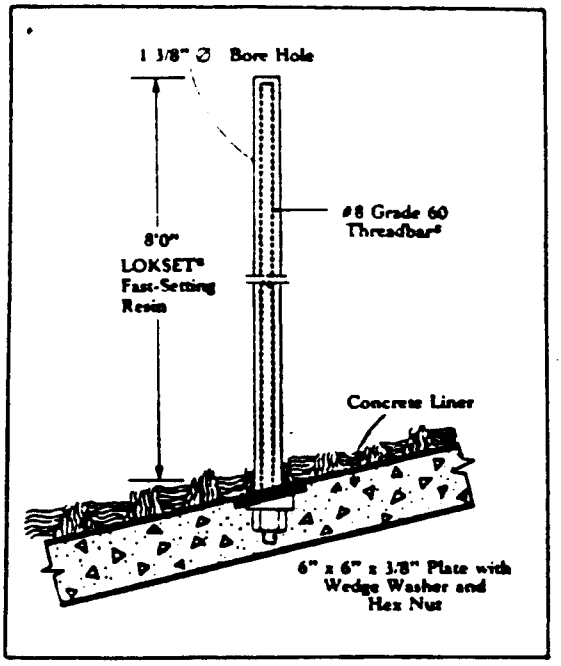
Lokset® cartridges should be stored in a cool, well-ventilated and dry area away from direct sunlight. High temperature conditions can reduce shelf life. Cartridges stored in extreme temperatures should be "normalized" at 50-70°F for at least two days prior to use to provide the expected gel time.

Pallets should not be stacked. Stock rotation is recommended so that the oldest stock is used first.

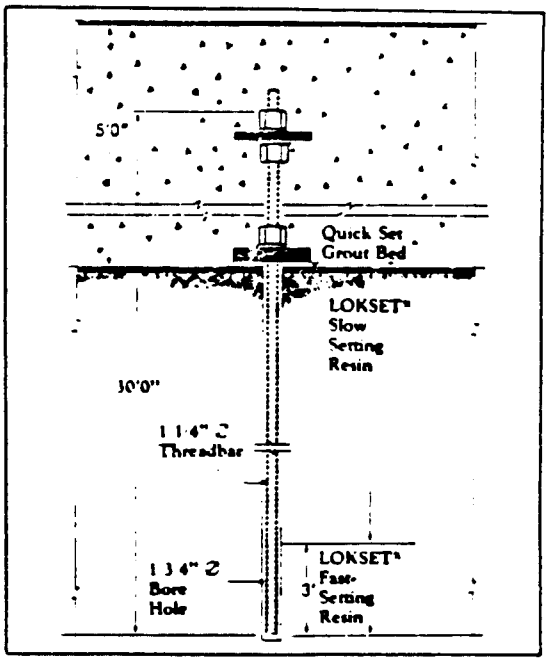
SAFE HANDLING PROCEDURES

REFER TO THE MATERIAL SAFETY DATA SHEET. Do not open or puncture cartridges prior to insertion. Contents of cartridges may cause mild irritation and contact should be avoided. Use in adequately ventilated area. Avoid prolonged inhalation of vapor. Eye protection should always be used when bolting. If resin contacts the eyes, flush immediately with water for at least 15 minutes. Seek medical attention.

Typical Lokset® resin anchor applications



Roof bolts in diversion tunnel.
Paintsville Lake Dam, Kentucky.
Army Corps of Engineers, Huntington District.



Stilling Basin Rock Anchors.
Smithland Dam, Ohio River.
Army Corps of Engineers, Nashville District.

The illustrations on page 3 and the engineering drawings above are reprinted courtesy of Dowidag Systems International, Lemont, Illinois.

UNDERGROUND DRILLING HOLE FIELD DATA SHEET

Date: _____

Room/Drift: _____

Purpose of Hole: _____

_____

_____

_____

_____

_____

_____

Location: _____

Depth at Completion: _____

Diameter: _____

Approximate Collar Coordinates _____

Vertical Angle: _____

Direction/Azimuth: _____

Remarks: _____

_____

_____

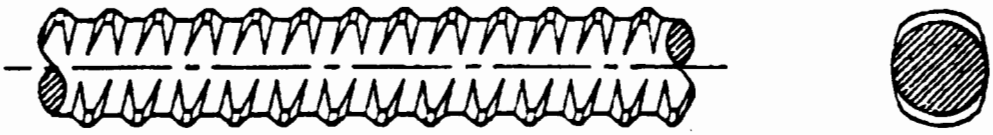
_____

Signature/Department: _____

WP Form 2014; 10/15/90
Page 1 of 1

10/15/90

LEFT HAND THREAD



PHYSICAL PROPERTIES	
EFFECTIVE AREA:	0.79 SQ.IN.
ULTIMATE STRENGTH:	90 KSI
ULTIMATE LOAD:	71.1 KIPS
YIELD LOAD:	47.4 KIPS
WEIGHT:	2.67 LBS./FT.
MAX. BAR Ø INCL. RIBS:	1.12 IN.
AVERAGE CORE Ø:	0.95 IN.
PITCH:	0.492 IN.

ATTACHMENT 4

DYWIDAG POST-TENSIONING SYSTEMS	DIMENSIONS: INCH/mm mm FOR REFERENCE ONLY	REV.	DATE	NAME	DRAWING NUMBER B08J0001
[redacted]	TOLERANCES: ± .02 UNLESS NOTED				
	MATERIAL: ASTM A615, GR 60 — SCALE NTS				
DATE: 08-23-90 DRAWN: B. WALECKI APPROVED:					

This drawing, the pertinent specifications, descriptions, conditions etc. and their contents are the

DYWIDAG SYSTEMS INTERNATIONAL U.S.A., INC.

TABLE: COMPARISON OF BAR MECHANICAL PROPERTIES

Bar Size/Grade ASTM	#7/GR 60 A 615			#7/GR 75 F 432			#8/GR 60 A615		
	Yield (kips)	Tensile (kips)	Elong. (%)	Yield (kips)	Tensile (kips)	Elong. (%)	Yield (kips)	Tensile (kips)	Elong. (%)
Guaranteed per ASTM	36	54	8	45	60	8	47.4	71.1	8
g. actual per mill cert.	42	65	12	47	72	9	55	83	13

6/24/91

ATTACHMENT 4.1

NORTH STAR STEEL MINNESOTA

P.O. Box 64189
1678 Red Rock Road
Saint Paul, Minnesota 55164

CERTIFIED TEST REPORT

Heat # B-42393
Size #8 Th.
Grade 7432-55
Date Rolled 5/3/90
P.O.
M.O. #

CHEMICAL ANALYSIS

C	MN	P	S	SI	SN	CU	NI	CR	MO	CB	V
.43	.98	.043	.035	.22	.017	.42	.13	.16	.032	.019	

AL				

JOMINY END-QUENCH HARDENABILITY RESULTS (HRc)

J1	J2	J3	J4	J5	J6	J7	J8	J9	J10	J11	J12
											-

J13	J14	J15	J16	J18	J20	J22	J24	J26	J28	J30	J32

PHYSICAL TEST REPORT

SPECIMEN AREA (in²)	YIELD KIPS	YIELD K.S.I.	TENSILE KIPS	TENSILE K.S.I.	GAUGE LENGTH (in)	% ELONG	BEND	% R.A.	Specification/Grade
.79	54.9	69.5	88.6	112.2	8.0	13.7	OK		7432-55
.79	54.8	69.4	83.6	105.8	8.0	13.7	OK		7606

Grain Size: Cleanliness:

Macro Etch: Specification: Results:

Impact Test	Temp (F)	ft./lb.	% Shear*	% L.E.*

* By specification or request only

This material has been produced and tested in accordance with the requirements of applicable specifications unless otherwise listed below. We hereby certify that the above test results are representative of those contained in the records of the company.

Any modification to this certification as provided by North Star Steel Minnesota without the expressed written consent of North Star Steel Minnesota negates the validity of this test report.

North Star Steel Minnesota is not responsible for the suitability of this material to meet specific applications.

SIGNED

DATE 5/3/90

SWORN AND SUBSCRIBED TO BEFORE ME

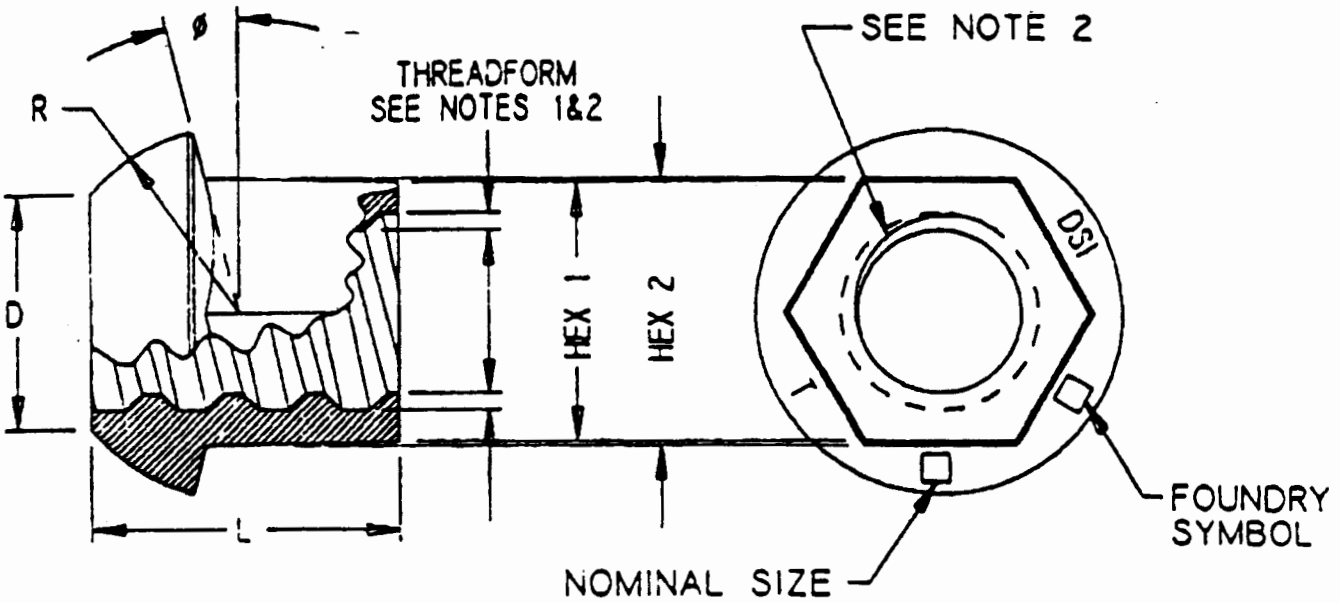
THIS _____ DAY _____

_____
Notary Public

MY COMMISSION EXPIRES _____

THIS CERTIFICATE IS NOTORIZED ONLY WHEN REQUESTED.

ATTACHMENT 4.2



NOMINAL SIZE	#6	20mm	#7	[redacted]	#9	#10	PRELIMINARY #11
PART NUMBER	B06U28810	B20U28810	B07U28810	B08U28810	B09U28810	B10U28810	B11U28810
L (IN./mm)	1.375/34.9	1.625/41.3	1.625/41.3	1.750/44.5	2.125/54.0	2.250/57.2	2.625/66.7
HEX 1 (IN./mm)							
HEX 2 (IN./mm)	1.250/31.8	1.375/34.9	1.375/34.9	1.500/38.1	1.688/42.9	1.813/46.0	2.063/52.4
R (IN./mm)	0.865/22.0	1.000/25.4	1.000/25.4	1.120/28.4	1.250/31.8	1.419/36.0	1.530/38.9
D (IN./mm)	1.063/27.0	1.188/30.2	1.188/30.2	1.313/33.4	1.438/36.5	1.641/41.7	1.821/46.3
Ø DEG.	14	14	14	14	14	14	14
THREADFORM: 8341/	743	785	744	745	748	747	724
MINOR Ø MIN. (IN./mm)	0.730/18.5	.782/19.9	0.858/21.7	0.974/24.7	1.096/27.8	1.238/31.4	1.406/35.7
MINOR Ø MAX. (IN./mm)	0.742/18.8	.794/20.2	0.868/22.0	0.986/25.0	1.108/28.1	1.252/31.8	1.420/36.1
WEIGHT LBS	0.27	0.54	0.5	0.72	0.89	1.20	1.77

4 → L FOR #11, AND WEIGHTS FOR #9, #10 & #11 CHANGED.

MATERIAL:

AUSTEMPERED DUCTILE IRON OR CAST STEEL
MIN. YIELD: 100 KSI
MIN. TENS: 120 KSI
MIN. EL. IN 2": 8%

TOLERANCES:

FOR HEX1, HEX2, R, D: +0/-0.040"

FOR L: +0.125/-0"

FOR Ø: ±1°

NOTES:

1. THREADFORM AS SPECIFIED WITH EXCEPTION OF MINOR DIAMETER (MINOR Ø) WHICH IS SPECIFIED IN TABLE.
2. AT BOTH ENDS OF NUT THREADFORM IS MODIFIED OVER LAST 1/2 TURN AS FOLLOWS: MINOR Ø OPENS UP TO MAJOR Ø AT CONSTANT RADIUS.

ATTACHMENT 5

DYWIDAG POST-TENSIONING SYSTEMS

[redacted]

DIMENSIONS: INCH/mm
mm FOR REFERENCE ONLY

MATERIAL: SEE ABOVE

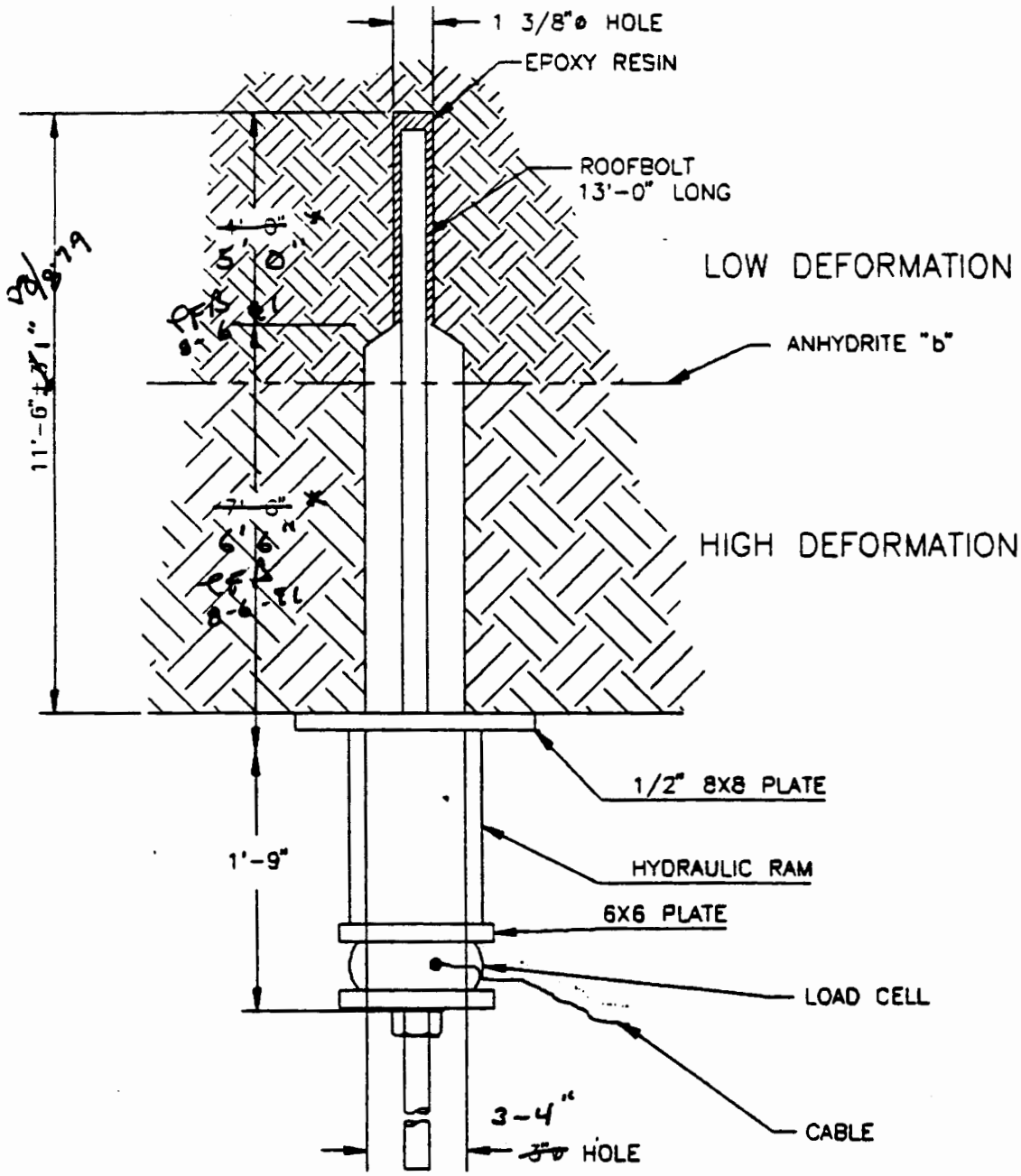
SCALE 1:1 FOR #7

REV.	DATE	NAME
1	02-20-90	K.L.
2	08-01-90	K.L.
3	08-28-90	K.L.
4	01-29-91	K.L.

DRAWING NUMBER B00U28810

DATE: 2-06-89 DRAWN: S. MALPOC APPROVED:

* CHANGE MADE DUE TO AVAILABLE EQUIPMENT
CRB
8/6/21



ROOFBOLT DETAIL
VERTICAL ALIGNMENT (TOL. 5°)

ATTACHMENT 6

ROCK BOLT CERTIFICATION TEST
DATA SHEET

Mine _____ Rock _____ Rock Bolt No. _____
Hole Location _____

Hole Length - ____ Dia. - @ Anchor _____/@ midpoint_____/@ collar ____
Orientation - _____

Bolt Grade - _____ Length - _____ Diameter - _____

Shell Manufacturer - _____ Model - _____ Diameter - _____
Length - _____

Resin Manufacturer - _____ Model - _____ Diameter - _____
Length - _____ No. Cartridges - _____

Installation Date - _____ Time - _____ Torque - _____ ft/lb

Test Date - _____ Time - _____ Torque - _____ ft/lb
Test Jack - _____ Test Pump - _____
Dial Indicator - _____ Calibrated - _____
Pressure Gauge - _____ Calibrated - _____

Pump Pressure	Bolt Tension	Extenso-meter Reading	Displace ment	Remarks

Test Results - _____ Maximum Pull Force - _____
Displacement at Maximum Pull Force - _____
Nature of Failure/Yield - _____
Other Remarks - _____
Tested By - _____ Approved By - _____

ATTACHMENT 7

APPENDIX C

DRAFT 09/10/91

WASTE ISOLATION PILOT PLANT
SUPPLEMENTARY ROOF SUPPORT SYSTEM

APPENDIX C

Support System Design

Anchor Design Detail

Channel Support Design Detail

Westinghouse Electric Corporation
Waste Isolation Division
Carlsbad, New Mexico
August 1991

DRAFT 09/10/91

WASTE ISOLATION PILOT PLANT
SUPPLEMENTARY ROOF SUPPORT SYSTEM

APPENDIX C

Support System Design

Anchor Design Detail

Westinghouse Electric Corporation
Waste Isolation Division
Carlsbad, New Mexico
August 1991

Check print #2

WESTINGHOUSE WASTE ISOLATION DIVISION

TITLE			PAGE
BOLT WORKING LOAD FOR 10' x 2' SPACING			1 OF 3
PROJECT	AUTHOR / DATE	CHK'D BY / DATE	CHK'D BY / DATE
	C. FRANCKE 9/5/91	J R Tybusk 9/6/91	
SO	CALC NO	FILE NO	REVISION

PROBLEM: DETERMINE THE MAXIMUM WORKING LOAD ON A BOLT IN A 2' x 10' SPACING.

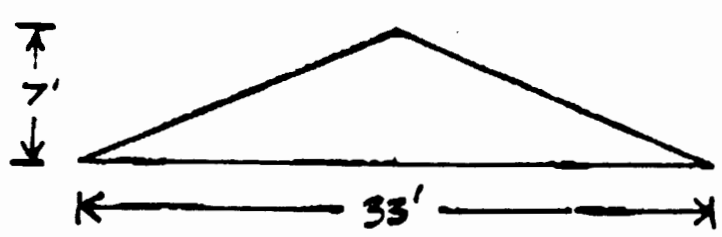
ASSUMPTIONS:

1) DENSITY OF WIPP SALT = 135 lb/ft³ = 2.17 gm/cm³ (DOC# ~~D-W-~~ D-81-W-01, Rev 6)
2) THE ROCK REQUIRING SUPPORT IS DELINEATED BY AN ISOCELES TRIANGLE WITH ITS APEX AT CLAY G AND ITS BASE AT THE ROOF. THE HEIGHT OF THE TRIANGLE IS 7' (DISTANCE FROM ROOF TO CLAY G). THE BASE OF THE TRIANGLE IS 33' (WIDTH OF THE ROOM).
3) THE ROCKBOLT PATTERN SPACING WILL BE 2' ACROSS THE WIDTH OF THE ROOM AND 10' ALONG THE LENGTH OF THE ROOM.

METHOD: THE MAXIMUM VOLUME OF ROCK SUPPORTED BY EACH BOLT WILL BE CALCULATED AND MULTIPLIED BY THE DENSITY OF THE ROCK TO DETERMINE THE LOAD ON THE BOLT.

CALCULATIONS:

THE FOLLOWING DIAGRAM ILLUSTRATES THE DIMENSIONS OF THE ROCK REQUIRING SUPPORT.



CROSS-SECTION VIEW

TITLE NO	REV DATE	AUTHOR	DATE	CHK'D BY	DATE	CHK'D BY	DATE

WESTINGHOUSE FORM 4025DA

TITLE: BOLT WORKING LOAD FOR 10' x 2' SPACING			PAGE 2 OF 3
PROJECT	AUTHOR C. FRANCKS 9/3/91	CHK'D BY J.R. Turbak 9/6/91	CHK'D BY / DATE
SID	CALC NO	FILE NO	GROUP

CALCULATIONS (CONT'D)

VOLUME OF ROCK

THE VOLUME OF ROCK SUPPORTED BY A BOLT IS DETERMINED BY THE FOLLOWING EQUATION:

$$V = S_L S_W t$$

WHERE V = VOLUME OF ROCK TO BE SUPPORTED

S_L = SPACING OF BOLTS IN THE LONG DIMENSION

S_W = SPACING OF BOLTS IN THE SHORT DIMENSION

t = THICKNESS OF ROCK TO BE SUPPORTED

THE VALUE OF "t" FOR A PARTICULAR BOLT IS THE AVERAGE THICKNESS OF THE ROCK ALONG A LINE "S_W" LONG CENTERED ON THE BOLT. THE MAXIMUM WORKING LOAD WILL BE AT THE CENTER OF THE SPAN WHERE $t = 7'$.

$$V = (10')(2')(7')$$

$$V = 140 \text{ ft}^3$$

REV NO	REV DATE	AUTHOR	DATE	CHK'D BY	DATE	CHK'D BY	DATE

WESTINGHOUSE WASTE ISOLATION DIVISION

TITLE				PAGE
BOLT WORKING LOAD FOR 10'x2' SPACING				3 OF
PROJECT	AUTHOR C. FRANCKE	DATE 9/5/91	CHK'D BY	DATE / CHK'D BY
B.O.	CALC NO		FILE NO	GROUP

CALCULATIONS: (CONT'D)

WORKING LOAD

THE WEIGHT OF THE ROCK SUPPORTED BY A BOLT IS DETERMINED BY THE FOLLOWING EQUATION:

$$W = \rho V$$

WHERE: W = WEIGHT OF ROCK (WORKING LOAD)
ρ = DENSITY OF ROCK
V = VOLUME OF ROCK

Checker Note:
Salt density changed to that used in the "Design Basis" report for consistency

$$W = (135\ lb/ft^3)(140\ ft^3)$$

$$W = 18.9\ kip = 84.1\ kN$$

DISCUSSION:

THE MAXIMUM LOAD ON A BOLT IN THE PROPOSED 2'x10' LAYOUT WILL BE ~~20~~ 18.9 kip.

REV NO	REV DATE	AUTHOR	DATE	CHK'D BY	DATE	CHK'D BY

FROM "DESIGN BASIS - UNDERGROUND EXCAVATIONS" BECHTEL DOC. # D-51-W-01

3.1.9 Excavation shall permit early installation and start up of experiments in one of more rooms while excavation continues in the remainder of the experimental areas. The design shall permit access to, and utilization of, both waste and nonwaste experimental areas.

3.2 Configuration and Essential Features

3.2.1 General Location Underground

All depths assume ERDA No. 9 drill hole surface elevation as datum. This is 3409 ft above mean sea level.

The storage area floor is approx. 2,150 ft below the ground surface. The final level of the excavations shall be as determined by the contracting officer. All excavations shall be parallel to the existing bedding planes within allowable tolerances. To the extent practical, a marker bed in the excavation rib will be defined as the reference plane from which the excavation shall be continued. In addition, holes shall be drilled into the floor and roof of the excavation for determination of the location of Marker Bed - 139 and the clay seams above the roof. Coring of these holes shall begin before the continuous miner advances more than 250 feet from the location.

Based on the existing, the average dip of the sedimentary beds in the underground areas will be approximately 2 percent to the south east.

3.2.2 Salt Composition and Density

The halite is relatively free of impurities with the exception of occasional thin but separate layers of lightly interspersed argillaceous materials (up to 4%) and of polyhalite (less than 1%). Halite is also widely interbedded with layers of anhydrite ranging from a few inches to several feet in thickness. The overall moisture content is 0.4%.

For purposes of excavation calculations, the salt in place is assumed to have a density of 135 lb /cu ft (1.8225 tons/cu yd).

3.2.3 Creep Allowance

Excavation dimensions shall include an allowance for salt creep of 1 ft in the vertical dimension and nine inches in the required width of all rooms.

WESTINGHOUSE WASTE ISOLATION DIVISION

TITLE			PAGE
FACTOR OF SAFETY – BOLT TENDON FAILURE			1 OF 2
PROJECT	AUTHOR C. FRANCKS DATE 9/5/91	CHK'D BY JR Tyburski DATE 9/6/91	CHK'D BY DATE
SO	CALC NO	FILE NO	GROUP

PROBLEM: DETERMINE THE FACTOR OF SAFETY (FofS) OF ROCKBOLTS AGAINST TENDON FAILURE.

ASSUMPTIONS:

1.) THE TENDON IS EXPECTED TO BE GRADE 60 STEEL WHICH MEETS ASTM STANDARD A615 (REF. 2).
2.) THE MAXIMUM WORKING LOAD ON THE TENDON IS 18.9 kip (84.1 KN).
3.) THE MINIMUM (GUARANTEED) ULTIMATE TENSILE STRENGTH OF THE TENDON IS 90,000 psi (REF. 2).
4.) THE TENDON IS 1" DIAMETER. Value not verified

REFERENCES:

1.) PTI (Post-Tensioning Institute), 1986, Recommendations for Prestressing Rock and Soil Anchors.
2.) ASTM Standard A615.

METHOD:

THE WORKING LOAD ON THE TENDON IS CALCULATED.

CALCULATIONS:

THE MAXIMUM LOAD THAT THE TENDON CAN SUPPORT IS DETERMINED BY MULTIPLYING THE ULTIMATE TENSILE STRENGTH OF THE TENDON BY THE CROSS-SECTIONAL AREA OF THE TENDON.

REV NO	REV DATE	AUTHOR	DATE	CHK'D BY	DATE	CHK'D BY	DATE

TITLE: FACTOR OF SAFETY - BOLT TENDON FAILURE			PAGE 2 OF 2
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CALCULATIONS (CONT'D)

$$P_t = T_t \pi d_t^2/4$$

WHERE: P_t = MAXIMUM ALLOWED WORKING LOAD
T_t = ULTIMATE TENSILE STRENGTH OF TENDON
d_t = DIAMETER OF TENDON

$$P_t = (90\ \text{kip/in}^2)\pi(1\ \text{in})^2/4$$
$$P_t = 70.7\ \text{kip}$$

FACTOR OF SAFETY

$$F_{of}S = \frac{\text{MAXIMUM ALLOWED TENSILE STRESS}}{\text{ACTUAL WORKING LOAD}}$$

$$F_{of}S = 70.7\ \text{kip}/18.9\ \text{kip}$$

$$F_{of}S = 3.7$$

DISCUSSION:

ACCORDING TO THE PTI RECOMMENDATIONS (REF 1): "The tendon size is determined such that the design load for the anchor does not exceed 60% of the guaranteed ultimate strength of the tendon." THIS WOULD BE EQUIVALENT TO A $F_{of}S$ OF 1.7 (i.e. $1/0.6 = 1.7$). ~~NORMALLY,~~ IN OTHER WORDS, THE WORKING LOAD SHOULD NOT EXCEED $0.6 * P_t = 42.4$ kip, WHICH IT DOES NOT. NORMALLY, ONE ONE DETERMINE A DESIRED SAFETY FACTOR, SUCH AS 1.7, AND DETERMINE THE REQUIRED BOLT DIAMETER. IN THIS CALCULATION, THE SAFETY FACTOR WAS NOT CALCULATED UNTIL THE END SINCE BOLT DIAMETER AND LOAD WERE KNOWN.

Checker Note: Agree, however when evaluating the FofS we should use 60% of Ult. or Max Allowed Tensile Stress 42.4/18.9 or 2.2 which is still greater than 1.7 i.e. referenced

REV NO	REV DATE	AUTHOR	DATE	CHK'D BY	DATE	CHK'D BY	DATE

Designation: A 615 - 90

American Association State Highway and Transportation Officials Standard
AASHTO No. M 31

Standard Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement[1]

This standard is issued under the fixed designation A 615; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This specification has been approved for use by agencies of the Department of Defense. Consult the DoD Index of Specifications and Standards for the specific year of issue which has been adopted by the Department of Defense.

1. Scope

1.1 This specification covers deformed and plain billet-steel concrete-reinforcement bars in cut lengths or coils. A deformed bar is defined as a bar that is intended for use as reinforcement in reinforced concrete construction. The surface of the bar is provided with lugs or protrusions (hereinafter called *deformations*) which inhibit longitudinal movement of the bar relative to the concrete which surrounds the bar in such construction and conform to the provisions of this specification. The standard sizes and dimensions of deformed bars and their number designations shall be those listed in Table 1.

NOTE 1—For coils of deformed bars, the capacity of industrial straightening units limits the maximum bar size that can be straightened.

1.2 Bars are of three minimum yield levels: namely, 40 000, 60 000, and 75 000 psi, designated as Grade 40, Grade 60, and Grade 75, respectively.

1.3 Hot-rolled plain rounds, in sizes up to and including 2 in. in diameter in coils or cut lengths, when specified for towels, spirals and structural ties or supports shall be furnished under this specification in Grade 40, Grade 60, and Grade 75 (Note 2). For bending properties, test provisions of the nearest nominal diameter deformed bar size shall apply. These requirements providing for deformations and marking shall not be applicable.

1.4 The weldability of the steel is not part of this specification.

NOTE 2—The weight for plain rounds smaller than ¾ in. in diameter shall be computed on the basis of the size in Specification A 510.

NOTE 3—A complete metric companion to Specification A 615 has been developed—A 615M; therefore, no metric equivalents are presented in this specification.

2. Referenced Documents

2.1 *ASTM Standards:*

A 370 Methods and Definitions for Mechanical Testing of Steel Products[2]

A 510 Specification for General Requirements for Wire Rods and Coarse Round Wire, Carbon Steel[2]

A 700 Practices for Packaging, Marking, and Loading Methods for Steel Products for Domestic Shipment[2]

E 29 Practice for Using Significant Digits in Test Data to Determine Conformance with Specifications[4]

2.2 *U.S. Military Standards:*

MIL-STD-129 Marking for Shipment and Storage[5]

MIL-STD-163 Steel Mill Products Preparation for Shipment and Storage[5]

2.3 *U.S. Federal Standard:*

Fed. Std. No. 123 Marking for Shipment (Civil Agencies)[5]

3. Ordering Information

3.1 Orders for material under this specification should include the following information:

3.1.1 Quantity (weight).

3.1.2 Name of material (deformed and plain billet-steel bars for concrete reinforcement).

3.1.3 Size.

3.1.4 Cut lengths or coils.

3.1.5 Deformed or plain.

3.1.6 Grade.

3.1.7 Packaging (see Section 19).

3.1.8 ASTM designation and year of issue, and

3.1.9 Certified mill test reports (if desired).

NOTE 4—A typical ordering description is as follows: 20 tons deformed and plain billet-steel bars for concrete reinforcement, No. 3 (10 mm), 40 ft 0 in. long, deformed, Grade 60, in secured lots, to ASTM A 615 - — Certified mill test reports are required.

4. Material and Manufacture

4.1 The bars shall be rolled from properly identified heats of mold cast or strand cast steel using the open-hearth, basic-oxygen, or electric-furnace process.

5. Chemical Composition

5.1 An analysis of each heat of steel shall be made by the manufacturer from test samples taken preferably during the pouring of the heats. The percentages of carbon, manganese, phosphorus, and sulfur shall be determined. The phosphorus content thus determined shall not exceed 0.06 %.

5.2 The chemical composition thus determined shall be reported on request to the purchaser or his representative.

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[1] This specification is under the jurisdiction of ASTM Committee A-1 on Steel, Stainless Steel and Related Alloys, and is the direct responsibility of Subcommittee A01.05 on Steel Reinforcement.

[2] Current edition approved Apr. 27, 1990. Published June 1990. Originally approved as A 615 - 54. Last previous edition A 615 - 89.

[3] *Annual Book of ASTM Standards*, Vol 01.04.

[4] *Annual Book of ASTM Standards*, Vol 14.02.

[5] Available from Naval Publications and Forms Center, 700 Robbins Avenue, Philadelphia, PA 19120.

TABLE 1 Deformed Bar Designation Numbers, Nominal Weights, Nominal Dimensions, and Deformation Requirements

Bar Designation No.[A]	Nominal Weight, lb/ft	Nominal Dimensions[B]			Deformation Requirements		
		Diameter, in.	Cross-Sectional Area, in.²	Perimeter, in.	Maximum Average Spacing, in.	Minimum Average Height, in.	Maximum Gap (Chord of Arc), in.
3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
4	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
6	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
7	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
8	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
10	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
11	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
14	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
18	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

[A] The nominal dimensions of a deformed bar are equivalent to those of a plain round bar having the same weight per foot as the deformed bar.
[B] Bar numbers are based on the number of eighths of an inch included in the nominal diameter of the bars.

5.3 An analysis may be made by the purchaser from finished bars. The phosphorus content thus determined shall not exceed that specified in 5.1 by more than 25 %.

6. Requirements for Deformations

6.1 Deformations shall be spaced along the bar at substantially uniform distances. The deformations on opposite sides of the bar shall be similar in size and shape.

6.2 The deformations shall be placed with respect to the axis of the bar so that the included angle is not less than 45°. Where the line of deformations forms an included angle with the axis of the bar of from 45 to 70° inclusive, the deformations shall alternately reverse in direction on each side, or those on one side shall be reversed in direction from those on the opposite side. Where the line of deformation is over 70°, a reversal in direction is not required.

6.3 The average spacing or distance between deformations on each side of the bar shall not exceed seven tenths of the nominal diameter of the bar.

6.4 The overall length of deformations shall be such that the gap between the ends of the deformations on opposite sides of the bar shall not exceed 12½ % of the nominal perimeter of the bar. Where the ends terminate in a longitudinal rib, the width of the longitudinal rib shall be considered the gap. Where more than two longitudinal ribs are involved, the total width of all longitudinal ribs shall not exceed 25 % of the nominal perimeter of the bar; furthermore, the summation of gaps shall not exceed 25 % of the nominal perimeter of the bar. The nominal perimeter of the bar shall be 3.14 times the nominal diameter.

TABLE 2 Tensile Requirements

	Grade 40[A]	Grade 60	Grade 75[B]
Tensile strength, min, psi	70 000	90 000	100 000
Yield strength, min, psi	40 000	60 000	75 000
Elongation in 8 in., min, %, Bar No.			
3	11	9	
4, 5, 6	12	9	
7, 8		8	
9, 10		7	
11, 14, 18		7	4

[A] Grade 40 bars are furnished only in sizes 3 through 6.
[B] Grade 75 bars are furnished only in sizes 11, 14, and 18.

6.5 The spacing, height, and gap of deformations shall conform to the requirements prescribed in Table 1.

7. Measurements of Deformations

7.1 The average spacing of deformations shall be determined by dividing a measured length of the bar specimen by the number of individual deformations and fractional parts of deformations on any one side of the bar specimen. A measured length of the bar specimen shall be considered the distance from a point on a deformation to a corresponding point on any other deformation on the same side of the bar. Spacing measurements shall not be made over a bar area containing bar marking symbols involving letters or numbers.

7.2 The average height of deformations shall be determined from measurements made on not less than two typical deformations. Determinations shall be based on three measurements per deformation, one at the center of the overall length and the other two at the quarter points of the overall length.

7.3 Insufficient height, insufficient circumferential coverage, or excessive spacing of deformations shall not constitute cause for rejection unless it has been clearly established by determinations on each lot (Note 5) tested that typical deformation height, gap, or spacing do not conform to the minimum requirements prescribed in Section 6. No rejection may be made on the basis of measurements if fewer than ten adjacent deformations on each side of the bar are measured.

NOTE 5—A lot is defined as all the bars of one bar number and pattern of deformation contained in an individual shipping release or shipping order.

8. Tensile Requirements

8.1 The material, as represented by the test specimens, shall conform to the requirements for tensile properties prescribed in Table 2.

8.2 The yield point or yield strength shall be determined by one of the following methods:

8.2.1 The yield point shall be determined by drop of the beam or halt in the gage of the testing machine.

8.2.2 Where the steel tested does not have a well defined yield point, the yield strength shall be determined at extension under load using an autographic diagram method or an

TABLE 3 Bend-Test Requirements

Bar Designation No.	Pin Diameter for Bend Tests[A]		
	Grade 40	Grade 60	Grade 75
3, 4, 5	3d[B]	3d	
6	4d	4d	
7, 8		5d	
9, 10		6d	
11		7d	7d
14, 18		8d	8d

[A] Test bends 180° unless noted otherwise.
[B] d = nominal diameter of specimen.

extensometer as described in 13.1.2 and 13.1.3 of Methods and Definitions A 370. The extension under load shall be 0.005 in./in. of gage length (0.5 %) for Grade 40 and Grade 60 and shall be 0.0035 in./in. of gage length (0.35 %) for Grade 75. When material is furnished in coils, the test sample must be straightened prior to placing it in the jaws of the tensile machine (see Note 6).

8.3 The percentage of elongation shall be as prescribed in Table 2.

NOTE 6—Straightening should be done carefully to avoid the formation of local sharp bends and to minimize cold work. Insufficient straightening prior to attaching the extensometer can result in lower-than-actual yield strength readings.

9. Bending Requirements

9.1 The bend-test specimen shall withstand being bent around a pin without cracking on the outside of the bent portion. The requirements for degree of bending and sizes of pins are prescribed in Table 3.

9.2 The bend test shall be made on specimens of sufficient length to ensure free bending and with apparatus which provides:

9.2.1 Continuous and uniform application of force throughout the duration of the bending operation.

9.2.2 Unrestricted movement of the specimen at points of contact with the apparatus and bending around a pin free to rotate.

9.2.3 Close wrapping of the specimen around the pin during the bending operation.

9.3 Other acceptable more severe methods of bend testing, such as placing a specimen across two pins free to rotate and applying the bending force with a fixed pin, may be used. When failures occur under more severe methods, retests shall be permitted under the bend test method prescribed in 9.2.

10. Permissible Variation in Weight

10.1 The permissible variation shall not exceed 6 % under nominal weight, except for bars smaller than ⅜ in. plain round, the permissible variation in weight shall be computed upon the basis of the permissible variation in diameter in Specification A 510. Reinforcing bars are evaluated on the basis of nominal weights. In no case shall the overweight of any bar be the cause for rejection.

10.2 The specified limit of variation shall be evaluated in accordance with Practice E 29 (rounding method).

11. Finish

11.1 The bars shall be free of detrimental surface imperfections.

11.2 Rust, seams, surface irregularities, or mill scale shall not be cause for rejection, provided the weight, dimensions, cross-sectional area, and tensile properties of a hand wire brushed test specimen do not violate the requirements of this specification.

11.3 Surface imperfections other than those specified in 11.2 shall be considered detrimental when specimens containing such imperfections fail to conform to minimum tensile or bending requirements.

12. Test Specimens

12.1 Tension test specimens shall be the full section of the bar as rolled. The unit stress determination shall be based on the nominal bar area.

12.2 The bend test specimens shall be the full section of the bar as rolled.

13. Number of Tests

13.1 For bar sizes No. 3 to 11, inclusive, one tension test and one bend test shall be made of the largest size rolled from each heat. If, however, material from one heat differs by three or more designation numbers, one tension and one bend test shall be made from both the highest and lowest designation number of the deformed bars rolled.

13.2 In the case of Nos. 14 and 18 bars, one tension test and one bend test shall be made of each size rolled from each heat.

14. Retests

14.1 If any tensile property of any tension test specimen is less than that specified, and any part of the fracture is outside the middle third of the gage length, as indicated by scribe scratches marked on the specimen before testing, a retest shall be allowed.

14.2 If the results of an original tension specimen fail to meet the specified minimum requirements and are within 2000 psi of the required tensile strength, within 1000 psi of the required yield point, or within two percentage units of the required elongation, a retest shall be permitted on two random specimens for each original tension specimen failure from the lot. If all results of these retest specimens meet the specified requirements, the lot shall be accepted.

14.3 If a bend test fails for reasons other than mechanical reasons or flaws in the specimen as described in 14.4 and 14.5, a retest shall be permitted on two random specimens from the same lot. If the results of both test specimens meet the specified requirements, the lot shall be accepted. The retest shall be performed on test specimens that are at an temperature but not less than 60°F.

14.4 If any test specimen fails because of mechanical reasons such as failure of testing equipment or improper specimen preparation, it may be discarded and another specimen taken.

14.5 If any test specimen develops flaws, it may be discarded and another specimen of the same size bar from the same heat substituted.

15. Inspection

15.1 The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacture-

er's works that concern the manufacture of the material ordered. The manufacturer shall afford the inspector all reasonable facilities to satisfy him that the material is being furnished in accordance with this specification. All tests (except product analysis) and inspection, shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

15.2 *For Government Procurement Only*—Except as otherwise specified in the contract, the contractor is responsible for the performance of all inspection and test requirements specified herein and may use his own or any other suitable facilities for the performance of the inspection and test requirements specified herein, unless disapproved by the purchaser at the time of purchase. The purchaser shall have the right to perform any of the inspections and tests at the same frequency as set forth in this specification, where such inspections are deemed necessary to assure that material conforms to prescribed requirements.

16. Rejection

16.1 Unless otherwise specified, any rejection based on tests made in accordance with 5.3, shall be reported to the manufacturer within 5 working days from the receipt of samples by the purchaser.

16.2 Material that shows injurious defects subsequent to its acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

17. Rehearing

17.1 Samples tested in accordance with 5.3 that represent rejected material shall be preserved for 2 weeks from the date rejection is reported to the manufacturer. In case of dissatisfaction with the results of the tests, the manufacturer may make claim for a rehearing within that time.

18. Marking

18.1 When loaded for mill shipment, bars shall be properly separated and tagged with the manufacturer's heat or test identification number.

18.2 Each producer shall identify the symbols of his marking system.

18.3 All bars produced to this specification, except plain round bars which shall be tagged for grade, shall be identified by a distinguishing set of marks legibly rolled into the surface of one side of the bar to denote in the following order:

18.3.1 *Point of Origin*—Letter or symbol established as the producer's mill designation.

18.3.2 *Size Designation*—Arabic number corresponding to bar designation number of Table 1.

18.3.3 *Type of Steel*—Letter *S* indicating that the bar was produced to this specification.

18.3.4 *Minimum Yield Designation*—For Grade 60 bars, either the number *60* or a single continuous longitudinal line through at least 5 spaces offset from the center of the bar side. For Grade 75 bars, either the number 75 or two continuous longitudinal lines through at least 5 spaces offset each direction from the center of the bar. (No marking designation for Grade 40 bars.)

19. Packaging

19.1 When specified in the purchase order, packaging shall be in accordance with the procedures in Practices A 700.

19.2 *For Government Procurement Only*—When specified in the contract or order, and for direct procurement by or direct shipment to the U.S. government, material shall be preserved, packaged, and packed in accordance with the requirements of MIL-STD-163. The applicable levels shall be as specified in the contract. Marking for shipment of such material shall be in accordance with Fed. Std. No. 123 for civil agencies and MIL-STD-129 for military agencies.

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WESTINGHOUSE WASTE ISOLATION DIVISION

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S.O.	CALC NO	FILE NO	GROUP

PROBLEM: DETERMINE FACTOR OF SAFETY (FofS) FOR A RESIN ANCHOR BOND TO SALT VS. WORKING LOAD ON THE ANCHOR.

ASSUMPTIONS:

1.) MAXIMUM WORKING LOAD ON THE ANCHOR IS 18.9 kip (84.1 kN).
2.) FIXED ANCHOR LENGTH IS 3' (0.91 m).
3.) AVERAGE HOLE (AND ANCHOR) DIAMETER IS 1 3/8" (0.035 m).
4.) UNCONFINED COMPRESSIVE STRENGTH (UCS) OF WIPP SALT IS TAKEN AS 25 MPa. (SEE DISCUSSION).
See below

REFERENCES:

1.) G.S. Littlejohn and D.A. Bruce, "Rock Anchors - State of the Art - Part 1: Design", Ground Engineering, May 1975.
2.) PTI (Post-Tensioning Institute), Recommendations for Prestressed Rock and Soil Anchors, 1986.
3.) BSI (British Standards Institute), BSI 8081, "British Standard Code of Practice for Ground Anchorages", 1989.
4.) SAND 78-1596, Geologic Characterization Report, Powers et al.

METHOD: THE ULTIMATE ANCHOR LOAD IS CALCULATED BASED ON THE SIZE OF THE ANCHOR AND THE ANCHOR BOND STRENGTH.

For most elastic rocks using the higher UCS value from lab testing may be appropriate, however, because the strength of salt is time dependent, I'm not certain this is a conservative approach. The original checker comment (using 1 N/mm²) was to use a lower value. This needs to be discussed. A reasonable field test to evaluate time dependent slippage might need be developed for confirmation.

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SO	CALC NO	SHEET NO	GROUP

CALCULATIONS:

ACCORDING TO REF 2., THE ULTIMATE ANCHOR LOAD IS CALCULATED AS FOLLOWS:

$$P_a = \pi d_a L_a \tau$$

WHERE: P_a = LOAD TO CAUSE BOND FAILURE
d_a = EFFECTIVE ANCHOR DIAMETER
L_a = FIXED ANCHOR LENGTH
τ = WORKING BOND STRENGTH

REF 2. FURTHER STATES THAT, IN THE ABSENCE OF PULL TEST DATA ON BOND STRENGTH, τ CAN BE ESTIMATED AS 1/10 OF THE UCS OF THE ROCK. THEREFORE:

$$\tau = 0.1\,(UCS)$$
$$= 0.1\,(25\,MPa)$$
$$\tau = 2.5\,MPa = 2500\,kN/m^2$$

This does not address the lack of relevant data, as suggested by Littlejohn & Bruce.

τ IS OFTEN REDUCED TO 1/3 τ TO ADD A SAFETY FACTOR. THIS WILL NOT BE DONE HERE BECAUSE THE 1/3 τ VALUE IS USED TO DETERMINE (DESIGN) ANCHOR LENGTH OR DIAMETER. SINCE ANCHOR LENGTH & DIAMETER ARE PRE-DETERMINED, THE FULL VALUE OF τ IS USED SO THAT THE REAL SAFETY FACTOR CAN BE DETERMINED.

THE LOAD TO CAUSE BOND FAILURE IS THEN:

$$P_a = \pi(0.025m)(0.9m)(2500\,kN/m^2)$$
$$P_a = 250\,kN = 56\,kip$$

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B.O.	CALC NO.	FILE NO	GROUP	

CALCULATIONS (CONT'D)

FACTOR OF SAFETY - ANCHOR BOND

$$FofS = \frac{\text{LOAD CAUSING FAILURE}}{\text{WORKING LOAD}}$$

$$FofS = 56\,kip / 18.9\,kip = 3.0$$

DISCUSSION

RECOMMENDED SAFETY FACTORS FOR ANCHOR BOND RANGE FROM 1.5 TO 4, DEPENDING ON ROCK TYPE AND APPLICATION (REF 1,2,3). SAFETY FACTOR IS NORMALLY INTRODUCED INTO THE CALCULATION BY REDUCING τ, WORKING BOND STRENGTH, TO 25% TO 50% OF THE ULTIMATE BOND STRENGTH (REF. 2). THIS WOULD BE EQUIVALENT TO A SAFETY FACTOR OF 2 TO 4. AS IS EXPLAINED ON PAGE 2 OF THIS CALC, τ WAS NOT REDUCED SO THAT THE TRUE FACTOR OF SAFETY COULD BE DETERMINED.

See Comment pg 1 of 3

THE VALUE USED FOR UNCONFINED COMPRESSIVE STRENGTH (UCS) IS BASED ON LABORATORY TESTING (REF. 4) AND ENGINEERING JUDGEMENT. BECAUSE UCS VALUES FROM LAB TESTING USUALLY UNDERESTIMATE THE IN SITU STRENGTH, THE HIGHEST VALUE FROM REF 4 WAS USED.

THE 3' FIXED ANCHOR LENGTH PROVIDES AN ACCEPTABLE FACTOR OF SAFETY AGAINST FAILURE OF THE ANCHOR BOND.

THE ABOVE ASSUMPTIONS WILL BE VERIFIED BY FIELD TESTING.

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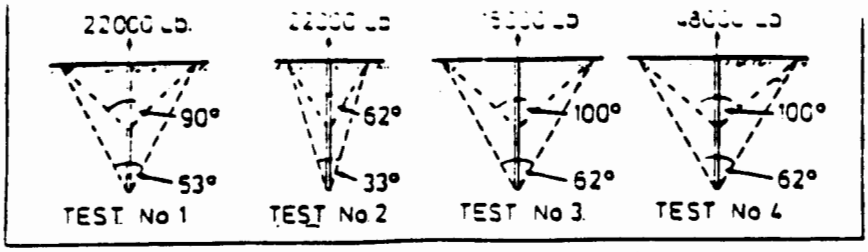

Fig. 3 Possible failure modes based on test results at Trinity Clear Creek
(after Saliman and Schaefer, 1968)

plex by Littlejohn and Truman-Davies (1974) where 2000kN anchors were spaced at 1m centres. Another method to dissipate load within the rock mass is simply to install anchors at different inclinations as in the design by Soletanche (1968) for the Zardesas Dam, Algeria. In some other countries a minimum distance between anchors is stipulated. Broms (1968), reviewing Swedish practice, confirmed a minimum spacing of 2.5m whilst the Czech Standard (1974) recommends 1.5m, one consideration being to reduce "inter-hole grouting", although this phenomenon is not necessarily a disadvantage in practice.

It is noteworthy that these guide rules or approaches are based on experience and engineering judgement, and not on an intimate knowledge of stress distribution around the anchor.

Remarks

With regard to uplift capacity no experimental or practical evidence and only very little theoretical data substantiate the methods currently used (Table I) to calculate the ultimate resistance to pull-out of individual, or groups of anchors. Indeed, there would appear to be results (Saliman and Schaefer (1968) and Brown (1970)) which indicate that failure in a rock mass does not generally occur in the form of an inverted 90 deg cone or wedge. However, it is reassuring to know that most designs are likely to be conservative in adopting a cone method with no allowance for the shear strength of the rock mass.

Nevertheless, some standardisation on safety factors for temporary and permanent anchors is desirable together with agreement on what allowances should be made for surcharge due to unconsolidated overburden and the effect of upper layers of weathered rock.

In general, effort should now be expended, in the form of field testing in a wide range of rock materials and masses which have been carefully classified, in order to study the shape and position of the rock zones mobilised at failure. Such programmes should accommodate single anchors and groups tested over a range of inclinations. Only in this way can anchor design in relation to overall stability be optimised both technically and economically.

BOND BETWEEN CEMENT GROUT AND ROCK

Introduction

Most designs to date concerning straight shaft fixed anchors have been successfully based on the assumption of uniform bond distribution over the fixed anchor surface area. In other words it has been generally accepted that the bond developed is merely a function of fixed anchor dimensions and applied load.

However, recent experimental and theoretical analyses have indicated that the character of the bond to the rock is more complex, and reflects additional parameters which often give rise to a markedly non-uniform stress distribution. Thus, in many cases the assumed mechanism of load transfer in the fixed anchor zone may be grossly inaccurate. For example, the situation could well arise where, for a high capacity anchor, the level of bond stress at the loaded (or proximal) end may be extremely high, possibly approaching failure, whereas the more distal parts of the fixed anchor may in effect be redundant. Clearly, such a situation will have a bearing on overall stability analyses, the interpretation of anchor extensions, and long-term creep behaviour.

Design criteria are reviewed relating to the magnitude and distribution of bond, fixed anchor dimensions, and factors of safety. For comparison, the results of relevant theoretical and experimental investigations are presented.

The straight shaft anchor is essentially on the development of bond or shear in the region of the rock/grout interface, and as described by Littlejohn (1972) it is usual in Britain to assume an equivalent uniform distribution of bond stress along the fixed anchor. Thus the anchor load, P, is related to the fixed anchor design by the equation

$$P = \pi d L \tau \qquad (1)$$

where L = fixed anchor length
d = effective anchor diameter
τ = working bond stress

This approach is used in many countries e.g. France (Fargeot, 1972), Italy (Massarani, 1973), Canada (Coates, 1970) and USA (White, 1973).

The rule is based on the following simple assumptions:

(*i*) Transfer of the load from the fixed anchor to the rock occurs by a uniformly distributed stress acting over the whole of the curved surface of the fixed anchor.
(*ii*) The diameter of the borehole and the fixed anchor are identical.
(*iii*) Failure takes place by sliding at the rock/grout interface (smooth borehole) or by shearing adjacent to the rock/grout interface in weaker medium (rough borehole).
(*iv*) There are no discontinuities or inherent weakness planes along which failure can be induced, and
(*v*) There is no local debonding at the grout/rock interface.

Where shear strength tests are carried out on representative samples of the rock mass, the maximum average working bond stress at the rock/grout interface should not exceed the minimum shear strength divided by the relevant safety factor (normally not less than 2). This approach applies primarily to soft rocks where the uniaxial compressive strength (UCS) is less than 7N/mm², and in which the holes have been drilled using a rotary percussive technique. In the absence of shear strength data or field pull-out tests, Littlejohn (1972) states that the ultimate bond stress is often taken as one-tenth of the uniaxial compressive strength of massive rocks (100 per cent core recovery) up to a maximum value of 4.2N/mm², assuming that the crushing strength of the cement grout is equal to or greater than 42N/mm². Applying an apparent safety factor of 3 or more, which is conservative bearing in mind the lack of relevant data, the work-

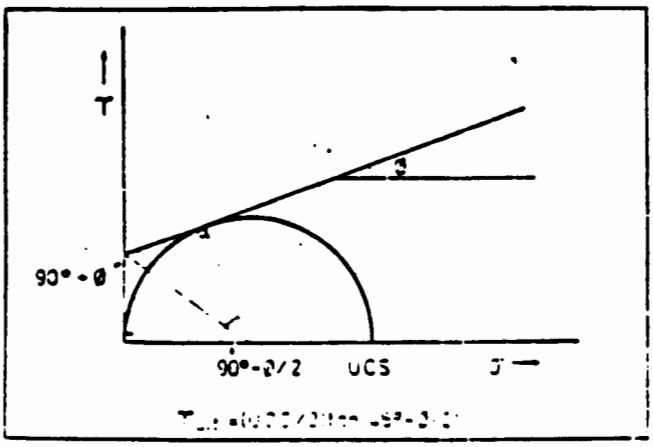

Fig. 4 Relationship between shear stress and uniaxial compressive strength

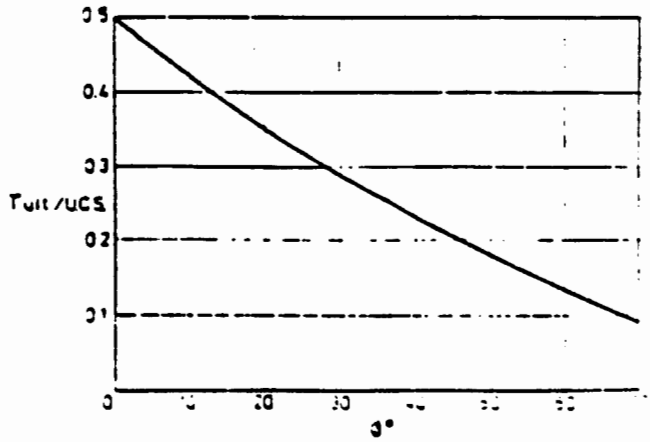

Fig. 5 Effect of Ø on τult/UCS ratio

[illegible] bond stress is therefore limited to 1.0N/mm².

In this connection it is noteworthy that Coates (1970) allows a maximum working bond of 2.45N/mm² but with a safety factor of 1.75 which indicates a value of τ_{ult} of 4.3N/mm². In some rocks, particularly granular, weathered varieties with a relatively low τ value, the assumption that τ_{ult} equals 10 per cent rock UCS may lead to an artifically low estimate of shear strength (Figs 4 and 5). In such cases the assumption that τ_{ult} equals 20-35 per cent UCS may be justified.

As a guide to specialists, bond values, as recommended throughout the world for wide range of igneous, metamorphic and sedimentary rocks, are presented in Table III. Where included, the factor of safety relates to the ultimate and working bond values, calculated assuming uniform bond distribution. It is common to find that the magnitude of bond is simply assessed by experienced engineers; the value adopted for working bond stress often lies in the range 0.35 to 1.4N/mm². Koch (1972) suggests bond stresses in this range for weak, medium and strong rock (Table III), and the Australian Code CA 35—1973 states that a value of 1.05N/mm² has been used in a wide range of igneous and sedimentary rocks, but confirms that site testing has permitted bond values of up to 2.1 N/mm² to be employed.

In this connection the draft Czech Standard (1974) concludes that since the estimation of bond magnitude and distribution is a complex problem, field anchor tests should always be conducted to confirm bond values in design, as there is no efficient or reliable alternative. Certainly, a common procedure amongst anchor designers is to arrive at estimates of permissible working bond values by factoring the value of the average ultimate bond calculated from test anchors, when available. Usually the recommended safety factor ranges from 2 to 3, but is frequently lower in very competent rocks, and higher in weaker, fissured, or weathered varieties.

The degree of weathering of the rock is

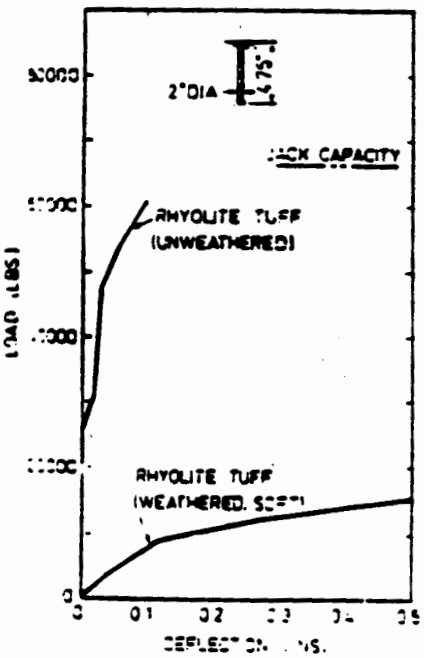

Fig 5 Effect of weathering at Currecanti Midway Transmission Line

(after Salmon and Schaefer, 1968)

TABLE III—ROCK/GROUT BOND VALUES WHICH HAVE BEEN RECOMMENDED FOR DESIGN

Rock type	Working bond (N/mm²)	Ultimate bond (N/mm²)	Factor of safety	Source
Igneous				
Medium hard basalt		5.73	3.4	India—Rao (1964)
Weathered granite		1.50-2.50		Japan—[illegible]
Basalt	1.21-1.38	3.46	[illegible]	Britain—[illegible]
Granite	1.38-1.55	4.83	[illegible]	Britain—[illegible]
Serpentine	0.45-0.59	1.55	[illegible]	Britain—[illegible]
Granite & basalt		1.72-3.10	[illegible]	USA—PCI (1974)
Metamorphic				
Manhattan schist	0.70	2.93	4.2	USA—[illegible]
Slate & hard shale		[illegible]	[illegible]	USA—PCI (1974)
Argillaceous sediments				
Limestone	1.00	2.53	[illegible]	Switzerland—[illegible]
Chalk—Grades I-III	0.01N/mm² (N = 5.9P/[illegible] blows 0.3m)	[illegible]	[illegible] (Temporary) [illegible] (Permanent)	Britain—[illegible]
Portsdown limestone	0.83-0.97	[illegible]	[illegible]	Britain—West-Hughes [illegible]
Chalk limestone	0.86-1.00	[illegible]	[illegible]	Britain—[illegible]
Bath limestone		[illegible]	[illegible]	USA—PCI (1974)
Dolomitic limestone		[illegible]	[illegible]	USA—PCI (1974)
Arenaceous sediments				
Hard coarse-grained sandstone	2.46		1.75	Canada—Coates (1970)
Weathered sandstone		0.69-0.86	3.0	New Zealand—[illegible]
Well cemented mudstones		0.69	2.0-2.5	New Zealand—[illegible]
Softer sandstone	0.40		3.0	Britain—Littlejohn (1973)
Softer sandstone (UCS > 2.0N/mm²)	0.60		3.0	Britain—Littlejohn (1973)
Hard fine sandstone	0.69-0.83	2.24	2.7-3.3	Britain—West-Hughes (1974)
Sandstone		0.83-1.73	[illegible]	USA—PCI (1974)
Argillaceous sediments				
Keuper marl		0.17-0.25 (0.46 C_u)	3.0	Britain—Littlejohn (1970) (C_u = undrained cohesion)
Weak shale		0.38		Canada—Golder Brawner (1970)
Soft sandstone & shale	0.10-0.14	0.37	2.7-3.7	Britain—West-Hughes (1974)
Soft shale		0.21-0.83	1.5-2.5	USA—PCI (1974)
General				
Competent rock (where UCS > 20N/mm²)	Uniaxial compressive strength—30 (up to a maximum value of 1.4N/mm²)	Uniaxial compressive strength—10 (up to a maximum value of 4.2N/mm²)	3	Britain—Littlejohn (1972)
Weak rock	0.35-0.70			Australia—Koch (1972)
Medium rock	0.70-1.05			
Strong rock	1.05-1.40			
Wide variety of igneous and metamorphic rocks	1.05		2	Australia—Standard CA35 (1973)
Wide variety of rocks	0.98			France—Fargeot (1972)
	0.50			Switzerland—Walther (1969)
	0.70			Switzerland—Comte (1945)
		1.20-2.50		Switzerland—Comte (1971)
	0.70		2.2.5 (Temporary) 3 Permanent)	Italy—Mascardi (1972)
	0.69	2.76	4	Canada—Golder Brawner (1970)
	1.4	4.2	3	USA—White (1973)
		15-20 per cent of grout crushing strength	3	Australia—Longworth (1971)
Concrete		[illegible]	[illegible]	USA—PCI (1974)

TABLE IV—ROCK/GROUT BOND VALUES WHICH HAVE SEEN EMPLOYED IN PRACTICE

Rock type	Working bond (N/mm²)	Test bond (N/mm²)	Ultimate bond (N/mm²)	τ_{ult} (test)	F_s (ultimate)	Source
Igneous						
Basalt	1.93		6.27		3.3	Britain—Parker (1958)
Basalt	1.10	3.80				USA—Eberhardt & Veltman (1965)
Tuff	0.80					France—Campefort (1966)
Basalt	0.63	0.72				Britain—Cementation (1962)
Granite	1.56	1.72				Britain—Cementation (1962)
Dolerite	1.56	1.72				Britain—Cementation (1962)
Very fissured felsite	1.56	1.72				Britain—Cementation (1962)
Very hard dolerite	1.56	1.72				Britain—Cementation (1962)
Hard granite	1.56	1.72				Britain—Cementation (1962)
Basalt & tuff	1.56	1.72				Britain—Cementation (1962)
Granodiorite	1.08					Britain—Cementation (1962)
Shattered basalt		1.01				USA—Salmon & Schaefer (1968)
Decomposed granite		1.24				USA—Salmon & Schaefer (1968)
Flow breccia		0.93				USA—Salmon & Schaefer (1968)
Mylonitised porphyrite	0.32-0.97					Switzerland—Descoeudres (1963)
Fractured diorite	0.95					Switzerland—Descoeudres (1963)
Granite	0.63	0.81				Canada—Barron et al (1971)
Metamorphic						
Schist	0.31					Switzerland—[illegible]
All types	1.20					Finland—[illegible]
Weathered fractured quartzite	1.56	1.72		1.1		Britain—Cementation (1962)
Blue schist	1.32	1.67		1.1		Britain—[illegible]
Weak meta sediments	1.10	1.23		1.1		Britain—Cementation [illegible]
Slate	0.43					Britain—Cementation [illegible]
Fine meta greywacke	1.57	1.73		1.1		Britain—Cementation [illegible]
[illegible]	0.35-0.39					[illegible]
Fractured quartzite	0.91					[illegible]
Weathered meta tuff		0.29				[illegible]
Greywacke	0.34					[illegible]
Quartzite	0.93-1.30	1.02-1.72		1.1		[illegible]
Microgneiss	0.35					[illegible]

Rock	Working bond (τ_{ll}/mm²)	Test bond (τ_{l}/mm²)	Ultimate bond (τ_{u}/mm²)	τ_{u}/τ_{l} (test)	τ_{u}/τ_{l} (ultimate)	Source
Metamorphic—contd.						
Sericite schist	0.05					Italy—Berardi (1972)
Quartzite schist	0.10					Italy—Berardi (1972)
Argillaceous & calcareous schist	0.63					Italy—Berardi (1972)
Slate	0.95	1.24		1.3		Switzerland—Meschler & Matt (1972)
Highly metased-iments	0.8[illegible]	1.08		1.3		USA—Bure (1972)
Slate & greywacke	1.08	1.40		1.3		Germany—Anon (1972)
Various metasediments			57			Germany—Abraham & Porzig (1973)
Micaceous schist & gneiss	0.53	0.83		1.5		USA—Nicholson Anchorage Co. Ltd (1973)
Slate	0.60	0.90	1.80	1.5	3.0	Britain—Littlejohn & Truman Davies (1974)
Sound Micaceous schist	1.74	2.16				USA—Feld & White (1974)
Micaceous schist	0.83-0.74			1.24		USA—Feld & White (1974)
Very poor gneiss & mud band	0.07					USA—Feld & White (1974)
Carbonate sediments						
Loamy limestone		0.63				Italy—Berardi (1980)
Fissured limestone & intercalations	1.08	1.19		1.1		Britain—Cementation Co. Ltd (1982)
Limestone	0.65					Switzerland—Muller (1986)
Poor limestone	0.32					France—Henneqiuin & Camberort (1966)
Massive limestone	0.38-0.78					France—Henneqiuin & Camberort (1966)
Karstic limestone	0.54					France—Henneqiuin & Camberort (1966)
Tertiary limestone	1.00		2.63		2.8	Switzerland—Losinger & Co. Ltd (1966)
Limestone			4.55-4.83			Switzerland—Ruttner (1966)
Marly limestone	0.03-0.07 (average) 0.21-0.38 (measured)					Italy—Berardi (1967)
Limestone			0.27			USA—Salman & Schaefer (1968)
Limestone	0.28					Italy—Berardi (1969)
Dolomitic limestone			1.80			Canada—Brown (1970)
Marly limestone	0.39-0.94					Italy—Berardi (1972)
Limestone	0.28					Italy—Berardi (1972)
Limestone	0.44					France—Soletanche Co. Ltd (1966)
puddingstone						
Limestone	1.18	1.42		1.2		USA—Bure (1972)
Chalk			0.70			Britain—Associated Tunnelling Co. Ltd (1973)
Dolomite		1.66				Canada—Golder Brawner (1973)
Dolomitic siltstone	0.43					USA—White (1973)
Limestone & marly bands	0.37	0.55		1.5		Italy—Mongilardi (1972)
Arenaceous sediments						
Sandstone	1.44	1.58		1.1		Britain—Morris & Garrett (1956)
Hard sandstone	1.42	1.56		1.1		Britain—Cementation Co. Ltd (1962)
Bunter sandstone	0.95	0.98		1.03		Britain—Cementation Co. Ltd (1962)
Sandstone	0.76	0.84		1.1		Britain—Cementation Co. Ltd (1962)
Sandstone	0.74					Czechoslovakia—Hobst (1968)
Sandstone	0.31	0.40	1.73	1.29	5.6	USA—Grosser (1970)
Sandstone	0.80					USA—Thompson (1970)
Poor sandstone	0.46					Germany—Brunner (1970)
Good sandstone	1.14					Germany—Brunner (1970)
Sandstone & Breccia	0.38					France—Soletanche (1968)
Sandstone		0.95				Australia—Williams et al. (1972)
Bunter sandstone	0.60	1.20		2.0		Britain—Littlejohn (1973)
Sandstone	1.17					Australia—McLeod & Hoadley (1974)
Argillaceous sediments						
Shale	0.62					Canada—Juergens (1968)
Marl	0.10	0.28		2.8		Italy—Berardi (1967)
Shale	0.30		0.63		2.1	Canada—Henne & Seaton (1967)
Very weathered shale			0.38			USA—Salman & Schaefer (1968)
Shale	0.13-0.24					USA—Kerisel (1970)
Grey siltstone	0.62					Britain—Universal Anchorage Co. Ltd (1972)
Clay marl	0.14-0.24	0.21-0.38		1.5		Germany—Schwarz (1972)
Shale	0.52					Canada—McRoberts et al. (1972)
Argillite	0.82					Canada—Golder Brawner (1973)
Mudstone	0.63	0.88		1.4		Australia—McLeod & Hoadley (1974)
Miscellaneous						
Bedded sandstone & shale	0.25-0.50					Italy—Beemonte (1981)
Porous, sound gossamer	1.57	1.72		1.1		Britain—Cementation Co. Ltd (1962)
Shale & sandstone	0.07	0.10		1.5		USA—Reti (1964)
Soft rocks	0.75					Sweden—Nordin (1966)
Sandstone & shale	1.82					Poland—Bulak et al. (1967)
Siltstone & mudstone	1.65					Australia—Maddes et al. (1967)
Fractured rock (75 per cent shale)						
Poor			0.24			USA—Salman & Schaefer (1968)
Average			0.38			USA—Salman & Schaefer (1968)
Good			0.75			USA—Salman & Schaefer (1968)
Limestone & clay breccia	0.20-0.25					Italy—Berardi (1972)

a major factor which affects not only the ultimate bond but also the load-deflection characteristics. Fig. 6 shows the results produced by test anchors in rhyolite tuff, of both sound and weathered varieties. No data are provided on grout or rock strengths but it is significant that the equivalent uniform bond stress at maximum jack capacity is scarcely 0.1N/mm². For design in soft or weathered rocks there are signs that the standard penetration test is being further exploited. For example, Suzuki et al (1972) state that for weathered granite the magnitude of the bond can be determined from the equation

$$\tau_{ult} = 0.007N + 0.12 \text{ (N/mm}^2\text{)}$$

where N = number of blows per 0.3m ..... (2)

Similarly, Littlejohn (1970) illustrates a correlation between N and ultimate bond for stiff hard chalk as follows

$$\tau_{ult} = 0.01N \text{ (N/mm}^2\text{)} \qquad (3)$$

In grades III, II and I of chalk he observed a range of τ_{ult} of 0.21—1.07N/mm² based on test anchors pulled to failure.

Although it would appear from evidence presented in subsequent sections that the assumptions made in relation to uniform bond distribution are not strictly accurate, it is noteworthy that few failures are encountered at the rock grout interface and new designs are often based on the successful completion of former projects that is, former working bond values are re-employed or slightly modified depending on the judgement of the designer.

Table IV contains data abstracted from reports of rock anchor contracts throughout the world. In addition to the working, test, and ultimate bond values, the measured and designed safety factors are provided where available. In certain cases the fixed anchor diameter has been inferred, to facilitate analysis of the data, as published.

It will be noted that, even for one rock type, the magnitude of bond used in practice is extremely variable. There are many reasons for this, the most important of which are:

(*i*) Different designers use different bond values and safety factors, which may be related to type of anchor and extent of the anchor testing programme.

(*ii*) "Standard" values for a certain rock type have often been modified to reflect local peculiarities or irregularities of the geology.

(*iii*) Factors related to the construction techniques e.g. drilling method, flushing procedure, and grout pressure will influence the results obtained. (The effect of these aspects will be discussed in Part 2—Construction.)

On the whole however, it would appear that the bond values employed are to a degree consistent with rock type and competency.

Fixed anchor dimensions

The recommendations made by various engineers with respect to length of fixed anchor are presented in Table V. Under certain conditions it is recognised that much shorter lengths would suffice, even after the application of a generous factor of safety. However, for a very short anchor the effect of any sudden drop in rock quality along the anchorage zone, and of constructional errors or inefficiencies, could induce a serious decrease in that anchor's capacity.

With regard to the choice of anchor diameter several considerations may be taken into account:

(*i*) Type and size of tendon.

(*ii*) The relation of diameter to perimeter area of fixed anchor and hence the anchor capacity, assuming uniform bond.

(*iii*) Ratio of steel area to cross-sectional area of borehole for efficient bond distribution and corrosion protection.

(*iv*) Drilling method and rig to be used, and

(*v*) Nature of rock in the anchor zone and presence of unconsolidated overburden, if any.

The authors find from a survey of several hundred commercial anchor tests that no direct relationship may be observed bearing in mind the range of anchor types, but that most anchors do follow the trend indicated in Table VI.

RECOMMENDATIONS

3.2.3 Factors of Safety

The design load P for the anchor equals the maximum anticipated load applied to the anchor times the factor of safety used in the design of the anchored structure. The value of the factor of safety depends upon the type of application, the degree of uncertainty in the design of the structure and the risk.

COMMENTARY

The engineer should not compound various factors of safety when designing an anchored structure. The uncertainty and risk associated with the work should be the primary criteria to determine the design load (including the factor of safety) and the engineer acts as a separate factor of safety in the added loads the design load (including the factor of safety) and the anchor test load (the actual factor of safety) for the anchor will have been compounded resulting in an overly conservative design.

3.2.4 Anchor Tendon Design

The tendon size is determined such that the design load for the anchor does not exceed 60 percent of the guaranteed ultimate tensile strength (GUTS) of the tendon. The lock off load, which shall be determined by the design engineer, may be larger or smaller than the design load. The recommendations for corrosion protection given in Section 6.0 "Corrosion Protection" shall be considered.

The load in an anchor tendon may either increase or decrease with time depending on the behavior of the structure.

3.2.5 Free Stressing Length

The free stressing length should not be less than 15 feet (4.572m).

The minimum stressing length recommended is to prevent significant reductions in transfer load due to stressing anchorage losses or movement.

3.2.6 Bond Length

The bond length can be estimated by the following equation:

$$Lb = \frac{P}{\pi \cdot d \cdot \tau_w}$$

Where:

- Lb = bond length
- P = design load for the anchor
- π = 3.14
- d = diameter of the drill hole
- τ_w = working bond stress in the interface between rock and grout

The bond length normally is not less than 10 feet. For normal applications the bond between the tendon and anchor grout is not critical.

Pull-out tests may be used to determine the ultimate in situ bond stress between the rock and the anchor grout. Pull-out tests usually require that the tendon capacity be increased or the bond length reduced in order to fail the anchor. Pull-out test should not be required if the anchors are tested as described in Section 3.7.

The working bond stress used to determine the bond length is normally 25 to 50 percent of the ultimate bond stress.

When selecting the working bond stress the engineer should consider the critical nature of the anchor application, variations in the rock properties and the installation procedures.

The ultimate bond stress depends on the:

1. Shear strength of the rock
2. Discontinuities in the rock mass

"British Standard Code of practice for Ground Anchorages" British Standards Institute, 1989

Table 25. Rock/grout bond values which have been recommended for design (after Littlejohn and Bruce [illegible])

Rock type	Working bond	Ultimate bond	Load factor of safety	Source
	N/mm²	N/mm²		
Igneous				
Medium hard basalt		5.73	3.4	India Rao (1964)
Weathered granite		1.50 to 2.50		Japan Suzuki et al. (1972)
Basalt	1.21 to 1.39	3.86	2.9 to 3.2	Britain Wycliffe-Jones (1974)
Granite	1.38 to 1.55	4.83	3.1 to 3.5	Britain Wycliffe-Jones (1974)
Serpentine	0.45 to 0.59	1.55	2.6 to 3.5	Britain Wycliffe-Jones (1974)
Granite and basalt		1.72 to 3.10	1.5 to 2.5	USA PCI (1974)
Metamorphic				
Manhattan schist	0.70	2.80	4.0	USA White (1973)
Slate and hard shale		0.83 to 1.38	1.5 to 2.5	USA PCI (1974)
Calcareous sediments				
Limestone	1.00	2.83	2.8	Switzerland Lozinger (1966)
Chalk – Grades I–III	0.01 N (N = SPT no. blows/0.3 m)	0.22 to 1.07	1.5 to 2.0 (Temporary) 3.0 to 4.0 (Permanent)	Britain Littlejohn (1970)
Tertiary limestone	0.83 to 0.97	2.76	2.9 to 3.3	Britain Wycliffe-Jones (1974)
Chalk limestone	0.88 to 1.00	2.76	2.8 to 3.2	Britain Wycliffe-Jones (1974)
Soft limestone		1.03 to 1.52	1.5 to 2.5	USA PCI (1974)
Dolomitic limestone		1.38 to 2.07	1.5 to 2.5	USA PCI (1974)
Arenaceous sediments				
Hard coarse-grained sandstone	2.45		1.75	Canada Coates (1970)
Weathered sandstone		0.69 to 0.85	3.0	New Zealand Irwin (1971)
Well-cemented mudstones		0.69	2.0 to 2.5	New Zealand Irwin (1971)
Bunter sandstone	0.40		3.0	Britain Littlejohn (1973)
Bunter sandstone	0.60		3.0	Britain Littlejohn (1973)
(Ultimate compressive stress > 2.0 N/mm²)				
Hard fine sandstone	0.69 to 0.83	2.24	2.7 to 3.3	Britain Wycliffe-Jones (1974)
Sandstone		0.83 to 1.73	1.5 to 2.5	USA PCI (1974)
Argillaceous sediments				
Keuper marl		0.17 to 0.25 (0.45 C_u)	3.0	Britain Littlejohn (1970) (C_u = undrained cohesion)
Weak shale		0.35		Canada Golder Brawner (1973)
Soft sandstone and shale	0.10 to 0.14	0.37	[illegible] to 3.7	Britain Wycliffe-Jones (1974)
Soft shale		0.21 to 0.83	1.5 to 2.5	USA PCI (1974)
General				
Competent rock (where ultimate compressive stress > 20 N/mm²)	Uniaxial compressive strength ÷ 10 (up to a maximum value of 1.4 N/mm²)	Uniaxial compressive strength ÷ 10 (up to a maximum value of 4.2 N/mm²)	3	Britain Littlejohn (1972)
Weak rock	0.35 to 0.70	–	–	Australia Koch (1972)
Medium rock	0.70 to 1.05			
Strong rock	1.05 to 1.40			
Wide variety of igneous and metamorphic rocks	1.05		2	Australia Standard CA35 (1973)

Table 25. concluded

Rock type	Working bond	Ultimate bond	Load factor of safety	Source
	N/mm²	N/mm²		
Wide variety of rocks	0.98			France (Fargeot 1972)
	0.50			Switzerland (Wetter 1969)
	0.70			Switzerland (Gamre 1965)
		1.20 to 2.50		Switzerland (Gamre 1971)
	10		2 to 2.5 (Temporary) 3 (Permanent)	Italy (Mascardi 1972)
	0.69	2.76	4	Canada (Golder Brawner 1973)
	1.4	4.2	3	USA (White 1973)
		15 % to 20 % of grout crushing strength	3	Australia (Langworth 1971)
Concrete		1.38 to 2.76	1.5 to 2.5	USA (PCI 1974)

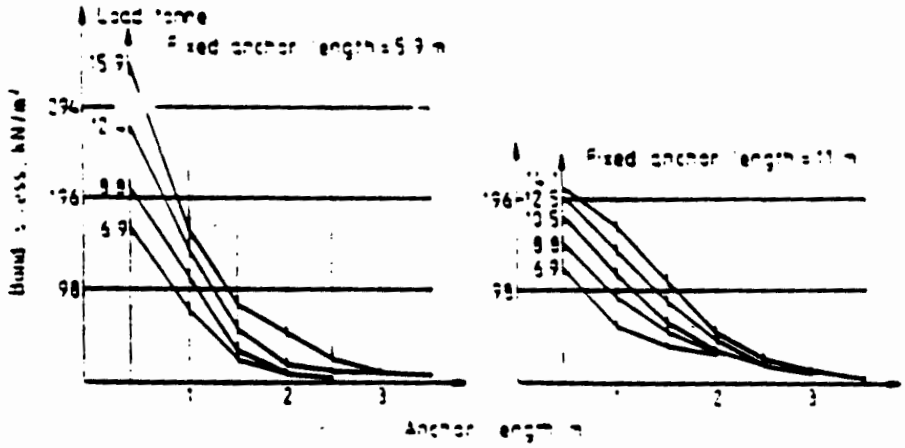

Figure 56. Distribution of bond along fixed anchor length (after Berardi 1967)

For compact sandy gravel ($\phi' = 40°$) at Vauxhall Bridge, London and compact dune sand ($\phi' = 35°$) at Ardeer, Scotland, values of β equal to 101 and 31 have been measured in the field (Littlejohn 1970) which are in good agreement with respective values of 99 and 35 estimated from figure 6.

The value of A depends to a large extent on construction technique and for the type B anchorage relevant to equation (3) values of 1.7 and 1.4 have been recorded in compact sandy gravel ($\phi' = 40°$) and compact dune sand ($\phi' = 35°$) respectively (Littlejohn 1970).

Littlejohn (1970) estimated for equation (3) that A lay in the range of 1 to 2, but that if the soil was not compacted or displaced during the casing installation and no residual grout pressure was left at the fixed anchor grout/soil interface on completion of the injection stage A might reduce to a value approximating to K_a. In the light of experience, this reduction is now considered unduly pessimistic since even with tremie grouting, the full hydrostatic head of the grout is applied at the fixed anchor interface, which creates a contact pressure greater than $K_a \gamma z$ in normally consolidated ground.

As a consequence, even for the tremie grouting method it is difficult to envisage a value of A less than 1 for design purposes.

WESTINGHOUSE WASTE ISOLATION DIVISION

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PROJECT	AUTHOR / DATE: C. FRANKE 9/5/91	CHK'D BY / DATE: J.R. Tyler 9/6/91	CHK'D BY / DATE
S.O.	CALC NO	FILE NO	GROUP

PROBLEM: DETERMINE THE FACTOR OF SAFETY (F of S) AGAINST PULLOUT OF A ROCK WEDGE BY A ROCKBOLT.

ASSUMPTIONS:

1.) THE GEOMETRY OF THE WEDGE IS AS SHOWN ON THE SKETCH ON PAGE 2.
2.) COHESION OF SALT IS $c = 1000 psi = 6.89$ MPa (REF 1)
3.) DENSITY OF WIPP SALT, $\rho = 135 lb/ft^3$ (REF 2)

REFERENCES:

1.) SAND 78-1596, "Geological Characterization Report", Powers, et al, 1978.
2.) D-51-W-01, "Design Basis - Underground Excavations" Bechtel

METHOD: THE SHEAR RESISTANCE OF THE WEDGE ABOVE CLAY G AND THE WEIGHT OF THE WEDGE ARE COMPARED.

THE SHEAR STRENGTH OF SALT WILL BE TAKEN AS EQUAL TO THE COHESION. THIS IS A VERY CONSERVATIVE VALUE SINCE

$$T = c + \sigma \tan\phi \qquad \text{(REF 1)}$$

WHERE T IS THE SHEAR STRENGTH, σ IS NORMAL FORCE (i.e. UCS for MAXIMUM) AND ϕ IS THE FRICTION ANGLE.

General Comment: I don't know that this calculation is significant to a bolt in salt.

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WESTINGHOUSE WASTE ISOLATION DIVISION

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	C. FRANCKE 7/26/91	JR Tyler 9/4/91		
BU	CALC NO	FILE NO	GROUP	

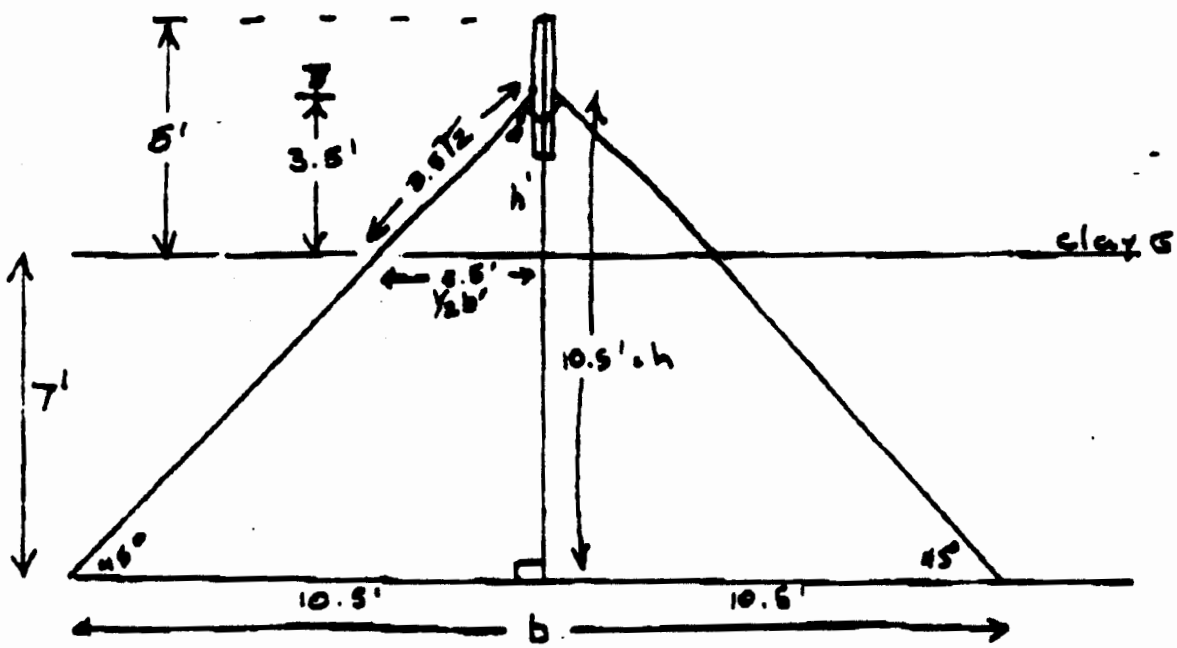

FIG. 1

W = WIDTH = 33'

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WESTINGHOUSE WASTE ISOLATION DIVISION

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W.O.	CALC NO	FILE NO	GROUP	

CALCULATIONS:

WEIGHT OF WEDGE

$$W = \tfrac{1}{2} b h w \rho$$

WHERE W = WEIGHT OF WEDGE
ρ = DENSITY OF SALT
b, h, w ARE BY FIGURE 1.

$$W = \tfrac{1}{2}(21')(10.5')(35')(135\ lb/ft^3)$$

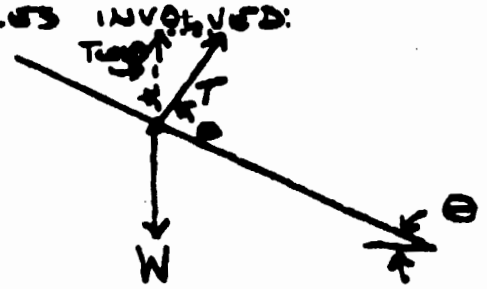
$$W = 491\ kip = 2.18\ MN$$

SHEAR RESISTANCE OF SMALL WEDGE

$$S = AT\cos\theta$$
$$= [\tfrac{1}{2} b'h' \cdot 2 + 2 \cdot \sqrt{2}\, h'w]\, T\cos\theta$$

WHERE: S = SHEAR RESISTANCE OF SMALL WEDGE
T = SHEAR STRENGTH OF ROCK
b', h', w ARE BY FIGURE 1
θ = ANGLE OF WEDGE FRACTURE

THE DIAGRAM BELOW ILLUSTRATES THE FORCES INVOLVED:



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CALCULATIONS: (CONT'D)

SHEAR RESISTANCE OF WEDGE (CONT'D)

$$S = [\tfrac{1}{2}(7')(3.5')(2) + (2)(\sqrt{2})(3.5')(33')](T\cos\theta)$$

$$S = (351\ ft^2)\left(\frac{144\ in^2}{1\ ft^2}\right)(1000\ psi)\cos\theta$$

$$S = (50{,}544\ kip)\cos\theta$$

$$\tan\theta = \frac{7'}{\frac{1}{2}(33')}$$

$$\theta = 23°$$

$$S = (50{,}544\ kip)\cos 23°$$

$$S = 46{,}530\ kip$$

FACTOR OF SAFETY

$$F_{of}S = \frac{46{,}530\ kip}{491\ kip} = 95$$

DISCUSSION:

THE CALCULATED F of S IS QUITE ADEQUATE

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"Geological Characterization Report, Waste Isolation Pilot Plant (WIPP) Site, Southeastern New Mexico" SAND78-1596 1978 Rogers, et al.

stress space (σ,T) where σ and T denote normal and shear stresses, and an envelope is drawn tangent to the circles representing the ultimate shear stress of any value of confining pressure.

When the data from Tables 9.2.4-1 and 9.2.4-2 are plotted, three Mohr's circles which are normal for SENM rock salt are obtained in Figure 9.2.4-2A in stress space (σ,T) . The ultimate stresses can be approximated by a straight line (Coulomb) envelope of the form $T = C + \sigma \tan \phi$. In conventional engineering terminology, C is called the cohesion and ϕ , the angle of internal friction. In this case, at ambient temperatures, rock salt from the 2,700 foot level has an apparent cohesion of approximately 1,000 psi and an angle of internal friction of 33°. Similar data for other rocks are being used for mine pillar design. However, it should be recognized that the validity of these ultimate stress analyses rests on two assumptions: (1) failure is independent of the intermediate principal stress, and (2) failure is defined solely in terms of stresses and independent of strain, strain rate and time. Both of these assumptions are currently being evaluated for rock salt.

In contrast to other rocks, it is important to remember that rock salt undergoes large deformations long before the ultimate stress is reached. Since these deformations can exceed 15% even at ambient temperature, it is conceivable that a practial failure condition might incorporate a maximum deformation criterion. To illustrate this case, a Coulomb envelope was constructed (Figure 9.2.4-2B) which defines the stress magnitudes at an arbitrarily chosen constant value of strain (ε_1 = 2.5%). This value is the average strain at the ultimate stress of samples tested in uniaxial compression at ambient temperature and a loading rate of 30 psi/min. It can be seen that Figure 9.2.4-2B is different from the ultimate stress envelope in Figure 9.2.4-2A. Clearly, the shapes of the Mohr envelopes are highly dependent on failure criteria. The values obtained also depend on the manner in which the Mohr's envelope is drawn. In Figure 9.2.4-2A, a "best fit" straight line tangent to the circles was drawn; while in B, a parabola was drawn tangent to the circles.

DRAFT 09/10/91

WASTE ISOLATION PILOT PLANT
SUPPLEMENTARY ROOF SUPPORT SYSTEM

APPENDIX C

Support System Design

Channel Support Design Detail

Westinghouse Electric Corporation
Waste Isolation Division
Carlsbad, New Mexico
August 1991

APPENDIX C - CHANNEL SUPPORT DESIGN

The final design of the steel beam support system was the result of an evolutionary process that started by considering an I beam. The original concept called for an I beam that would be held in place by eight rock anchors and four yieldable steel posts. This design had several difficulties with it:

- Each I beam would weigh about 2,000 pounds, making the installation process difficult and potentially dangerous.
- The supporting rock anchors could not be attached to the centerline of the I beam. Instead, the rock anchors would have to be attached by means of a separate plate to the flanges of the I beam. This would have generated excess moments in the beam as well as introducing torque forces, which would have been difficult to calculate.
- The yieldable posts would have been difficult to test, and as they were not performing any function that the rock anchors could not provide, they were eliminated from the design.

The final design calls for a 15 x 40 channel with 11 rock anchors that are fastened through the centerline of the channel.

The channel will be made of three 9 foot sections bolted together with four 7.5 inch by 3 inch splice plates which allow for greater ease in the handling and placement of the channel.

The beam has been designed to accommodate the unequal distribution of the rock load, and the rock anchors will be tensioned to account for the fact that most of the detached load is in the middle of the room.

The support channel design calculations are given below.

WESTINGHOUSE WASTE ISOLATION DIVISION
CALCULATION COVER SHEET

System: AU00	Design Class: III-A.
PWR: N/A	Sheet No.: 1 of 9
	Ref. EDT: N/A

FSAR Checked: ✓ FSAR Change Required: yes

DOE Order 6430.1A Checked: ✓

Source of Data:
1. Geotech. Expert Report - WIPP, June 1991
2. —
3. —

Source of Formulas and References: AISC - Manual of Steel Construction
Blodget - Design of Weldments

Type of Calculation: Prelim.: _____ Final: ✓

References:	P&ID/Dwg. No./Rev.:	Entitled:
Westinghouse Drawings	54-D003-W1 -W2 -W3 -W4 -W5	Supplementary Roof Support, Room 1, Panel 1 General Arrangement, Sections and Details

RECORD OF ISSUES

Rev.	Description	By	Date	Chk'd	Date	App'd	Date
0	Issued for external review	JMK	9/5/91	HG	9/6/91	MJP	9/6/91

4918:0230

TITLE U/G. Room 1 - Roof Supp. System			PAGE 2 OF 9
PROJECT	AUTHOR JMX 8/29/91	CHK'D. BY HS 9/6/91	CHK'D. BY DATE
S.O.	CALC. NO. 6-91-JMX	FILE NO.	GROUP M&C.E.

Data and assumptions for the below calculations:

1. Density of salt material is 135 lb/ft^3 as per Design Basis - U/G Excavations, D-51-W-01, page 6.
2. The total load overburden is based on 10 ft spacing of the rows of Rock Bolts in the longitudinal direction of the room.
3. Triangular distribution of the load is assumed across the width of the room (33'-0).
4. Material of the Rock Bolts is as per ASTM A 615 - Grade 60. Yield stress Fy = 60 000 psi, allowable working

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PROJECT	AUTHOR JHX 8/20/91	CHK'D. BY HS 9/6/91	CHK'D. BY DATE
S.O.	CALC. NO. G-91-JHX	FILE NO.	GROUP H&C.E.

stress in tension $F_t = 0.60 F_y$ =

= 36000 psi. (See PTI-1986).

5. Allowable load on the Rock Bolt is

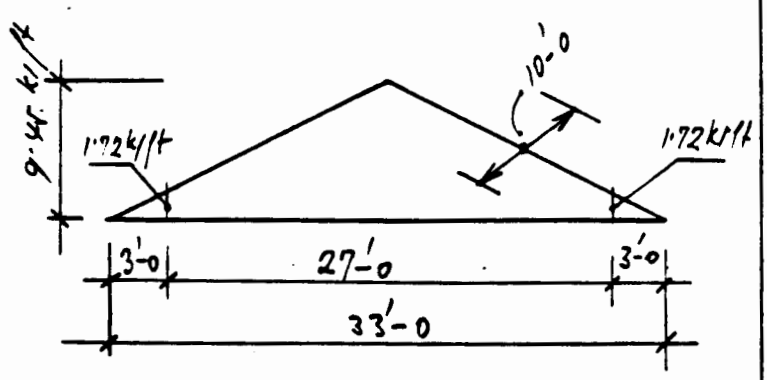
$$P_W = A \times F_t = 0.79 \times 36.0 = \underline{28.4^k}$$

Triangular Load:

Weight of the overburden

$$w = (\tfrac{1}{2}\, 10.0 \times 33.0 \times 7.0) \times 135\ lb/ft^3 = \underline{156\,000\ lb.}$$

Load per linear foot across the room $q = \dfrac{2 \times 156.0}{33.0} = \underline{9.45\ k/ft}$



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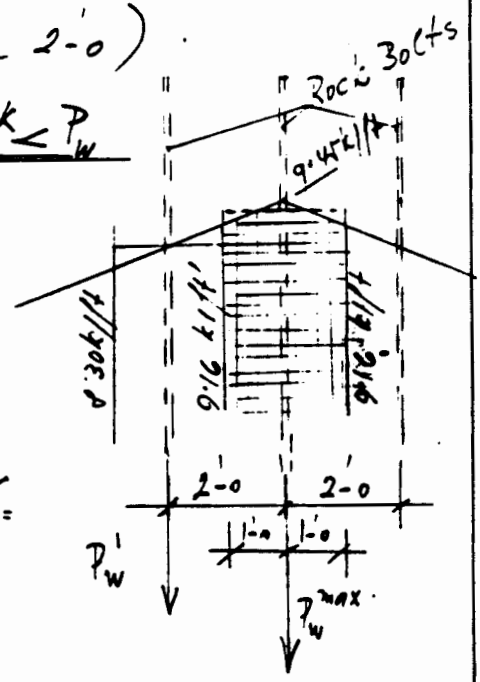
Load on the critical Rock Bolt @ center of the room: (spacing @ 2'-0)

$P_W^{max} = 2'0 \times 9.16 = \underline{18.33^k < P_W}$



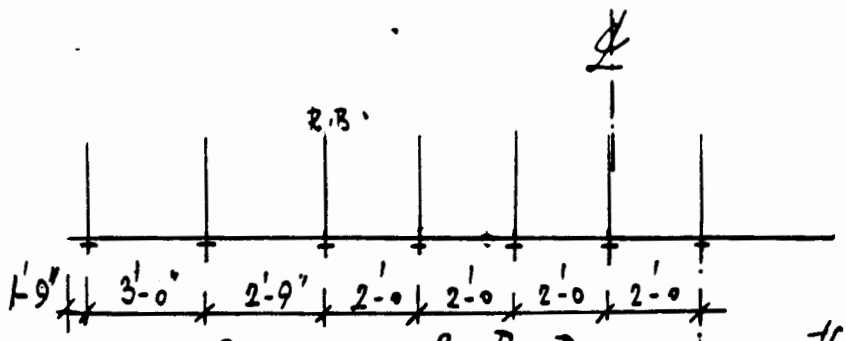
Adjacent Rock Bolt:

$P_W' = 2'0 \times 8.30 = \underline{16.6^k}$

In case the center R.B. fails half of the load i.e. $\frac{1}{2}$ 18.33^k = 9.16^k will be taken by the adjacent R.B.

$^2P_W = 16.6 + 9.16 = \underline{25.76^k < P_W = 28.4^k}$ - see page 3

$\Rightarrow$ no overloading occurs.



Spacing of R.Bs. across the room

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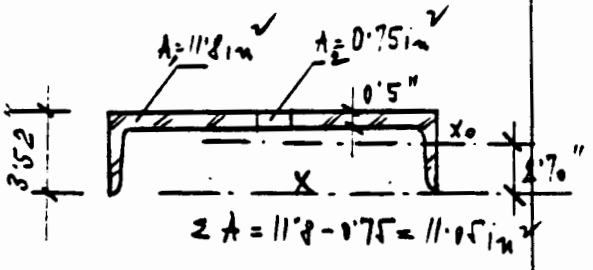
Check of stress in the channel from bending. ([15x40; $S = 3.37 in^3$).

Assume conservatively $M = 1/10\, w l^2$

$M_{\ell} = 1/10\ 8.9 \times 2.0^2 = 3.56$ k/ft ~ @ center of the room

$$\sigma = \frac{3560 \times 12}{3.37} = 12676 \text{ psi} < \sigma_{allow}$$

<u>Section modulus "S" of</u> the [15x40 with a ϕ 1.5" hole.



Static moment M_x:

$11.8 \times 2.74 \ldots\ldots 32.33 in^3$

$- 0.75 \times 3.27 \ldots\ldots - 2.45$

$M_x = 29.88 in^3$

$$X_0 = \frac{M_x}{A_0} = \frac{29.88}{11.05} = 2.70 \text{ in}$$

$$I = I_0 + A_1 x_1 - I_\phi - A_2 x_2 = 9.23 + 11.8 \times 0.043^2 -$$

$$- 1/12 \times 1.5 \times 0.5^3 - 0.75 \times 0.82^2 = \underline{\underline{8.71 \text{ in}^4}}$$

$$S = \frac{I}{C} = \frac{8.71}{2.70} = \underline{\underline{3.21 \text{ in}^3}}$$

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Check new stress in the [15 (see page 5)

$M = 3560\ k/ft \Rightarrow \sigma = \frac{3560 \times 12}{3.21} = 13308\ psi < \sigma_{allow}$

Shear in the web:

$P_w^{max.} = 18.33^k$

$A_{req.} = \frac{18330}{14000} = 1.31\ in^2$

$r_{req.} = \frac{1.31}{0.5 \times 2\pi} = 0.42\ in \Rightarrow r_{used} \sim 1.00 > 0.42\ in.$

Use $\phi 2''$ washer under the rock bolt (see below $2\ 5\frac{1}{2} \times 5\frac{1}{2}$)

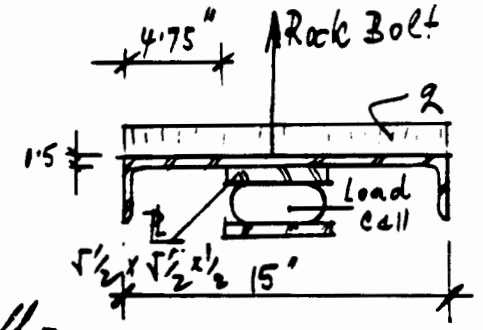
Bending of the [15 perpendicular to the length of the beam @ support

$q = \frac{2P_w}{15'' \times 24''} = \frac{28400}{15 \times 24} =$

$\doteq 78.9\ psi$

$M = \frac{1}{2} 78.9 \times 4.75^2 = 890\ lb.in$

$\sigma = \frac{890}{1.042} = 21192\ psi \doteq \sigma_{allow}.$



$S = \frac{1}{6} 1.0 \times 0.5^2 = 1.042\ in^3$

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Shear (in the 1/4" web) (along the ½ perimeter)

$A = 4 \times 5.5 \times 0.5 = 11 \text{ in}^2$

$\tau = \frac{28400.0}{11.0} = \underline{2582 \text{ psi}} < \sigma_{allow} = 14000 \text{ psi}$

Combined stress: (bending + shear)

$$\left(\frac{f_b}{F_{cr}}\right)^2 + \left(\frac{\tau}{\tau_{cr}}\right)^2 \leq 1$$

$$\left(\frac{21.19}{23.76}\right)^2 + \left(\frac{2.58}{14.0}\right)^2 = \underline{0.725 + 0.033 = 0.828 < 1.0}$$

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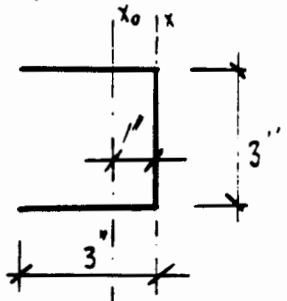
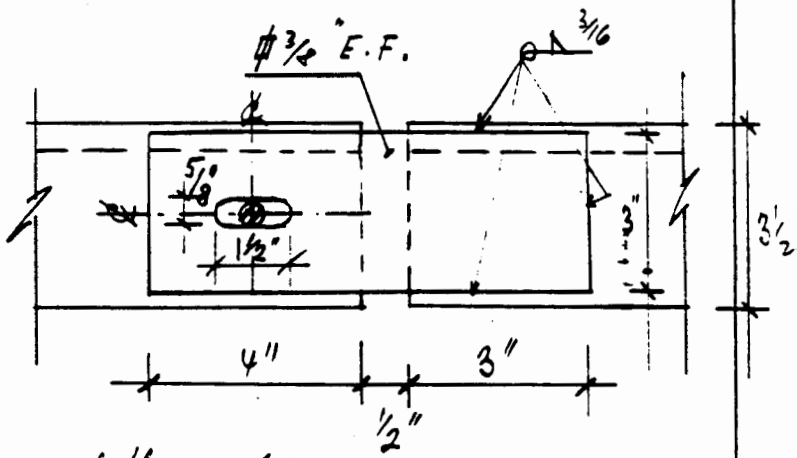
Design of splice:

- assumed as a flexible pin, allowing small horizontal movement

C.G. of the weld:





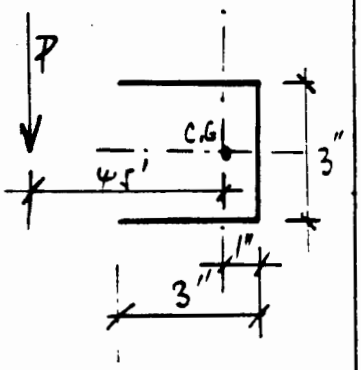
$S_{M_x} = 2(3 \times 1.5) = 9 in$

$L = 3 \times 3 = 9 in$

$X_0 = \frac{9.0}{9.0} = 1''$

Design of the weld:

See table XXIII, page 4-90 AISC.



$k = 1.0 \quad a = 1.5$

$l = 3'' \quad P = 5.66^k$

$C = 0.775$

$$D = \frac{P}{C\,C_1\,l} = \frac{5.66^k}{0.775 \times 1.0 \times 3.0} = 2.43$$

Use 3/16 weld

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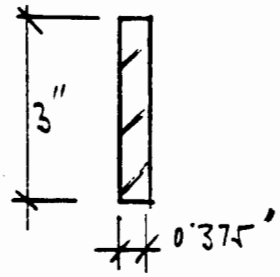
Splice plate - stresses from bending and shear

Assume ≥ 3/8" thick



Moment:

$M = \frac{1}{2} \times 5.66 \times 2.5 = 7075$ in-lb (one plate)

$\sigma = \frac{7075.0}{0.562} = 12589$ psi => OK

$S = \frac{1}{6} 0.375 \times 3.0^2 = 0.562 \text{ in}^3$

Shear:

$\tau = \frac{\frac{1}{2} 5660}{3 \times 0.375} = 2515$ psi < 14000

Combined stress:

$\left(\frac{12589}{23760}\right)^2 + \left(\frac{2515}{14000}\right)^2 = 0.28 + 0.03 = 0.31 < 1.0 \Rightarrow$ OK

Bolt at the sliding end.

$\tau = \frac{\frac{1}{2} 5.66}{0.306} = 9.25$ ksi

Assume ϕ 5/8"

$A = 3.14 \times 0.31^2 = 0.306 \text{ in}^2$

Use Bolts A 325 ($F_v = 17.5$ ksi)

Finger tight.

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APPENDIX D

DRAFT 09/10/91

WASTE ISOLATION PILOT PLANT
SUPPLEMENTARY ROOF SUPPORT SYSTEM

APPENDIX D

Geomechanical Monitoring Program

Westinghouse Electric Corporation
Waste Isolation Division
Carlsbad, New Mexico
August 1991

GEOTECHNICAL MONITORING PROGRAM

1.0 INTRODUCTION

A system to support the roof in Room 1, Panel 1 has been designed on the basis of the rock mechanics data that is given in the Geotechnical Design Summary Report (Westinghouse, 1991a). The design itself, is presented in the Design Report for the Supplementary Roof Support System, Underground Storage Area, Panel 1, Room 1 (Westinghouse, 1991b). The support system accommodates a controlled yield of the roof rock as the creep of the salt takes place. The success of the planned system relies heavily on a monitoring program that will determine not only the geomechanical performance of the room but will also assess the structural performance of the support system.

The support system is designed to carry the dead weight of a rock wedge that is forming in the roof of Room 1. The development of this wedge has been established from the rock fall that occurred in SPDV Test Room 1 and from observations of fracture development in other parts of the underground facility. The wedge is not yet fully formed but experience in Room 1, Panel 1 would indicate that it will form within the next 7 years unless fracture development in the roof can be controlled. The support system in Room 1 has two purposes. It is design to minimize the development and propagation of roof fractures thereby, ensuring that the rock remains self supporting for as long as possible, and secondly the system must have an capacity to carry the dead weight of the rock wedge once it forms in the roof while accommodating both vertical and lateral displacements due to far field creep effects.

The geotechnical monitoring program will establish the loads that are developing in the support and the deformations of the rock that are taking place around the room. The geotechnical data will be used to ensure that the support system is performing in a controlled manner and to establish the load adjustments required to the support system in order to accommodate the creep movements of the salt, and to confirm that room performance remains within satisfactory bounds.

This plan describes the geotechnical monitoring program that will be implemented to evaluate both the room performance and the performance of the support system. The plan describes the instrumentation that will be installed in the room, and it discusses the criteria that will be applied to ensure that the support system is adjusted in a controlled manner and that room performance remains satisfactory. It should be noted that as more data becomes available, especially on the interaction of the support with the room, then the criteria that are the basis for adjusting the loads on the support may require modification.

The plan has been developed and will be implemented in accordance with the general requirements for the control of test activities as described in the Geotechnical Engineering Program Plan (Westinghouse, 1991c). They cover the eighteen criteria that are defined in the Quality Assurance Program for Nuclear Facilities (ANSI/ASME NQA-1-1986). The Geotechnical Engineering Program Plan supports the Quality Assurance Program implemented at the site (Westinghouse, 1990).

The geomechanical monitoring of a room can give indications of its deterioration. Monitoring of the performance of excavations at the WIPP has already provided early identification of such conditions. SPDV Test Room 1 showed evidence of worsening conditions at least 3 years prior to the roof failure in that room. In addition, the Geotechnical Expert Panel has expressed confidence that instrumentation in Panel 1 can give adequate notification of deteriorating conditions (US DOE, 1991).

2.0 MONITORING OF ROOM PERFORMANCE

The program for monitoring room performance has already been developed and implemented in Room 1, Panel 1. The basis for the monitoring is that the measurements and observations are simple to make; That minimal maintenance of instrumentation is required, that instrumentation is easily replaced if it malfunctions; and that conditions throughout the room are known. The data should also provide data on geomechanical performance features that have been identified elsewhere in the underground facility, especially in the SPDV Test Rooms as features that should be to give good comparison with other data collected at the site. Room performance is being characterized from the following:

- the development of bed separations and lateral shifts at the interfaces of the salt and the clays underlying the anhydrites "a" and "b".
- the establishment of the room closure rates and determine if they are accelerating with time or exceeding expected rates.
- the assessment of the behavior of the pillars.
- the assessment of fracture development in the roof and floor.

The instrumentation in Room 1, Panel 1, was upgraded during the summer of 1991 from the original monitoring program established for the panel in 1988. At that time, limitations were imposed on penetrations through the anhydrite "b" in the roof. These limitations were in effect for waste storage considerations and no longer apply to the use of the room as the location for the bin scale tests. Roof conditions are now assessed from observation boreholes and from extensometer measurements. Measurements of room closure, rock displacements and observations of fracture development in the immediate roof beam can now be made and used to evaluate the performance of the room. The upgraded monitoring program was presented to the Geotechnical Expert Panel who considered that it was adequate to determine deterioration within the room and could provide early warning of deteriorating conditions in the room (US DOE, 1991).

The location of the instrumentation monitoring room performance is shown in Figure 2.1. The specifications for the instruments are given in Table 2.1. A summary of the installation requirements including tolerances, workmanship and national codes and standards that the instrumentation must meet are given in Appendix A. Figures 2.2 and 2.3 provided the instrumentation cabling layout for the convergence meters and extensometers respectively.

2.1 ROOM CONVERGENCE MEASUREMENTS

Vertical and horizontal convergence stations will be installed at seven cross sections throughout the room to monitor roof/floor and wall/wall room closure. The locations for the instruments are provided in Figure 2.1. At each cross section, roof/floor convergence will be measured at mid span and at room quarter points and wall/wall convergence will be measured at mid room height. The convergence measurements will establish the rates of room closure for comparison with predicted rates and will be evaluated to determine assymmetric roof/floor closure of the room.

Each convergence station will consist of a mechanical anchor fixed about 150 mm below the rock surface. Details of a typical convergence anchor installation are given in Figure 2.4. An extensometer consisting of a wire or tape stretched under a constant tension and with an accurate distance measuring device is attached between the anchors . Changes in length between the anchors will be monitored periodically to determine room closure. The convergence measurements can be made manually or remotely. For manual measurements the the extensometer is put in place only for a reading and is subsequently removed. For the remote readings, the extensometer remains in position and the manual extensometer measuring device is replaced by a electronic length measuring device. In our application in Room 1, a potentiometer readout with a range of 36 inches will be used where remote readings can be obtained. Remote readings cannot be made at all locations because the permanent installation of wires across the room will interfere with access into the room.

2.2 EXTENSOMETER MEASUREMENTS

Borehole extensometers will be installed in the roof and the walls of Room 1. Roof extensometers will monitor bed separations at anhydrites "a" and "b", and dilation and creep movements within the immediate roof beam of salt. Wall extensometers will monitor the lateral deformations within the pillars.

Within each borehole, five measuring points will be anchored to the rock to monitor rock movements towards the room. Details of a typical borehole extensometer installation are shown in Figure 2.5. In the roof holes the anchors are nominally fixed at depths into the hole of 0, 6.5, 7.5, 13, and 14 feet, for the purpose of monitoring bed separation across the anhydrites "a" and "b". In the wall holes, the anchors are fixed at depths of 0, 5, 10, 15, 25 and 50 feet. The specifications for the drilling of boreholes, the installation of extensometers and for the instruments are given in Table 2.2.

Calibration of the measuring device for the multiple point extensometers will be carried out either by the manufacturer or by the Site Calibration Laboratory. Calibration will be traceable to N.I.S.T.

Readings will either be taken manually with a readout device provided by the manufacturer or will be performed remotely through the automatic data acquisition system that is maintained by the Manager and Operating Contractor in the underground. Measuring frequency, once the room is in use as a laboratory for the bin scale tests will be carried out every week. This frequency may be adjusted to meet any changes that develop.

2.3 SURVEY MEASUREMENTS

Survey measurements will be made in the room by the surveyors from Mine Engineering. These measurements will be used to separate roof and floor deformations. The measurements will be taken on a routine basis, probably at intervals of about 3 months.

2.4 FRACTURE MAPPING OF OBSERVATION BOREHOLES

Three observation boreholes have been drilled into the roof of Room 1, Panel 1. Observations of bed separation and lateral strata shifts will be made on a routine basis at intervals of about 3 months. The boreholes will be monitored using a scratch probe that has been used for the Excavation Effect Program (U.S. DOE, 1987). The holes can also be viewed with a borehole camera if the fractures require visual observation.

In addition, the boreholes for the rock anchors will be observed from fractures immediately following their drilling. This will be carried out using the scratch probe.

2.5 DATA ACQUISITION

The instrumentation monitoring room performance, is currently read manually. Conversion to remote reading of instrumentation is planned. This conversion will take place when the data acquisition system for the monitoring of the support system is installed in the room. It may not be practical to convert all the instrumentation to remote readings. The roof extensometers will be converted to remote reading. The ancillary equipment to allow remote reading of the quarter point convergence stations will be installed but a final decision on installation of the wires will depend on establishing that they will not be damaged by personnel maintaining or sampling the bins. The roof/floor convergence stations at midspan i.e. down the middle of the central access way and the wall/wall convergence stations will not be monitored remotely as they would interfere with access.

The instrumentation will be monitored from a data logger located in an alcove in S1950 of Panel 1 between Rooms 4 and 5. The data logger is part of the Geomechanical Instrumentation System installed in the underground. The system is controlled from a computer located on the surface. The data logger that will be used to remotely read the instrumentation monitoring room performance in Room 1 is already in place. The specifications to which the datalogger is manufactured are provided in Appendix A.

The results from the instrumentation in the room will be evaluated on a continuous basis. Documentation of the results will be provided annually in the Geotechnical Field Data and Analysis Reports.

3.0 MONITORING OF SUPPORT SYSTEM PERFORMANCE

The monitoring of the support system performance provides an assessment of the manner in which the support is controlling roof movements including the breakup of the immediate roof. The monitoring program in Room 1, Panel 1 will evaluate the following:

- o the performance of the structural system that supports the roof.
- o the load that develops in each rock anchor for the purpose of adjusting loads so that the build up is controlled in a consistent manner.

The basis for the instrumentation must be that the measurements are simple; that instrumentation is easily replaced if it malfunctions; and that the performance of each anchor can be continuously monitored and readily compared with performance of other anchors. The instrumentation layout for monitoring the support system performance is provided in Figure 2.6. Cabling layouts are provided in Figures 2.7, 2.8, and 2.9.

The most important of these measurements are those that determine the anchor loads. These measurements will be used to adjust the anchors to ensure that the anchors are not stressed beyond the allowable working stresses and that the roof is lowered in a controlled manner that accommodates the continued creep of the solid salt.

The measurements of cable elongation and pressures developing on the sheeting are taken to determine how these components of the support system are performing. No adjustments are planned on the basis of these measurements. However, if they show load buildup additional actions may be considered. It is not expected that breakup of the roof rock will be excessive, it appears more likely that the rock will remain primarily self supporting until the detached wedge in the roof fully forms. It is not expected that this will occur within the next two years based on the experience obtained from SPDV Test Room 1 Therefore, it is not believed that loads approaching the full weight of the detached wedge will develop on the expanded metal sheeting and the cables.

3.1 ROCKBOLT LOAD CELLS

The rockbolt load cells monitor the axial loading on the rock anchors. The measurements will be made on each anchor and will be the basis for adjusting the load on each anchor should an adjustment become necessary.

Each load cell consists of a cylindrical body with a central annulus for the rock anchor. The load measuring element is a spool of high strength steel or aluminum on which electrical resistance strain gauges are bonded in a full bridge configuration that provides temperature stability and compenstation for off center loading. A steel outer cover and O ring seals protect the strain gauges from mechanical damage and water penetration.

The load cells shall have sufficient capacity to measure up to 50 kips with a sensitivity of 0.02 kips. In order to maximize the vertical adjustment on the tendons, the height of the load cells shall not exceed 75 mm. The typical load cell installation is shown in Figure 2.10.

3.2 PRESSURE CELLS

Pressure Cells that will monitor the pressures that develop between the expanded metal sheeting and the salt roof. The measurements will be made in selected areas within the room that are expected to have the greatest roof movements and hence, be more susceptible to the development of loads due to the breakup of the immediate roof rock. Typical pressure cell installation is shown in Figure 2.11.

The pressure cell is manufactured from two steel plates welded together. The space between the two plates is filled with de-aired antifreeze solution or hydraulic fluid and is connected via a high-pressure stainless steel tube to a pressure gage and/or pressure transducer. A pump is used to inflate the pressure cell and press the cell against the rock. A change in load on the the cell will cause a deflection of the diaphragm which results in a change in the fluid pressure. The pressure cells, in Room 1, will be installed between the rock and the mesh to monitor the pressure distribution on the cable lacing and mesh.

The pressure cells should be constructed from corrosion resistant materials such as stainless steel. Each pressure cell will have pressure gage. The pressure cell can be modified for remote monitoring by replacing the pressure gage with or adding a pressure transducer.

The most important factor to take into consideration when installing pressure cells is to ensure a good contact with the surrounding material and to avoid localized or point loading of the cell. To avoid point loading, the pressure cells will be encapsulated with a concrete based grout. After the grout has set up, the pressure cells are placed between the rock surface and support mesh. The pressure cells are pumped up so that the cell is completely filled with fluid.

Pressure in the cells will be monitored using pressure gages. Monitoring of the pressure cells can be change from manually read to remotely read with the addition of a pressure transducer to the cell.

3.3 CABLE ELONGATION

Crack meters will monitor the elongation of the cables that support the mesh and expanded metal sheeting. The measurements will be made on selected sections within the room that are expected to have the largest deformations and be more susceptible to breakup of the surficial rock. Typical crack meter installation is shown in Figure 2.12.

3.4 DATA ACQUISITION SYSTEM

The data acquisition system shall provide for remote multiplexing of the load cells at locations within the room. The data acquisition system shall be capable of handling the required number of multiplexers. The data acquisition system shall be configured to monitor 33 rows of load cells, each row containing 11 loads cells.

The data logger will consist of a programmable controller, switching units, and a readout device. To prevent thermal deterioration, the switching units must multiplex all signal functions for each instrument. Continuous connection to a constant-voltage power bus is not allowed.

The data logger will include a Racal-Vadic Model VA1251G/K modem for data link connection to the surface data logging computer.

To facilitate compatibility with existing GIS equipment, existing communication parameters, protocol, and programming must be incorporated into the data logging components.

A Racal-Vadic Modem Model VA1251G/K will exchange ASCII character data over the datalink cable via the following parameters:

- Baud Rate - 300
- Parity - Even
- Stop Bits - One
- Word Length - Seven Bits

The two panel switches on the Racal Vadic Modem are to be set as follows:

- o Analog/Digital Loopback - OFF
- o Transmit Reversal - OFF

The modem's RS-232 interface will connect with the supplier-provided control units to ensure proper data communications.

The surface datalogging computer has been programmed to communicate with all underground control units through an exchange of ASCII character data. The computer sends a two character address sequence down the datalink cable through the surface modem. Each control unit then demodulates the character sequence through its modem. Each control unit is uniquely programmed (via a PROM chip) to respond to its own address sequence. Upon receipt of its address sequence, each control unit will poll its instruments, perform any necessary data reduction, and send instrument readings through the modem as a string of ASCII characters.

4.0 ADJUSTMENT OF SUPPORT SYSTEM

The most important part of the support system performance involves the controlled yield of the roof. It is expected that the interaction of the support system with the roof will pass through several stages.

The anchors will be set to a nominal load of 1000 lb after proof testing to 1.33 times their working load. The purpose of the preload is to ensure that the lacing and meshing under the channel is secured firmly in position. As the loads change they will be compared with an estimate of conditions. There are two cases to be considered. These are the control of load during the detachment of the wedge when the full loads have not developed and the case when the wedge has detached and the working loads have been reached and any continued build up would be dependent on the creep of the solid salt onto the wedge that creates a stress build up in the support system that must be relieved by the controlled yield of the support.

Initially, the roof will be self supporting as the fractures will not have developed sufficiently to define a detached wedge. It is likely that this condition will be maintained for a period of years, especially if the bolting systems are able to reduce the widening and propagation of the fractures that do develop. However, for worst case conditions, it must be assumed that fractures will propagate and that gradually the degree of self support of the roof will be lost. As this occurs, the rock anchors will provide increasing roof support and loads will build up in the anchors. Once the roof wedge becomes detached, then the rock anchors will be fully supporting the wedge and will have reached their working loads. Control of anchor loads must consider the adjustments needed during load build up when the wedge is not fully detached and load distributions may not be as expected, and those required once the wedge has detached and is subject to both vertical and lateral movements due to the creep of the solid salt.

In addition, the wedge shape must be taken into account when estimating the adjustments that must be made to the anchor loads. Two possible geometric shapes have been proposed to define the wedge that develops in the roof of excavations. A triangular distribution identified from visual observation of the roof fall in SPDV Test Room 1, and a parabolic distribution based on survey data of the roof of the room after the fall. For the purpose of assessing the adjustments to the anchors in Room 1, both distributions will be compared with the field data to determine which is more appropriate. The comparison will be carried out on a row by row basis and also over time since the geometry of the wedge may depend on location within the room and load distribution within a row of anchors may change with time as fractures develop.

4.1 CRITERIA FOR LOAD ADJUSTMENT

The criteria for adjusting the loads in the anchors are as follows for the two cases that have been identified:

CASE 1: Load distribution below Maximum Working Load

This case will occur as the load develops from the nominally applied loads due to the increasing support provided to the wedge as the fractures develop. During this stage it is not obvious exactly how the loads will build up, but it is expected that they will develop slowly because the rock is still self supporting. Based on these assumptions, the following criteria will be applied to load adjustments for a row during the build up to maximum working loads:

- No adjustments will be considered necessary to a row of anchors until the load in one anchor exceeds 4 kips.

- If the load distribution within a row of anchors is consistent with a triangular or a parabolic load distribution, then no adjustment is necessary. Consistent is taken to mean, variations from the load distribution of less than 20 percent for all anchors in the row.

- If the loading for a row of anchors is consistent with a triangular or a parabolic distribution but with a variation from 20 to 25 percent for an individual anchor, then no adjustment is necessary, but an analysis shall be made to establish the rate of load increase for all anchors within the row and to estimate whether the variation is increasing and the time that it will take to reach a value of 25 percent above the remainder of the distribution.

- If the load distribution for a row of anchors is consistent with a triangular or a parabolic load distribution but with a variation of 25 percent for an individual anchor, then an adjustment to that anchor will be carried out. The load on the anchor will be reduced by not more than 50%.

- If the load in one anchor exceeds 4 kips and the load distribution within the row is not consistent with either a triangular or a parabolic distribution, then a study will be carried out to establish whether an alternative plausible load distribution can be established. If this is possible, then this distribution will be used to determine the adjustment to the anchor load.

For example, if the loads develop on one side of the room due to assymmetric room closure, then an assymmetric load distribution may be found to be a more appropriate basis for load redistribution.

CASE 2: Load Distribution at Maximum Working Loads

Load distribution at maximum working load is considered to have developed when controlled adjustment of a row of bolts cannot reduce the anchor loads below levels that are consistent with the weight of a detached rock wedge. Once this stage has been reached, then the following criteria will be used to adjust the anchors in each row:

- If the measured load in an anchor is 10 percent or more over the allowable working load for that anchor, an adjustment to the load will be made. The load on the anchor will be reduced by not more than 50% of its allowable working load.

- If the load distribution does not conform with a triangular or parabolic distribution, a study will be carried out to determine whether the measured distribution is reasonable and can be explained in terms of a geometric wedge shape that is appropriate.

These criteria are based on our expectation of the performance of the roof rock and of the support system and their interaction. A mock up demonstration is planned in another room in Panel 1. During the demonstration, loads in the anchors will be adjusted to establish the effects of changing loads by a controlled amount on the loads that develop on nearby bolts. Should the data from the demonstration indicate that the criteria do not provide adequate control for support system adjustments, then alternative criteria will be developed. The application of the modified criteria to the adjustment of the support system in Room 1 will require the approval of the Manager of Engineering for the Managing and Operating Contractor for the WIPP with concurrence from the Managers of Operations, Safety, and Quality Assurance.

4.2 ANALYTICAL EVALUATION FOR LOAD ADJUSTMENTS

In parallel with the monitoring of actual loads in the rock anchors, a study will be carried out to determine the load transfer that can occur between anchors. The study will include field tests and analytical computations. The field tests will investigate how load changes in one bolt affect adjacent bolts. Computational analyses will look at load transfer effects between bolts. These studies will be completed before adjustments to anchor loads are required in Room 1, Panel 1.

Computer simulations will assess the effects of adjusting the loads within the tendons . This will be done on a row basis, since the available software codes are based on two dimensional modelling and it is assumed that in eraction effects between rows spaced nominally 10 feet apart will not be significant. The codes that will be used are VISCOT, a finite element code and FLAC, a finite difference code. Both codes were developed for the structural analysis of geologic media. The VISCOT code which is a version of a publicly available code originally developed by Hinton and Owen (1982) was modified for used in the Salt Repository Program for the disposal of high level radioactive wastes. The FLAC code is a propietary code developed with funding from the Nuclear Regulatory Commission for application to repository projects.

The codes will be used to determine interaction effects between bolts supporting the isolated rock wedge. They will establish if adjusting the load in one anchor within a row will change the loads in other bolts within the row and by how much. They will also assess whether assymmetric load distributions can develop due to lateral or differential vertical displacements of the salt and how these effects can be compensated for or minimized by adjusting the anchor loads.

A preliminary assessment of load redistribution has been carried our using VISCOT. For the case of the fully detached wedge reduction is bolt loading of will be redistributed among the other bolts in a row without overloading of any bolt. The redistributions for a number of cases are shown in Figures 4.1. It should be noted that the study of bolt load adjustment will be an ongoing activity and that field data will be assessed to determine the effectiveness of the analytical evaluations for load adjustments.

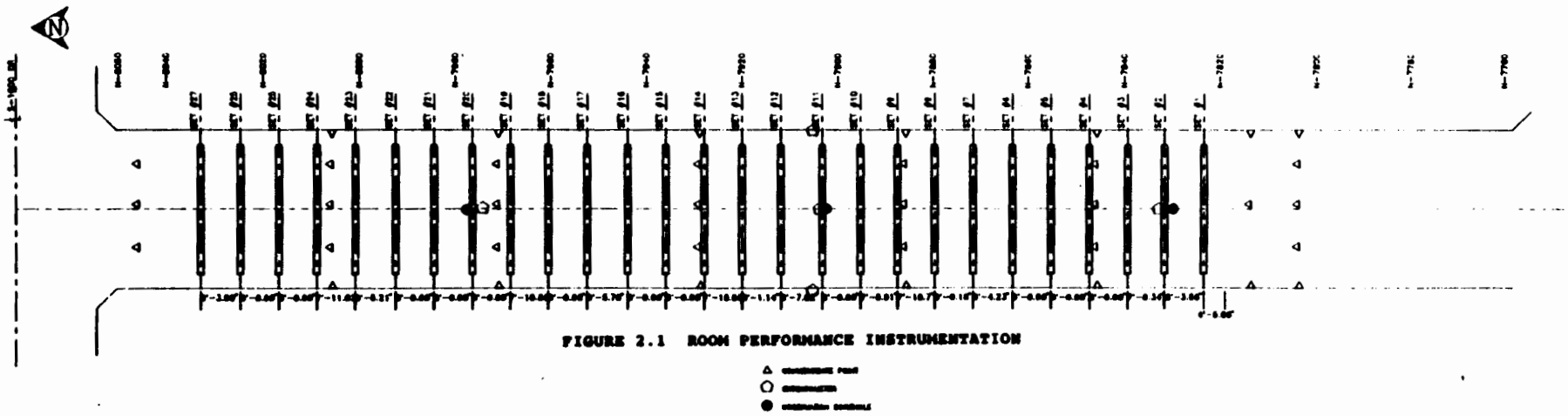


FIGURE 2.1 ROOM PERFORMANCE INSTRUMENTATION

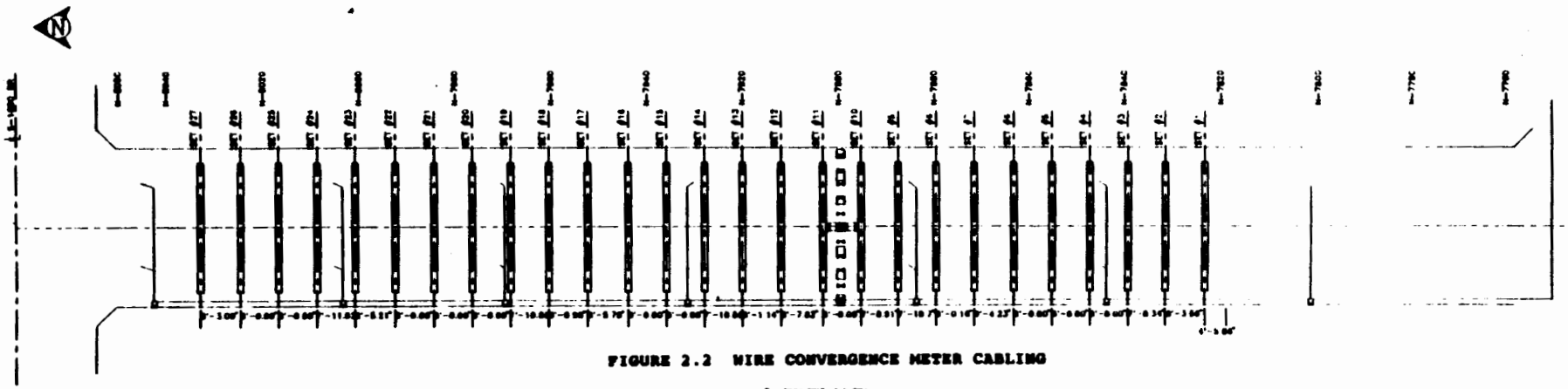


FIGURE 2.2 WIRE CONVERGENCE METER CABLING

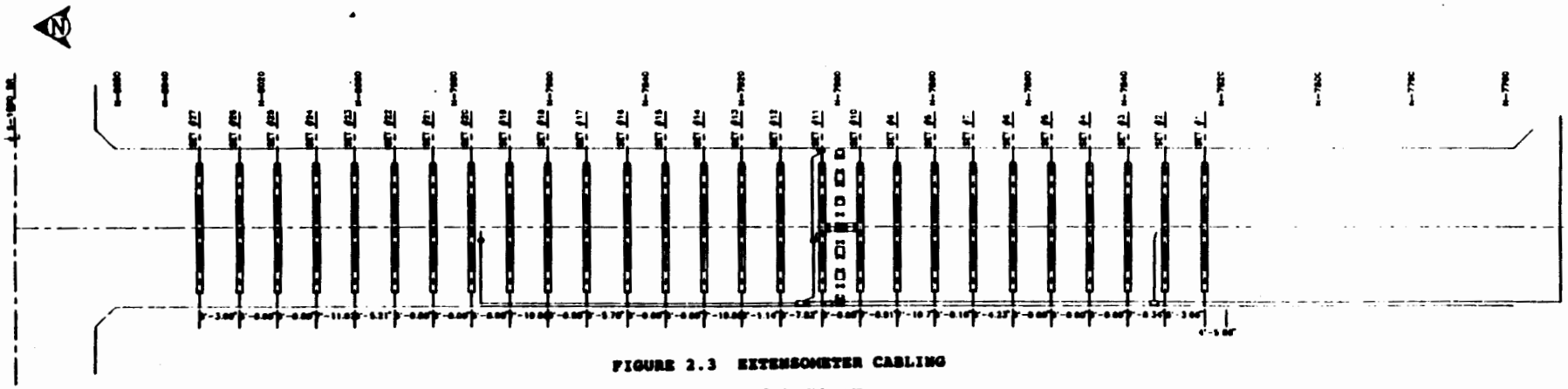
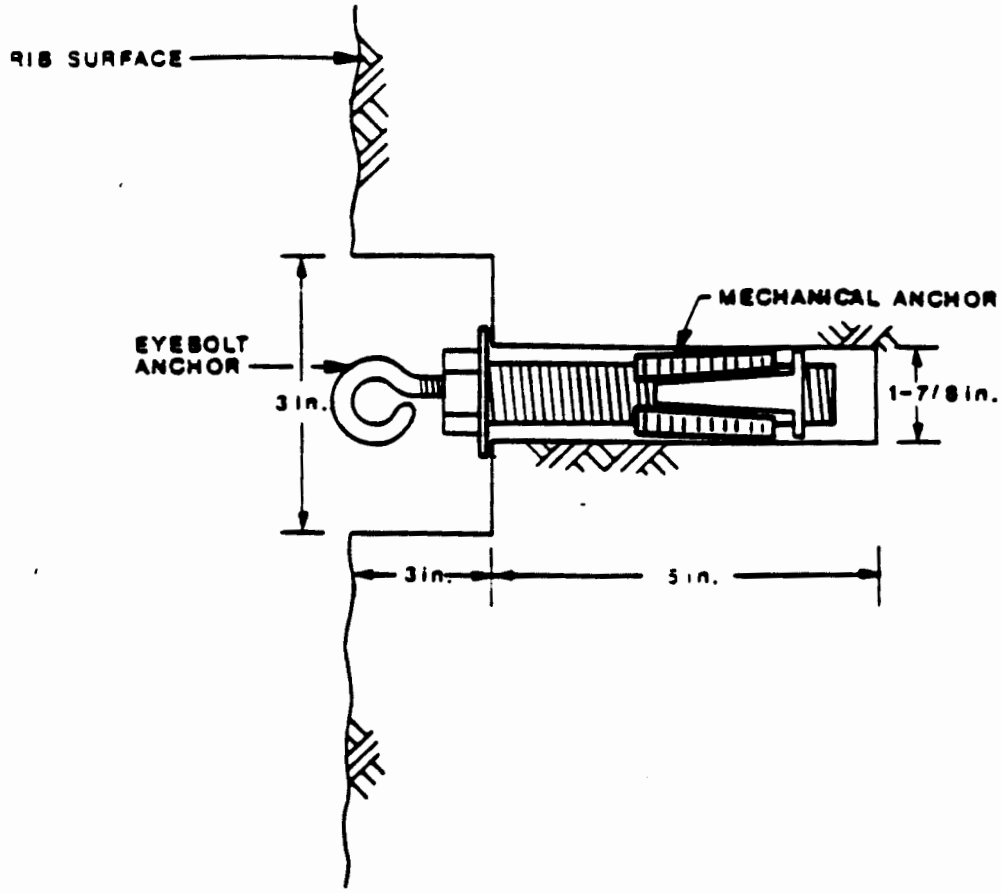


FIGURE 2.3 EXTENSOMETER CABLING

301001 88 84 08-A6



Note: • Floor installations may be recessed greater than 3 in. and protected with coverplate.

• Dimensions approximate

FIGURE 2.4 TYPICAL RADIAL CONVERGENCE POINT

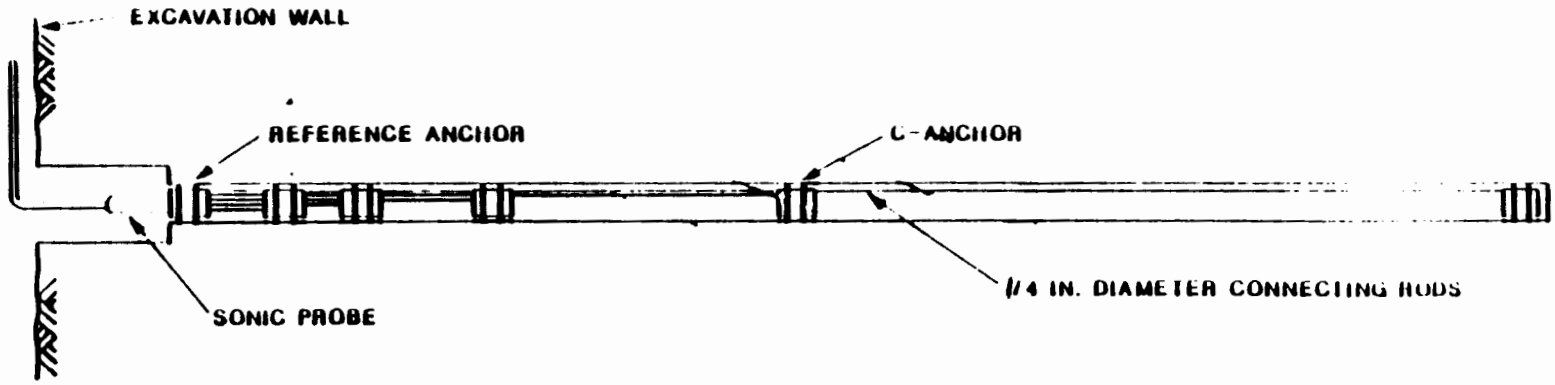


FIGURE 2.5 TYPICAL ROD-TYPE EXTENSOMETER INSTALLATION

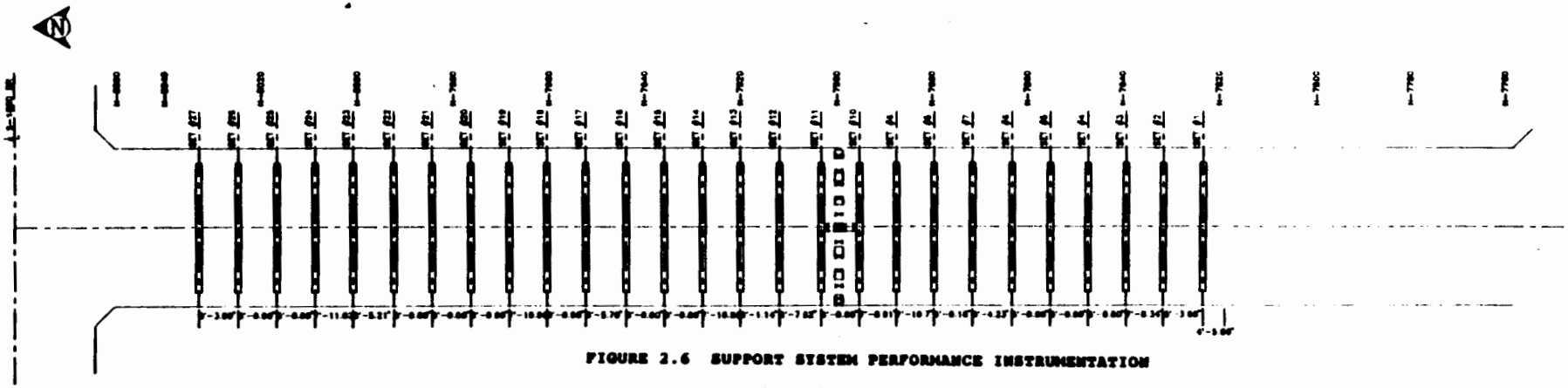


FIGURE 2.6 SUPPORT SYSTEM PERFORMANCE INSTRUMENTATION

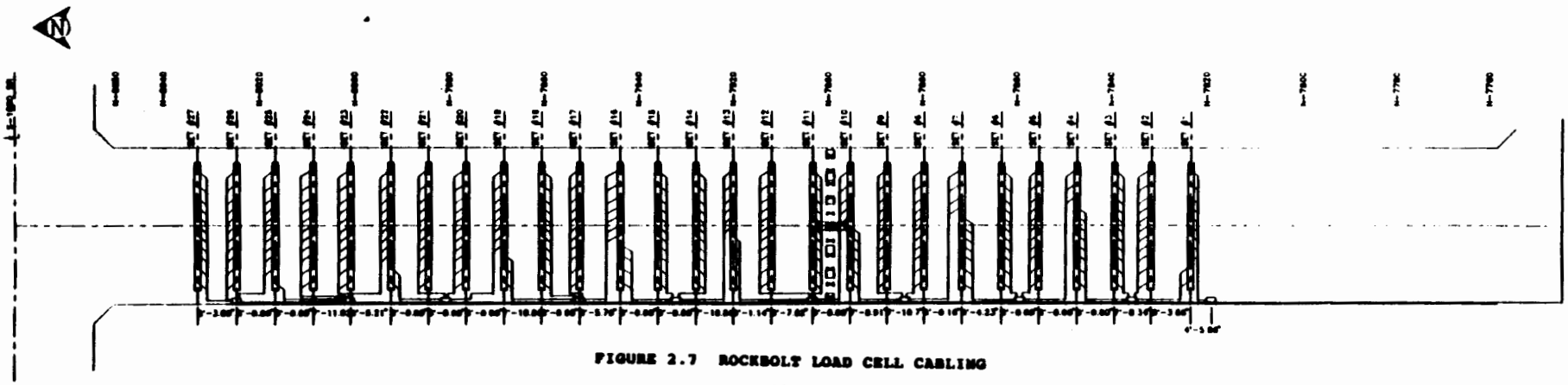


FIGURE 2.7 ROCKBOLT LOAD CELL CABLING

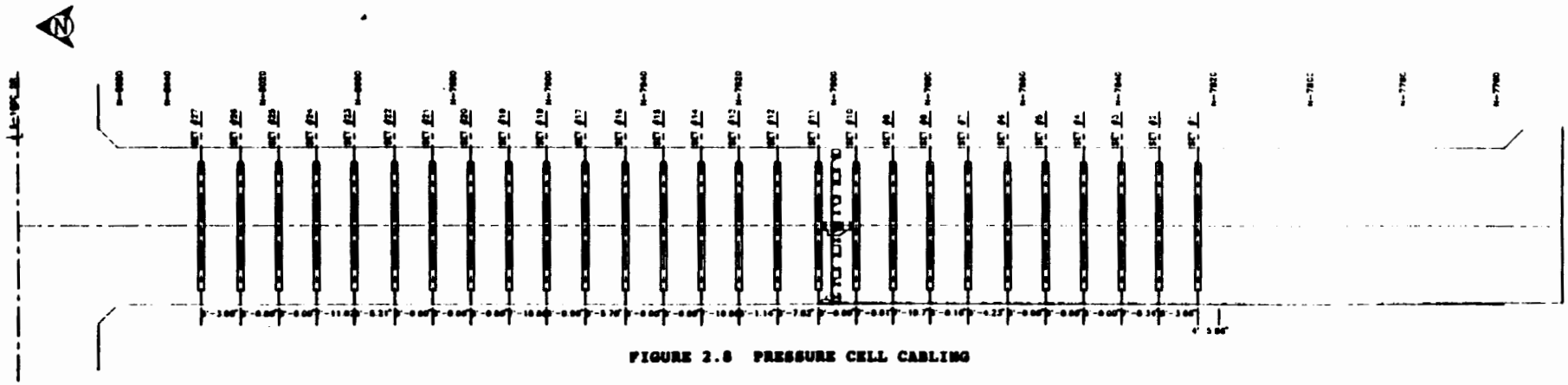


FIGURE 2.8 PRESSURE CELL CABLING

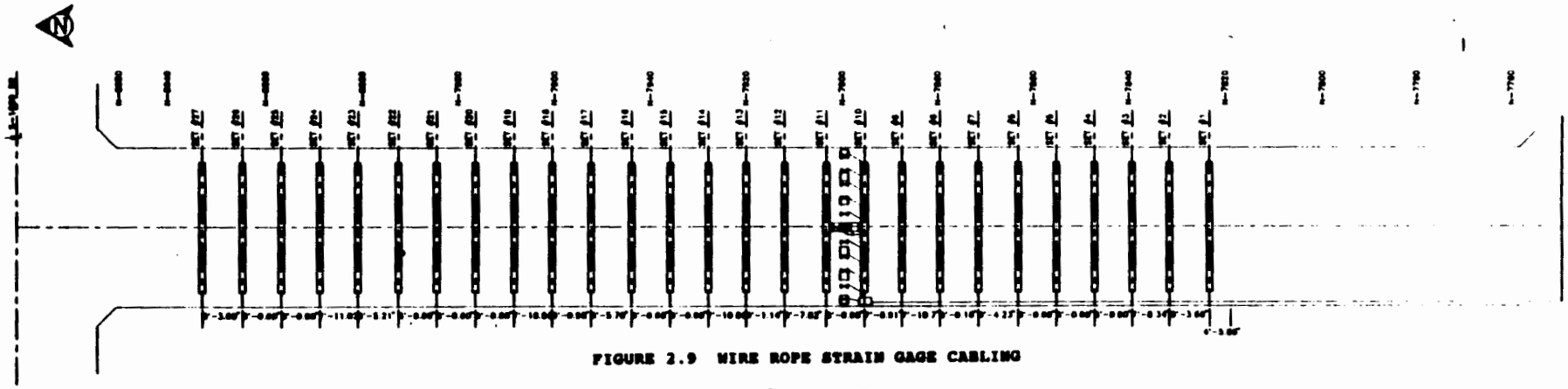


FIGURE 2.9 WIRE ROPE STRAIN GAGE CABLING

- ROCK BOLT LOAD CELL
- WIRE ROPE STRAIN GAGE
- PRESSURE CELL

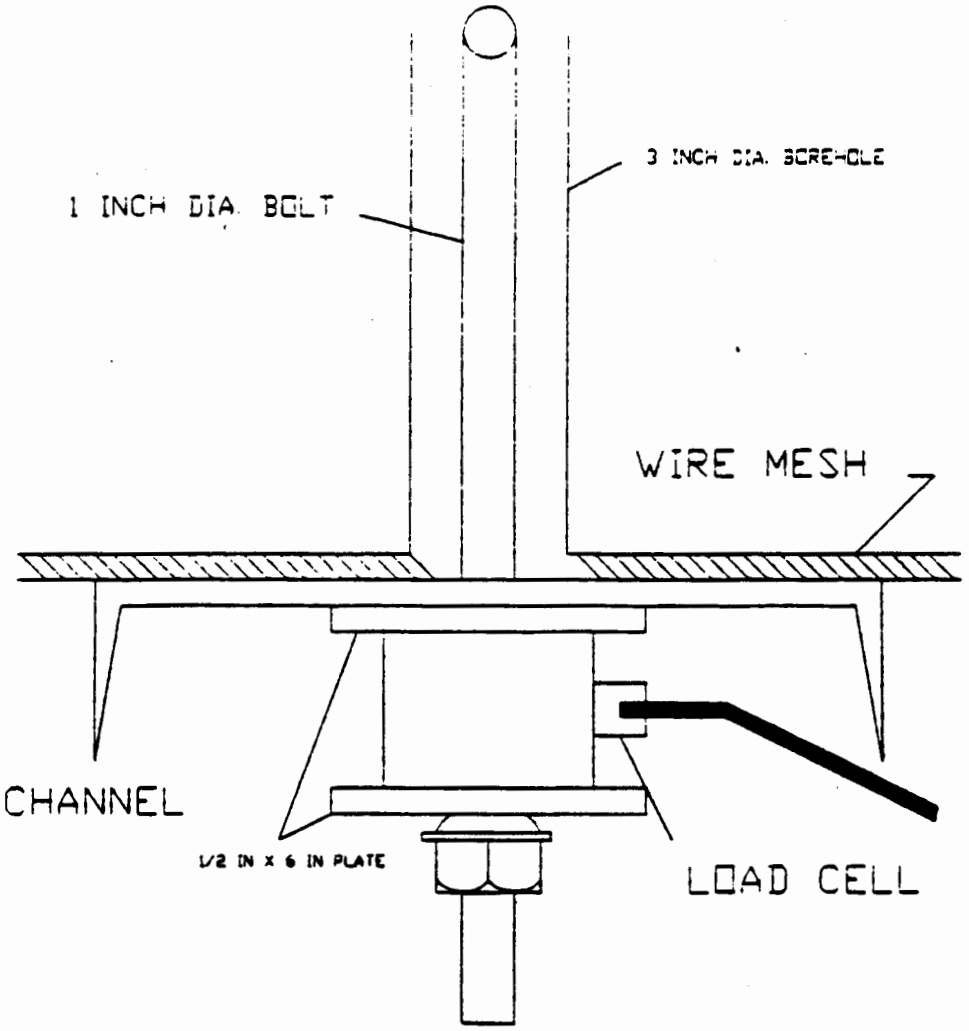


FIGURE 2.10 TYPICAL LOAD CELL INSTALLATION

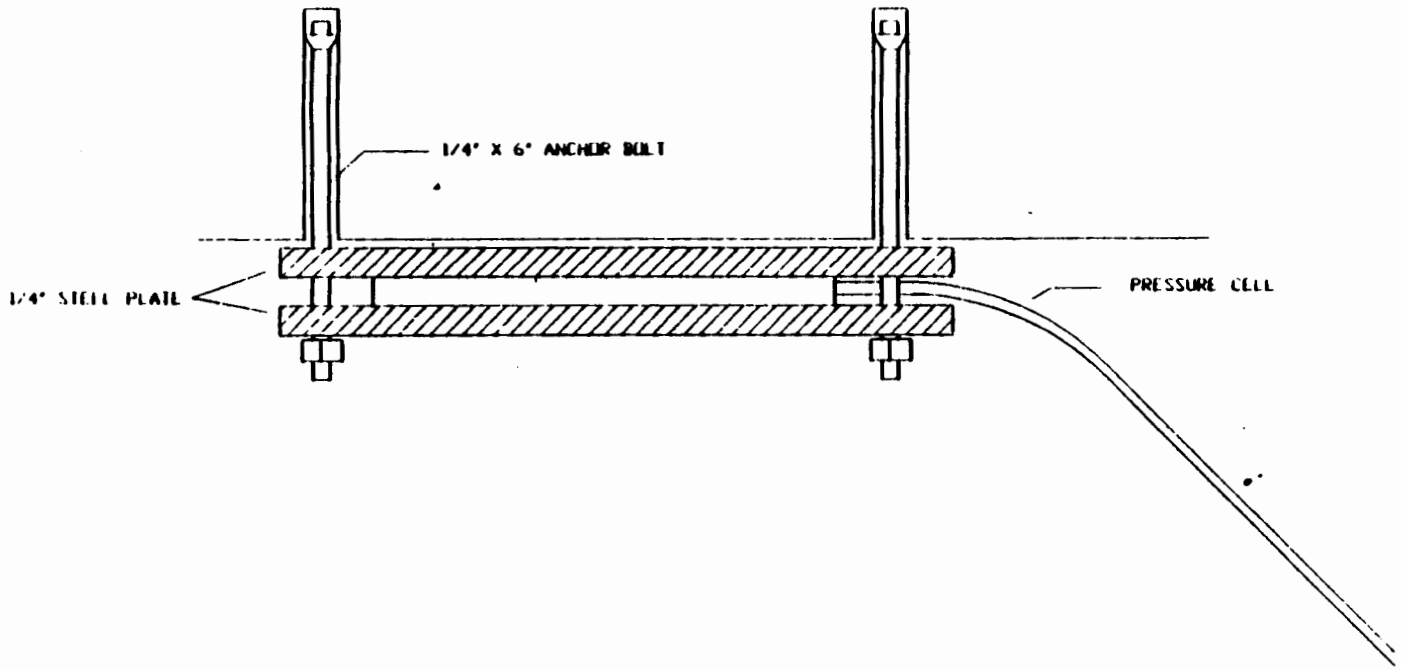


FIGURE 2.11 TYPICAL PRESSURE CELL INSTALLATION

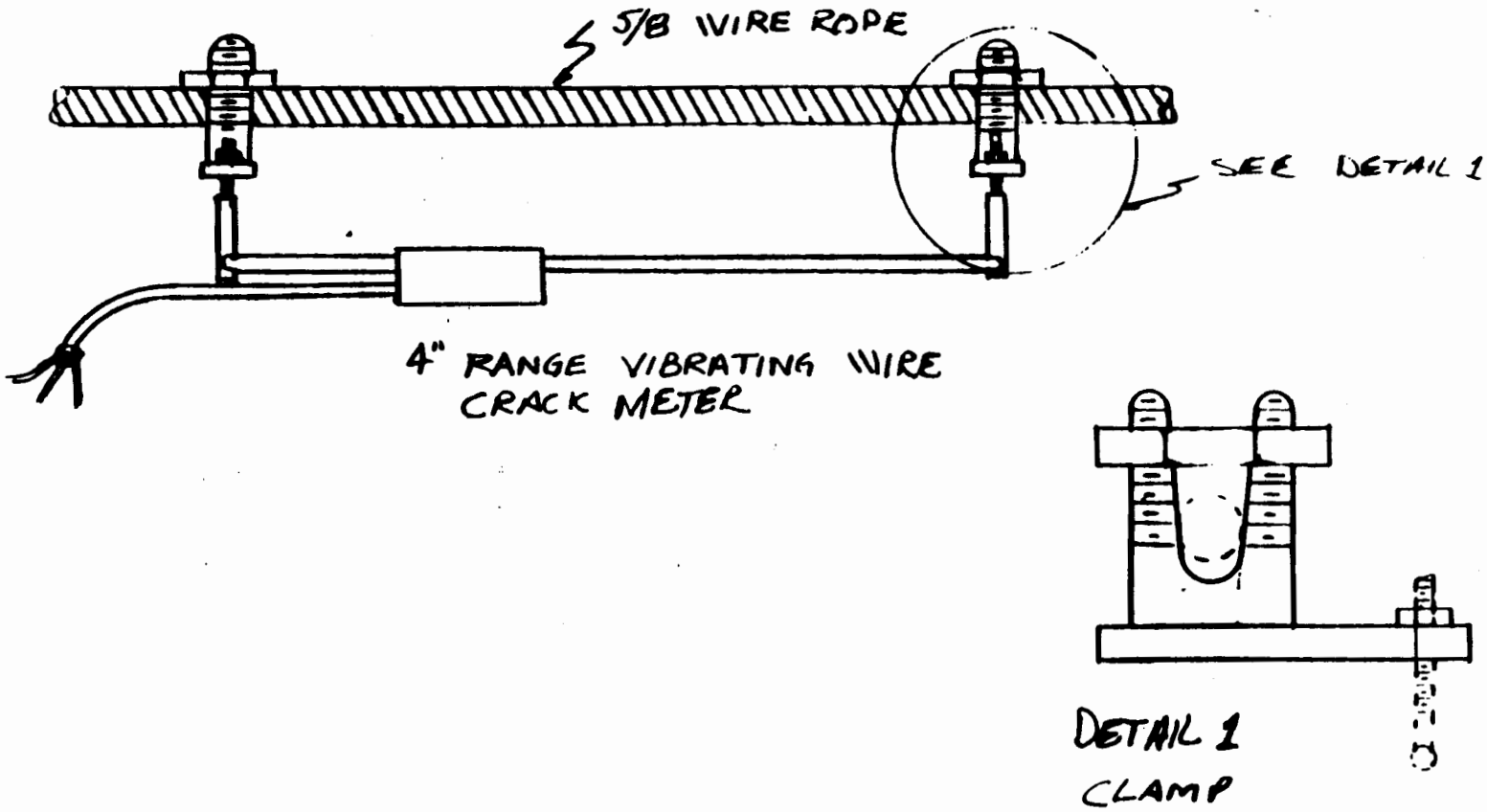
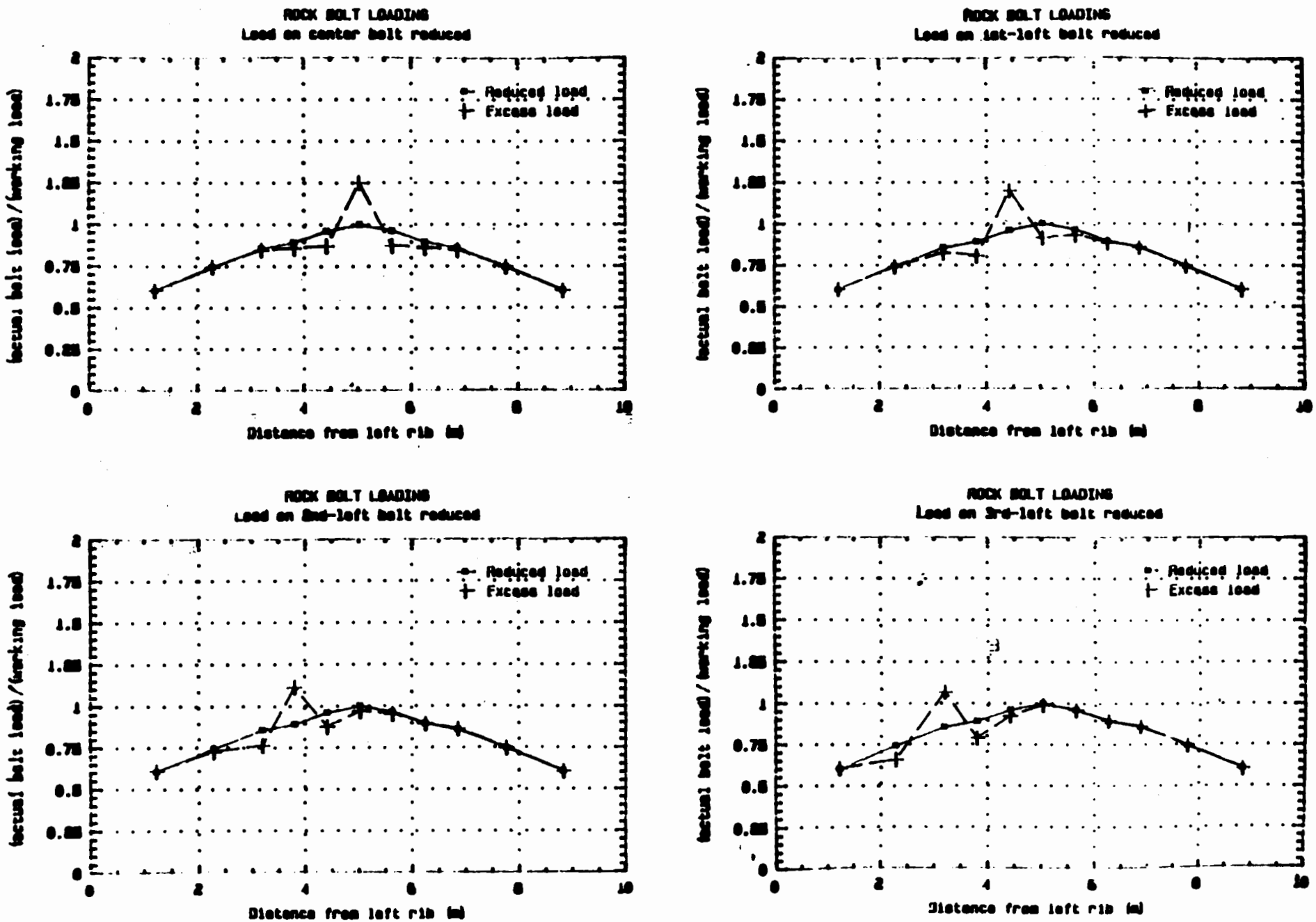


FIGURE 2.12 TYPICAL CRACKMETER INSTALLATION

FIGURE 4.1 ROCK BOLT LOADING REDISTRIBUTION



SUMMARY OF AFFIDAVIT OF MARCUS A. WILEY

Mr. Wiley is a registered professional engineer with nineteen years of mining industry experience. He is president of a mining consulting firm and has been responsible for the development and maintenance of dozens of project schedules for mining and related construction activities.

Mr. Wiley reviewed DOE planning documents concerning Test Phase activities. He employed project management computer analysis and concludes that ten years, and in any event no less than nine and one-half years, would be required to complete the Test Phase activities. This period is significantly beyond the term of the administrative land withdrawal.

AFFIDAVIT OF MARCUS A. WILEY

STATE OF COLORADO)
) ss.
COUNTY OF ARAPAHOE)

MARCUS A. WILEY, states as follows:

1. I am a registered professional engineer in several states (New Mexico license number 8384, Colorado license number 14650, Oklahoma license number 14203). I have nineteen years of mining industry experience since graduation from college. My Bachelor of Science degree in Mining Engineering was obtained in 1972 from the New Mexico Institute of Mining and Technology located at Socorro, New Mexico.

2. I make this affidavit for the purpose of setting forth, in accordance with accepted techniques, the time schedule or "time line" that demonstrates how the proposed Department of Energy (DOE) nuclear waste tests at the Waste Isolation Pilot Plant (WIPP) will be conducted and how much time they would, in fact require. My conclusion is that ten years (and in any event, no less than nine and one-half years) would be required - a period significantly beyond the term of the administrative land withdrawal involved here.

PERSONAL BACKGROUND

3. I was employed by Consolidation Coal Company from 1972 through 1981

in increasingly responsible engineering and management positions throughout the United States. Among those positions were mining engineer, project engineer, senior mining engineer, production foreman, maintenance foreman, maintenance superintendent, mine superintendent and project manager. I worked in mines and/or engineering offices located in Utah, New Mexico, Colorado, Montana, Illinois and Ohio. I worked in various phases of mining including exploration, geologic mapping, mine planning, scheduling, mine economics, permitting, government and public relations, equipment selection, employee selection, training, construction, maintenance and operations.

4. Wiley Engineering, Inc. is a mining consulting firm organized in October 1981 to provide professional services to the industry. Its office is located at: 9137 E. Mineral Circle, Suite 380, Englewood, Colorado 80112. I am its president. The firm provides clients with assistance in exploration, geological mapping, reserve evaluation, mine planning, mine design, project scheduling, permitting, operations review, feasibility studies, economic analysis, computer applications, bid evaluations, contract evaluations, project management, contract mining, contract reclamation, conveyor belt design and construction, and litigation support.

5. Between August 1983 and August 1985, I also served as the President of Ranchers Coal, Inc., located in Miami, Oklahoma. In this position, I managed an investor acquisition of an Oklahoma coal company and supervised a four-fold increase of production. I had overall responsibility for financing, marketing, lease acquisition, geology, engineering, permitting, scheduling, reclamation, safety,

operations, transportation and public relations.

CREATION OF TIME SCHEDULE

6. A comprehensive plan and time schedule is critical to the success of any project. A great deal of my experience has been in the planning and scheduling of operations. I have been responsible for the development and maintenance of dozens of project schedules for mining and related construction activities.

7. I conducted the time line analysis with the assistance of Darlene K. Sherrod. Ms. Sherrod received a Bachelor of Business Administration degree in Business Management and a Bachelor of Business Administration degree in Business Analysis from Texas A&M University located in College Station, Texas. Her study in Business Analysis included numerous courses in scheduling, computer science, operations research, statistical analysis and project management. After college, she worked on various scheduling and project management assignments utilizing numerous computer applications. Ms. Sherrod acted pursuant to my direction and reported directly to me.

8. I reviewed materials prepared by the Department of Energy, Sandia National Labs, Westinghouse Waste Isolation Division, and the Environmental Evaluation Group of New Mexico relating to the WIPP Project. I accepted and utilized the assumptions and specifications set forth therein as to the amount of time each element of the proposed test would require. I then prepared time line schedules for the testing of nuclear waste proposed for the WIPP site.

9. The following documents among others were reviewed for the preparation

of the WIPP project time lines:

A. 40 C.F.R. §§ 190-91

B. Test Plan: Bin CH TRU Tests (SAND 90 8500)

C. Test Plan Addendum #1: Bin Scale Tests (SAND 90-2082)

D. Status of the WIPP Project (Neill & Chaturvedi) (Waste Mgmt. '91)

E. Critical Experiments and Time Lines (W.D. Weart, SNL) (Aug. 12, 1991)

F. Room Stability Expert Panel (April 1991)

G. Waste Retrieval Plan January 1990 (DOE/WIPP 89-022)

10. PERT and Gantt schedules were prepared using computer application software. The schedules prepared are a Gantt chart titled "WIPP Project Schedule of Activities" (exhibit 1) and a PERT chart titled "WIPP Project PERT Chart" (exhibit 2). These schedules incorporate the Wet and Dry Bin-Scale testing, the Alcove testing, laboratory testing, solubility/leachability tests, waste retrieval and safety margin for retrieval. The time line starts with the issuance of notice to proceed on October 3, 1991, and assumes receipt of nuclear waste on October 10, 1991. Furthermore, the assumption is made that specified and required start-up testing has been completed as described in test plan documents. The time line incorporates the phasing and time requirements for other various activities as discussed in the documents reviewed.

11. The computer software used to develop the PERT and Gantt charts which describe the WIPP project scheduling is Time Line published by Symantec

Corporation located in Cupertino, California. This software package is designed specifically for use in project management and related applications.

12. The attached Gantt (exhibit 1) and PERT (exhibit 2) charts show a total time requirement of almost 10 years (October 3, 1991 to August 8, 2001) to complete the proposed testing and retrieval of nuclear waste as discussed in the documents reviewed. This time period extends four years beyond the date for the end of the administrative withdrawal (June 29, 1997). (Note: if it were assumed that the last bins delivered to the site were tested only for the duration of the first bins received, then the total time requirement might be reduced to about nine and one half years - still well beyond the administrative withdrawal period.)

Statutory verification:

I declare under penalty of perjury that the foregoing is true and correct.

Executed on October 5, 1991.

Marcus A. Wiley

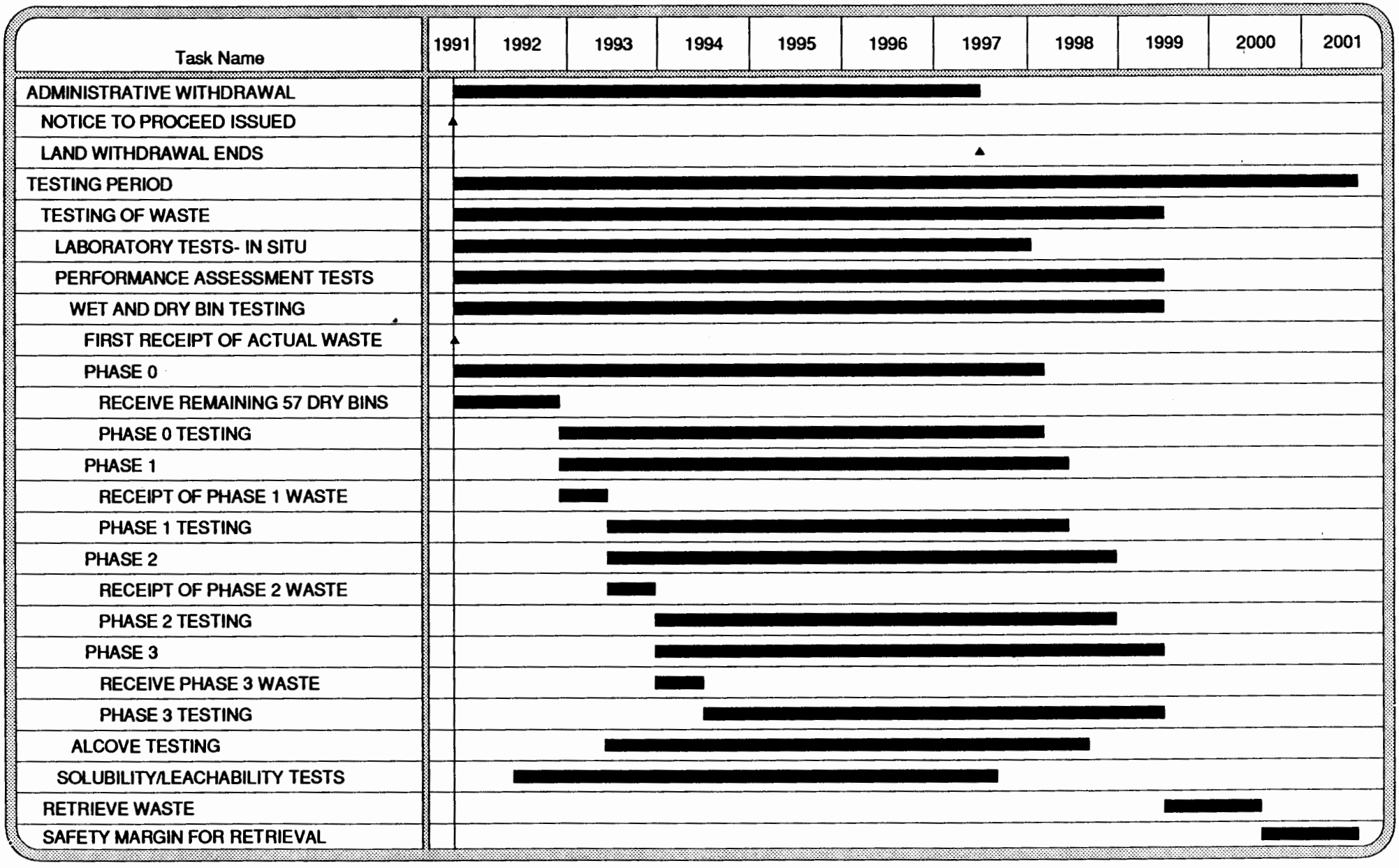
Marcus A. Wiley

X. Wiley 1

WIPP PROJECT SCHEDULE OF ACTIVITIES

WILEY ENGINEERING, INC.

OCTOBER 5, 1991

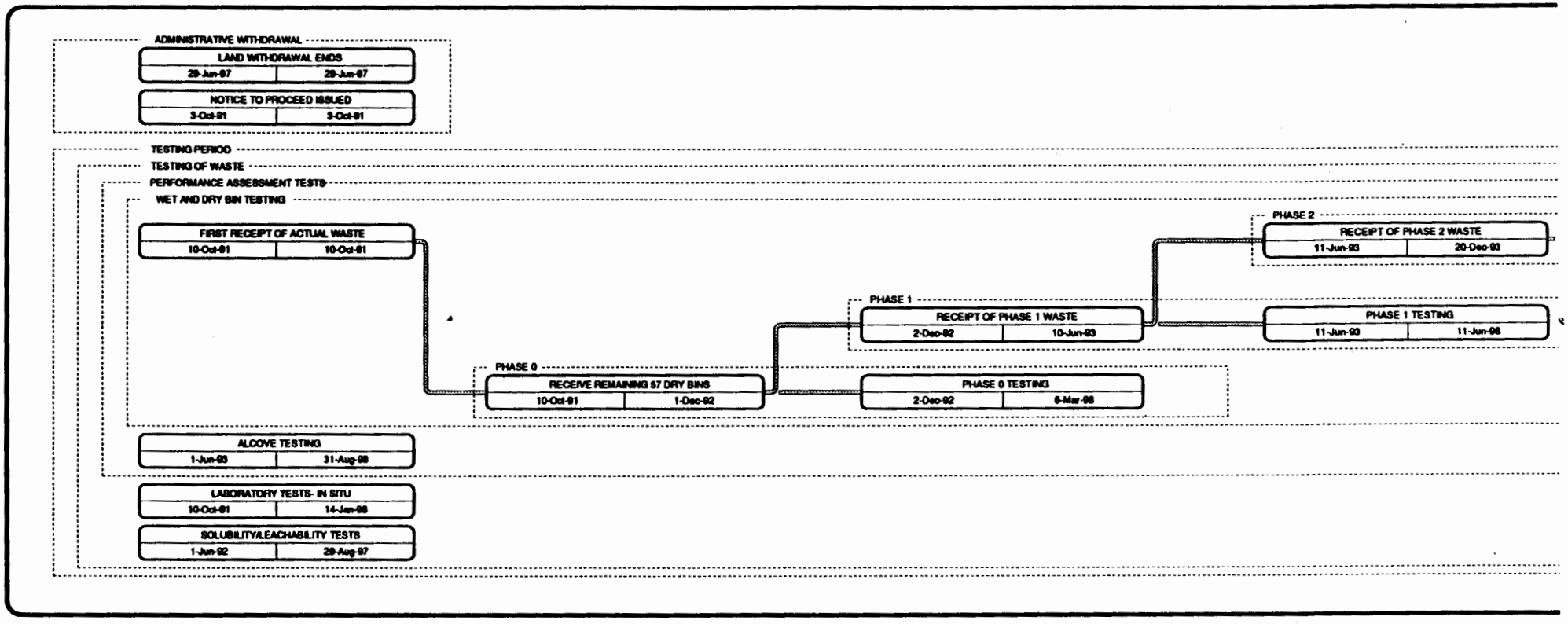


MARCUS A. WILEY AFFIDAVIT

EXHIBIT 1

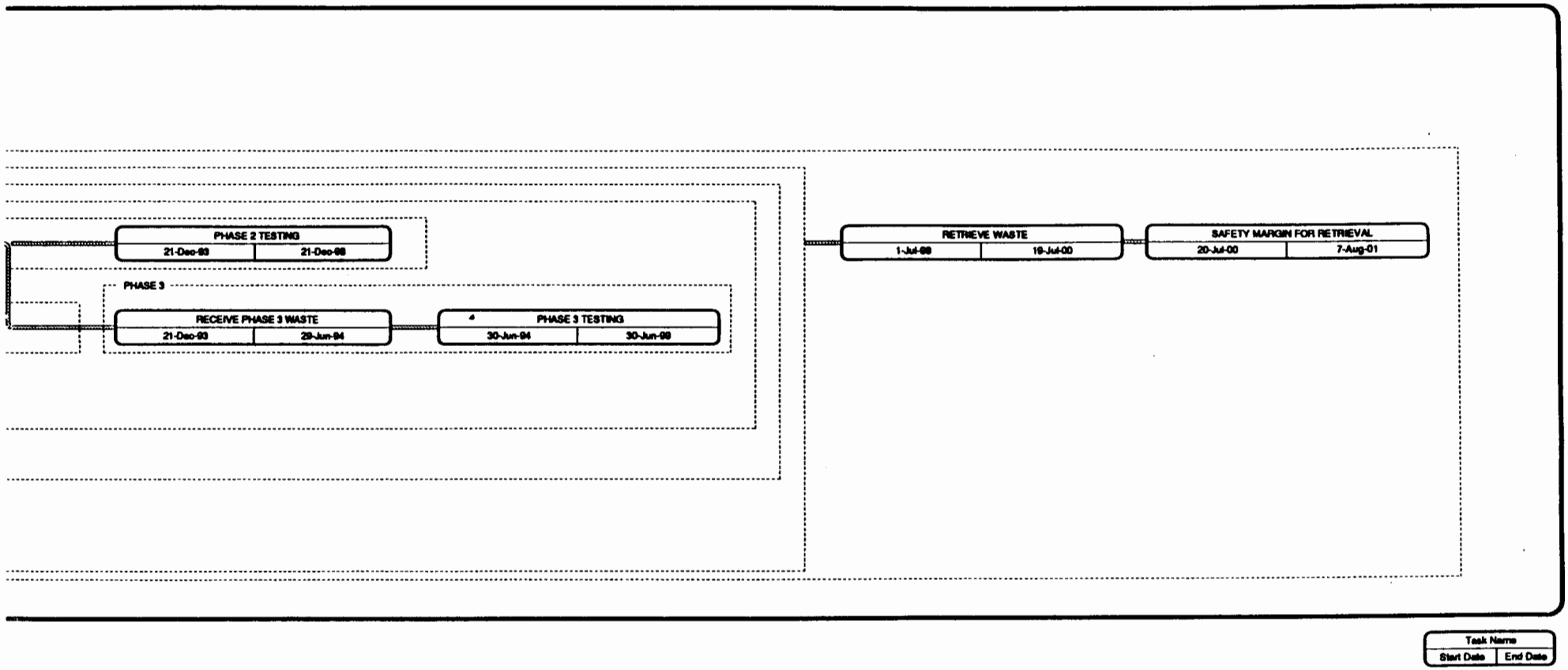
X. Wiley 2

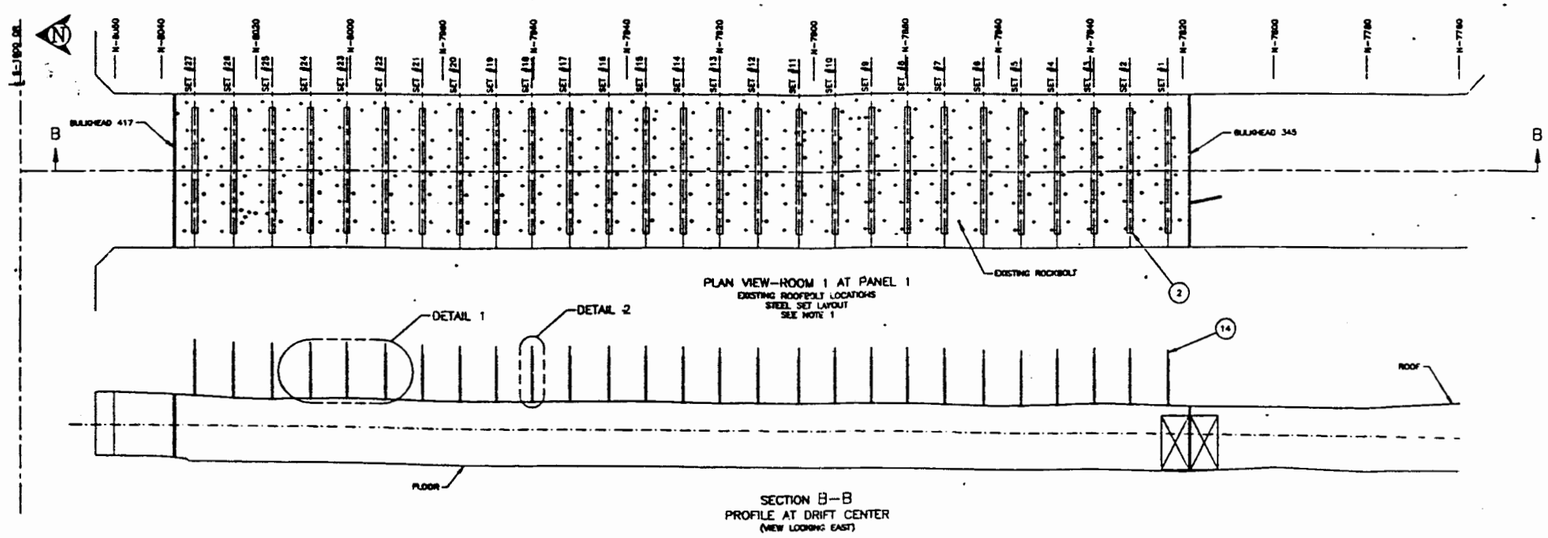
WIPP PROJEC
WILEY ENGIN
OCTOBE



MARCUS A. WILEY AFFIDAVIT
EXHIBIT 2

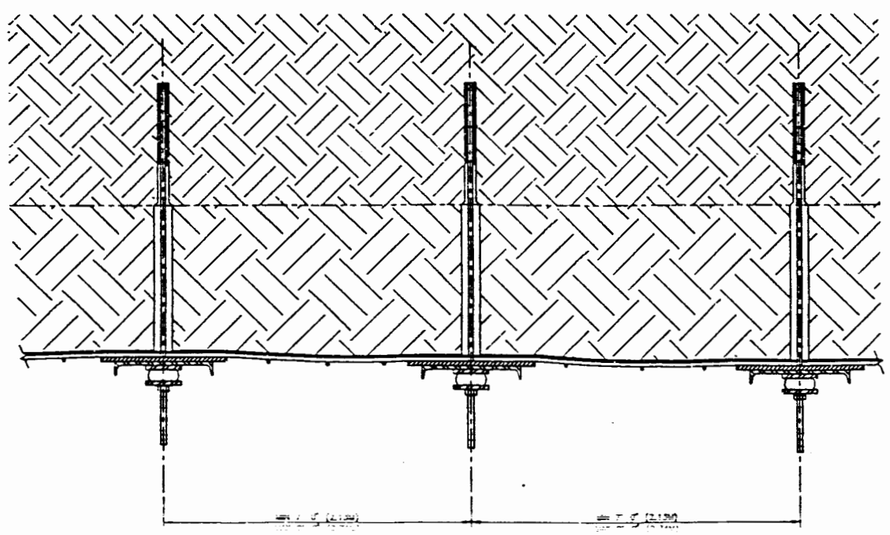
CT PERT CHART
INEERING, INC.
ER 5, 1991



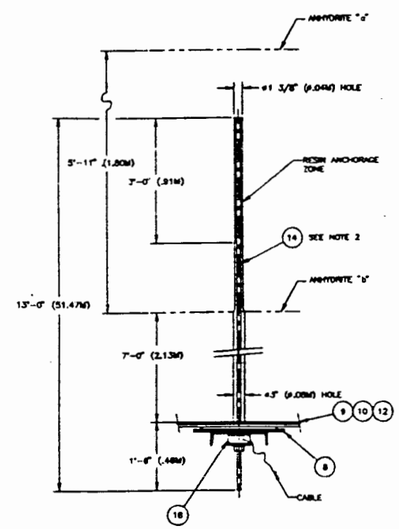


PLAN VIEW–ROOM 1 AT PANEL 1
EXISTING ROOFBOLT LOCATIONS
STEEL SET LAYOUT
SEE NOTE 1

SECTION B–B
PROFILE AT DRIFT CENTER
(VIEW LOOKING EAST)



DETAIL 1
WIRE MESH AND LACING
N.T.S.



DETAIL 2
ROCKBOLT INSTALLATION & ASSEMBLY

NOTES:
1. LOCATION OF STEEL SETS (ITEM 2) TO BE DETERMINED IN FIELD.
2. INSTALL ROCKBOLTS (ITEM 14) PER MANUFACTURERS INSTRUCTIONS.

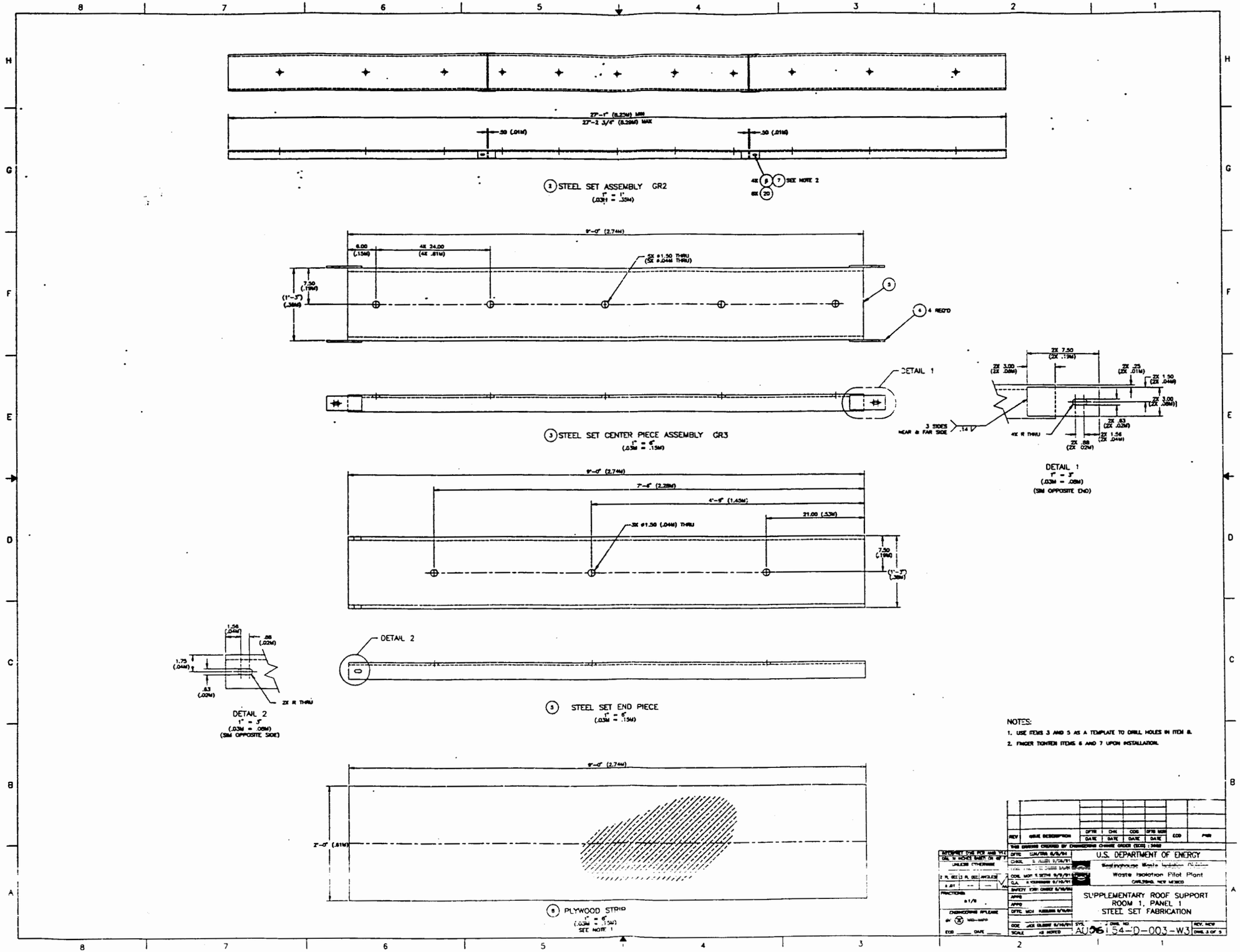
REV.	ISSUE DESCRIPTION	DFTR DATE	CHK DATE	CDS DATE	DFTR MGR DATE	ECO	PWR

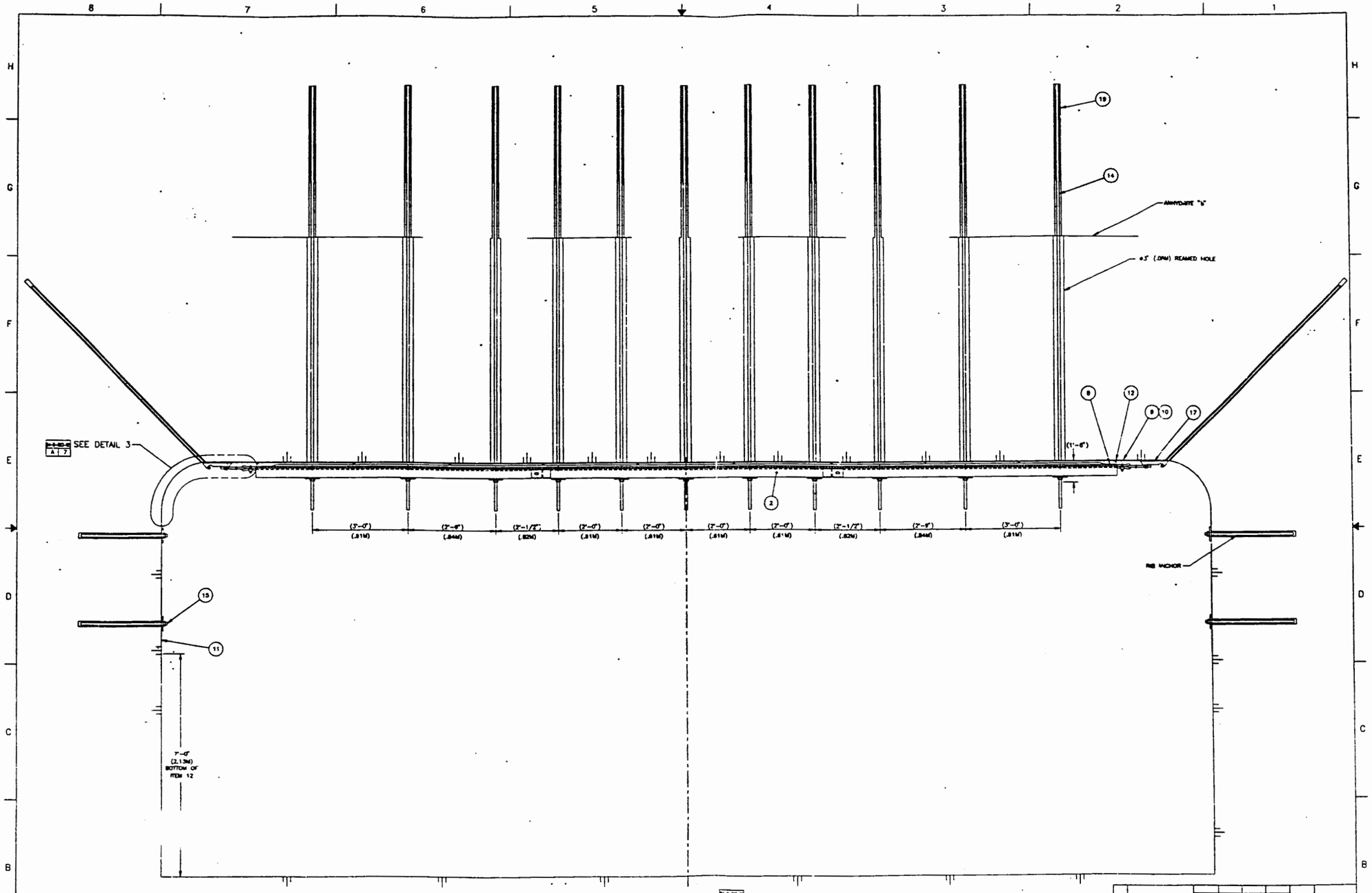
THIS DRAWING CREATED BY ENGINEERING CHANGE ORDER (ECO) : 5492

U.S. DEPARTMENT OF ENERGY
Westinghouse Waste Isolation Division
Waste Isolation Pilot Plant
CARLSBAD, NEW MEXICO

SUPPLEMENTARY ROOF SUPPORT
ROOM 1, PANEL 1
STEEL SET LOCATIONS & DETAILS

SYS. AU06 | DWG. NO. 54-D-003-W2 | REV. NO.


27'-1" (8.25M) MIN
27'-2 3/4" (8.29M) MAX
.50 (.01M)
.50 (.01M)
4X 6 7 SEE NOTE 2
8X 20
3 STEEL SET ASSEMBLY GR2
1" = 1'
(.03M = .35M)
9'-0" (2.74M)
6.00 (.15M)
4X 24.00 (4X .61M)
3X ø1.50 THRU (3X ø.04M THRU)
7.50 (.19M)
(1'-3") (.38M)
5
4 REQ'D
DETAIL 1
3 STEEL SET CENTER PIECE ASSEMBLY GR3
1" = 6"
(.03M = .15M)
2X 7.50 (2X .19M)
2X 3.00 (2X .08M)
2X .25 (2X .01M)
2X 1.50 (2X .04M)
2X 3.00 (2X .08M)
2X .63 (2X .02M)
2X 1.56 (2X .04M)
2X .88 (2X .02M)
3 SIDES NEAR & FAR SIDE
.14
4X R THRU
DETAIL 1
1" = 3"
(.03M = .08M)
(SIM OPPOSITE END)
9'-0" (2.74M)
7'-6" (2.28M)
4'-9" (1.45M)
21.00 (.53M)
3X ø1.50 (.04M) THRU
7.50 (.19M)
(1'-3") (.38M)
DETAIL 2
1.56 (.04M)
.88 (.02M)
1.75 (.04M)
.63 (.02M)
2X R THRU
DETAIL 2
1" = 3"
(.03M = .08M)
(SIM OPPOSITE SIDE)
3 STEEL SET END PIECE
1" = 6"
(.03M = .15M)
9'-0" (2.74M)
2'-0" (.61M)
8 PLYWOOD STRIP
1" = 6"
(.03M = .15M)
SEE NOTE 1
NOTES:
1. USE ITEMS 3 AND 5 AS A TEMPLATE TO DRILL HOLES IN ITEM 8.
2. FINGER TIGHTEN ITEMS 6 AND 7 UPON INSTALLATION.
REV
ISSUE DESCRIPTION
DFTR
CHK
COG
DFTG MGR
ECO
PKG
DATE
THIS DRAWING CREATED BY ENGINEERING CHANGE ORDER (ECO) : 34468
DFTR
CHECK
COG. MGR
C.A.
SAFETY
APPR
APPR
DFTG. MGR
DOE
SCALE
AS NOTED
U.S. DEPARTMENT OF ENERGY
Westinghouse Waste Isolation Division
Waste Isolation Pilot Plant
CARLSBAD, NEW MEXICO
SUPPLEMENTARY ROOF SUPPORT
ROOM 1, PANEL 1
STEEL SET FABRICATION
DWG. NO.
REV. NEW
SHT. 2 OF 3
AU06 154-D-003-W3



SECTION A–A
ROOM 1 AT PANEL 1
(VIEW LOOKING SOUTH)
STEEL SUPPORT

REV	ISSUE DESCRIPTION	DFTR DATE	CHK DATE	CDC DATE	DFTG MGR DATE	ECO	PMR

THIS DRAWING CREATED BY CHANGE ORDER (ECO): 5602

DIMENSIONS ARE PER ANSI Y14.5
DIM. IN INCHES BASED ON 68°F.
UNLESS OTHERWISE SPECIFIED TOLERANCES
7 PL DEC ± PL DEC ± ANGLES ±1°

DFTG DJH/TRB 8/5/91
CHKR S. ALEN 9/20/91
CDC ENG
CDC MGR 9/10/91
C.A. 9/10/91

MATERIAL: ø.5"

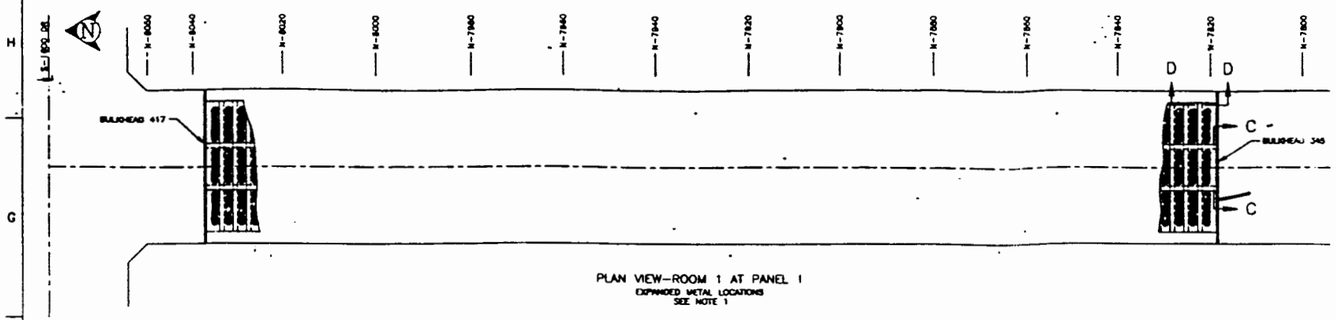
CHANGE/FINAL RELEASE
BY (W) WIO-SSMP
ECO ____ DATE ____

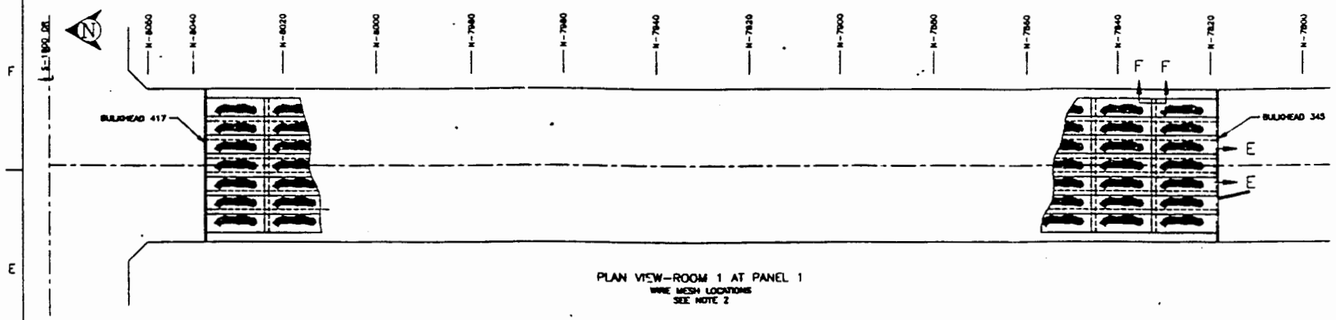
APPR
APPR
DFTG MGR
CDC MGR 9/10/91
SCALE

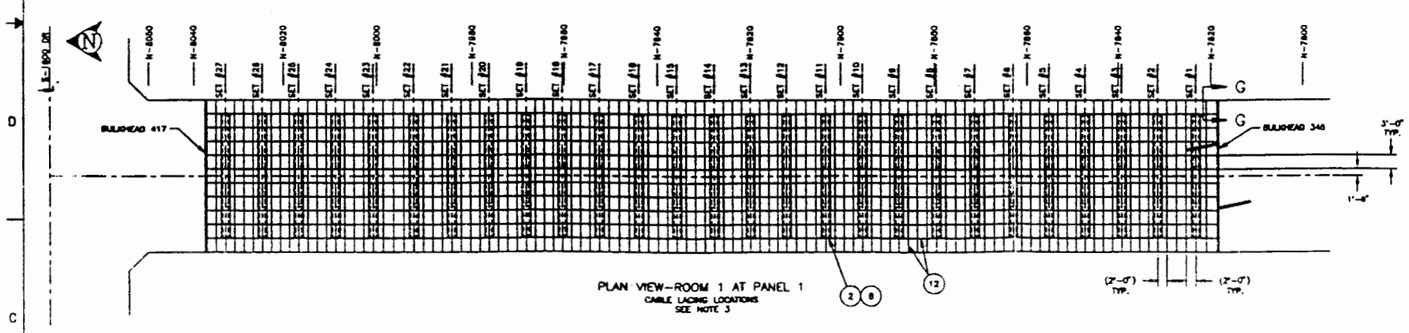
U.S. DEPARTMENT OF ENERGY
Westinghouse Waste Isolation Division
Waste Isolation Pilot Plant
CARLSBAD, NEW MEXICO

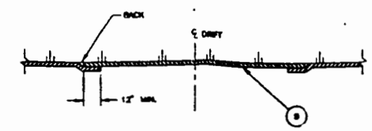
SUPPLEMENTARY ROOF SUPPORT
ROOM 1, PANEL 1
ROOM 1 CROSS SECTION

SYS AUC6 | DWG NO 54-D-003-W4 | REV NEW | DWG # OF #

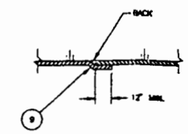




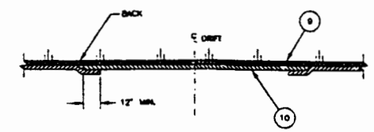




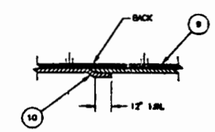
SECTION C-C
EXPANDED METAL OVERLAP
N.T.S.
SEE NOTE 4



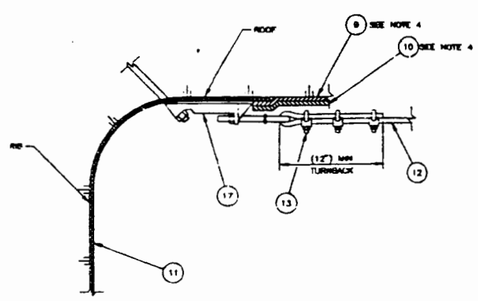
SECTION D-D
EXPANDED METAL OVERLAP
N.T.S.
SEE NOTE 4



SECTION E-E
WIRE MESH OVERLAP
N.T.S.
SEE NOTE 4



SECTION F-F
WIRE MESH OVERLAP
N.T.S.
SEE NOTE 4



SECTION G-G
WIRE MESH OVERLAP
N.T.S.
DETAIL 3

NOTES:

1. PLACEMENT OF EXPANDED METAL TO BE DETERMINED IN FIELD.
2. LOCATION OF WIRE MESH SHEETS TO BE DETERMINED IN FIELD.
3. LOCATION OF CABLE LACING TO BE DETERMINED IN FIELD.
4. EXPANDED METAL AND WIRE MESH OVERLAPPING EXAGGERATED. ACTUAL FIELD INSTALLATION MAY VARY.

REV	ISSUE DESCRIPTION	DFTR DATE	CHK DATE	COD DATE	DFTR MGR DATE	COD	PMR

THIS DRAWING CREATED BY CHANGEORDER CHANGE ORDER (COD) : 5488

DFTR. D.A.V/TM 2/3/91
CHKR. J. ALBI 5/16/91
COD. THE ORIGINAL SIGNATURE
COD. SEE ORIGINAL SIGNATURE

SHEET 1 OF 1 CHG ORDER 5/16/91
APPD
APPD
DFTR. MGR. ALBIGES 5/16/91
DOE REC CLOSE 5/16/91
SCALE 1" = 10' U.N.O.

U.S. DEPARTMENT OF ENERGY
Westinghouse Waste Isolation Division
Waste Isolation Pilot Plant

SUPPLEMENTARY ROOF SUPPORT
ROOM 1, PANEL 1
WIRE MESH & LACING DETAILS

SYM. AU06 DWG. NO. S4-D-003-W5 REV. NEW DWG. 0 OF 1

government in investigating the violations, use of environmental audits and other procedures to ensure compliance with all applicable environmental laws and regulations, and use of measures to remedy expeditiously and completely any violations and the harms caused thereby.

This guidance and the examples contained herein provide a framework for the determination of whether a particular case presents the type of circumstances in which lenience would be appropriate.

II. Factors to be Considered

Where the law and evidence would otherwise be sufficient for prosecution, the attorney for the Department should consider the factors contained herein, to the extent they are applicable, along with any other relevant factors, in determining whether and how to prosecute. It must be emphasized that these are examples of the types of factors which could be relevant. They do not constitute a definitive recipe or checklist of requirements. They merely illustrate some of the types of information which is relevant to our exercise of prosecutorial discretion.

It is unlikely that any one factor will be dispositive in any given case. All relevant factors are considered and given the weight deemed appropriate in the particular case. See Federal Principles of Prosecution (U.S. Dept. of Justice, 1980), Comment to Part A.2; Part B.3.

A. Voluntary Disclosure

The attorney for the Department should consider whether the person[1] made a voluntary, timely and complete disclosure of the matter under investigation. Consideration should be given to whether the person came forward promptly after discovering the noncompliance, and to the quantity and quality of information provided. Particular consideration should be given to whether the disclosure substantially aided the government's investigatory process, and whether it occurred before a law enforcement or regulatory authority (federal, state or local authority) had already obtained knowledge regarding noncompliance. A disclosure is not considered to be "voluntary" if that disclosure is already specifically required by law, regulation, or permit.[2]

B. Cooperation

The attorney for the Department should consider the degree and timeliness of cooperation by the person. Full and prompt cooperation is essential, whether in the context of a voluntary disclosure or after the government has independently learned of violation. Consideration should be given to the violator's

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1/ As used in this document, the terms "person" and "violator are intended to refer to business and nonprofit entities as we as individuals.

2/ For example, any person in charge of a vessel or of an on shore facility or an offshore facility is required to notify appropriate agency of the United States Government of any discharge of oil or a hazardous substance into or upon inter the navigable waters of the United States. Section 311(b)(5) the Clean Water Act, 33 U.S.C. 1321(b)(5), as amended by the Pollution Act of 1990, Pub. L. 101-380, § 4301(a), 104 Stat. 533 (1990).

willingness to make all relevant information (including the complete results of any internal or external investigation and the names of all potential witnesses) available to government investigators and prosecutors. Consideration should also be given to the extent and quality of the violator's assistance to the government's investigation.

C. Preventive Measures and Compliance Programs

The attorney for the Department should consider the existence and scope of any regularized, intensive, and comprehensive environmental compliance program; such a program may include an environmental compliance or management audit. Particular consideration should be given to whether the compliance or audit program includes sufficient measures to identify and prevent future noncompliance, and whether the program was adopted in good faith in a timely manner.

Compliance programs may vary but the following questions should be asked in evaluating any program: Was there a strong institutional policy to comply with all environmental requirements? Had safeguards beyond those required by existing law been developed and implemented to prevent noncompliance from occurring? Were there regular procedures, including internal or external compliance and management audits, to evaluate, detect, prevent and remedy circumstances like those that led to the noncompliance? Were there procedures and safeguards to ensure the integrity of any audit conducted? Did the audit evaluate all sources of pollution (i.e., all media), including the possibility

of cross-media transfers of pollutants? Were the auditor's recommendations implemented in a timely fashion? Were adequate resources committed to the auditing program and to implementing its recommendations? Was environmental compliance a standard by which employee and corporate departmental performance was judged?

D. Additional Factors Which May Relevant

1. Pervasiveness of Noncompliance

Pervasive noncompliance may indicate systemic or repeated participation in or condonation of criminal behavior. It may also indicate the lack of a meaningful compliance program. In evaluating this factor, the attorney for the Department should consider, among other things, the number and level of employees participating in the unlawful activities and the obviousness, seriousness, duration, history, and frequency of noncompliance.

2. Internal Disciplinary Action

Effective internal disciplinary action is crucial to any compliance program. The attorney for the Department should consider whether there was an effective system of discipline fo employees who violated company environmental compliance polici Did the disciplinary system establish an awareness in other employees that unlawful conduct would not be condoned?

3. Subsequent Compliance Efforts

The attorney for the Department should consider the exte of any efforts to remedy any ongoing noncompliance. The promptness and completeness of any action taken to remove th source of the noncompliance and to lessen the environmental

resulting from the noncompliance should be considered. Considerable weight should be given to prompt, good-faith efforts to reach environmental compliance agreements with federal or state authorities, or both. Full compliance with such agreements should be a factor in any decision whether to prosecute.

III. Application of These Factors to Hypothetical Examples[3/]

These examples are intended to assist federal prosecutors in their exercise of discretion in evaluating environmental cases. The situations facing prosecutors, of course, present a wide variety of fact patterns. Therefore, in a given case, some of the criteria may be satisfied while others may not. Moreover, satisfaction of various criteria may be a matter of degree. Consequently, the effect of a given mix of factors also is a matter of degree. In the ideal situation, if a company fully meets all of the criteria, the result may be a decision not to prosecute that company criminally. Even if satisfaction of the criteria is not complete, still the company may benefit in terms of degree of enforcement response by the government. The following hypothetical examples are intended to illustrate the operation of these guidelines.

Example 1:

This is the ideal case in terms of criteria satisfaction and consequent prosecution leniency.

3/ While this policy applies to both individuals and organizational violators, these examples focus particularly upon situations involving organizations.

1. Company A regularly conducts a comprehensive audit of its compliance with environmental requirements.

2. The audit uncovers information about employees' disposing of hazardous wastes by dumping them in an unpermitted location.

3. An internal company investigation confirms the audit information. (Depending upon the nature of the audit, this follow-up investigation may be unnecessary.)

4. Prior to the violations the company had a sound compliance program, which included clear policies, employee training, and a hotline for suspected violations.

5. As soon as the company confirms the violations, it discloses all pertinent information to the appropriate government agency; it undertakes compliance planning with that agency; and it carries out satisfactory remediation measures.

6. The company also undertakes to correct any false information previously submitted to the government in relation to the violations.

7. Internally the company disciplines the employees actually involved in the violations, including any supervisor who was lax in preventing or detecting the activity. Also, the company reviews its compliance progr to determine how the violations slipped by and corrects t weaknesses found by that review.

8. The company discloses to the government the names of the employees actually responsible for the violations, and it cooperates with the government by providing documentation necessary to the investigation of those persons.

Under these circumstances Company A would stand a good chance of being favorably considered for prosecutorial leniency, to the extent of not being criminally prosecuted at all. The degree of any leniency, however, may turn upon other relevant factors not specifically dealt with in these guidelines.[4/]

Example 2:

At the opposite end of the scale is Company Z, which meets few of the criteria. The likelihood of prosecutorial leniency, therefore, is remote. Company Z's circumstances may include any of the following:

1. Because an employee has threatened to report a violation to federal authorities, the company is afraid that investigators may begin looking at it. An audit is undertaken, but it focuses only upon the particular violation, ignoring the possibility that the violation may be indicative of widespread activities in the organization.

2. After completing the audit, Company Z reports the violations discovered to the government.

4/ For example, if the company had a long history of noncompliance, the compliance audit was done only under pressure from regulators, and a timely audit would have ended the violations much sooner, those circumstances would be considered.

3. The company had a compliance program, but it was effectively no more than a collection of paper. No effort is made to disseminate its content, impress upon employees its significance, train employees in its application, or oversee its implementation.

4. Even after "discovery" of the violation the company makes no effort to strengthen its compliance procedures.

5. The company makes no effort to come to terms with regulators regarding its violations. It resists any remedial work and refuses to pay any monetary sanctions.

6. Because of the non-compliance, information submitted to regulators over the years has been materially inaccurate, painting a substantially false picture of the company's true compliance situation. The company fails to take any steps to correct that inaccuracy.

7. The company does not cooperate with prosecutors in identifying those employees (including managers) who actually were involved in the violation, and it resists disclosure of any documents relating either to the violations or to the responsible employees.

In these circumstances leniency is unlikely. The only positive action is the so-called audit, but that was so narrow focused as to be of questionable value, and it was undertaken only to head off a possible criminal investigation. Otherwise the company demonstrated no good faith either in terms of

compliance efforts or in assisting the government in obtaining a full understanding of the violation and discovering its sources.

Nonetheless, these factors do not assure a criminal prosecution of Company Z. As with Company A, above, other circumstances may be present which affect the balance struck by prosecutors. For example, the effect of the violation (because of substance, duration, or amount) may be such that prosecutors would not consider it to be an appropriate criminal case. Administrative or civil proceedings may be considered a more appropriate response.

Other examples:

Between these extremes there is a range of possibilities. The presence, absence, or degree of any criterion may affect the prosecution's exercise of discretion. Below are some examples of such effects:

1. In a situation otherwise similar to that of Company A, above, Company B performs an audit that is very limited in scope and probably reflects no more than an effort to avoid prosecution. Despite that background, Company B is cooperative in terms of both bringing itself into compliance and providing information regarding the crime and its perpetrators. The result could be any of a number of outcomes, including prosecution of a lesser charge or a decision to prosecute the individuals rather than the company.

2. Again the situation is similar to Company A's, but Company C refuses to reveal any information regarding the individual violators. The likelihood of the government's prosecuting the company are substantially increased.

3. In another situation similar to Company A's, Company D chooses to "sit on" the audit and take corrective action without telling the government. The government learns of the situation months or years after the fact.

A complicating fact here is that environmental regulatory programs are self policing: they include a substantial number of reporting requirements. If reports which in fact presented false information are allowed to stand uncorrected, the reliability of this system is undermined. They also may lead to adverse and unfair impacts upon other members of the regulated community. Fo example, Company D failed to report discharges of X contaminant into a municipal sewer system, discharges tha were terminated as a result of an audit. The sewer authority, though, knowing only that there have been excessive loadings of X, but not knowing that Company D a source, tightens limitations upon all known sources o Thus, all of those sources incur additional treatment expenses, but Company D is unaffected. Had Company D revealed its audit results, the other companies would have suffered unnecessary expenses.

In some situations, moreover, failure to report is a crime. See, e.g., 33 U.S.C. § 1321(b)(5) and 42 U.S.C. § 9603(b). To illustrate the effect of this factor, consider Company E, which conducts a thorough audit and finds that hazardous wastes have been disposed of by dumping them on the ground. The company cleans up the area and tightens up its compliance program, but does not reveal the situation to regulators. Assuming that a reportable quantity of a hazardous substance was released, the company was under a legal obligation under 42 U.S.C. § 9603(b) to report that release as soon as it had knowledge of it, thereby allowing regulators the opportunity to assure proper clean up. Company E's knowing failure to report the release upon learning of it is itself a felony.

In the cases of both Company D and Company E, consideration would be given by prosecutors for remedial efforts; hence prosecution of fewer or lesser charges might result. However, because Company D's silence adversely affected others who are entitled to fair regulatory treatment and because Company E deprived those legally responsible for evaluating cleanup needs of the ability to carry out their functions, the likelihood of their totally escaping criminal prosecution is significantly reduced.

4. Company F's situation is similar to that of Company B. However, with regard to the various violations shown by the audit, it concentrates upon correcting only the easier, less

expensive, less significant among them. Its lackadaisical approach to correction does not make it a strong candidate for leniency.

5. Company G is similar to Company D in that it performs an audit and finds violations, but does not bring them to the government's attention. Those violations do not involve failures to comply with reporting requirements. The company undertakes a program of gradually correcting its violations. When the government learns of the situation, Company G still has not remedied its most significant violations, but claims that it certainly planned to get to them. Company G could receive some consideration for its efforts, but its failure to disclose and the slowness of its remedial work probably mean that it cannot expect a substantial degree of leniency.

6. Comprehensive audits are considered positive efforts toward good faith compliance. However, such audits are not indispensable to enforcement leniency. Company H's situation is essentially identical to that of Company A, except for the fact that it does not undertake a comprehensive audit. It does not have a formal audit program, but, as a part of its efforts to ensure compliance does realize that it is committing an environmental violation. It thereafter takes steps otherwise identical those of Company A in terms of compliance efforts and cooperation. Company H is also a likely candidate for leniency, including possibly no criminal prosecution.

In sum, mitigating efforts made by the regulated community will be recognized and evaluated. The greater the showing of good faith, the more likely it will be met with leniency. Conversely, the less good faith shown, the less likely that prosecutorial discretion will tend toward leniency.

IV. Nature of this Guidance

This guidance explains the current general practice of the Department in making criminal prosecutive and other decisions after giving consideration to the criteria described above, as well as any other criteria that are relevant to the exercise of criminal prosecutorial discretion in a particular case. This discussion is an expression of, and in no way departs from, the long tradition of exercising prosecutorial discretion. The decision to prosecute "generally rests entirely in [the prosecutor's] discretion." Bordenkircher v. Hayes, 434 U.S. 357, 364 (1978).[5] This discretion is especially firmly held by the criminal prosecutor.[6] The criteria set forth above are intended only as internal guidance to Department of Justice attorneys. They are not intended to, do not, and may not be relied upon to create a right or benefit, substantive or procedural, enforceable

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[5] Although some statutes have occasionally been held to require civil enforcement actions, see, e.g., Dunlop v. Bachowski, 421 U.S. 560 (1975), those are unusual cases, and the general rule is that both civil and criminal enforcement is at the enforcement agency's discretion where not prescribed by law. Heckler v. Chaney, 470 U.S. 821, 830-35 (1985); Cutler v. Hayes, 818 F.2d 879, 893 (D.C. Cir. 1987) (decisions not to enforce are not reviewable unless the statute provides an "inflexible mandate").

[6] Newman v. United States, 382 F.2d 479, 480 (D.C. Cir. 1967).

at law by a party to litigation with the United States, nor do they in any way limit the lawful litigative prerogatives, including civil enforcement actions, of the Department of Justice or the Environmental Protection Agency. They are provided to guide the effective use of limited enforcement resources, and do not derive from, find their basis in, nor constitute any legal requirement, whether constitutional, statutory, or otherwise, to forego or modify any enforcement action or the use of any evidentiary material. See Principles of Federal Prosecution (U.S. Dept. of Justice, 1980) p. 4; United States Attorneys' Manual (U.S. Dept. of Justice, 1986) 1-1.000.

SUMMARY OF AFFIDAVIT OF LOKESH CHATURVEDI, Ph.D.

Dr. Chaturvedi is a geologist and civil engineer who is Deputy Director of the New Mexico Environmental Evaluation Group ("EEG"), which provides independent technical evaluation of the WIPP project to the State of New Mexico. Dr. Chaturvedi has worked for EEG since 1982. He has published over fifty research papers, is familiar with the numerous published EEG reports dealing with scientific and technical aspects of the WIPP and has visited the WIPP site on about 50 occasions including about 40 underground inspections. (¶¶ 2-6).

Dr. Chaturvedi discusses the background and proposed use of the WIPP and the EPA standards which ultimately will be used to decide whether the WIPP is suitable for the safe and permanent disposal of transuranic radioactive waste. (¶¶ 8-14). He concludes that:

- The EPA standards for demonstrating the suitability of the WIPP do not require underground testing with radioactive waste. (¶ 14).
- DOE is unable at the present time to perform any of its proposed tests at the WIPP site except for the Dry Bin Scale Tests. (¶ 15).
- The Bin Scale Tests (both Dry and Wet) clearly do not need to be conducted in the facility and could be done elsewhere, in which event the experiments would be safer and the test data would be more reliable and available sooner than if the tests are performed at the WIPP. (¶¶ 15 and 17).

- There are serious problems in performing the Dry Bin Scale Tests at the WIPP including upheaving salt floors which are unsuitable for holding bins, collapsing roofs in the underground excavations and the expected generation of flammable gasses within the bins. (¶ 15(a)-(d)).

- Although DOE has acknowledged that the Dry Bin Scale Tests do not have to be performed at the WIPP site, the decision by DOE not to do this appears more logistical (work force available at WIPP) and symbolic (emphasis on bringing radioactive waste to WIPP) than scientific (need to experiment in the mine, even though there may be delays in obtaining data and the quality of data may not be reliable). (¶ 17).

- EEG has various public health and safety concerns relating to Public Land Order 6826 (January 28, 1991), including the absence of any numerical limit on the amount of radioactive waste brought by DOE to the WIPP and the absence of a requirement for disposal of the waste after the experiments. (¶ 16).

SUMMARY OF AFFIDAVIT OF MARION DEMING, R.N.

Marion Deming is a Registered Nurse and a member of Local 1199, National Union of Hospital and Health Care Employees, AFL-CIO, currently serving as the National Union Health and Safety Coordinator for the approximately 30,000 union members across the country. At the invitation of the New Mexico affiliate union, Ms. Deming travelled to New Mexico during September 1991 and personally conducted a survey of the hospitals located along the designated New Mexico routes for transportation of radioactive waste to the WIPP. Ms. Deming concludes (¶ 3) that none of the hospitals along the WIPP route are currently ready to respond to potential emergencies related to waste transport to the WIPP. None of the hospitals have WIPP-specific emergency response plans in place. Hospital facilities lack separate entrances for contaminated patients. They lack decontamination facilities for both patients and employees, separate facilities to dispose of contaminated waste water, and separate ventilation systems. In addition, none of the employees in hospitals along the WIPP route have received hands-on training that focuses specifically on the hazards associated with WIPP emergencies. The employees in the hospitals were unaware of the availability of radiation detectors used to measure plutonium contamination, with the exception of one hospital, whose detector was broken. Only one hospital had the medicatio necessary to combat plutonium contamination.

SUMMARY OF THE AFFIDAVIT OF JUDITH M. ESPINOSA

Judith Espinosa is the Cabinet Secretary of the New Mexico Environment Department (NMED). Secretary Espinosa has statutory authority to enforce the New Mexico Hazardous Waste Act (HWA) which is the analog to the federal Resource Conservation and Recovery Act (RCRA). Since 1985 New Mexico has had federal authorization to administer and enforce a hazardous waste management program.

Secretary Espinosa testifies that the DOE submitted a mixed waste Part A permit application for WIPP on or about January 18, 1991 and that DOE submitted a mixed waste Part B permit application for WIPP on or about February 28, 1991. By letter dated July 1, 1991, NMED preliminarily concluded that DOE did not make a timely HWA mixed waste permit application for the WIPP. NMED further preliminarily concluded that the WIPP may not qualify for interim status under the HWA.

SUMMARY OF AFFIDAVIT OF GABRIEL FERNANDEZ-DELGADO

Dr. Fernandez is a civil and geotechnical engineer and a Research Engineer at the University of Illinois in Urbana and has taught there as Visiting Assistant Professor of Civil Engineering.

Dr. Fernandez concludes that the waste emplaced for tests cannot be expected to be retrievable after 18 months and may be nonretrievable within about a year. Specifically:

1. There can be no assurance that Room 1, Panel 1, the test room, will remain stable for more than 18 months from this date. (¶ 9).
2. SPDV Room 1, where a major roof fall occurred in February 1991, was barred to access after slightly more than six years, and Room 1, Panel 1 is slightly over five years old and if it follows the pattern of SPDV Room 1 would be closed to access within about a year. (¶ 9).
3. The DOE's proposed roof support system is defective in the following respects:
   a. The bolt length which is anchored in resin is too short. (¶ 17).
   b. The system is too limited in ability to deform to adjust load distribution; thus, bolts may become overloaded. (¶ 18).
   c. The system fails to deal with the shear forces which will bear upon the roof bolts at the point of rock

fracture and where the bolt connects to a support channel. (¶¶ 19-20).

d. The system must bear the weight of buckling forces, which exceed the design load. (¶ 15).

e. Essential equipment clearance may be lost due to breakup of the roof beam. (¶ 21).

f. Failure can occur or accelerate at an unpredictable rate. The system may result in sudden accelerations. (¶ 22).

SUMMARY OF AFFIDAVIT OF DON HANCOCK

Mr. Hancock is the Administrator of the Southwest Research and Information Center ("SRIC") in Albuquerque, New Mexico and Director of SRIC's Nuclear Waste Safety Project. Since 1976, he has participated in all public proceedings involving the evaluation under NEPA of the potential environmental impacts of the WIPP.

Mr. Hancock concludes:

1. Based on internal documents which he obtained from the agencies (attached as exhibits), DOI requested DOE to analyze the alternative of delaying underground experimentation with radioactive waste at the WIPP site pending legislative withdrawal of the land by Congress. DOI stated that DOI preferred a legislative withdrawal and wanted to be assured that the environmental impacts of an administrative withdrawal would be "considerably less" than those of a legislative transfer of land. (¶ 8).

2. The Final Supplement to Environmental Impact Statement ("FSEIS") and the DOI and DOE Records of Decision do not consider or analyze the alternative of awaiting Congressional withdrawal before proceeding with underground testing with radioactive waste. (¶ 9).

3. Comments submitted by the SRIC and other parties, including the comment that legislative withdrawal should be analyzed, were addressed insufficiently or not at all in the FSEIS. (¶ 10).

4. He is familiar with the WIPP legislation currently being considered by Congress. Based on the bills and amendments the environmental impacts of legislative land withdrawal might be significantly different from the environmental effects of administrative withdrawal, particularly with respect to determination of compliance with disposal standards and the terms and conditions of on-site experimentation with radioactive waste. (¶ 11).

SUMMARY OF AFFIDAVIT OF LINDA L. LEHMAN

Ms. Lehman is a geologist and hydrogeologist who specializes in nuclear waste disposal issues and performance assessments of nuclear waste disposal facilities.

Ms. Lehman concludes that the DOE will not be prejudiced by an inability to conduct tests with radioactive waste at the WIPP. Specifically:

1. There is no necessity for the dry bin scale tests to be conducted underground at the WIPP. The tests could be conducted on the surface at a laboratory facility at or near the present storage location, probably with less risk and inconvenience. (¶¶ 18-23).
2. The remaining elements of the DOE test plan - the wet bin-scale tests, bin-scale solubility tests, and alcove tests - are not now prepared to be conducted. Moreover, all the bin-scale tests can be conducted at an off-site laboratory facility. (¶¶ 24-27).
3. The information to be obtained from the dry bin-scale tests is of unknown importance and appears to be of marginal importance. (¶¶ 28-49).

SUMMARY OF THE AFFIDAVIT OF JOHN D. LESHY

Professor Leshy teaches law at Arizona State University and is former Associate Solicitor at the DOI from 1977 to 1980. He has taught, written and consulted extensively about the FLPMA and has specialized knowledge of the FLPMA with respect to the WIPP based on his experience at the DOI.

Professor Leshy concludes that a legislative withdrawal would be required in order to permanently store nuclear waste at the WIPP site. (paragraph 23)

As Associate Solicitor at the DOI, Leshy wrote a memorandum to the BLM stating, "We have suggested informally to DOE's counsel that DOE seek specific legislation which would reserve the necessary core areas (surface and subsurface) for the project. DOE appears to favor such an approach..." (paragraph 15, Exhibit C)

Congress passed the FLPMA in part to prevent the executive from making land withdrawals in an uncontrolled and haphazard manner and on the basis of the Public Land Law Review Commission's recommendation that withdrawals of a permanent or indefinite term should be accomplished only by an act of Congress. (paragraphs 18 and 19)

The reporting required in the FLPMA is mandatory for land withdrawals (paragraph 20). An extension of an existing administrative withdrawal must also be reported to Congress. Moreover FLPMA mandates that an administrative withdrawal shall not be extended for a purpose which is different from the purpose of the original withdrawal. An extension of a withdrawal cannot be for a period longer than the original withdrawal period. (paragraph 22)

SUMMARY OF AFFIDAVIT OF JACK PARKER

Mr. Parker is a geologist and mining engineer, specializing in rock mechanics. He is an independent consultant who was selected by the DOE in 1991 as a member of the DOE's expert panel reviewing the room stability problems encountered at the WIPP.

Mr. Parker concludes that the waste emplaced for tests will become non-retrievable within less than a year. Specifically:

1. Room 1, Panel 1, the test room where the waste will be placed, exhibits fractures similar to those which gave rise to the roof collapse in SPDV Room 1 in February 1991. (¶31).
2. Failure of the test room is highly probable. (¶31).
3. If the test room follows the pattern of SPDV Room 1, it can be expected to be closed to access by October 1992. (¶31).
4. The DOE's proposed roof support system, relying upon rock bolts, is exposed to shearing and breaking of the bolts. There is insufficient bolt clearance to accommodate shear movement. (¶¶ 37-42).
5. The equipment clearance necessary to retrieve the waste will be lost within less than a year. (¶¶ 44-45, 47).
6. The DOE's roof support system will probably impair warning of a roof failure by masking the acceleration of closure which provides a warning signal. (¶ 47).

SUMMARY OF AFFIDAVIT OF CHRISTOPHER J. WENTZ

Christopher J. Wentz is employed by the New Mexico Department of Energy, Minerals and Natural Resources ("EMNRD"). He is Coordinator of EMNRD's Radioactive Waste Task Force which provides technical and policy analysis concerning the WIPP and is responsible for coordinating emergency response preparedness.

Mr. Wentz has calculated that the State of New Mexico would lose at least $50 million in mineral revenues as a result of administrative withdrawal of public lands for the WIPP (which calculation is based only on oil and gas resources and does not include foregone revenues from any of the other known mineral resources at the WIPP site). (¶¶ 3-10). He states that on October 3, 1991, DOE notified the State of New Mexico that the first shipment of radioactive waste would arrive at the WIPP "as early as October 10, 1991." He subsequently has been informed that the truck will depart from the Idaho National Engineering Laboratory in Idaho Falls, Idahc at 10:00 a.m. MST (Noon EST) on October 10, 1991. (¶ 11). Upon commencement of the first shipment of radioactive waste, the State and its local governments will have to mobilize a substantial amount of personnel and resources, ranging from mechanical and radiological inspections to emergency medical services to State and local law enforcement. (¶¶ 12-13).

SUMMARY OF AFFIDAVIT OF MARCUS A. WILEY

Mr. Wiley is a registered professional engineer with nineteen years of mining industry experience. He is president of a mining consulting firm and has been responsible for the development and maintenance of dozens of project schedules for mining and related construction activities.

Mr. Wiley reviewed DOE planning documents concerning Test Phase activities. He employed project management computer analysis and concludes that ten years, and in any event no less than nine and one-half years, would be required to complete the Test Phase activities. This period is significantly beyond the term of the administrative land withdrawal.

SUMMARY OF AFFIDAVIT OF LOKESH CHATURVEDI, Ph.D.

Dr. Chaturvedi is a geologist and civil engineer who is Deputy Director of the New Mexico Environmental Evaluation Group ("EEG"), which provides independent technical evaluation of the WIPP project to the State of New Mexico. Dr. Chaturvedi has worked for EEG since 1982. He has published over fifty research papers, is familiar with the numerous published EEG reports dealing with scientific and technical aspects of the WIPP and has visited the WIPP site on about 50 occasions including about 40 underground inspections. (¶¶ 2-6).

Dr. Chaturvedi discusses the background and proposed use of the WIPP and the EPA standards which ultimately will be used to decide whether the WIPP is suitable for the safe and permanent disposal of transuranic radioactive waste. (¶¶ 8-14). He concludes that:

- The EPA standards for demonstrating the suitability of the WIPP do not require underground testing with radioactive waste. (¶ 14).
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- Although DOE has acknowledged that the Dry Bin Scale Tests do not have to be performed at the WIPP site, the decision by DOE not to do this appears more logistical (work force available at WIPP) and symbolic (emphasis on bringing radioactive waste to WIPP) than scientific (need to experiment in the mine, even though there may be delays in obtaining data and the quality of data may not be reliable). (¶ 17).

- EEG has various public health and safety concerns relating to Public Land Order 6826 (January 28, 1991), including the absence of any numerical limit on the amount of radioactive waste brought by DOE to the WIPP and the absence of a requirement for disposal of the waste after the experiments. (¶ 16).

STATE OF NEW MEXICO)
) ss.
COUNTY OF BERNALILLO)

AFFIDAVIT OF LOKESH CHATURVEDI, Ph.D.

I, Lokesh Chaturvedi, Ph.D., do hereby depose upon my oath and state as follows:

1. My name is Lokesh Chaturvedi and I reside in the City of Albuquerque, County of Bernalillo, State of New Mexico.

2. I am employed as the Deputy Director of the Environmental Evaluation Group ("EEG") and have been so employed since March 1988. From June 1982 to March 1988 I was employed by EEG as Senior Engineering Geologist.

3. As detailed in my resume, a copy of which is attached hereto as Exhibit A-1, I received a Ph.D. in Geological Sciences from Cornell University in 1969, a M.S. in Civil Engineering from Purdue University in 1965, a M.Sc. in Applied Geology from University of Roorkee in Roorkee, India in 1963 and a B.Sc.in Geology, Physics and Mathematics from Maharaja's College in Jaipur, India in 1960. I have taught courses in Introductory Geology, Physical Geology, Geology for Engineers, Introduction to Geological Engineering, Site Investigation, Engineering Geology, Subsurface Exploration, Environmental Geology, Hydrogeology, Soil Mechanics, Rock Mechanics, Geomorphology and Geological Oceanography at both undergraduate and graduate levels. Since 1968, I have authored or coauthored 56 published research papers on the subjects of Radioactive Waste Disposal,

Remote Sensing, Geothermal Hydrology and Mechanical Properties of Rocks. I have performed several funded research projects in site evaluation for radioactive waste disposal, geothermal hydrology and multispectral remote sensing and I have performed several professional consulting projects in the areas of hydrogeology, subsurface exploration, engineering materials and nuclear waste disposal site investigation.

4. The State of New Mexico established EEG in 1978 to provide independent technical evaluation of the Waste Isolation Pilot Plant ("WIPP") located near Carlsbad, New Mexico. The United States Congress provides 100% federal funding for EEG through appropriations to the U.S. Department of Energy ("DOE"). EEG has offices in both Albuquerque and Carlsbad. In addition to multi-disciplinary technical evaluation of the project, EEG performs independent environmental monitoring of air, water and soil, both on-site at WIPP and in surrounding communities.

5. Since 1978, EEG has published 49 EEG Reports and many other papers dealing with scientific and technical aspects of WIPP. I have read and am generally familiar with the contents of all of these published EEG Reports, a list of which is attached hereto as Exhibit A-2.

6. I personally have visited the WIPP site on about 50 occasions including about 40 underground inspections.

7. The Attorney General of the State of New Mexico has requested that I provide this affidavit in order to reflect my

assessment of various scientific and technical aspects of the WIPP Project.

8. The WIPP Project was authorized by Public Law 96-164 (December 29, 1979) "for the express purpose of providing a research and development facility to demonstrate the safe disposal of radioactive wastes resulting from the defense activity and programs of the United States exempted from regulation by the Nuclear Regulatory Commission." DOE manages WIPP, and Sandia National Laboratory provides scientific and technical services.

The WIPP facility is intended for permanent disposal of over 6 million cubic feet of transuranic waste, containing about 15 million curies of radioactivity. Transuranic ("TRU") wastes consist of radioactive elements heavier than uranium which remain radioactive for thousands of years. TRU includes radioactive isotopes of plutonium, thorium, americium, uranium, neptunium, curium, californium, cobalt, strontium, ruthenium, antimony, tin cesium, cerium, and europium. About 97% by volume of the waste planned for WIPP consists of contact-handled ("CH-TRU") radioactive waste packed in ordinary 55-gallon steel drums. Ultimately, about 850,000 drums of CH-TRU waste are expected to be emplaced in the WIPP repository. The drums of CH-TRU waste also contain various kinds of hazardous waste which, for all but approximately a dozen drums, has not yet been characterized as required for experimental purposes. The

remaining waste--only 3% by volume but approximately 30% by curie content--is highly radioactive remote-handled ("RH-TRU") waste, about 7,500 containers of which will be shipped in casks not yet designed.

Because of the need to permanently isolate radioactive wastes from the environment, the WIPP facility must be carefully designed and tested to ensure that the probability of escape of radionuclides to the biosphere (the Earth's air, water and soil) does not exceed the limits mandated in regulations which are to be promulgated by the U.S. Environmental Protection Agency ("EPA").

9. The WIPP repository is located in southeastern New Mexico about 25 miles east of Carlsbad, New Mexico. The WIPP site is located on 10,240 acres, constituting a 4 mile by 4 mile piece of land. When completed, WIPP will consist of 56 underground "rooms" excavated in an ancient salt bed 2,150 feet beneath the surface. Salt deforms plastically under pressure and is expected to close around the waste after emplacement. Each room will be 300 feet long, 33 feet wide and 13 feet high. To date, seven of the planned 56 waste rooms have been completed. CH-TRU waste is planned to be stored in 55-gallon drums stack three high in the rooms and drifts. RH-TRU waste is expected to be placed in containers 10 feet long and 2 feet in diameter in horizontal holes about three feet in diameter drilled into the walls of the various rooms.

10. The geohydrology of the WIPP site is an important parameter that needs to be understood for reliable predictions of future behavior of the repository. The WIPP site is located in the lower part of a 2,000 foot thick salt formation known as the Salado Formation, which dates to the Permian Age (225 million years ago). Overlying the Salado Formation is the Rustler Formation, consisting of anhydrite and siltstone with two water-bearing dolomite members. Underlying the Salado Formation is the Castile Formation. Thirteen boreholes have encountered pressurized brine in the upper Castile Formation in the northern Delaware Basin, where the WIPP site is located.

11. Groundwater provides a potential pathway for radionuclide migration to the biosphere. Because of this, WIPP site assessment to date has focused largely on the Rustler Formation's water-bearing members and the Castile Formation's pressurized brine reservoirs. Two previous WIPP sites in the area are abandoned by DOE after exploratory boreholes (ERDA-6, drilled in 1975, and WIPP-12 in 1981) encountered pressurized brine beneath or adjacent to each proposed site. The present WIPP site is located about 1.25 miles to the south of the WIPP-12 borehole.

12. In addition to the water present in the Rustler Formation's water-bearing members and the Castile Formation's brine reservoirs, the amount of brine expected to seep directly from the Salado Formation into the excavations is another

important parameter that needs to be understood. The Salado salt may be saturated with brine, and the brine inflow from the salt, albeit at very low permeability, may contribute an unknown quantity of brine to the repository once the ventilation of the facility ceases to remove moisture. DOE is conducting a series of tests to assess the permeability of the salt beds surrounding the WIPP facility and to measure the amount of brine inflow. None of these ongoing tests requires the emplacement of radioactive waste in the repository.

13. The rapid closure of excavations in salt is another important parameter. The WIPP facility was designed for a 25 year operation because of the predicted difficulties in keeping it operationally safe much beyond that period. Geomechanical measurements in the WIPP excavations show that the closure rate due to the creep of salt is three to four times faster than initially predicted. Because of closure and the presence of layers of clay and anhydrite in the salt, some parts of the facility excavated in 1983 have collapsed or are already unsafe and closed to entry. On June 19, 1990, a slab of rock weighing an estimated 100 tons fell from the ceiling in heated Room A-2. On February 4, 1991, the roof of Site and Preliminary Design Validation ("SPDV") Room 1, excavated in 1983, failed; a slab of rock weighing an estimated 1,200 tons separated from the roof and crashed onto the floor. Entry to SPDV Room 1 is now forbidden. SPDV Room 1 is similar in dimensions and in geology

to the rooms where DOE proposes to conduct its experiments with radioactive waste. If the roof of a test room fails and collapses while test waste is present, the possible consequences to the workers and the waste bins could be severe. DOE is therefore designing an elaborate system to prevent roof failure and to provide warning of an impending failure.

14. The design and long-term safety of the WIPP repository, and the ultimate decision whether to use WIPP for permanent disposal of TRU waste, will be judged through an assessment of WIPP's compliance with standards to be promulgated by EPA ("EPA Standards," codified in 40 CFR 191). The EPA Standards require a probabilistic assessment of potential scenarios for release of radionuclides from the repository to the biosphere (groundwater, air or soil) for 10,000 years. The EPA Standards also set limits of probabilities and magnitudes of such releases.

The EPA Standards contain two subparts. Subpart A limits the radiation exposure of members of the public from the management and storage of radioactive waste and also applies to facilities designed for temporary storage of the waste. Subpart B was developed to assure long-term integrity of a geologic repository for nuclear waste. Standards contained in Subpart B apply to the proposed Nevada repository for high-level waste and to the WIPP. Since waste containers are expected to be received, handled, examined and transported underground prior to

permanent emplacement, Subpart A provisions apply to WIPP during waste handling operations. Compliance with Subpart B is required prior to a decision to leave the waste underground for permanent disposal.

Subpart B of the EPA Standards was vacated by the First Circuit Court of Appeals in 1987 and was remanded to EPA for revision and repromulgation. As of this date, EPA has not promulgated revised Subpart B Standards. Sandia National Laboratory on behalf of DOE expects to complete its assessment of compliance with the old, vacated Subpart B Standards in 1994. If the revised and repromulgated Subpart B Standards modify substantially the vacated Subpart B Standards, DOE's demonstration of compliance may require additional years.

Neither Subpart A nor Subpart B of the EPA Standards nor the latest draft of the revised Subpart B EPA Standards requires experimentation with actual radioactive or other waste in a repository.

15. As of this date, DOE has announced plans to ship a maximum of 105 bins of CH-TRU waste to the WIPP site to be used in the Dry Bin Scale Tests. Each bin will contain the contents of between 4 and 6 drums of CH-TRU waste. In my opinion, it is not necessary for DOE to experiment in the WIPP repository with actual waste, and in fact DOE has acknowledged that Bin Scale Tests do not have to be performed at the WIPP site.

DOE's plans for experimentation with waste at WIPP

have changed substantially during the past four years. In 1987, before any plans for waste experiments were available, DOE stated that it needed to ship 125,000 drums of waste (15% of the total volume capacity of WIPP) to WIPP for research and development purposes. In 1988, a draft of the first report outlining DOE's plans for experiments with TRU wastes at WIPP was issued. The report proposed filling four of the WIPP rooms with CH-TRU waste to monitor gas generation. The specific quantity of waste was not identified, but at approximately 6,000 drums per room, it would have been about 24,000 drums (2.8% of the total volume). In 1990, DOE published a new plan proposing to perform Laboratory Scale Tests, Bin Scale Tests and Alcove Scale Tests relating to the production, depletion and composition of gases from TRU waste. DOE proposed to commence these tests "in parallel."

With respect to the Laboratory Scale Tests, DOE intends to conduct the tests in laboratories away from the WIPP site.

With respect to the Alcove Scale Tests, DOE is unable to perform the tests at the present time. DOE engineers have attempted to test inflatable seals for the alcoves, but these have not been successful. Without an effective seal, gas would leak out of the alcoves, making accurate measurement of gas generation impossible and the data unreliable. DOE now plans to design and test a rigid concrete seal to contain the gases. DOE

expects to perform its "initial" gas barrier acceptance tests in May 1993. Because of the nature of the WIPP geologic strata where fractures rapidly develop all around the excavations in roofs, floors and walls within a year or two after excavation, it may not be possible to maintain the alcove seals for the duration of the Test Phase, even if a seal appears effective immediately after its emplacement.

With respect to the Bin Scale Tests, DOE determined in 1990 that the proposed Wet Bin Scale Tests, which involve sampling of liquids from the bins, could not be performed at WIPP, because the WIPP repository lacks the necessary containment to handle safely liquid plutonium contaminated samples. DOE is now prepared to perform only the Dry Bin Scale Tests, using a maximum of 105 bins or 630 drum-equivalent. (A "bin" is a rectangular steel box which holds 4 to 6 drums of CH-TRU waste.) The current DOE plan is to emplace the bins in rows along the walls of Rooms 1 and 2 of Panel 1, with two bins stacked in each row. Gas measurements from the bins would continue for five years. However, there remain several serious problems in performing the Dry Bin Scale Tests at the WIPP site.

(a) The upheaving salt floor in a mined room is not a suitable location for placing double-stacked bins, each weighing up to 2 tons. Even without any loading, the floor periodically needs to be dug up with heavy equipment and "reconstituted" with compressed crushed salt. There is no

published analysis of loading on this floor. It appears that at a minimum, the bins would have to be removed periodically to reconstitute the floor which will upset the test environment and possibly affect adversely the collection of data.

(b) Because the roofs of the SPDV rooms, which were excavated in 1983, became unstable within six years, the Panel 1 rooms where Bin Scale Tests are planned were rock-bolted with 10-foot mechanical rockbolts. Because of the mechanism of failure, however, the 10-foot rockbolts in Panel 1 rooms are not expected to extend significantly the life of Panel 1 rooms. Room 1 of Panel 1 was excavated in May to August 1986 and is five years old. The other rooms in Panel 1 were excavated between 1986 and 1988.

(c) EPA has placed various conditions upon DOE's underground experiments with waste, including the requirement that DOE demonstrate that the concentrations of flammable gases are less than 50% of the lower explosive limit ("LEL") in air. Periodic purging of the bins, to comply with this condition, is expected to affect adversely the reliability of the test data.

(d) The Dry Bin Scale Tests are going to be conducted in closed, pressurized vessels, thus, the underground atmosphere of the WIPP is not a factor in the experiment. Thus, as DOE has acknowledged, the Dry Bin Scale Tests do not have to be performed at WIPP.

16. EEG has several concerns relating to protection of the

public health and safety under the terms of the Public Land Order 6826 (issued on January 28, 1991):

(a) The Public Land Order does not establish a numerical limit on the amount of TRU waste DOE may bring to WIPP prior to demonstrating compliance with the EPA Standards governing safe disposal of radioactive materials. The Public Land Order requires DOE not to ... "exceed the amount that can feasibly be removed should the site not be selected as a permanent repository." While the DOE identified a need of 0.5% by volume (4500 drums) of the CH-TRU waste for experiments, the most recent version of the DOE WIPP decision plan (August 9, 1991) has deleted the solubility tests and the alcove tests. At this time DOE has concrete plans to experiment with a maximum of 105 bins (630 drums), that is 0.08% of the total volume or a total of 17.5 truckloads.

(b) The Public Land Order does not preclude the introduction of RH-TRU waste.

(c) The Public Land Order does not close the surface to mineral leasing.

(d) The Public Land Order deletes the prohibition on "burial of radioactive materials" found in the 1983 Public Land Order 6403. Hence, it appears that "burial" is no longer precluded.

(e) The Public Land Order is unclear whether operational demonstration with waste is allowed.

(f) The Public Land Order does not require any plans as to the disposition of the waste after the Test Plan.

(g) The Public Land Order does not require that experimentation with actual waste be of value in performance assessment.

17. The WIPP facility has been designed and constructed as a full-scale repository for permanent disposal of up to 850,000 drums of CH-TRU waste and 7,500 containers of RH-TRU waste. DOE plans, however, to use the WIPP facility for experimentation with TRU wastes over the course of five years or longer. With at least two years required for emplacement of waste and two years for retrieval, the total time necessary to conduct the Dry Bin Scale Tests is at least nine years after the initial emplacement.

The WIPP repository is not an ideal place for performing experiments with radioactive waste. Operational problems are repeatedly being encountered, and the solutions to these problems are expected to become more difficult with the aging of the facility. In contrast, the Bin Scale Tests planned by DOE to be conducted in the WIPP facility clearly do not need to be conducted in the facility and could be done elsewhere, in which event the test data would be available sooner than if the tests are performed at WIPP. The decision by DOE not to do this appears more logistical (work force available at WIPP) and symbolic (emphasis on bringing radioactive waste to WIPP) than

scientific (need to experiment in the mine, even though there may be delays in obtaining data and the quality of data may not be reliable).

In my opinion, it would be safer and faster to further define gas generation rates (the purpose of the Bin Scale Tests) by experimenting with waste in locations other than WIPP. In addition, DOE should accelerate its schedule for conducting the other tests that do not require the use of radioactive waste, the results of which are necessary to assess compliance with the EPA Standards. DOE's primary goal should be to complete the calculations necessary to assess WIPP's compliance with the EPA Standards, to create the basis for an informed decision about whether to use the site as a permanent repository for the Nation's TRU waste. No underground experiments with waste are needed to accomplish that goal.

Lokesh Chaturvedi

LOKESH CHATURVEDI, Ph.D.

Dated: _____

SUBSCRIBED AND SWORN TO before me by Lokesh Chaturvedi, Ph.D., on this 20th day of September, 1991.

Hari Khalsa

NOTARY PUBLIC

My Commission Expires:

April 23, 1994

LOKESH CHATURVEDI
5401 Vista Lejana N.E.
Albuquerque, New Mexico 87111
(505) 292-2140 home
(505) 828-1003 office

PRESENT POSITION: Deputy Director
Environmental Evaluation Group
7007 Wyoming Blvd. N.E., Suite F-2
Albuquerque, New Mexico 87109

Responsible for an independent technical evaluation of the Waste Isolation Pilot Plant (WIPP) - the first planned deep geological repository for radioactive waste in the US - as part of an interdisciplinary team of scientists and engineers.

SUMMARY OF PROFESSIONAL EXPERIENCE

Teaching: Have taught courses in Introductory Geology, Physical Geology, Geology for Engineers, Introduction to Geological Engineering, Site Investigation, Engineering Geology, Subsurface Exploration, Environmental Geology, Hydrogeology, Soil Mechanics, Rock Mechanics, Geomorphology and Geological Oceanography at both undergraduate and graduate levels.
Research: Have published research papers in Radioactive Waste Disposal, Remote Sensing, Geothermal Hydrology and Mechanical Properties of Rocks. Have performed several funded research projects in site evaluation for radioactive waste disposal, geothermal hydrology and multispectral remote sensing.
Consulting: Have performed several professional consulting projects in the areas of hydrogeology, subsurface exploration, engineering materials and nuclear waste disposal site investigation.

EDUCATION

Ph.D., Geological Sciences, 1969, Cornell University, Ithaca, NY.
M.S., Civil Engineering, 1965, Purdue University, Lafayette, IN.
M.Sc., Applied Geology, 1963, University of Roorkee, Roorkee, India.
B.Sc., Geology, Physics, Mathematics, 1960, Maharaja's College, Jaipur, India.

PROFESSIONAL EXPERIENCE

Deputy Director, Environmental Evaluation Group, Albuquerque, NM, March, 1988 to present.

Senior Engineering Geologist, Environmental Evaluation Group, Santa Fe, NM, June 1982 to March 1988.

PROFESSIONAL EXPERIENCE (Cont.)

Associate Professor in Geological Engineering, Departments of Earth Science and Civil Engineering, New Mexico State University, Las Cruces, NM, September 1976 - May 1982.

Assistant Professor in Geology, City University of New York, Hunter College, New York, NY, September 1974 - June 1976.

Senior Lecturer in Engineering Geology, Department of Civil Engineering, Indian Institute of Technology, New Delhi, India, September 1969 -June 1974.

Visiting Assistant Professor in Geological Engineering, Michigan Technological University, Houghton, MI, January 1969 - June 1969.

Research Assistant, Water Resources Research Institute and the Department of Geological Sciences, Cornell University, Ithaca, NY, January 1968 - December 1968.

Visiting Research Scientist, National Energy Authority of Iceland (Orkustofnun), Department of Natural Heat (Jardhitadeld), Reykjavik, Iceland, June 1967 - December 1967.

Teaching Assistant, Department of Geological Sciences, Cornell University, Ithaca, NY, September 1965 - June 1967.

Research Assistant, Multispectral Remote Sensing Research Project, Department of Soil Science, Purdue University, Lafayette, IN, January 1965 - May 1965.

Teaching Assistant, Department of Engineering Geology, Purdue University, Lafayette, IN, September 1963 - December 1965.

PROFESSIONAL SOCIETY AFFILIATIONS

Memberships: Geological Society of America
American Geophysical Union
American Society of Civil Engineers
Association of Engineering Geologists
Assoc. of Groundwater Scientists and Engineers
New Mexico Geological Society

Committees:
- Corresponding Member, Committee on Groundwater, American Society of Civil Engineers, 1982 - present.
- GSA Representative, Joint ASCE-GSA-AEG Committee on Engineering Geology, 1984 - present.
- Member, International Association of Engineering Geology (IAEG) Panel on Engineering Geology Research In USA, 1984 - 1985.

PROFESSIONAL SOCIETY AFFILIATIONS (Cont.)

Convenor and Chairman, GSA Symposium on Geologic Disposal of Radioactive Wastes, Geological Society of America 96th Annual Meeting, Indianapolis, IN, November 1983.

Member, Technical Program Committee, International Association of Engineering Geology (I.A.E.G.)

Symposium on Hazardous Waste Disposal, Winston-Salem, NC, October 1985.

Co-chairman, Session on Environmental Geology, Geological Society ofAmerica, 100th Annual Meeting, Phoenix, AZ, October 26 - 29, 1987.

Member, Program Committee, International Conferences on High-Level Radioactive Waste Management (ASCE/ANS), 1988.

Co-Chairman, Symposium on "Site Characterization for Radioactive Waste Remediation and Disposal," 28th International Geological Congress, Washington, D.C., July, 1989.

U.S. Delegate, INTRAVAL (International Validation of Hydrologic Models) Workshop, Cologne, Germany, October 15-19, 1990.

U.S. Delegate, Organization for Economic Cooperation and Development/Nuclear Energy Authority (OECD/NEA) Workshop on Site Evaluation and Design of Experiments (SEDE '90), Paris, France, October 22-24, 1990.

SELECTED PRESENTATIONS

Several invited presentations at the Radioactive and Hazardous Material Committee of New Mexico Legislature, Governor's Radioactive Waste Consultation Task Force, Universities, National Academy of Sciences, and Public Forums plus the following Congressional Hearings:

Testimony at the Joint Hearing on WIPP before the Subcommittees of the Committee on Energy and Commerce and the Committee on Interior and Insular Affairs of U.S. House of Representatives, May 8, 1990.

Testimony at the Hearing on WIPP before the Environment, Energy, and Natural Resources Subcommittee of the Committee on Government Operations of U.S. House of Representatives, June 12, 1989.

Testimony at the Hearing on WIPP before a Subcommittee of the Committee on Government Operations of U.S. House of Representatives, September 13, 1988.

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Chapman, Jenny B. and L. Chaturvedi, 1988, Lithologic Control on the Hydrology and Water-chemistry of the Permian Rustler Formation in the Vicinity of the WIPP Site, Southeastern New Mexico, (abs.), Geol. Soc. of America 1988 Centennial Abstracts With Programs, p. A365.

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Chaturvedi, L., et. al., 1987, Performance Assessment for a Nuclear Waste Repository--The WIPP Experience, in R. G. Post and M. E. Wacks (eds.), Waste Management '87, Vol. 3, pp. 141-145.

Chaturvedi, L. and J. B. Chapman, 1986, Geological issues in the siting and construction of a nuclear waste repository, Geol. Soc. of America, 96th Annual Meeting, Abstracts Vol., p. 563.

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Chaturvedi, L., 1986, Site Characterization for High Level Nuclear Waste Repositories - Lessons to be Learned from WIPP, in R. G. Post and M. E. Wacks (eds.), Waste Management '86, Vol. 2, pp. 107-111.

Chaturvedi, L., 1986, Evaluation of Dissolution for Radioactive Waste Repositories in Salt Deposits, Bulletin of the International Association of Engineering Geology, No. 34, 1986, pp. 11-14.

Chaturvedi, L., 1985, Rock Salt as a Medium for Long-term Isolation of Radioactive Wastes - a Reassessment, Abstract Vol., Geological Society of America Annual Meeting, Orlando, FL, p. 543.

Chaturvedi, L. and J. K. Channell, 1985, The Rustler Formation as a Barrier to Migration of Contaminated Groundwater, Southeastern New Mexico, Environmental Evaluation Group Report (EEG-32), December 1985, 85 p.

Chaturvedi, L., 1984, Occurrence of Gases in the Salado Formation, Environmental Evaluation Group Technical Report, EEG-25, 76 p.

Chaturvedi, L. and K. Rehfeldt, 1984, Groundwater Occurrence and the Dissolution of Salt at the WIPP Radioactive Waste Repository, EOS, Trans. American Geophysical Union, Vol. 65, No. 31, pp. 457-459.

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Chaturvedi, L., et. al., 1983, Multispectral Remote Sensing of Saline Seeps, IEEE Trans. on Geoscience and Remote Sensing, Vol. GE-21, No. 3, pp. 239-251.

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Chaturvedi, L., 1981, Geothermal Hydrology, in State-coupled Low Temperature Geothermal Resource Assessment Program/Final Technical Report, DOE/ID/01717-1, Chap. 8, 22 p.

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Chaturvedi, L., 1981, Geological Predictability and the Disposal of Nuclear Wastes - Experience at WIPP Site, Abstract Vol., 24th Annual Meeting, Association of Engineering Geologists, p. 28.

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EEG-1 Goad, Donna, A Compilation of Site Selection Criteria Considerations and Concerns Appearing in the Literature on the Deep Disposal of Radioactive Wastes, June 1979

EEG-2 Review Comments on Geological Characterization Report, Waste Isolation Pilot Plant (WIPP) Site, Southeastern New Mexico SAND 78-1596, Volumes I and II, December 1978.

EEG-3 Neill, Robert H., et al, (eds.) Radiological Health Review of the Draft Environmental Impact Statement (DOE/EIS-0026-D) Waste Isolation Pilot Plant, U. S. Department of Energy, August 1979.

EEG-4 Little, Marshall S., Review Comments on the Report of the Steering Committee on Waste Acceptance Criteria for the Waste Isolation Pilot Plant, February 1980.

EEG-5 Channell, James K., Calculated Radiation Doses From Deposition of Material Released in Hypothetical Transportation Accidents Involving WIPP-Related Radioactive Wastes, November 1980.

EEG-6 Geotechnical Considerations for Radiological Hazard Assessment of WIPP. A Report of a Meeting Held on January 17-18, 1980, April 1980.

EEG-7 Chaturvedi, Lokesh, WIPP Site and Vicinity Geological Field Trip. A Report of a Field Trip to the Proposed Waste Isolation Pilot Plant Project in Southeastern New Mexico, June 16 to 18, 1980, November 1980.

EEG-8 Wofsy, Carla, The Significance of Certain Rustler Aquifer Parameters for Predicting Long-Term Radiation Doses from WIPP, September 1980.

EEG-9 Spiegler, Peter, An Approach to Calculating Upper Bounds on Maximum Individual Doses From the Use of Contaminated Well Water Following a WIPP Repository Breach, September 1981.

EEG-10 Radiological Health Review of the Final Environmental Impact Statement (DOE/EIS-0026) Waste Isolation Pilot Plant, U. S. Department of Energy, January 1981.

EEG-11 Channell, James K., Calculated Radiation Doses From Radionuclides Brought to the Surface if Future Drilling Intercepts the WIPP Repository and Pressurized Brine, January 1982.

EEG-12 Little, Marshall S., Potential Release Scenario and Radiological Consequence Evaluation of Mineral Resources at WIPP, May 1982.

EEG-13 Spiegler, Peter., Analysis of the Potential Formation of a Breccia Chimney Beneath the WIPP Repository, May, 1982.

EEG-14 Not published.

EEG-15 Bard, Stephen T., Estimated Radiation Doses Resulting if an Exploratory Borehole Penetrates a Pressurized Brine Reservoir Assumed to Exist Below the WIPP Repository Horizon, March 1982.

EEG-16 Radionuclide Release, Transport and Consequence Modeling for WIPP. A Report of a Workshop Held on September 16-17, 1981, February 1982.

EEG-17 Spiegler, Peter, Hydrologic Analyses of Two Brine Encounters in the Vicinity of the Waste Isolation Pilot Plant (WIPP) Site, December 1982.

EEG-18 Spiegler, Peter, Origin of the Brines Near WIPP from the Drill Holes ERDA-6 and WIPP-12 Based on Stable Isotope Concentration of Hydrogen and Oxygen, March 1983.

EEG-19 Channell, James K., Review Comments on Environmental Analysis Cost Reduction Proposals (WIPP/DOE-136) July 1982, November 1982.

EEG-20 Baca, Thomas E., An Evaluation of the Non-radiological Environmental Problems Relating to the WIPP, February 1983.

EEG-21 Faith, Stuart, et al., The Geochemistry of Two Pressurized Brines From the Castile Formation in the Vicinity of the Waste Isolation Pilot Plant (WIPP) Site, April 1983.

EEG-22 EEG Review Comments on the Geotechnical Reports Provided by DOE to EEG Under the Stipulated Agreement Through March 1, 1983, April 1983.

EEG-23 Neill, Robert H., et al., Evaluation of the Suitability of the WIPP Site, May 1983.

(Continued on Back Cover)

EEG-48

AN ASSESSMENT OF THE FLAMMABILITY AND EXPLOSION POTENTIAL OF TRANSURANIC WASTE

Matthew Silva

Environmental Evaluation Group
New Mexico

June 1991

SUMMARY OF AFFIDAVIT OF GABRIEL FERNANDEZ-DELGADO

Dr. Fernandez is a civil and geotechnical engineer and a Research Engineer at the University of Illinois in Urbana and has taught there as Visiting Assistant Professor of Civil Engineering.

Dr. Fernandez concludes that the waste emplaced for tests cannot be expected to be retrievable after 18 months and may be nonretrievable within about a year. Specifically:

1. There can be no assurance that Room 1, Panel 1, the test room, will remain stable for more than 18 months from this date. (¶ 9).
2. SPDV Room 1, where a major roof fall occurred in February 1991, was barred to access after slightly more than six years, and Room 1, Panel 1 is slightly over five years old and if it follows the pattern of SPDV Room 1 would be closed to access within about a year. (¶ 9).
3. The DOE's proposed roof support system is defective in the following respects:
   a. The bolt length which is anchored in resin is too short. (¶ 17).
   b. The system is too limited in ability to deform to adjust load distribution; thus, bolts may become overloaded. (¶ 18).
   c. The system fails to deal with the shear forces which will bear upon the roof bolts at the point of rock fracture and where the bolt connects to a support channel. (¶¶ 19-20).

d. The system must bear the weight of buckling forces, which exceed the design load. (¶ 15).

e. Essential equipment clearance may be lost due to breakup of the roof beam. (¶ 21).

f. Failure can occur or accelerate at an unpredictable rate. The system may result in sudden accelerations. (¶ 22).

Affidavit of Gabriel Fernandez-Delgado

Gabriel Fernandez-Delgado, deposes and says:

1. I am a civil and geotechnical engineer and currently hold the position of Research Engineer at the University of Illinois in Urbana, Illinois. In that position I engage in teaching, research, and consulting. My resume is attached as Exhibit A.

2. I was educated in Colombia through my undergraduate years. I graduated in 1970 from the Universidad de los Andes as a Civil Engineer. I received my M.S. degree from the University of Illinois in Soil and Rock Mechanics in 1972. I received my Ph.D. from the University of Illinois in Geotechnical Engineering in 1976.

3. In 1971 though 1976 I worked as Research Assistant in the Department of Civil Engineering at Urbana. In 1976 through 1984 I taught at Urbana as a Visiting Assistant Professor of Civil Engineering. Since 1984 I have concentrated primarily in research as a Research Engineer at the University.

4. In my practice I have considerable experience in the rock mechanics of underground evaporite deposits:

a. I was associated with Dr. A. J. Henderon, Jr. in projects at Bayou Choctaw, West Hackberry, and Bryan Mound, Louisiana, which concerned the development of general criteria for the acceptance of existing and proposed salt cavities as oil storage vessels. The

caverns were located at depths ranging from 1500 to 3800 ft.

b. In Windsor, Canada, I evaluated the structural stability of an underground storage cavern dissolved out of salt materials. The cavern is located 1300 ft deep in a salt layer overlaid by a sequence of sandstone, dolomite and shale layers.

c. In Pugwash, Nova Scotia, I evaluated the proposed expansion of the present salt mining works located at two levels 600 and 800 ft below the ground surface. The project involved structural stability analysis and specific recommendations (room and pillar sizes, monitoring program, excavation sequence) regarding excavation of an intermediate mine level connecting the present mining levels.

d. In Fort Saskatchewan, Canada, I acted as consultant with regard to a deep salt storage site for Northwestern Utilities Limited in the evaluation, design and development of a storage cavern system in 600-ft-deep salt deposits in the Fort Saskatchewan area. The project involves implementation of field testing to determine most relevant engineering properties of salt in-situ; development and redistributions around the cavern as well as corresponding short and long-term (creep) of the rock mass surrounding the openings.

e. I have also examined the solution mining potential of salt formations at the Cargill Salt Mine in Hutchinson, Kansas and other salt mines in the same vicinity.

5. I have been retained as a consultant to the State of New Mexico in connection with certain room stability problems encountered at the Waste Isolation Pilot Plant ("WIPP").

6. I examined certain geotechnical data reflecting the origins of the room stability problem. These include:

a. Geotechnical Field Data and Analysis, DOE WIPP 91-012 (June 1990)

b. Interim Geotechnical Field Data Report, DOE WIPP 86-012 (Fall 1986) (extracts)

c. Design Validation Final Report, DOE WIPP 86-010 (1986) (extracts)

d. Borns, D.J., and J.C. Stormont, An Interim Report on Excavation Effects Studies at the Waste Isolation Pilot Plant: Delineation of the Disturbed Rock Zone, SAND 87-1375 (1988)

e. Stormont, J.C., Discontinuous Behavior Near Excavations in a Bedded Salt Formation, SAND 89-2403 (1989)

f. Design Criteria Waste Isolation Pilot Plant, Revised Mission Concept -IIA, WIPP DOE-71, Rev. 4 (extracts)

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

h. Brunwald, H.P., and H.C. Howarth, Compression Tests of Roof-Salt Slabs Supported by Potash Salt Pillars, RI-3386, U.S. Bureau of Mines (1938)

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

7. I also reviewed the Report of the Geotechnical Panel on the Effective Life of Rooms in Panel 1, DOE/WIPP 91-023 (June 1991), and the report entitled Waste Isolation Pilot Plant, Supplementary Roof Support System, Underground Storage Area, Room 1, Panel 1 (August 1991). The first report (the June 1991 report) contains the initial assessments of members of an expert panel which was convened to examine the room stability problems that the DOE has encountered at the WIPP. The second report (the August 1991 report) contains the proposed roof support system which is to be installed in an effort to prolong the life of one of the underground rooms. It is my understanding that the DOE plans to conduct tests with radioactive waste in that room.

8. I also visited the WIPP site on September 27, 1991, in the company of attorneys from the Office of the Attorney General of New Mexico. There was an extensive underground tour. In addition, I consulted during the tour with Mr. Tom Schultheiss of Sandia National Laboratories, Mr. Harry Bibby, who is in charge of mining operations, and Mr. Jim Mewhinney, in

charge of environmental compliance. I also spoke at length with Mr. Hamish Miller, who consulted in connection with the design of the proposed roof support system.

9. I have not yet completed my analysis of the problem and the proposed solution. However, at present it seems clear that the proposed support system is not likely to extend the useful life of the test room, Room 1 of Panel 1, much beyond the projected life without such support. I would concur with the consensus of the expert panel report that there can be no assurance that Room 1, Panel 1 will remain stable for more than two years from the date of their report, or eighteen months from this date. I would add that predicting the date when an underground room will fail is extremely difficult. For that reason it is very possible that the roof of Room 1, Panel 1 may either fail, or give such warning of failure that access to the room must be barred, well before the eighteen month point. I note that the data on SPDV Room 1, which experienced a major roof failure on February 4, 1991, show that access to that room was suspended when that room was slightly over six years old. (August 1991 report, at Fig. 5.1). Room 1 of Panel 1 is now slightly over five years old. Thus, if it follows the pattern of SPDV Room 1, the room would be closed to access within about one year.

10. My examination of the WIPP facility provided data about the origins of the stability problem as well as the deficiencies of the proposed solution.

11. I observed the magnitude of the 1991 roof fall in SPDV Room 1, where a segment of the roof weighing an estimated 1400 tons separated from the roof and fell. SPDV Room 1 is now closed to any access, and I was not permitted to enter the room.

12. I also observed the roof fractures in Room 1, Panel 1 -- the room where test waste will be placed -- and concluded that they follow the same pattern as the fractures which led to the fall of the roof in SPDV Room 1.

13. In SPDV Room 4 I noted in observation holes in the floor and the roof that underlying and overlying strata have moved between three and six inches relative to the opening in approximately two years. These are substantial shear movements which are part of the cause of the failure of the underground rooms.

14. The proposed support system relies upon 13 foot Dywidag rock bolts anchored with resin grout above the anhydrite "b" clay layer. These bolts are designed to protrude 18" from the roof to accommodate horizontal and vertical deflection, i.e., the lowering of the roof rock. The rock bolts are anchored in and placed in three inch diameter clearance holes running to the anhydrite "b" layer. The bolts are fed through a 1-1/2" hole in a steel channel 15" wide by 3-1/2" deep. The channel is 27' long, comprised of three nine foot sections joined by fishplates. Eleven bolts secure each channel set, and the sets are placed at eight to ten foot intervals. There is an additional lacing system comprised of 5/8" wire rope fastened

with diagonal eight foot resin grouted Dywidag rock bolts. The lacing is placed laterally and longitudinally on three foot centers. In addition, there is 3/4" x 1" expanded steel mesh and 4" x 4" mesh formed of welded 1/4" steel wire.

15. In my discussions with Hamish Miller and his staff, I learned that the proposed support structure is not expected to stop separation of the strata from occurring. Rather, it is expected to support the fractured roof beam once it comes loose. The system, however, is only designed to absorb the gravitational load, not any additional stresses. Mr. Miller stated to me that he recognized that the downward force of the roof beam could exceed the force of gravity. This is because there is a tendency to buckle due to the roof beam being subject to horizontal stresses.

16. In my opinion the proposed support system suffers from several deficiencies which could render it ineffective to resist the forces which give rise to a roof fall.

17. The bolt length which is anchored in resin--a three foot segment--seems too short. The load, with time, will travel up the bolt. Shear forces will develop and weaken the bond in the lower parts of the bolt and move upwards. (creep of salt material in anchor zone). In addition, the bolt will be subjected to potential lateral deflection, which diminishes its load carrying capacity. The designers have only tested the rapid load capacity of the bolt -- i.e., testing to failure at

about 47 thousand pounds. This may not reflect the long-term steady-state capacity of the bolt system.

18. To equalize the bolt load one must pay the price of deformation of the structure. That is, if you slacken a nut, the roof comes down lower in the given area. The ability to deform appears to be limited by the rigidity of the channel structure. Therefore, there may be no choice but to overload a given bolt.

19. Most important for the support system to deal with is the shear force on the bolt. The system does not deal with this force. In practice, the seven foot roof beam will first compress inward after the excavation, as lateral stress bears on the intact member. This causes the ends to shear <u>inward</u> toward the center. Thereafter, fractures will continue to develop and migrate toward the weak anhydrite - clay seam lying seven feet above the roof, which seam cannot transmit stress and so concentrates it. We saw such fractures along the edges of the roof; how deep they go is not known. It appears to me also that the three inch diameter clearance hole is inadequate to deal with the expected horizontal movement. This will lead to excessive bolt stress and potential bending. These shear stresses mean that the bolt at the point of shear -- at or below the anhydrite layer -- is bent and loses load carrying capacity.

20. Another potential location for stress concentration is the connection between the bolt and the channel, because the

stiffness of the channel may not be able to accommodate the non-uniformity of the load that initially develops on the bolt.

21. The sagging of the roof will cause breakage of the salt, which in turn may result in substantial loads on the lacing system. There is considerable stretch in this system, but it will have to bear a considerable load. The lacing may bulge so low between the horizontal members that the necessary clearance is lost. Uniform behavior is not to be relied upon.

22. The design review panel advised that the operator set up go-no-go criteria in advance for abandonment of the room. At present, there are no such criteria. The operator will simply monitor through various means and continuously reevaluate the performance. I have a fundamental disagreement with this approach. Failure can occur or accelerate at an unpredictable rate. Our predictive ability may be off by several months. The system may result in sudden accelerations. The various rooms are not identical, since the strata, although they seem to be fairly uniform, are not identical. Therefore, criteria in terms of closure acceleration or the like should be set down in advance as guidelines for retreat.

23. The monitoring system is elaborate but not uniformly effective. Load cells on each bolt and pressure cells may not provide all the required information to assess the structural performance of the bolt (e.g., bending stresses at the anhydrite level). Stresses are not simply in tension.

24. It would be much more prudent to scale off the dead weight of the roof beam up to the anhydrite "b" layer, make an arch, and bolt the new roof. This was done at two shaft stations (waste shaft and salt handling shaft), and they are now reasonably stable. The system adopted by the DOE contains many uncertainties that cannot be predicted and that may lead to failure at a rate that is unpredictable. Proven systems have been ignored.

I declare under penalty of perjury under the laws of the United States of America that the foregoing is true and correct.

Executed on: October 7, 1991

GABRIEL FERNANDEZ-DELGADO

IV. F-Delgado A

PERSONAL DATA AND PROFESSIONAL RECORD

Name: GABRIEL FERNANDEZ-DELGADO

Address: 2230 Civil Engineering Building
University of Illinois
Urbana, Illinois 61801

Marital Status: Married

Date of Birth: November 13, 1949

EDUCATION

Civil Engineer 1970	Universidad De Los Andes Bogota, Colombia South America
M.S.-Soil and Rock Mechanics 1972	University of Illinois Urbana, Illinois 61801
Ph.D.-Geotechnical Engineering 1976	University of Illinois Urbana, Illinois 61801

AWARDS

U.S. National Committee for Rock Mechanics case histories award for a significant, original contribution, 1987.

ACADEMIC EXPERIENCE

1984-present	Research Engineer, University of Illinois at Urbana-Champaign. - Organizer and speaker at a University of Illinois short course on the Use of Shotcrete for Underground Support. Course sponsored by Urban Mass Transportation Administration.
1976-1984	Visiting Assistant Professor of Civil Engineering, University of Illinois at Urbana-Champaign.
8/71-1976	Research Assistant in the Department of Civil Engineering at the University of Illinois. Principal research involved:

a) development of design criteria for thin shotcrete linings and mix design considerations of conventional shotcrete.
b) a field-oriented investigation of conventional and experimental shotcrete to tunnels. The field program was carried out in the Washington Metro System, Dupont Station.
c) large-scale testing of innovative tunnel support systems with monolithic concrete rings made with steel fiber concrete and horseshoe shaped steel sets fabricated from hollow and concrete-filled tubular box sections.

Teaching Assistant in the Department of Civil Engineering. CE 280 (Section B) Introduction to Soil Mechanics and Foundation Engineering. CE 384 Applied Soil Mechanics and CE 497 (Section CD) Deep Foundations.

RESEARCH ACTIVITIES

- Member of a National Academy of Engineering team sent to investigate the geotechnical aspects of the mudslides generated during the 1985 explosion of the Ruiz Volcano in Colombia, South America.

- Consultant for SMRI (Salt Mining Research Institute) in different studies to evaluate causes and failure mechanisms resulting in large sinkhole formation. These studies include investigation of in-situ subgrade conditions, evaluation of the structural stability of underground materials and monitoring of the behavior of these materials. Criteria has been developed to predict response of the subgrade materials and to provide design guidelines for development of large underground, deep, salt caverns in different geological set-ups.

- Consultant for Tudor Engineering Company for planning and conducting a study of in-situ structural support capabilities of different shotcrete types - Underground Research Chamber -Atlanta.

CONSULTING EXPERIENCE

Examples of HYDROPOWER PROJECTS

TARBELA DAM -- Pakistan. Associate of Dr. A. J. Hendron Jr. in the stability analysis of the service spillway of Tarbela Dam as well as the left slope of the plunge pool. Evaluation of the rock slope stability for proposed changes at outlet tunnels 3 and 4. It included development of different approaches for a more realistic evaluation of hydrodynamic forces against rock slopes.

VAIONT LANDSLIDE -- Italy. Associate of Dr. A. J. Hendron Jr. for review and evaluation of the failure mechanisms involved in the Vaiont landslide.

CHATUGE AND NOTTELY DAMS -- North Carolina and Georgia. Dynamic stability analysis. Chatuge Dam is located in North Carolina on the Hiwassee River and is a homogeneous dam with a maximum height equal to 144 ft and 2840 ft long. Nottely Dam, located in Georgia on the Nottely River, is a rockfill dam with a large central core. The dam has a maximum height of 180 ft and is 1940 ft long.

SUAREZ RIVER HYDROELECTRICAL DEVELOPMENT -- Consultant for Contecol (Consultoria tecnica Colomgiana) in the pre-feasibility and feasibility studies of the hydro-electrical development of the Suarez River, Colombia, South America.

KPONG HYDRO PROJECT -- Republic of Ghana. Associate of Dr. Hendron in the evaluation of the static and dynamic stability of a three mile long dike in the Volta River.

EL CAJON DAM -- Honduras. Evaluation of the structural stability of the underground Powerhouse chamber, approximately 30x40x175m, and located in cavernous limestone materials.

AMBROSIA LAKE FACILITY -- New Mexico. Consultant for Woodward & Clyde Engineers in the evaluation of effectiveness of a cutoff wall to mitigate seepage flows and the influence of the wall in the overall stability of the existing failing dams.

GUAVIO DAM -- Colombia. Consultant to the Empresa de Energia de Bogota during construction of a 920 ft high rockfill dam, with 15 kilometers of power conduits and 2200 ft deep underground powerhouse.

RIOGRANDE II PROJECT -- Colombia. Member of the Board of Consultants for a combined energy and water supply scheme for the City of Medellin (Colombia). Project involves an earthfill dam, 27 kilometers of power conduits and a 2200 ft deep underground powerhouse.

JIGUEY AND AGUACATE DAMS -- Dominican Republic. Member of the Board of Consultants for two concrete arch-gravity dams (340 ft high and 100 ft high) on the Nizao River Basin. Project also involves 15 kilometers of tunnels, two underground powerhouses; and two cofferdams, approximately 35 meters high built with roller compacted concrete (RCC).

PORCE II -- Colombia. Member of the Board of Consultants for a 130 meter high roller-compacted concrete (RCC) dam on the Porce river. The project includes a 6 kilometers long power tunnel and a surface power house.

<u>Examples of SLOPE STABILITY</u>

CLINCH RIVER BREEDER REACTOR PLANT -- Tennessee. Associate of Dr. Hendron on review of slope stability analysis carried out in vertical rock cuts up to 120 ft deep. Design of the required support and review of the rock bolt, blasting and instrumentation specifications.

BUS TERMINAL -- PORT AUTHORITY -- New York and New Jersey. Associate of Dr. Hendron on wedge stability analysis for a vertical rock cut, 60 ft deep, with building loads 10 ft away from the edge.

CLIMAX MINE -- Climax, Colorado. Associate of Dr. Hendron in the stabilization of a 1500 ft tall, 500 ft wide slope in residual soils.

RUCK-A-CHUCKY BRIDGE -- American River, California. Hanson Engineers, Springfield, Illinois. Stability analysis of rock slopes securing highly loaded anchor cables supporting the bridge.

LA PAZ - COTAPATA HIGHWAY -- Bolivia. Evaluation of slope stability problems along existing road alignment east of La Paz. Special emphasis was made in analyzing and developing remedial measures for two road sections where massive landslides developed in rock and colluvial materials, respectively.

Examples of TUNNELS

QUINCY SEWER TUNNEL -- Quincy, Illinois. Associate of Dr. Hendron for evaluation of ground and hydraulic loads on a 7.52 ft by 4.62 ft horseshoe-shaped tunnel, 40 ft deep, driven in limestone. Assessment of the required support and evaluation of the reinforcement needed.

ESLLIRP TUNNEL PROJECT - SABROOKE GENERATING PLANT PROPERTY -- Rockford, IL. Associate of Dr. Hendron on geotechnical report on geology and soil conditions, ground behavior, anticipated tunnel conditions, dewatering and support requirements. The report also included the results and analysis of a field pumping test as well as cost estimate and bidding procedures. Specifications for excavation, support, and performance during construction were also written. Monitoring of performance and construction actually being carried out. Tunnel is 9 ft in diameter, 2000 ft long and located 40 ft below ground surface.

I-40 HIGHWAY TUNNEL -- Overton Park, Memphis, Tennessee. Associate of Dr. R. E. Heuer in the feasibility of constructing highway tunnel through Overton Park.

O'DONNELL MINE -- West Virginia. Consultant for CONSOL, Consolidation Coal Company, Pittsburgh, on the design of a support system for an inclined, 2000 ft long, access tunnel through sandstone and shales. The project included the design of a rock bolt-mesh-shotcrete support lining as well as development of shotcrete specifications.

MOUNT ADAMS ANCHORAGE TUNNEL -- Cincinnati, Ohio. ARMCO, Inc. Analysis and design of a 9-ft diameter tunnel which will be used to provide anchorage for a high capacity tieback system supporting a cylindrical pile wall.

MT. BAKER RIDGE TUNNEL -- Seattle, Washington. Analysis of a 60-ft diameter highway tunnel in glacial till; multiple drift perimeter tunnels filled with concrete.

AURORA RAMPARTS -- Colorado. Analysis and design of a 10-ft dameter outlet in sandstone materials. Study of internal hydraulic pressure effects on bare portions of the tunnel.

GUADALUPE IV -- Medellin, Colombia, South America. Consultant for the Contractor in the analysis and design of liner reinforcement in a 6 Km-long water pressure tunnel. The 15-ft diameter tunnel was excavated in Quarzite and Hornblende and had internal pressures ranging from 140 psi to 300 psi.

CHINGAZA PROJECT -- Bogota, Colombia, South America. Consultant of the Empresa de Acueducto de Bogota, in the evaluation of remedial measures to be implemented in the failed sections of the Palacio-Rioblanco tunnel stretch. This tunnel excavated in shales and sandstones had an average internal pressure of 100 psi.

BATH PROJECT -- Virginia. Consultant for Harza Associates in the evaluation of the effects of nearby quarry blasting on the liner of drainage tunnels and shafts of the upper dam of the Bath Project.

PEHEUNCHE PROJECT -- Chile. Consultant to the consortium CBPO-Techint in the evaluation of ground behavior and support requirements for the 8 1/2 Kilometer long pressure tunnel (tunnel Comun).

MESITAS PROJECT -- Colombia, South America. Consultant to the Empresas de Energia Electrica on the design of reinforced concrete liners to repair several sections of a 10 kilometer long power tunnel excavated in the Easter Andes Cordillera.

BI-COUNTY WATER TUNNEL -- Washington, D.C. Consultant to the Washington Suburban Sanitary Commission in the design of remedial measures to upgrade the liner of a 6 1/2 miles long water-supply pressure tunnel that has shown excessive leakage.

HONOLULU SEWER TUNNEL -- Honolulu Hawaii. Planning and Implementation of field exploratory program for a 6000 ft long, 10 ft diameter sewer expansion tunnel in downtown Honolulu. Evaluation of field exploratory program and preparation of geotechnical report and specifications for potential bidders. Specifications included excavation, dewatering, tunnel support, ground movement criteria - effect of blasting vibrations on nearby structures.

SUPERCONDUCTING SUPERCOLIDER Consultant to EBASCO Engineering Company, to evaluate tunnel ground conditions and tunnel requirements at 3 different sites in the state of New York, to excavate a 60 miles long tunnel and large underground chambers.

Examples of FOUNDATIONS

KILLEN POWER STATION -- Ohio. Analysis of soil exploration data for Dayton Power and Light Co. Job done in association with Dr. M. T. Davisson.

OVERPASS SYSTEM -- Bogota, Colombia. AREAS, Ltd. Design of foundations at four different intersections in overpass system at Bogota. The project involved review of the soil conditions and proposed design foundations; it also included wave equation analysis and planning and interpretation of load tests on 40 cm x 40 cm x 4.25 m long concrete piles.

EXXON RESEARCH LABS -- Clinton, New Jersey. EXXON Corporation. Review of subsurface conditions as well as foundation design. It included design of caissons in sound and partially weathered rocks; study of subsurface drainage systems and development of criteria for proof testing of rock underneath the drilled piers.

ALLIED HOSPITAL -- Scranton, Pennsylvania. Loewe and Associates. Design of foundations for a hospital located above excavated coal mines. It included subsidence studies of the area and evaluation of the structural stability of the coal mine roof.

WOLSUNG NUCLEAR POWER PLANT -- Korea. Associate of Dr. Hendron in the review and analysis of load-deflection data measured at the reactor foundation; involved analysis of extensometer data and development of a model to predict foundation settlements and potential implications in the structural behavior of the reactor. Foundation materials consisted of dacite and agglomerate.

Examples of LARGE UNDERGROUND OPENINGS AND MINES

PEACHTREE SUBWAY STATION -- Atlanta, Georgia.
Parsons, Brinckerhoff, Quade and Douglas. Evaluation of the structural stability of the rock cavern and proposed liner designs.

MERCY HOSPITAL -- Scranton, Pennsylvania. Loewe and Associates. Design and construction of a 50 ft high and 800 ft long retaining wall at Mercy Hospital. Project included design and long term monitoring of a permanent rock-bolt anchor system to support the wall; it also included preparation of specifications for the support system and other excavation aspects (i.e. blasting procedures, protection of nearby structures, etc.)

HARVARD SQUARE EXCAVATION -- Boston. Parsons, Brinckerhoff, Quade and Douglas. Preparation of geotechnical report on geology, rock conditions and ground behavior - anticipated excavation condition, ground water and support requirements - protection of sensitive adjacent structures -preparation of specifications for excavation, support, performance and monitoring of the underground opening.

BAYOU CHOCTAW, WEST HACKBERRY AND BRYAN MOUND -- Louisiana. Associate of Dr. Hendron in the development of general criteria for the acceptance of existing and proposed salt cavities as oil storage vessels. Caverns were located at depths ranging from 1500 to 3800 ft.

SULPHUR MINES -- Louisiana. Associate of Dr. Hendron in oil storage feasibility study.

CALGARY -- Canada. Associate of Dr. Hendron in a feasibility study for the development of storage caverns in Calgary.

WARREN PETROLEUM STORAGE CAVERNS -- Windsor, Canada. Evaluation of structural stability of an underground storage cavern dissolved out of salt materials. The cavern is located 1300 ft deep in a salt layer overlaid by a sequence of sandstone, dolomite and shale layers.

PUGWASH SALT MINE -- Nova Scotia, Canada. Canadian Rock Salt. Evaluation of proposed expansion of the present salt mining works located at two levels 600 and 800 ft below the ground surface. The project involved structural stability analysis and specific recommendations (room and pillar sizes, monitoring program, excavation sequence) regarding excavation of an intermediate mine level connecting the present mining levels.

LUCKY FRIDAY SHAFT -- Idaho. J. S. Redpath Corporation. Design of a 700-ft deep, 20-ft diameter concrete shaft at the Hecla Mine, Idaho. Project involved evaluation of in-situ stress conditions, design of shaft lining, evaluation of the interaction between lining and surrounding rock materials and predictions of lining deformations. This shaft will be the deepest mine shaft in the western hemisphere when completed.

DEEP SALT STORAGE - FORT SASKATCHEWAN -- Canada. Consultant for Northwestern Utilities Limited in the evaluation, design and development of a storage cavern system in 600-ft-deep salt deposits in the Fort Saskatchewan area. Project involves implementation of field testing to determine most relevant engineering properties of salt in-situ; development and redistributions around the cavern as well as corresponding short- and long-term (creep) of the rock mass surrounding the openings.

Examples of SUBSIDENCE

CARGILL SINKHOLE -- Hutchinson, Kansas. Associate of Dr. Hendron on subsurface exploration to investigate the formation of a 300 ft diameter, 33 ft deep sinkhole.

SOLUTION MINING RESEARCH INSTITUTE (SMRI) -- KANSAS. Consultant to study sinkhole formation in Cargill Salt Mine, Hutchinson, Kansas.

SOLUTION MINING RESEARCH INSTITUTE (SMRI) -- Kansas. Study of sinkhole formation in Carey Salt Mine, Hutchinson, Kansas. ORTON SALT -- Ohio. Evaluation of present brine fields and development of new fields at the salt plant in Rittman, Ohio.

MORTON SALT -- Kansas. Evaluation of structural stability and future development of dissolution caverns at their salt plant in Hutchinson, Kansas.

Examples of DYNAMIC SOIL AND ROCK RESPONSE

GREEN COUNTY NUCLEAR REACTOR -- New York. Evaluation of ground motions at proposed nuclear facilities resulting from potential surface explosions at a nearby quarry.

PROYECTO HERCULES II -- Mexico. Evaluation of ground motions at proposed coal mine facilities, resulting from blasting at nearby open pit mine.

THORNTON QUARRY -- Chicago, Illinois. Evaluation of the stability of a 300 ft high railroad ridge through the middle of the quarry under the dynamic loading produced by nearby blasting.

MONDAWMIN SUBWAY STATION - Baltimore. Evaluation of blasting effects on the stability of the subway excavation slopes. Analysis of blasting - induced ground motions and vibrations. Preparation for potential litigation.

LEHIGH BUILDING - Kankakee, Illinois. Evaluation of the impact of demolition-induced vibrations and dynamic ground motions on the structural behavior of adjacent building. Preparation for litigation.

Examples of PORT ENGINEERING

PORT DEVELOPMENT PROJECT IN GUAYAQUIL -- Ecuador, South America. Review of soil conditions and proposed design of a wharf that included dredging, fill, retaining wall and piles. Slope stability analysis and suggestions of alternative solutions.

PORT DEVELOPMENT IN PUERTO MONT -- Chile, South America. Evaluation of pile capacity to support new wharf facilities.

CERREJON COAL MINE LOADING FACILITIES -- Bahia Portete, Colombia. Engineering consultant to evaluate foundation design for large oil storage tanks and for the coal transportation and loading facilities.

PUBLICATIONS

Fernandez-Delgado, G., and J. Dumar (1970): ENGINEERING PROPERTIES OF BOGOTA CLAY, Final Report to the Colombian Science Foundation.

Fernandez-Delgado, G., J. W. Mahar, and E. J. Cording (1975): SHOTCRETE STRUCTURAL TESTING OF THIN LINERS, UIUC Final Report for U.S. Department of Transportation, Federal Railroad Administration.

Parker, H. W., G. Fernandez-Delgado and L. Lorig (1975): FIELD-ORIENTED INVESTIGATION OF CONVENTIONAL AND EXPERIMENTAL SHOTCRETE FOR TUNNELS, UIUC Final Report for U.S. Department of Transportation, Federal Railroad Administration.

Parker, H. W., G. Fernandez-Delgado and L. Lorig (1976): A PRACTICAL NEW APPROACH TO SHOTCRETE REBOUND LOSSES, Proceedings of the Engineering Foundation Conference on Shotcrete for Ground Support, American Concrete Institute, Publication SP-54, October, 1976.

Fernandez-Delgado, G., J. W. Mahar, and H. W. Parker (1976): STRUCTURAL BEHAVIOR OF THIN SHOTCRETE LINERS OBTAINED FROM LARGE SCALE TESTS, Proceedings of the Engineering Foundation Conference on Shotcrete for Ground Support, American Concrete Institute, Publication SP-54, October, 1976.

Fernandez-Delgado, G., J. W. Mahar, and E. J. Cording (1976): SHOTCRETE, LARGE SCALE TESTING OF THIN LINERS WITH A FLAT ARCH GEOMETRY, UIUC Final Report for U.S. Department of Transportation, Federal Railroad Administration.

Fernandez-Delgado, G. (1977): LARGE SCALE TESTING OF THIN SHOTCRETE LINERS, Ph.D. Dissertation, University of Illinois, Urbana-Champaign, Department of Civil Engineering.

Fernandez-Delgado, G., E. J. Cording, J. W. Mahar, and M. L. Van Sint Jan (1979): THIN SHOTCRETE LININGS IN LOOSENING ROCK, 4th Rapid Excavation and Tunneling Conference, Atlanta.

Fernandez-Delgado, G., E. J. Cording, J. W. Mahar, and M. L. Van Sint Jan (1981): ATLANTA RESEARCH CHAMBER MONOGRAPH - APPLIED RESEARCH FOR TUNNELS, U.S. Department of Transportation, Report NO. UMTA-6A-06-0007-81-1, March, 1981.

Hendron, Jr., Alfred J. and G. Fernandez (1983): DYNAMIC AND STATIC DESIGN CONSIDERATIONS FOR UNDERGROUND CHAMBERS, Proceedings, ASCE National Convantion, Philadelphia, Pennsylvania, pp 157-197.

Fernandez-Delgado, G., and A. J. Hendron, Jr. (1984): INTERPRETATION OF A LONG TERM IN-SITU BOREHOLE TEST IN A DEEP SALT FORMATION. Bulletin of the Association of Engineering Geologists, Vol. XXI, No. 1, 1984, pp 23-38.

Fernandez-Delgado, G. (1985): EVALUATION OF DYNAMIC STABILITY OF EARTH AND ROCKFILL DAMS, Proceedings, Seminar on Earthquake Engineering Research Institute, University of Puerto Rico, Mayaguez, May, 1985.

SUMMARY OF AFFIDAVIT OF LINDA L. LEHMAN

Ms. Lehman is a geologist and hydrogeologist who specializes in nuclear waste disposal issues and performance assessments of nuclear waste disposal facilities.

Ms. Lehman concludes that the DOE will not be prejudiced by an inability to conduct tests with radioactive waste at the WIPP. Specifically:

1. There is no necessity for the dry bin scale tests to be conducted underground at the WIPP. The tests could be conducted on the surface at a laboratory facility at or near the present storage location, probably with less risk and inconvenience. (¶¶ 18-23).
2. The remaining elements of the DOE test plan - the wet bin-scale tests, bin-scale solubility tests, and alcove tests - are not now prepared to be conducted. Moreover, all the bin-scale tests can be conducted at an off-site laboratory facility. (¶¶ 24-27).
3. The information to be obtained from the dry bin-scale tests is of unknown importance and appears to be of marginal importance. (¶¶ 28-49).

AFFIDAVIT OF LINDA L. LEHMAN

Linda L. Lehman, being duly sworn, deposes and says:

1. I am a principal of L. Lehman & Associates, Inc., a counseling firm specializing in hydrologic and nuclear waste matters located in Burnsville, Minnesota.

2. My resume is attached as exhibit A. I graduated from the Florida Atlantic University in 1974 with a B.S. in Geology and from the University of South Florida in 1978 with a M.S. in Hydrogeology.

3. I am a Registered Geologist in the State of Indiana; a Professional Hydrogeologist, certified by the American Institute of Hydrology; and a Certified Ground Water Professional of the Association of Ground Water Scientists and Engineers.

4. I was a Hydraulic Engineer with the Nuclear Regulatory Commission for three years. I have worked as a consultant in both high-level and low-level radioactive waste disposal issues for the past nine years.

5. My low level waste experience includes evaluating the low-level and mixed waste disposal facilities at the DOE uranium processing facility in Fernald, Ohio; as well as the disposal and remediation of low-level, mixed, transuranic and high-level DOE defense wastes at the Hanford Reservation. In addition, I have directed efforts for State and private clients in the area of low-level radioactive waste disposal.

6. My working knowledge of nuclear waste regulations is extensive. I participated in the drafting and development of the NRC's 10 CFR 60 regulations relating to high-level radioactive disposal, and in the development of Minnesota's Radioactive Waste Management Act (MN Statute 116C.71-116C.74), and the Rules Relating to Exploratory Drilling for the Disposal of High-Level Radioactive Waste (MN Part 4410.7900-4410.7934).

7. As principal of L. Lehman & Associates, Inc., I was Technical Coordinator for the Yakima Indian Nation review of the Basalt Waste Isolation Project (BWIP). I was the technical lead under contract to the Minnesota Governor's Nuclear Waste Council regarding siting of a nuclear waste repository in crystalline rock.

8. I am currently a contractor to the State of Nevada's Nuclear Waste Project Office providing hydrogeologic and regulatory analyses to the State's high-level waste program. Through programmatic and technical review of technical and procedural documents and attendance at DOE/NRC sponsored meetings and workshops I have acquired broad familiarity with radioactive waste problems.

9. As a Hydraulic Engineer with the U.S. Nuclear Regulatory Commission (NRC), I provided technical review of the U.S. Department of Energy (DOE) High-Level Radioactive Waste Disposal Program at the Nevada Test Site and the Hanford Site with respect to hydrology, particularly ground water and solute transport modeling. I developed the NRC conceptual

hydrogeologic model at the Hanford Site in the Pasco Basin, and applied the SWIFT code to evaluate site suitability for a high-level radioactive waste repository. I also originated, implemented, and managed a large private contract for the NRC which provided a comparative analysis of computer codes used to license repositories and their applicability to specific sites.

10. I have been retained by the State of New Mexico to assist in analyzing the current status of the performance assessment by the Department of Energy ("DOE") concerning the Waste Isolation Pilot Plant ("WIPP").

11. Before the WIPP may be employed for disposal of radioactive waste, the DOE must establish its compliance with certain standards promulgated by the Environmental Protection Agency. These are the Standards for Management and Disposal of Spent Nuclear Fuel, High-Level and Transuranic Radioactive Waste. The 1985 version of these regulations is codified at 40 CFR §191. Subpart B of these standards contains the criteria from long-term disposal of radioactive waste. The Subpart B standards were vacated by the Court of Appeals for the First Circuit and are currently under consideration by the EPA; however, no new standards have been proposed. With the consent of the State of New Mexico, the DOE has in the meantime continued to evaluate compliance with the 1985 standards.

12. I have undertaken a review of the current status of the DOE's effort to demonstrate compliance with the EPA disposal standards and the proposed dry bin-scale tests and other planned

tests in an attempt to determine the contribution that such tests may make to a performance assessment of the WIPP's compliance with the EPA disposal standards.

13. The EPA standards include the Containment Requirement, which provides as follows:

> "Disposal systems for spent nuclear fuel or high-level or transuranic radioactive wastes shall be designed to provide a reasonable expectation, based upon performance assessment, that the cumulative release of radionuclides to the accessible environment for 10,000 years after disposal from all significant processes and events that may affect the disposal system shall:
>
> (1) Have a likelihood of less than one chance in 10 of exceeding the quantities calculated according to Table 1 (Appendix A); and
>
> (2) Have a likelihood of less than one chance in 1,000 of exceeding ten times the quantities calculated according to Table 1 (Appendix A)." 40 C.F.R. §191.13(a)).

The section continues, explaining that the performance assessment need not provide "complete assurance" of compliance. Rather, "what is required is a reasonable expectation, on the basis of the record before the implementing agency, that compliance with §191.13(a) will be achieved." (40 C.F.R. §191.13(b)).

14. The EPA Standards also contain Individual Protection Requirements. (40 C.F.R. §191.15). This section requires that disposal systems for transuranic waste "provide a reasonable expectation that, for 1,000 years after disposal, undisturbed performance of the disposal system shall not cause the annual dose equivalent from the disposal system to any member of the public in the accessible environment to exceed 75 millirems to the whole body or 25 millirems to any critical organ." (Id.)

15. Compliance with the above requirements is to be evaluated by a process of analysis known as a "performance assessment." The term is defined in the EPA Standards:

> ""Performance assessment" means an analysis that: (1) identifies the processes and events that might affect the disposal system; (2) examines the effects of these processes and events on the performance of the disposal system; and (3) estimates the cumulative releases of radionuclides, considering the associated uncertainties, caused by all significant processes and events. These estimates shall be incorporated into an overall probability distribution of cumulative release to the extent practicable." (40 C.F.R. §191.12 (q)).

16. I have reviewed pertinent materials concerning the performance assessment at the Waste Isolation Pilot Plant ("WIPP"), including the following:

a. 40 C.F.R. Part 191

b. Test Plan: Alcove CH TRU Tests (SAND 90 8499)

c. Test Plan: Bin CH TRU Tests (SAND 90 8500)

d. Test Plan: Performance Assessment (DOE WIPP 89-011)

e. Test Plan Addendum #1: Bin scale tests (SAND 90-2082)

f. Status Report: Potential for Long-Term Isolation (SAND 90-0616)

g. Preliminary Comparison with 40 C.F.R. Part 1 191 B (SAND 90-2347)

h. WIPP Performance Assessment: 1990 Snapshot (SAND 90-2338)

i. Status of the WIPP Project (Neill & Chaturvedi) (Waste Mgmt. '91)

j. WIPP FSAR Addendum (July 1991) (WP 02-9 Rev. 0) (excerpts)

k. Letter, EEG to Hunt (DOE), Aug. 9, 1991

l. Critical Experiments and Time Lines (W.D. Weart, SNL) (Aug. 12, 1991)

17. From an examination of these materials, discussions with personnel of the Environmental Evaluation Group ("EEG"), and my knowledge of the performance assessment process, three principal conclusions can be reached:

a. First, there is no necessity for the dry bin scale tests to be conducted underground at the WIPP, or at any other specific location. These tests could be conducted on the surface at laboratory facility at or near the present storage location, and there might well be less risk and inconvenience than if they were conducted at the WIPP.

b. Second, the remaining elements of the DOE test plan --the wet bin scale tests, bin-scale solubility tests, and alcove tests--are not prepared to be conducted at the present

time, so that a prohibition on their conduct will not impair any interests of the DOE. Moreover, all the bin-scale tests can be conducted at a laboratory facility and do not need to be conducted at the WIPP.

c. Third, the information to be obtained from the dry bin-scale tests is of unknown importance to the DOE in the process of performance assessment and appears to be of marginal importance in the present context. Thus, again, the DOE's interests will not be impaired if these tests are not immediately begun at the WIPP.

18. The proposed tests are outlined in four principal documents. These are the January 1990 Alcove test plan; the January 1990 Bin scale test plan; the April 1990 Test plan, covering all proposed tests; and the December 1990 Bin scale test addendum.

19. The January 1990 Bin-scale test plan (exhibit B) expressly states that the tests do not have to be conducted at the WIPP: "It is not mandatory on a scientific basis that these bin-scale tests be conducted at the WIPP. The waste-filled test bins do not directly experience the impacts of the repository environment on waste degradation..." (at 28).

20. In fact, the bin-scale tests are designed to be conducted in bins which are loaded with waste at the place of storage, e.g., Idaho National Engineering Laboratory, sealed and maintained under pressure throughout the test period. The only occasion for communicating with the bin atmosphere is to

extract samples of gas for testing or safety monitoring purposes, or to relieve excessive gas pressure, or to inject inert gas to purge potentially explosive gases. These operations could be conducted at any location. To conduct them at the WIPP site or underground adds nothing to the experiment.

21. Indeed, the January 1990 bin-scale test plan acknowledges the existence of "options for conducting these bin-scale tests at other U.S. DOE sites, e.g., the Rocky Flats Plant, the Idaho National Engineering Facility, and possibly others..."(at 29). The plan adds: "The following possibilities are also being evaluated: (a) conducting portions of the bin-scale test program at alternative sites, then moving them to the WIPP as appropriate, or (b) conducting bin tests at alternative sites for waste forms that are not currently transportable to the WIPP, e.g., high-activity wastes from the Savannah River Site." (Id.)

22. Based on my current involvement in the Yucca Mountain project on behalf of the State of Nevada, I can state that there are no current plans to conduct on-site tests with radioactive waste as part of the performance assessment of that site.

23. From an examination of the test plans it is apparent that there are several special constraints imposed upon tests conducted at the WIPP which would not necessarily apply, were the tests conducted at an appropriate laboratory location. Because the WIPP is not designed to function as a laboratory, it does not possess a multiple containment structure, and any

release of radioactivity in the mine environment must be scrupulously avoided. Therefore, there are constraints which limit the concentration of flammable gases, the concentration of oxygen, and the ambient gas pressure in the test bins. These limitations must be enforced by means of pressure relief valves, purging with inert gases, and use of a "getter" substance to extract oxygen. Such devices are cumbersome, will affect the data generated in an experiment, at best will require use of corrective formulas, and may so add to the calculations that usable results are substantially delayed. Some of these constraints need not be applied in an appropriate laboratory setting.

24. The proposed tests, other than the dry bin tests, are simply not prepared to be conducted at present. These proposed tests comprise the following:

a. "wet bin" tests

b. leaching/solubility tests

c. alcove tests

The wet bin tests involve the addition of up to 120 liters of brine to the test bins (see test plan Addendum ex.C.,at 19). However, the WIPP waste acceptance criteria do not allow disposal of waste containing more than 1% of free liquids, and the added brine would exceed the 1% limit. Therefore, the brine at some point must be removed. No procedure has been developed to accomplish this removal. (Id. 84-85) The task is not a minor one, since up to 120 liters of radioactive liquid must be

removed from each of approximately 120 "wet" bins (Id. 23). This is approximately equal to 3804 gallons, or about 20 automobile gas tanks full. Normally, the transfer of radioactive liquid must be carried out within a containment (see DOE Order 6430.1A, Sec. 1325), but no such containment exists at the WIPP or has been incorporated into the test bins. This problem is unresolved. Before wet bin tests may be conducted at the WIPP a Final Safety Analysis Report Addendum must be completed concerning such tests, and it must analyze safety problems in connection with each procedure in the tests. This has not been done, and the wet bin tests cannot be conducted.

25. The leaching/solubility tests involve determination of the extent to which radionuclides become dissolved in brine, which permeates the repository. Such tests will involve addition of brine to transuranic waste and the periodic sampling of brine to determine the dissolved radionuclide content. Such extraction of radioactive liquid, with no containment to protect against the release of radioactivity in event of a spill, is prohibited by at least one DOE order (DOE 6430.1A, Sec. 1325) and by good laboratory practice. No procedure exists to extract liquids safely. A Final Safety Analysis Report Addendum would have to be prepared to analyze the safety implications of whatever procedure may be adopted. This has not been done. With these unsolved problems the DOE is not prepared to conduct the leaching/solubility tests at the WIPP.

26. The alcove tests likewise have not been the subject of a Final Safety Analysis Report Addendum. Furthermore, a seal must be developed to close the alcove to passage of gas. No seal has been developed. The DOE does not expect to develop such a seal until possibly 1993. Consequently, the DOE is not now prepared to conduct the alcove tests at the WIPP.

27. The August 12, 1991 Sandia presentation of Critical Experiments and Time Lines (exhibit D) shows that even by the DOE's schedule the wet bin, solubility, and alcove tests cannot be conducted at the present time or for many months in the future.

28. Turning again to the dry bin tests, there is a fundamental lack of any strong justification for such tests. The dry bin tests will generate data reflecting the rate of gas generation only during the operational phase of the repository, i.e., prior to the sealing of the repository for permanent disposal. During the operational phase, it is not anticipated that significant quantities of brine will enter and remain in the repository horizon, because brine will be removed by the ventilation system. The dry bin tests will therefore measure only the gas which can be expected to be generated while the repository is in use and being ventilated, inter alia, to remove gas. Such information is not material to the determination of whether the amount of gas generated from brine-soaked waste in a sealed repository after permanent disposal will affect the repository's compliance with long-term disposal standards.

29. The original bin-scale test plan (January 1990) set down the following objectives for the bin-scale tests.

1. Quantify with a high degree of control gas generation - and depletion - rates, and compositions from actual TRU wastes, as a function of waste type, time, and interactions with brines and other repository natural and engineered barrier materials. Experimental conditions will represent, primarily, the longer-term, post-operational phase of the repository as well as the operational-phase. With the exception of VOC's, these tests will not quantify total gas generation potentials (quantities).

2. Provide a larger-scale evaluation and extension of the laboratory-scale test results, using actual TRU wastes under repository relevant, expected conditions. The use of accelerative, overtest conditions could bias interpretations and will not be permitted.

3. Evaluate the synergistic impacts of microbial action, varying degrees of brine saturation, waste compaction, degradation-product contamination, etc., on the gas-generation capacity and geochemical environment of TRU waste.

4. Incorporate representative long-term impacts of room closure and waste compaction on gas generation by including supercompacted wastes.

5. Evaluate effectiveness for minimizing overall gas generation by incorporating getter materials, waste form modifications, and/or engineered fixes into the CH TRU waste test system.

6. Measure solution-leachate, source-term radiochemistry and hazardous-constituent (i.e., organics, toxic metals) chemistry of brine-saturated TRU wastes, as a function of many credible environmental variables.

7. Determine the amount of volatile organic compounds and other hazardous gases released from the TRU wastes under realistic repository conditions, to quantify how EPA hazardous waste regulations will impact the WIPP.

8. Conduct detailed pretest and posttest waste characterizations of all wastes used in this program to quantify radioactive species, hazardous waste constituents, and overall waste matrix components.

These characterizations are necessary to demonstrate both to what extent test wastes are representative of the behavior of all CH TRU wastes and to provide information needed in test data interpretations. Posttest waste characterizations will specifically quantify the total VOC source-term available in the tested waste materials.

9. Specifically determine to what extent the test wastes are "representative" of, and/or bracket, the RCRA constituent concentrations of the CH TRU wastes in storage at DOE waste generator sites that are to be isolated at WIPP. Wastes to be considered for WIPP emplacement and tested in this program must meet specifications of both the WIPP waste acceptance criteria and applicable transportation requirements.

10. Provide necessary gas-generation and -depletion data and source-term information in direct support of WIPP PA analyses, predictive modeling, and related evaluations, as well as for related EPA RCRA characterizations.

11. Help establish an acceptable level of confidence in the WIPP PA calculations. Help evaluate the validity of pertinent assumptions used in modeling. Help eliminate most "what if" questions and concerns.

(ex. B January 1990 Test Plan, at 14-15)

30. Almost none of the stated objectives can be attained by the dry bin tests as now envisioned. I discuss the objectives in order:

1. The dry bin test results will reflect only the operational, not the disposal, phase. Moreover, the dry bin tests are not representative for hazardous constituents.

2. The previous laboratory tests and the bin-scale tests cannot be extended to saturated or partially saturated conditions, where agents affecting solubility or chelation could be accurately evaluated. Moreover, the repository conditions do not affect these tests.

3. There is no synergism to evaluate, because nothing will be added to the waste (unless inert gases are required to purge explosive gases), and there will be no compaction.

4. Availability of compacted wastes at present is unknown. The Rocky Flats supercompactor is not operational.

5. Form modifications and engineered "fixes" are not now contemplated.

6. Leaching/solubility tests are not part of the dry bin tests.

7. No realistic repository conditions are present in dry bin tests. Results may not be representative for inventory of hazardous constituents.

8. Characterization must be done, but the test waste may not be representative for hazardous constituents.

9. Results may not be representative for hazardous constituents.

10. Initial dry bin tests will provide only data about the variability of gas generation data for waste within a stated TRUCON code. Dry bin test data does not relate to disposal conditions.

11. Dry bin tests relate to the operational phase, not the disposal phase. They are not useful for the performance assessment under Part 191.

31. In summary, the dry bin tests can achieve little to establish compliance with the long-term disposal standards.

32. The dry bin tests must also be examined in terms of the current status of performance assessment modeling. The performance assessment of the WIPP is being conducted by Sandia National Laboratories ("Sandia"). This task incorporates numerous segments, not all of which are at the same stage of completion. Certain of the tasks involve the conduct of experiments to develop data for use in the performance assessment. Certain other tasks involve the development of computer modeling systems to represent segments of the disposal system whose behavior is to be studied in connection with the possible release of radionuclides.

33. The overall approach to performance assessment taken by Sandia is based upon the identification of the events that can occur to affect the disposal system, the likelihood of their occurrence, and the consequence of their occurrence. The analysis also includes the remaining uncertainties and their effect on the confidence in the predicted results.

34. The EPA's Guidance for Implementation of Subpart B assumes that the results of the performance assessments will be assembled in a complementary cumulative distribution function (CCDF), which graphically reflects the cumulative release of radioactivity which relates to the relevant levels of probability.

35. Several CCDF's are developed individually in the process of performance assessment, when various possible release "scenarios" are examined and their likelihood and consequences

are expressed. Ultimately, the CCDF's for the individual scenarios will be summed to express in a single CCDF the likelihood of release of radioactivity from the repository.

36. The compliance assessment methodology proceeds by first describing the disposal system in terms of the geologic and hydrologic characteristics of the area, the repository design, and the waste. This process results in the development of numerous conceptual models, which are then translated into mathematical descriptions, i.e., computer model descriptions of the behavior of segments of the physical world under varying input circumstances.

37. The compliance assessment methodology also involves the identification and development of release "scenarios." The fact situations comprised within such scenarios are then applied to the mathematical models to determine the consequences of the scenarios.

38. Each scenario is supposed to be mutually exclusive, so that the probability and consequence of each may be summed to determine the probability and consequence of release from the repository.

39. Each scenario itself embraces the full range of probabilities and consequences with respect to the uncertain events and processes contained within the scenario.

40. Scenarios are analyzed to identify those parameters that are important to the probability and the consequence and to determine the effect of a change in the parameters upon the

mathematical model result. This is the process of sensitivity analysis.

41. Mathematical models can also be verified and possibly validated by comparison against available data concerning physical events and processes.

42. The compliance assessment process is carried out by linked computer programs controlled by an overall executive process, the CAMCON.

43. In the 1990 iteration, entitled Preliminary Comparison with 40 C.F.R. Part 191, Subpart B for the Waste Isolation Pilot Plant, December 1990, analysis of probabilities and consequences was carried out by employing 29 imprecisely known variables, which are listed in Table C-2 thereof. These variables do not include a value for gas generation by waste.

44. Sensitivity analysis of the importance of gas generation would demonstrate whether that process is more or less important than other processes in determining the consequences of each of the pertinent scenarios. If the factor of the rate of gas generation is less important, it may require less effort to decrease the uncertainty in the range and distribution of the factor to an acceptable level. A sensitivity analysis of the gas generation rate will also provide information governing the priorities of data collection.

45. As stated in Review and Discussion of Code Linkage and Data Flow in Nuclear Waste Compliance Assessments, SAND 87-2833 (1978), the "use of sensitivity analysis is to gain

understanding and insight about the system. Hence, its primary usefulness is in the early phases of an assessment to help produce necessary understanding and allocate resources to develop a credible compliance assessment." (at 43). Part of the process of compliance assessment is to assume that "concerns raised both within and outside the project remain in the proper perspective; without this system viewpoint, any one concern has the potential to take the project off on a tangent." (at 48).

46. To my knowledge, no sensitivity analysis of the factor of the rate of gas generation has been conducted with respect to any of the release scenarios. Thus, it is not possible to say at present that data concerning gas generation is important or unimportant to the performance assessment process.

47. Furthermore, at present the status of the compliance assessment conceptual models of gas generation by corrosion and gas generation by biological means are said by Sandia to be "preliminary," which means that the "understanding of the component or subsystem is intuitive and incomplete" (Preliminary Comparison at V-107, Table V-7).

48. At the end of 1990 Sandia reported that "[s]imulations that incorporate gas are preliminary, and cannot be used to quantify sensitivity of the modeling system to gas generation." (Id. VI-1).

49. Therefore, based on the 1990 Preliminary Comparison, it is not possible to state that data concerning gas generation-

-the principal purpose of the bin tests--are important to the performance analysis.

FURTHER AFFIANT SAITH NOT

Linda L. Lehman

Linda L. Lehman

ACKNOWLEDGMENT

STATE OF MINNESOTA
COUNTY OF DAKOTA

The foregoing instrument was acknowledged before me by Linda L. Lehman on October 6, 1991.

Vicki K. Ingraham
Notary Public

VICKI K. INGRAHAM
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My Commission Expires 1/31/95

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RESUMÉ

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University of South Florida, M.S., 1978, Hydrogeology
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President/Principal Hydrogeologist
L. Lehman & Associates, Inc.; 1985 - Present

Private Consultant
Hydrogeology; 1983 - 1985

Hydraulic Engineer
U.S. Nuclear Regulatory Commission; 1979 - 1982

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Parsons, Brinkerhoff, Quade & Douglas, Inc.; 1977 - 1979

EXPERIENCE:

Ground Water Modeling

- Currently developing modeling efforts as the representative of the State of Nevada at the international flow and transport model validation effort for nuclear waste repository performance codes (INTRAVAL).
- Directed the development of conceptual flow models at solid and hazardous waste sites contaminated with volatile organic contaminants and other pollutants.
- Performed ground water flow and contaminant transport modeling of high-level nuclear waste sites (the Hanford Site Washington).
- Performed time series analyses using computerized data bases to establish baseline ground water conditions at high-level nuclear waste sites.

SELECTED PUBLICATIONS:

Nguyen, V.V., G.V. Abi-Ghanem and L.L. Lehman; Fractal Mixing in a Class of Composite Media; Preprints of Proceedings of the Stochastic Approach to Subsurface Flow, Montvillargenne, France; 6/85.

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Lehman, L.L.; Model Comparison; Comments of the Yakima Indian Nation on the Draft Environmental Assessment for the Hanford Site, Washington under the Nuclear Waste Policy Act, Volume 2; 3/85.

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TEST PLAN: WIPP Bin-Scale CH TRU Waste Tests

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smaller-scale laboratory data on simulated wastes. It must be emphasized that it is the combined suite of CH TRU waste test programs, laboratory, bin-scale, and alcove, that is required to provide the full spectrum of information and expertise needed for the WIPP PA program. The three experimental programs must be linked with both geochemical modeling and studies of the response of the WIPP to elevated gas pressures, should these be generated. Each test program has its own significant advantages and disadvantages. None of the three test programs alone can credibly produce the required information.

The laboratory tests [Brush, 1989; Bertram-Howery and Hunter, 1989] were initiated in FY 89 and will be conducted in parallel with the WIPP bin-scale and in situ alcove [Molecke, 1989b] tests, both of which will begin in FY 90. These parallel test programs will proceed concurrently and will be sequenced to permit the early laboratory results to have some impact on the configuration of the bin-scale tests, and vice versa. For example, backfill getter additives to be evaluated in laboratory tests for gas and brine sorption capability would be selected and evaluated by the end of FY 90 [Lappin, 1989a], then subsequently evaluated for in situ efficacy in Phases 2 and 3 of the bin-scale tests (to be described later). Also, preliminary brine-leachate results from the bin-scale tests could be used to help "focus" laboratory evaluations of radionuclide chemistry into specific ranges of test conditions as quickly as possible [Lappin, 1989a]. Initial results from both the laboratory and bin-scale tests could be used to help redefine the starting test parameters of the alcove tests on an alcove by alcove basis, assuming that the wastes and other test materials had not already been loaded, the alcove sealed (from access), and testing in that specific alcove initiated. Results from the alcove tests are not currently anticipated to have much feedback to the laboratory and bin-scale tests because of their later schedule sequencing and emplacement — with the possible exception of later contingency additions to Phase 3 of the bin-scale tests.

5.5 OPTIONS ON BIN-SCALE TEST LOCATION

It is not mandatory on a scientific basis that these bin-scale tests be conducted at the WIPP. The waste-filled test bins do not directly experience the impacts of the repository environment on waste degradation, as do the parallel in situ alcove CH TRU waste tests [Molecke, 1989]. It is mandatory,

however, that these tests provide most of the required data to the WIPP Performance Assessment modeling effort in the necessary time frame, before the end of FY92 [Bertram-Howery and Hunter, 1989]. Due to uncertainties in current WIPP opening and waste availability schedules, options for conducting these bin-scale tests at other U.S. DOE sites, e.g., the Rocky Flats Plant, the Idaho National Engineering Facility, and possibly others have been investigated. The merits, technical relevance, schedule feasibility, and expenses for the other site options are still being evaluated. The following possibilities are also being evaluated: (a) conducting portions of the bin-scale test program at alternate sites, then moving them to the WIPP as appropriate, or (b) conducting bin tests at alternate sites for waste forms that are not currently transportable to the WIPP, e.g., high-activity wastes from the Savannah River Site.

Conducting the bin-scale tests underground at WIPP is by far the best choice or option based on the deciding factors listed in Table 5.2. The WIPP site and other sites are compared in this Table; deciding factors are listed in approximate descending order of importance.

Table 5.2 Deciding Factors and Options For Bin-Scale Test Location

Favored Site	Deciding Factors:
W (= WIPP)	1. Time Availability to Meet WIPP PA Needs
W	2. Test Set-up and Instrumentation Time
W	3. Isolation from the Accessible Environment
W	4. In Situ Temperature Control ($2^{o}C$ range)
W	5. Availabilities of Test Facilities, Buildings
W	6. Minimization of Overall Test Expenses
W	7. Programmatic Concerns, Site Relevance
W provided	8. SNL-WIPP PI and Instrumentation Control
W provided	9. Data Acquisition and Control Systems
W	10. Minimization of Travel, Key Personnel
W, O (= Other)	11. Test Radiological Safety and Control
W, O	12. Technical Personnel, Training & Availability
W, O	13. Analytical Instrumentation and Availability
O, W	14. Legislation and Permitting Uncertainties
O	15. Waste Transportation Concerns

VI. Lehman C

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TEST PLAN ADDENDUM #1: WIPP BIN-SCALE CH TRU WASTE TESTS

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quate to support microbial colonies and provide gas production from microbial degradation mechanisms [Molecke 1979, Caldwell et al 1987]. However, the extent and duration of activity that can be supported by this initial moisture content remains to be determined.

c. "Wet" - This term is used to mean tests with up to 120 liters of Salado brine(s) and added excess WIPP rock salt. The salt (backfill material, described in Section 10.1.1 of the Test Plan), 3 $ft.^3$/bin, equivalent to 110 kg, is added to provide a repository relevant halite-saturated chemical environment. The salt also ensures that microbial inoculation takes place, in addition to the microbes present in the natural, underground-collected Salado brines [Brush 1990b]. As presently planned, and as reiterated by the DOE/WPO, the underground bin-scale gas testing program specifically includes bins with up to 120 liters of added brine.

d. "Phase 0" - Initial gas testing, in which we evaluate the bin-to-bin variability of gas production potentials for similar wastes (of the same TRUCON codes) under "uniform" anaerobic, "dry" experimental conditions — with no added brine, salt, or other backfill materials. The only exceptions to the anaerobic (only) conditions include: testing of two high-organic, high-cellulosic-content, TRUCON codes (116/216 and 119/219) under both anaerobic and aerobic conditions; and, testing of all sludge codes under initially aerobic conditions only. Note: Phase 0 testing is described in detail in Section 3.2; the statistical implications of bin-to-bin variability are reported separately [Lappin et al 1990].

From a procedural point of view, we feel that a limited number of "wet" bins (Phase 1 tests) could be incorporated into early testing as soon as such bins became available, so long as the Phase 0 testing of the same TRUCON codes had been initiated for at least 6 months. Because of this intentional delay in the use of "wet" bins, future editions of the Integrated System Checkout Plan [WID 1990b and revisions thereto] must include capabilities (i.e., interface requirements) for the upcoming handling, and both emplacement and posttest retrieval operations for "wet" bins. However, these capabilities do not need to be included for the initial, planned starting schedule.

failed pump in the oxygen-sensor system.

d) An appreciable, i.e., > 5° C, upward trend in bin-waste temperature, indicative of potential microbial activity or the onset of spontaneous combustion.

5.14.1 Posttest Waste Handling

No further data gathering requirements are placed on the test bins and wastes following the obtainment of the final, t = termination gas sample ("final" as determined by the Principal Investigator).

5.14.1.1 Vacuum Distillation Deletion

The previous requirement to subject each posttest waste bin to the procedure of vacuum distillation at the WIPP is being deleted. This deletion has been discussed with the Environmental Protection Agency [US EPA 1990b] and is not an issue of contention at present. The major piece of data from such distillation, the "source term" for bin total VOC content, has been determined not to be necessary or required to demonstrate "representativeness" with respect to waste VOC content. Refer to the "representativeness" discussion in Section 4.2.4.

5.14.1.2 Removal of Posttest Brine

The added brine in the "wet" bins needs to be removed at the conclusion of the gas testing phase, to obtain "dry" wastes that would again meet the WIPP waste acceptance criteria (WAC). This drying was another stated goal of the (previously proposed, now deleted) vacuum distillation procedure, to remove free liquids/brine.

The brine remaining in test bins at the conclusion of the gas testing phase can be either drained or pumped from the bin(s), and/or solidified in place. The brine will not be used for any (currently planned) data gathering activity. All waste brine-related data will be obtained from the parallel WIPP CH TRU Waste Leaching/Solubility Test Program, as described in Section 2. The procedures and hardware required for this brine removal and/or solidi-

fication need to be designed and provided by WID, as part of waste retrieval activities, and addressed in a future revision to the FSAR Addendum.

Following final brine removal from the bins (if not solidified in place), some residual, sorbed brine may still be present in the TRU wastes, probably more than the amount allowed by the WIPP WAC. The procedures and equipment required to dry or solidify the brine in place also need to be designed and provided by WID, as part of waste retrieval activities. Options mentioned in the past by WID include injection of drying agents. Other options are to be developed, as appropriate.

5.16.2 Additional Test Safety Related Concerns

5.16.2.1 Periodic Instrumentation Monitoring of Potentially Flammable/Explosible Gases

The topic of on-line, periodic monitoring of bin gases for potentially flammable and/or explosible concentrations of hydrogen and methane (assuming an adequate concentration of oxygen) was discussed at the WIPP project RECAP meeting on October 5, 1990. The major option discussed at this meeting was the possible use of specific gas monitors in addition to the current oxygen sensors, i.e., possibly for hydrogen, methane, LEL (lower explosibility limit), etc. All of these gages would be operated solely for their safety related function; they would have the benefit of having a local readout (and/or remote readout at the site Computer Monitoring System) which could be checked periodically, independent of the DAS computer. Because such gas monitors would have safety functions only, they would be provided, monitored, and maintained by WID.

Such specific-gas monitors would be in addition to the gas concentration data to be obtained from the periodic gas samples (and GC-MS analyses), as described in the Test Plan.

SNL personnel (lead: Paul Cahill, division 1811) have investigated the availability and applicability of such specific gas monitors. Initial survey results (from about 10 suppliers) indicate that essentially all of these (off-the-shelf) gas (safety) monitors are designed for use in an air environment,

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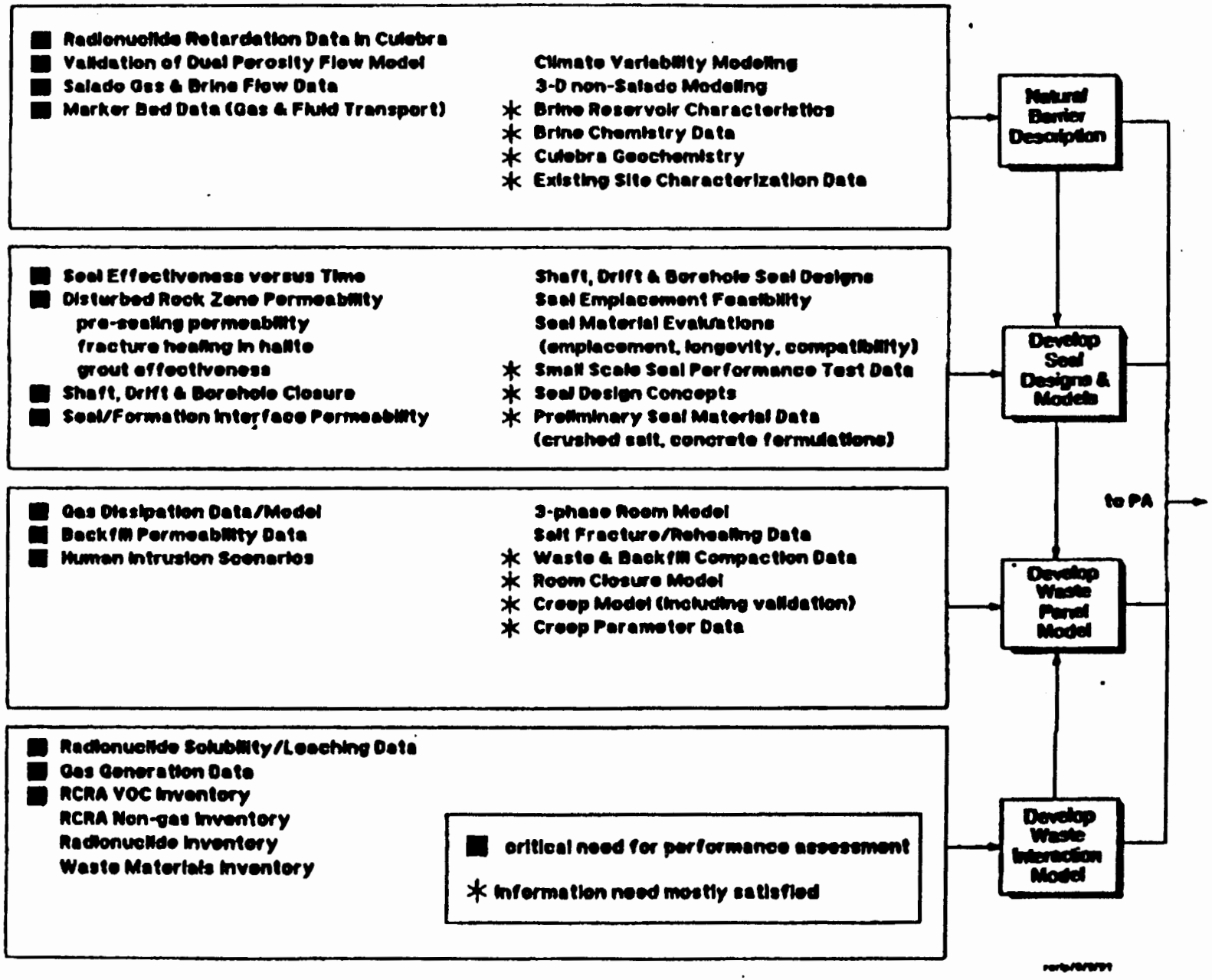
CRITICAL EXPERIMENTS
and
TIME LINES

August 12, 1991

Wendell D. Weart

SNL

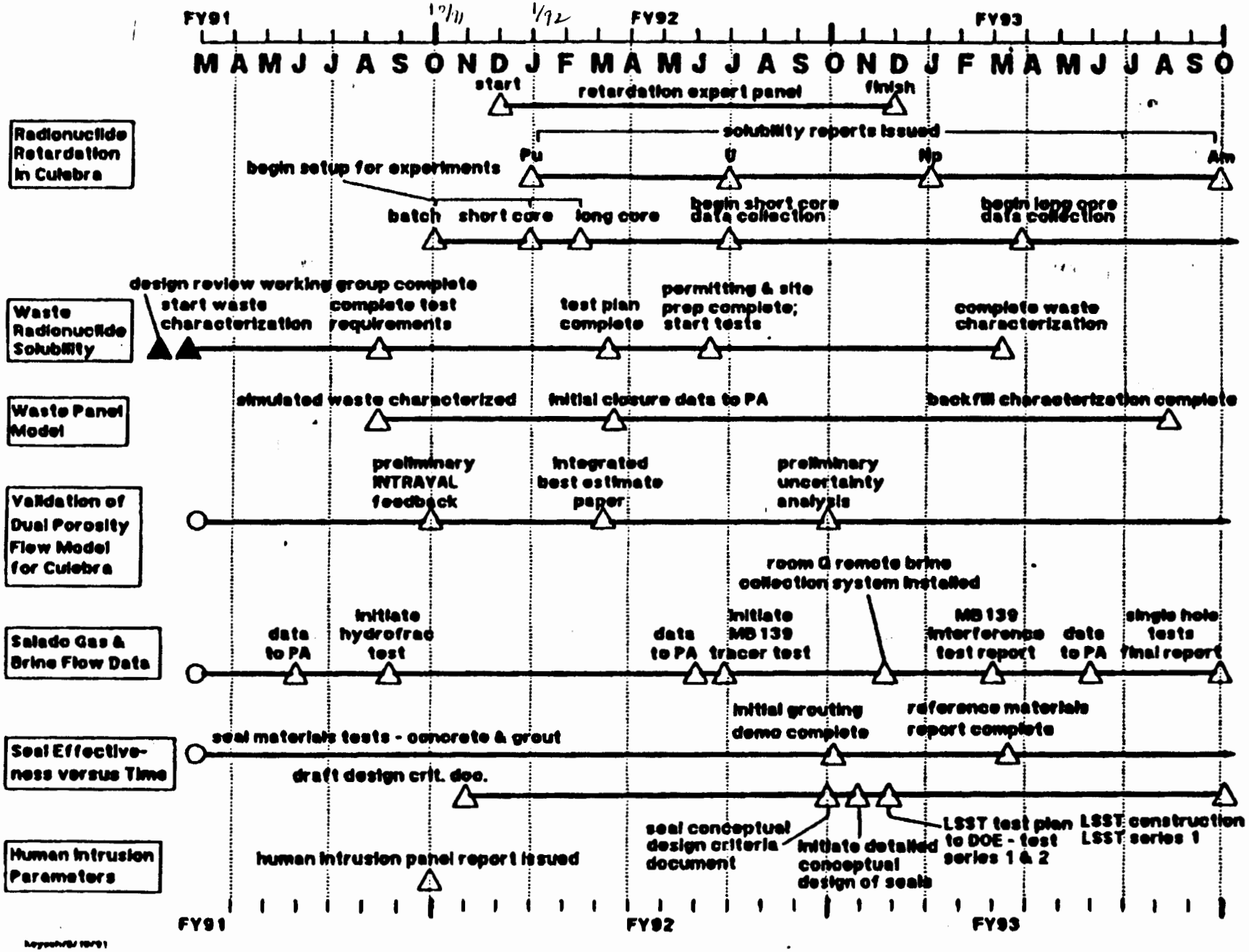
Necessary Information Needs for 191 & RCRA PA



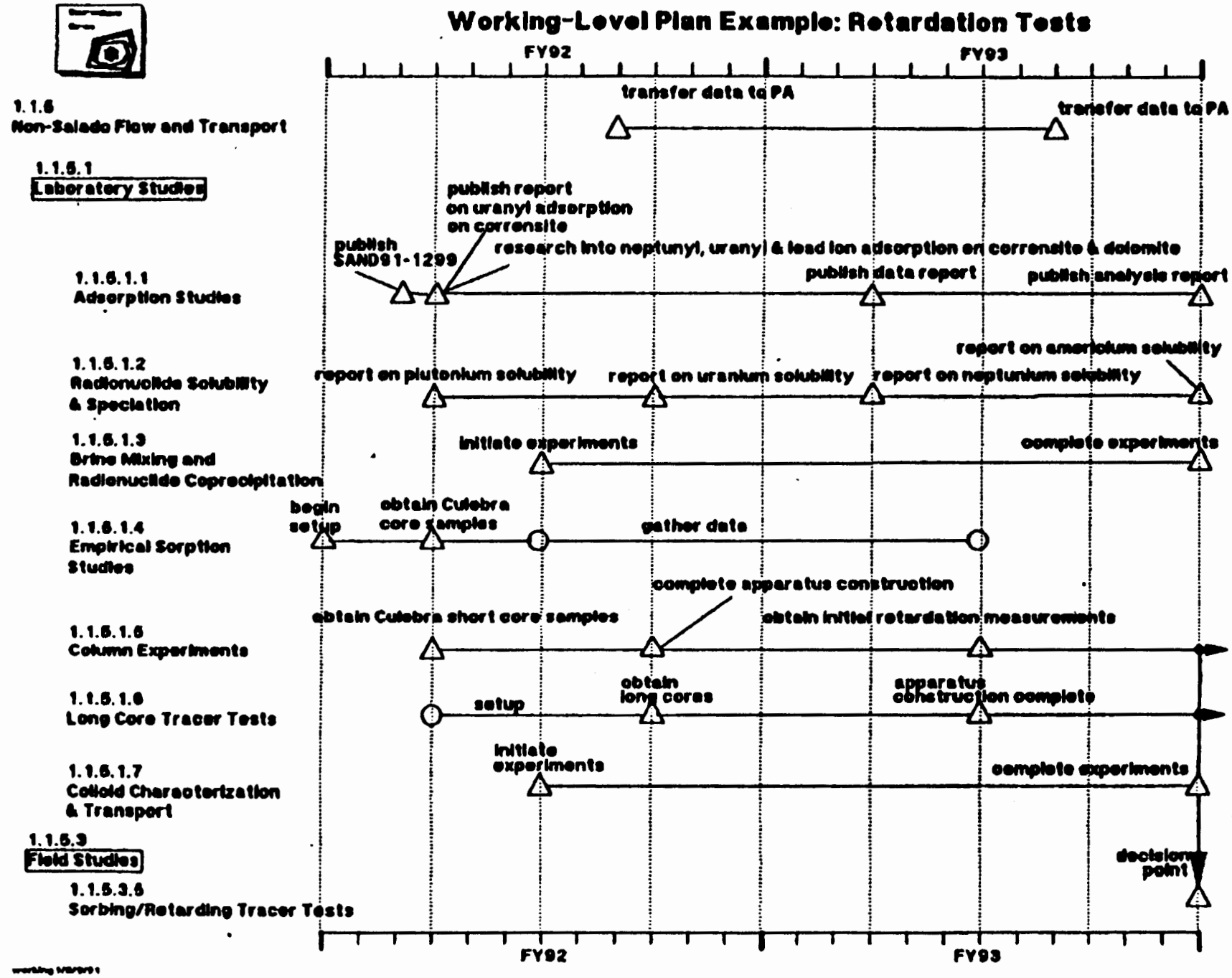
Information Need	Activities Producing Information
Radionuclide Retardation in Culebra	Expert Panel & Laboratory Retardation Experiments
Waste Radionuclide Solubility	Lab Tests (surrogate & radioactive nuclides) Bench & Field Scale Tests (TRU wastes)
Waste Panel Model	Laboratory/In-Situ Experiments/Analysis/Model Development
Validation of Dual Porosity Flow Model for Culebra	Analysis & Evaluation of Existing Data
Salado Gas & Brine Flow Data	Field Experiments/Analysis
Seal Effectiveness versus Time	Field & Laboratory Experiments/Modeling
Human Intrusion Parameters	Expert Panel on Human Intrusion

Figure 5. Relationship of Information Needs to Data Gathering Activities for Categories Having High PA Sensitivity

HIGHEST PRIORITY ACTIVITIES

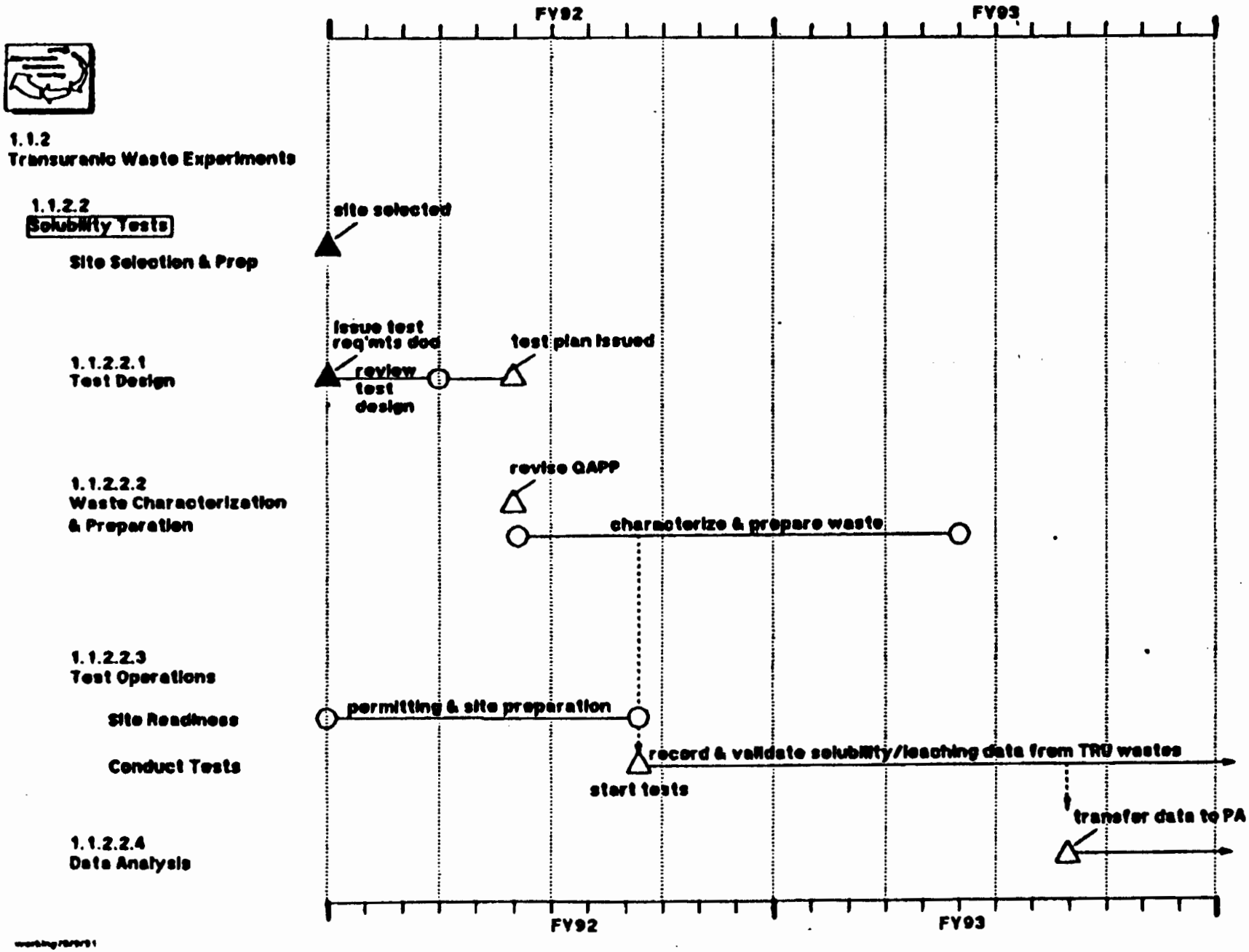


Working-Level Plan Example: Retardation Tests



1991 1992 1993

Working-Level Plan Example: Solubility Tests



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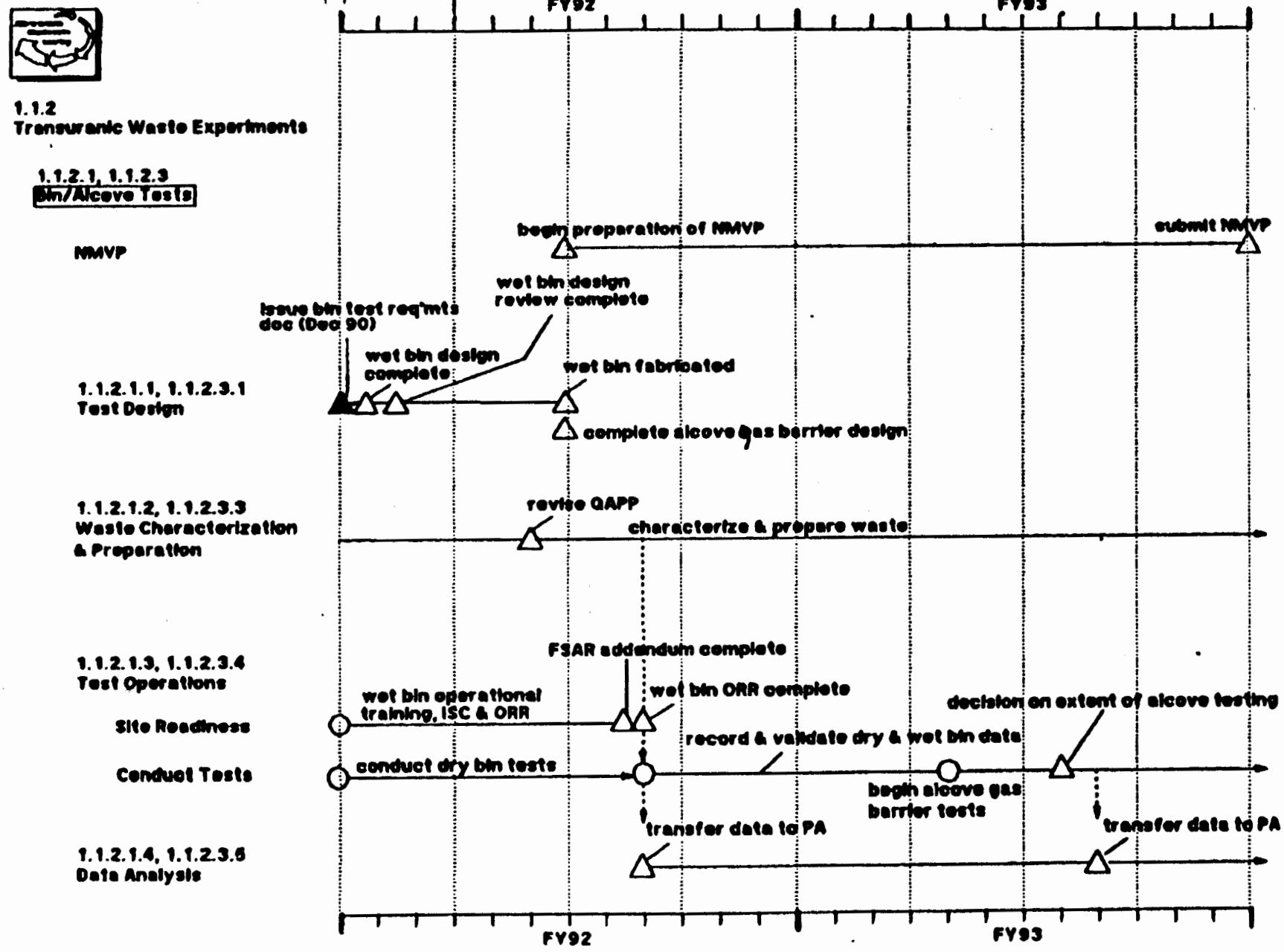
Working-Level Plan Example: Bin/Alcove Tests



Performance Assessment Requirements for Bin Tests
40CFR191

- High gas generation rates lead to gas-dominated rooms
  - PA indicates less radioactive release for human intrusion scenarios
- Low gas generation may lead to brine saturated waste
  - PA indicates more release for human intrusion
- Neither case leads to releases in 10,000 years without human intrusion
- Conclusion: Lower gas generation rates can lead to more severe waste room source term conditions

Performance Assessment Requirements for Bin Tests
40CFR268 (RCRA/ No Migration Determination)

- High gas generation rates will cause gases to migrate farther along interbeds
  - RCRA gases (VOCs) can be carried along with these gases toward RCRA boundary
- RCRA standard does <u>not</u> require consideration of human intrusion
- Conclusion: High gas generation rates can lead to a more severe waste room source term for VOC migration

Performance Assessment Requirements for Bin Tests
Conclusion

- Neither a high nor a low bounding assessment of gas generation rate is sufficient to assure the most severe long-term conditions for both WIPP standards since the bounds act in opposite directions
- Conclusion: Realistic values of TRU waste gas generation must be determined to adequately represent the waste room source term for both standards
- Bin tests are the most realistic simulation of repository/waste gas generation interactions and should be conducted to provide our best understanding of future waste room source term conditions

Technical Concerns Often Raised Regarding Radioactive Tests at WIPP

Is gas generation really a major issue?

- Performance Assessment to date has not addressed RCRA, an area where high gas generation increases concern for compliance
- Evaluation of both 40CFR191 and RCRA requires best estimates of gas; bounds are not sufficient
- Lack of knowledge on the gas generation issue will not be acceptable to the public

Technical Concerns Often Raised Regarding Radioactive Tests at WIPP

Radioactive waste tests are commencing later than originally expected

- Wasteform complexity and regulatory & safety requirements do lengthen times
- Characterization will be required to ship waste to WIPP-- tests or not
- Bin tests will provide timely data to PA on present schedule

Test results will be difficult to interpret and extend to WIPP

- Test data will provide statistical knowledge of gas generation in WIPP
- Lab data will supplement bins for phenomenological interpretation
- Extensive test matrix will allow extrapolation to the WIPP repository

Technical Concerns Often Raised Regarding Radioactive Tests at WIPP

Solubility experiments have been removed from the WIPP bin tests

- DOE is committed to accelerating solubility experiments with TRU waste
- Other facilities are being actively considered to speed solubility tests
- New bin design to allow solubility testing at WIPP is being investigated

SUMMARY OF CRITICAL EXPERIMENTS

Retardation in the Culebra Aquifer

- Expert Panel Deliberations in Early 1992
- Laboratory Tests Accelerated

Solubility

LA ~ W/PO

- TRU Solubility Tests to Commence in Mid-'92

Bin Test

HOND OAY ↑

- Tests with Some High-Organic TRU Earlier in Program
- Redesign of Bins to Accelerate Wet Bin Testing into Mid-'92
- Decision on Alcoves in Early 1993 Based on Results of Bin Tests & Alcove Gas Barrier

SUMMARY OF AFFIDAVIT OF JACK PARKER

Mr. Parker is a geologist and mining engineer, specializing in rock mechanics. He is an independent consultant who was selected by the DOE in 1991 as a member of the DOE's expert panel reviewing the room stability problems encountered at the WIPP.

Mr. Parker concludes that the waste emplaced for tests will become non-retrievable within less than a year. Specifically:

1. Room 1, Panel 1, the test room where the waste will be placed, exhibits fractures similar to those which gave rise to the roof collapse in SPDV Room 1 in February 1991. (¶31).
2. Failure of the test room is highly probable. (¶31).
3. If the test room follows the pattern of SPDV Room 1, it can be expected to be closed to access by October 1992. (¶31).
4. The DOE's proposed roof support system, relying upon rock bolts, is exposed to shearing and breaking of the bolts. There is insufficient bolt clearance to accommodate shear movement. (¶¶ 37-42).
5. The equipment clearance necessary to retrieve the waste will be lost within less than a year. (¶¶ 44-45, 47).
6. The DOE's roof support system will probably impair warning of a roof failure by masking the acceleration of closure which provides a warning signal. (¶ 47).

Affidavit Of Jack Parker

Jack Parker, being duly sworn, deposes and says:

1. I am the principal of Jack Parker and Associates, Inc., a rock mechanics and mining consulting firm located in White Pine, Michigan. I have provided one-time and ongoing consulting services to the mining and related industries since 1971. My resume is attached as Exhibit A to this affidavit.

2. I was born in England and educated there through secondary schools. At age 16, when many English students leave school, I did so and began work in the coal mines, first as an office boy and then in functions involving surveying, engineering and planning at four coal mines held by the National Coal Board.

3. I was selected to participate in the National Coal Board education scheme, which provided night classes and part-time day classes for promising students in the scientific fields. Under that system I received my secondary education.

4. In 1953 I migrated to Canada and thereafter worked as an engineer and surveyor on mine development projects at Hudson's Bay and at Sudbury, Bancroft, Haileybury, and North Bay, Ontario.

5. In 1954 I enrolled in Michigan Tech to study both mining engineering and geological engineering in a dual degree program. I received a B.S. in geological engineering and a B.S. in mining engineering in 1958 and a M.S. in geology in 1960. My

grade-point average was 3.75 out of a possible 4.00, and I ranked fourth in a class of 384.

6. After I received my B.S. degrees I taught at Michigan Tech in the Geology Department while working toward my M.S. I also undertook various work at mines in Ontario and Quebec.

7. In 1961 I joined R.L. Loofbourow, a mining consulting firm in Minneapolis.

8. Later in 1961 I received an offer to work as a foreman at the White Pine Copper Mine in White Pine, Michigan. I accepted the position. First I worked as a mine foreman in connection with underground mine development. After a year, I was given responsibility, as a research engineer, for the design of drilling plans, blasting, and bolting. After two further years I was made Director of Rock Mechanics and supervised three engineers and ten technicians. In this position, I had responsibility for the geotechnical aspects of mining methods, mine layouts, and overall questions of rock stability. I held this position for seven years until an industry downturn.

9. Since 1971 I have acted as a self-employed mining and geologic consultant, emphasizing problems of rock mechanics. The science of rock mechanics, as I practice it, involves a practical understanding of the properties and behavior of rocks in place and how to control them to attain stated objectives. Most of my practice involves clients in the mining and civil engineering fields. Typically, I am requested to address a problem which is described as involving rock stability or

anticipated instability. Normally, I visit the location to observe the precise nature of the problem and to apply observation techniques and basic instrumentation. I normally study available geologic data reflecting the nature of the strata in question and the physical properties of the rocks. It is usually necessary to review the plans of the underground workings involved to obtain an understanding of the probable stresses to which they are exposed. Based upon such information, and using lessons learned at previous sites, I will normally recommend one or more solutions, with the principal objectives being practicality, safety, and efficiency in time and money.

10. I have specific experience in dealing with rock mechanics problems involving evaporite deposits. A partial list of clients is attached to my resume. Representation involving evaporite deposits includes the following:

a. Allied Chemical, Wyoming: A trona deposit.

b. Amax, Inc., New Mexico: Potash deposit in Carlsbad area.

c. Canadian Rock Salt, Ltd., Ontario and Nova Scotia: Rock salt deposits.

d. Cargill Salt, Louisiana and New York: Salt mines.

e. Cliffs Engineering, Colorado: Nahcolite deposit.

f. Domtar Minerals, Canada, Michigan: Gypsum deposits.

g. Domtar Minerals, Louisiana: Rock salt deposit.

h. International Salt Co., Ohio and New York: Rock salt deposits.

i. Kerr McGee, New Mexico: Potash deposit in Carlsbad area.

j. Morton Salt, Louisiana and Ohio: Rock salt deposits.

k. Texasgulf, Inc., Wyoming: Trona deposit.

l. Westroc, Ltd., Ontario and Manitoba: Gypsum deposits.

11. I conducted a major project for the United States Bureau of Mines concerning roof control in coal mines and, specifically, resin bolting techniques. The study included mines in the United States, England, and France and generated a handbook on the design, installation and operation of resin bolted roof control systems.

12. I have been recognized in court as an expert in the fields of geology, mine engineering and rock mechanics.

13. In April 1991 I was requested by Dr. Roy Cook of Westinghouse Electric Corp. ("Westinghouse") to be a member of a panel of geotechnical experts which was convened to provide outside review and evaluation of geologic conditions at the Waste Isolation Pilot Plant ("WIPP") and, specifically, to provide an estimate of the life expectancy of the underground rooms in Panel 1, where a test program was to be conducted, and if necessary to recommend additional remedial actions that would

improve the longevity of Panel 1 rooms to allow the tests to be successfully completed. I agreed to join the panel.

14. As a panel member, I was provided with certain geotechnical data. These included:

a. Geotechnical Field Data and Analysis, DOE WIPP 91-012 (June 1990)

b. Interim Geotechnical Field Data Report, DOE WIPP 86-012 (Fall 1986)(extracts)

c. Design Validation Final Report, DOE WIPP 86-010 (1986)(extracts)

d. Borns, D.J., and J.C. Stormont, An Interim Report on Excavation Effects Studies at the Waste Isolation Pilot Plant: Delineation of the Disturbed Rock Zone, SAND 87-1375 (1988)

e. Stormont, J.C., Discontinuous Behavior Near Excavations in a Bedded Salt Formation, SAND 89-2403 (1989)

f. Design Criteria Waste Isolation Pilot Plant, Revised Mission Concept -IIA, WIPP DOE-71, Rev. 4 (extracts)

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

h. Brunwald, H.P., and H.C. Howarth, Compression Tests of Roof-Salt Slabs Supported by Potash Salt Pillars, RI-3386, U.S. Bureau of Mines (1938)

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

15. The panel met on April 9-10, 1991, in Carlsbad, New Mexico. At that time I visited the underground mine workings at the WIPP and made personal observations of the nature of the roof stability problems.

16. In particular, I observed the result of the roof fall in SPDV Room 1. In that room a segment of the roof weighing more than 1000 tons separated from the roof and fell. SPDV Room 1 is now closed to any access, and I was not permitted to enter the room.

17. Moreover, in an adjacent SPDV Room I studied observation holes cut in the floor and the roof of the room. These holes are bored so that one can observe the change over time in the horizontal and vertical relationships of the strata that are penetrated by the hole. I noted that one hole in the roof had penetrated strata that had moved sufficiently to obscure the entire diameter of a six inch hole.

18. The panel conferred after hearing presentations from personnel of Westinghouse and the DOE. At the time I expressed the view that Room 1 of Panel 1 --the proposed test location --would not last more than an additional two years from that date, and quite possibly less based upon the unpredictability of roof instability and the apparently faster rate of closure of Room 1, compared to SPDV Room 1. An estimate of two years would mean that the room is vulnerable to failure at any time after approximately April 1993.

19. At the panel meeting I also stated my opinion that any attempt to provide additional roof support without relieving the stresses which give rise to a roof fall would fail to prolong the life of Room 1 as required.

20. I continue to maintain the opinions which I expressed to other members of the expert panel. In my opinion, one cannot assume that Room 1, Panel 1, will remain stable beyond April 1993. In my opinion, a roof support system, like the system depicted in the report entitled Waste Isolation Pilot Plant Supplementary Roof Support System, Underground Storage Area, Room 1, Panel 1, DOE WIPP 91-9230, cannot be expected to extend the useful life of Room 1 as required.

21. At the request of Westinghouse, I submitted my report on the issues presented to the panel and made a brief follow-up letter report. My reports are published in the Report of the Geotechnical Panel on the Effective Life of Rooms in Panel 1, DOE WIPP 91-023 (June 1991). This Report is attached as Exhibit B.

22. In my opinion the roof failure observed at the WIPP has its origin in the design of the mine itself. The combination of wide (33 feet) rooms with flat roofs and unusually wide, stiff pillars (100 feet) created a situation that was prone to failure. This is for the reason that the wide, stiff pillars concentrate stresses at the weak points of a room.

23. At the depth of the WIPP rooms, which is 2150 feet, underground lithostatic pressures of more than 2000 pounds per square inch are to be expected. When salt is extracted at that depth, the open space remaining cannot absorb stress any longer. The stress is, accordingly, diverted to the walls, floor, and roof. Under stress, the opening tends to prefer the most stress-resistant form, which is a circular cross section. In this particular situation, the anhydrite "b" layer is located about seven feet above the roof, separated from the salt by a layer of clay, which is weak and cannot transmit stress. In consequence, the horizontal stress is largely concentrated in the seven foot roof beam, which is unsupported in its 33 foot width. Had the pillars been narrower--say, 33 feet--they would not have been able to transmit high vertical stresses, and the roof beam would not be exposed to such forces. However, such a design was not adopted.

24. In the design adopted for the WIPP, the horizontal forces cause the roof to buckle and to fracture. The parting of the fracture will tend to proceed from the upper corner of the room upward toward the anhydrite "b" layer, following a shallow angle. In Room 1, Panel 1 I observed the beginnings of such fractures in the roof along both walls. Such fractures will ultimately weaken the roof beam to the point that it will be unable to sustain its own weight, and it will fall, as it did in SPDV Room 1. But other forces are involved too.

25. Before it falls, the roof beam will demonstrate shear movement. First, before fracturing, the roof will buckle downwards in the center, weakening the bond of the salt with the overlying clay and anhydrite seam. When fracturing begins, it normally occurs earlier in one side than in the other. At that point the continuing horizontal stress will force the more fractured side of the beam to move laterally and downward, shearing against the rock above it. This behavior is evident in the roofs of the WIPP rooms, which show substantial horizontal movement relative to overlying strata.

26. Thus, the stresses around the mine opening cause it to tend to assume the most favorable shape for an underground tunnel in salt, which is a more or less circular cross-section.

27. The shear fractures will propagate, coinciding with an increase in the measured rate of convergence of the roof and the floor. The rate of convergence, and any acceleration of that rate, provides a warning of the impending occurrence of roof failure in unsupported conditions. Acceleration in the convergence rate apparently results from acceleration in the propagation of roof fractures, so that it provides evidence of a forthcoming failure.

28. The rapid failure of the openings at the WIPP parallels experience elsewhere. Most of the roof falls that I have examined in potash mines in the Carlsbad area have been in rooms adjacent to pillars that were unusually wide and stiff. The design of the WIPP follows local potash mines but <u>reduces</u>

the extraction ratio, and thus widens the pillars. This design places significant additional stress on the roof beam, leading to failure.

29. Similarly, in the Cayuga Salt Mine in upstate New York, there were numerous roof failures at depths of 2000 to 2700 feet in rooms 32 feet wide and 12 feet high. Bolting was adopted to prevent such falls, but without success. The ultimate solution was to adopt yielding pillars only about 20 feet square, which would yield rather than exert high vertical and horizontal stresses.

30. By contrast with the WIPP rooms, access drifts at the WIPP show comparatively long lives and low convergence rates. It is typical of most mines to have access drifts that are narrower than the rooms themselves. Such a configuration leads to better long-term stability. A study of the convergence rates of drifts at the WIPP shows that the amount of convergence is considerably less in drifts than in rooms. Interaction of multiple and parallel openings, as in the panels, accentuates the adverse stress conditions.

31. As to the useful life of Room 1, Panel 1, the eight year life of SPDV Room 1 provides a useful indication of the life span which can be expected. The rooms are not identical. However, the fracture pattern which led to the failure of SPDV Room 1 appears again in the roof of Room 1 and in the shear movements visible in the observation holes in the other SPDV rooms. The failure of all of these rooms is highly probable.

The Panel 1 rooms may, in fact, fail sooner than the SPDV rooms, as suggested by the higher convergence figures for the Panel 1 rooms compared to the SPDV Rooms at a similar age, e.g.:

CONVERGENCE AT 2-3 YEARS, RATE IN INCHES PER YEAR

Panel 1,	Room 1	4.60
	Room 2	3.33
	Room 3	3.34
	Room 4	2.56
	Room 5	2.28
	Room 6	2.76
	Room 7	2.70
SPDV	Room 1	2.93
	Room 2	2.39
	Room 3	2.64
	Room 4	2.65

These data are from the report, Waste Isolation Pilot Plant Supplementary Roof Support System, Underground Storage Area, Room 1, Panel 1, Table 5.1 DOE WIPP 91-9230 (Aug. 1991) (Exhibit C hereto). The rate is particularly high in Room 1, Panel 1. Moreover, the data for that room fail to show the decline in rate, below three inches per year, seen in other rooms before the eventual increase. Further, the eight-year span for SPDV Room 1 covers the time span from excavation to total failure. The room was, in fact, closed to access from a time well before its failure. From the materials submitted to me, I infer that the room was closed approximately eighteen months before the

failure (Report at Fig. 5.1, August 1991). To estimate maximum useful life the time span of six years, two months is a more nearly correct comparison. Under this approach the useful life of Room 1, Panel 1 will end in approximately October 1992.

32. I have also studied the proposed roof support system described in the August 1991 report. Briefly, the proposed system employs 13 foot Dywidag rock bolts anchored with resin grout above the anhydrite "b" clay layer. These bolts are designed to protrude 18" from the roof to accommodate sagging of the roof rock. The rock bolts are placed in three inch (possibly four inch) diameter holes reamed to the anhydrite "b." The bolts are fed through a 1-1/2" hole in a steel channel 15" wide by 3-1/2" deep. The channel is 27' long, comprised of three nine-foot sections joined by fishplates. Eleven bolts secure each channel set, and the sets are placed at eight to ten foot intervals. An additional lacing system is comprised of 5/8" wire rope fastened with diagonal eight-foot resin grouted Dywidag rock bolts. The lacing is installed laterally and longitudinally on three foot centers. In addition, 3/4" x 1" expanded steel mesh and 4" x 4" mesh formed of welded 1/4" steel wire is installed next to the roof salt.

33. The proposed support system relies upon numerous assumptions that are demonstrably wrong. (See Report, at pp.3-2 through 3-3 (Aug. 1991)). For example, there is no basis to assume that the zone of rock where the bolts are anchored is "sufficiently stable to provide a good anchoring base for the

support system rock anchors." In fact, inclinometer measurements show that this zone and the area above it are already subject to deflection in the SPDV area (See Fig. 5.4), as stands to reason, since they are subject to added stress. Lateral differential displacements have been observed up to 50 feet into the roof (See pp. 4-3, A-6). Nor is it correct to assume without data that the lateral stress is no greater than the vertical stress; it may well be much greater. Again, it is incorrect to imply that "creep" ~~or elastic~~ deformations are the principal result of differential stress. In my experience by far the major deformation effect at depth is fracturing, not creep, and the DOE seems to acknowledge this (see p. A-4), but its model totally fails to account for fracture behavior (Id.). Yet again, it is incorrect to assume that once fractures occur, roof movements "are increasingly associated with gravity rather than salt creep." They are associated with both, but, more significantly, they are the result of stress-induced fracturing. Nor can it be assumed that a three-inch clearance hole will be "sufficient to prevent shearing of the tendons that may arise from differential lateral deformations that might take place in the roof rock." Deformation <u>has</u> and <u>will</u>--not might--occur, and three-inch holes to provide play is not enough, since the DOE's own inclinometers show more movement than three-inch holes would allow (see Fig. 5-4). Nor can lateral deformation be assumed to be 1/2" per year; the data show more than 2 1/2" over a four year period (Fig. 5.4), and it cannot be assumed to proceed

symmetrically when fracture takes place. The proposed system is predicated on false assumptions and cannot, therefore, ensure stability.

34. With reference to the possibility of extending as required the life of Room 1 by roofbolting methods such as the roof support system recently adopted, I have little assurance that it can be done. Mechanically anchored ten-foot roof bolts were installed in Room 1 two years after it was excavated. They have failed to arrest closure, probably because the anchorage is slipping. The bolts may also be subjected to severe lateral stress and tension at the plane of failure. The probability of bolt failure will be greatest at the point of greatest shear, i.e., the edges of the room. Bolts failure will probably occur there first.

35. In fact, most of the closure experienced in rock salt at the depth we are concerned with takes place through fracturing, rather than through "creep." This fact bears upon the nature of the roof supports which are required. The movements of the roof anticipated in the design of the proposed WIPP support system are assumed to consist of creep, but the actual movements to be expected will probably involve large fractured slabs of salt, which generate shear forces. The planned support system does not anticipate shear forces to any significant degree.

36. The proposed support system is predicated on the assumption that a large mass of roof rock will become loose from

the overlying strata and must be suspended simply as dead weight. In other words, ~~that~~ forces other than gravity alone are not anticipated.

37. In fact, as observed at the Cayuga Salt Mine and in the SPDV rooms at the WIPP, a principal component of movement before failure is in shear. The roof bolts are to be anchored in strata above the anhydrite "b" layer. Beneath the "b" layer there is to be a three-inch (possibly four inches) diameter clearance hole. This permits a lateral movement of at most two inches (and possibly as little as one inch) before shear forces are applied to the bolt. This distance is the difference between the bolt diameter (one inch) and the clearance (at most three inches, but less if the shear is applied close to the resin anchor point). The stated anticipated lateral movement is 1/2" per year, which means 2-1/2" in five years--exceeding the available clearance in a time much shorter than the time that the DOE has stated was required for the tests. Moreover, 1/2" per year understates the actual lateral movement to be expected. Shear forces are frequently asymmetrical, and we have insufficient data to draw from.

38. There is also the possibility of shear failure at the point where the roof bolts go through the channels. There is essentially no play at that point, yet the roof rock can be expected to move against the bolt at that point also.

39. The shear movement will also force the fractured roof beam downward against the channel supports, adding to the

gravitational load. In fact, the bolting will increase this force, because it renders the beam more competent and thus more likely to be thrust downward under stress. The system, however, is designed only to absorb the gravitational load.

40. The Dwyidag bolts have been tested, but only in momentary static load and under unrealistic conditions. For example, it is said that three 12" resin cartridges were used to anchor the bolt. By my calculations this much resin will create a 60" resin bond length. That is considerably longer than the approximately 36" length above the anhydrite "b" layer that can be counted upon to provide firm support. In other words, in the design, there will only be about 36" of anchorage, not 60", and the tests are not representative of actual conditions.

41. Moreover, the tests measure the holding strength of the bolts in tension, whereas they will probably be loaded in shear.

42. I anticipate that failure of the support system or such threat of failure that entry into Room 1, Panel 1 must be barred, will occur well before the end of the required time period, largely by reason of shear failure of the roof bolts.

43. The report on the proposed support system says that the support must permit up to 50 mm (two inches) per year of overall roof lowering and adds: "The overall roof lowering over a nine-year period will therefore, be 450 mm." (p.A-10) An overall lowering of 450 mm, or 18", will exceed the available bolt length, which is only 14" after the mesh, plywood, channel,

load cell, and nut are added. Moreover, it cannot be assumed that the 18" average lowering will apply uniformly; some areas will have less, and some more than 18" of lowering. Thus, the system, even if it works entirely as planned, cannot endure the nine year period that the DOE stated was required for testing and retrieval.

44. In the latest report it is said that clearance of 11' 4" is required (Report at 3-1, Aug. 1991). At present the roof height is approximately 13'. The support system reduces clearance by at least 18" by reason of the protruding roof bolts; thus, the initial clearance becomes 11'6". The available 2" of additional clearance will disappear in one year under the DOE's own "working assumptions for room closure" in the report concerning the support system. At that time the waste bins would not be retrievable, because equipment could not get access to them. Note too that the support system itself allows only 11' 6" of clearance immediately after installation, and those critical two inches are almost certain to be lost to floor heave and roof closure combined.

45. Another problem with clearance arises from the lacing system placed between the channels. This system is designed to be flexible, but because of the number of rock bolts installed and to be installed, the roof beam will probably break into pieces, which must be supported by the lacing. I know of no way to calculate the extent to which the stretched lacing, and progressive closure, would interfere with essential access. In

some Canadian mines I have observed such lacing overloaded with broken rock protruding as much as 18" into the room. It cannot be assumed that this will not occur at the WIPP.

46. There is a further difficulty with the manner in which the proposed support system impairs the warning of roof failure. The expert panel was advised by Westinghouse that six months would be required to remove the waste bins, in an emergency. I do not believe that it is possible to predict the occurrence of rock falls with precision in the circumstances anticipated. In unsupported areas there will normally be evidence of acceleration of closure some months before a roof fall. But the situation is different after a roofbolt system is installed. Roof support systems can mask the accelerated movement which serves as a warning signal. At the Cayuga Salt Mine, for example, such masking made it difficult to predict time of failure. Instead of acceleration, the closure simply maintained a steady rate until something became overloaded, and the roof fell suddenly. The proposed support system will probably have the same effect at the WIPP.

47. In my professional opinion it is not prudent to assume that the roof of Room 1, Panel 1, will escape failure or imminent failure before the end of the required test period, and it is apparent that the required headroom will be lost before the end of one year from this date. In light of the availability of more economical alternatives, such as the creation of new test rooms or the conduct of tests at another

location, I consider it imprudent to conduct the tests at the WIPP in Room 1, Panel 1, even equipped with the proposed supplementary support system.

FURTHER AFFIANT SAYETH NOT

Jack Parker 10/6/91
JACK PARKER

SUBSCRIBED AND SWORN TO before me on this 6 day of October, 1991.

NOTARY PUBLIC
Michael White

MY COMMISSION EXPIRES:

3-28-94

VIII. Parker A

PHONE 906-885-5445
OR 906-885-5384

BOX 465

JACK PARKER AND ASSOCIATES, INC.
ROCK MECHANICS . MINING . GEOLOGY
WHITE PINE, MICHIGAN 49971

JACK PARKER

RESUME

1. Born 1930, in England. Naturalized in U.S.A. 1974.

2. EXPERIENCE:

a) Since 1971, Self-Employed. A consultant in rock mechanics, mining and geology. The work requires a practical understanding of the properties and behaviour of rocks in place - and what to do about them.

Some are one-time, trouble-shooting projects; others are on-going, with regular inspections and reports.

Most jobs begin with a rock stability problem, then a telephone call, a visit, and a remedy, followed by further cooperation to ensure that other rock-related problems do not get out of hand.

The work involves design of underground mining methods and layouts, including consideration of roof, floor, pillars and supports. Much of the information needed is gained from observations, and by projection from similar circumstances in other mines. Simple, low-cost instrumentation is used at some mines for further evaluation. The emphasis is on practical approaches to problems and practical solutions.

Expert witness in several law cases.

Assisted in teaching rock mechanics seminars at Michigan Tech, U of Nevada at Reno, U of Missouri at Rolla, U. of Missouri at Kansas City, and at Cambrian College, Sudbury, Ontario.

At the request of industry many seminars stressing the practical aspects of rock mechanics have been presented at White Pine, Michigan; Beckley, W. Va.; Kansas City, and at several mining properties.

Initially taught rock mechanics and rock fragmentation part-time at Michigan Tech.

The attached list of clients, past and present, illustrates the variety of mines, minerals, locations and companies served.

b) 1961 - 1971, 10 years at the White Pine Copper Mine, White Pine, Michigan.

First year as a mine foreman, on underground development.

Two years as research engineer, on drilling, blasting, bolting, etc.

Seven years as Director of Rock Mechanics, with 3 engineers and 10 technicians. Dealing with mining methods, mining layouts and mine stability.

Some part-time teaching at Michigan Tech.

c) 1961, One year with R. L. Loofbourow, Mining Consultant, in Minneapolis.

d) 1954 - 1961, At Michigan Tech. Four years on BS degrees, followed by 3 years on MS plus teaching in Geology Dept. Worked during vacations at mines in Ontario and Quebec.

e) 1953 - 1954. Migrated to Canada. Worked as engineer/surveyor on mine development projects on Hudson's Bay, at Sudbury, Bancroft, Haileybury and North Bay, Ontario.

f) 1946 - 1953, Worked in coal mines in England. First as office boy, then 6 years in surveying/engineering/planning at a group of four mines, under the National Coal Board.

3. EDUCATION:

High School in England

Night school and part-time day school in England, under National Coal Board education scheme.

1954, to Michigan Tech.

1958, BS Geological Engineering
BS Mining Engineering
1960, MS Geology

Gradepoint average 3.75/4.00
Position in class 4/384

4. PUBLICATIONS AND AWARDS:

4.1: Parker and Scott, Instrumentation of Room-and-Pillar Workings, Proceedings 6th Symposium on Rock Mechanics, Rolla, Missouri - 1964.

4.2: Cummings and Parker, The White Pine Hydraulic Cell, Proceedings 6th Symposium on Rock Mechanics, Rolla, Missouri, 1964.

4.3: Parker, The White Pine Sagmeter, E/MJ - May 1965.

4.4: Scott, Michels and Parker, The Role of Rock Mechanics in Mine Opening Stability, National Council Newsletter - March 1966.

4.5: Garfield and Parker, Rock Mechanics, An Operations Tool at White Pine, Min Cong Journ - June 1966.

4.6: Parker, How Moisture Affects Mine Openings, E/MJ - November 1966.

4.7: Parker, Mining in a Lateral Stress Field at White Pine, Can. I.M.M. Trans. v LXIX 1966.

4.8: Barrientos and Parker, Use of the Pressure Arch in Mine Design AIME preprint 69-AM-369, September 1969, SME Transactions, September 1974. S.M.E. Robert Peele award for "significant achievement in authorship in the fields of Mining & Exploration, 1975".

4.9: Agapito and Parker, Development of a Better Rockbolt Assembly at White Pine, AIME Annual Meeting, Denver - February 1970.

4.10: Parker, Temperatures and Humidity Affect Strength of Rock Structures at White Pine, SME Transactions - June 1970.

4.11: Caverson and Parker, Roofbolts Hold Best With Resin, Mining Eng. May 1971

4.12: Parker, Practical Rock Mechanics for Miners. A series of seven articles in E/MJ, beginning June 1973.

4.13: The First Copper Miners in Michigan, Compressed Air magazine, January 1975.

4.14: Petersen, Plumeau and Parker, Yielding Pillars and Pressure Arches at Cayuga Rock Salt Mine, SME Preprint - Keystone, June 1977, also EM/J May 1979.

AIME Award for Outstanding Contributions in Rock Mechanics, in yield pillar and arch design. 1981.

4.15: Parker, Pillar Design - Problems or Opportunities? 1st Int. Conf. on Stability in Underground Mining, Vancouver. August 1982.

A partial list of clients: EVAPORITES HOTLINED

1. Allied Chemical, Wyoming (Trona)
2. Amax, Mich, N Mex, Wyo, (Copper, Potash, Trona)
3. American Electric Power, Utah (Coal)
4. Anaconda, Montana (Copper)
5. Black River Mining, Kentucky (Limestone)
6. Boatmens Bank, KC (Limestone)
7. Boone Quarries, Columbia, MO (Limestone)
8. Canadian Rock Salt, Ontario and Nova Scotia
9. Cargill Salt, Louisiana and New York
10. Celtite, Ohio (Roofbolting Resin)
11. Cliffs Engineering, Colorado (Nahcolite)
12. Commercial Distribution, Kansas City (Underground Space)
13. Continental Minerals, Nevada (Talc)
14. Dickenson Red Lake, Ontario (Gold)
15. Domtar Minerals, Canada, Michigan (Gypsum) Louisiana(Rock Salt)
16. Dravo, Kentucky (Limestone)
17. Earth Sciences, Inc., New Mex (Uranium)
18. East Malartic, Quebec (Gold)
19. Environment One, Maine (Mine Fire Detection)
20. Exxon, Wyoming (Uranium)
21. Fairmount Development, KC (Underground Development)
22. Garney Companies - KC Quarries, KC
23. Geneva-Pacific, Alaska (Copper)
24. Grasis Corp, KC (Underground Space)
25. Great Midwest, KC (Underground Space)
26. Inland Steel Coal, Illinois
27. Inland Storage Distribution, KC (Underground Space) now Americold
28. International Salt, Ohio, New York (Rocksalt)
29. Kemmerer Coal, Wyoming
30. Kerr McGee, Illinois, New Mex (Coal, Potash)
31. Kimberley Clark, Mich. (mineral leases)
32. Lone Star Cement, W.Va (limestone)
33. Louisiana Land, Mich (Copper)
34. Louisville Crushed Stone, KY (Underground Space & Limestone Mining)
35. Macassa Mines, Ontario (Gold)
36. Maple Meadow, W.Va. (Coal)
37. Marley Engineering, KC (Underground Space)
38. Missouri Limestone Co., MO
39. Monterey Coal, W. Va. and Illinois
40. Morton Salt, Louisiana, Ohio (Rocksalt)
41. National Park Services, Utah (Tunnel Stability)
42. Noranda Mines Ltd., Quebec (Gold)
43. Ozark Lead, Missouri (Lead, Zinc)
44. Parsons-Jurden, Idaho (Vanadium)
45. Pfizer, Inc., Nevada, California (Talc, Limestone)
46. Reocin Mines, Spain (Zinc, Lead)
47. Riverside Cement, California (Limestone)
48. Soiltest, Inc., Illinois (Instrumentation)
49. S.E. Public Services, KC, KS (Underground Space)
50. Southern Utah Fuel (Coal)
51. Stoneco, Indiana, Ohio (Limestone)
52. Texasgulf, Wyoming, Nebraska (Trona, Limestone)

53. 3M, Minnesota (Roof Support Systems)
54. Timna Mines, Israel (Copper)
55. Underground Developers Associates, KC
56. Unisil, Minn and Wis (Silica)
57. US Bureau of Mines - numerous projects
58. US Steel, W. VA (Coal)
59. Unocal, Colorado (Underground Oil Shale)
60. UV Industries, N. Mex (Copper, Zinc)
61. Westinghouse, Colorado (Instrumentation)
62. Westroc, Ontario, Manitoba (Gypsum)
63. White Pine Copper Co, Mich (Copper)
64. Wilroy Mines, Ontario (Copper, Zinc, Lead) Quebec (Gold)
65. Winchester Center, KC (Underground Space)
66. Yava Mines, Nova Scotia (Lead)
67. Calmat Corp CA (Aggregates)
68. Bardon-Trimount, Mass (Crushed rock)
69. Cyprus Mines, CO, WY, PA (Coal)
70. Rogers Group, TN (Limestone)
71. American Mine Services, WI (Copper-gold)
72. Holland Quarries, KC, MO - (Limestone)
73. ASARCO, TN (Zinc)
74. Griesemer Quarries - Springfield, MO (Limestone)
75. WIPP (Waste Isolation Pilot Project) New Mexico

A major project with the U.S. Bureau of Mines involved the practical aspects of roof control, primarily resin bolting. It has included visits to about 40 coal mines in the United States, France and England, to study roof support problems and solutions. The outcome was a practical handbook on the design of resin bolting systems.

DOE/WIPP 91-023

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

June 1991

Waste Isolation Pilot Plant

6918:0438

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

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 sealing 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

OBSERVERS

Dr. C. B. Cox, WID
Dr. J. A. Mewhinney, DOE
Mr. S. C. Sethi, WID
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
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
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Mr. A. T. Iannacchione

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Ms. R. Molgaard, WID
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Ms. J. L. Francke, WID
Mr. S. C. Sethi, WID
Dr. L. Chaturvedi, WID
Dr. J. A. Mewhinney, DOE
Mr. E. K. Hunter, DOE
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

OBSERVERS

Mr. D. Galbraith, WID
Mr. H. D. Ripley, WID
Mr. C. T. Francke, WID
Mr. T. F. Brockman, WID
Mr. S. C. Sethi, WID
Mr. J. A. Gonzalez, WID
Mr. E. K. Hunter, DOE
Dr. K. M. Chua, UNM
Mr. R. C. Carrasco, WID
Ms. J. L. Francke, WID
Mr. M. G. W. Phillips, DOE, HQ
Dr. L. Chaturvedi, EEG
Mr. H. L. Bibby, WID
Dr. J. A. Mewhinney, DOE
Mr. R. Batra, DOE
Mr. B. R. Pleau, WID

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) <u>SPDV TEST ROOM FAILURE</u>

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) RESPONSE TO STATEMENTS

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 present 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.

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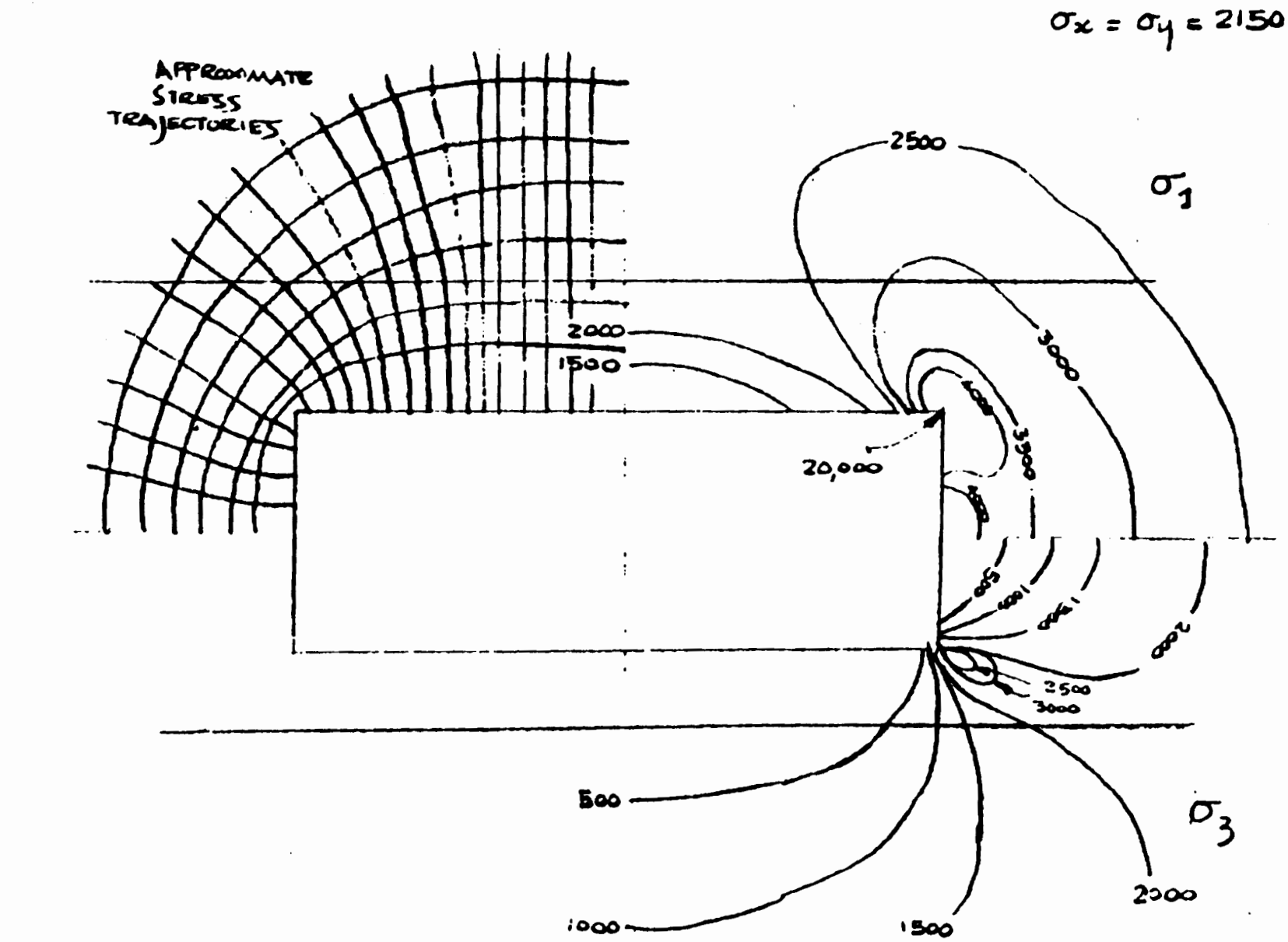


Fig 1. APPROXIMATE CONTOURS OF MAJOR & MINOR PRINCIPAL STRESS (psi) AROUND A 33 ft × 13 ft

0 1 2 3 4 5 ft.

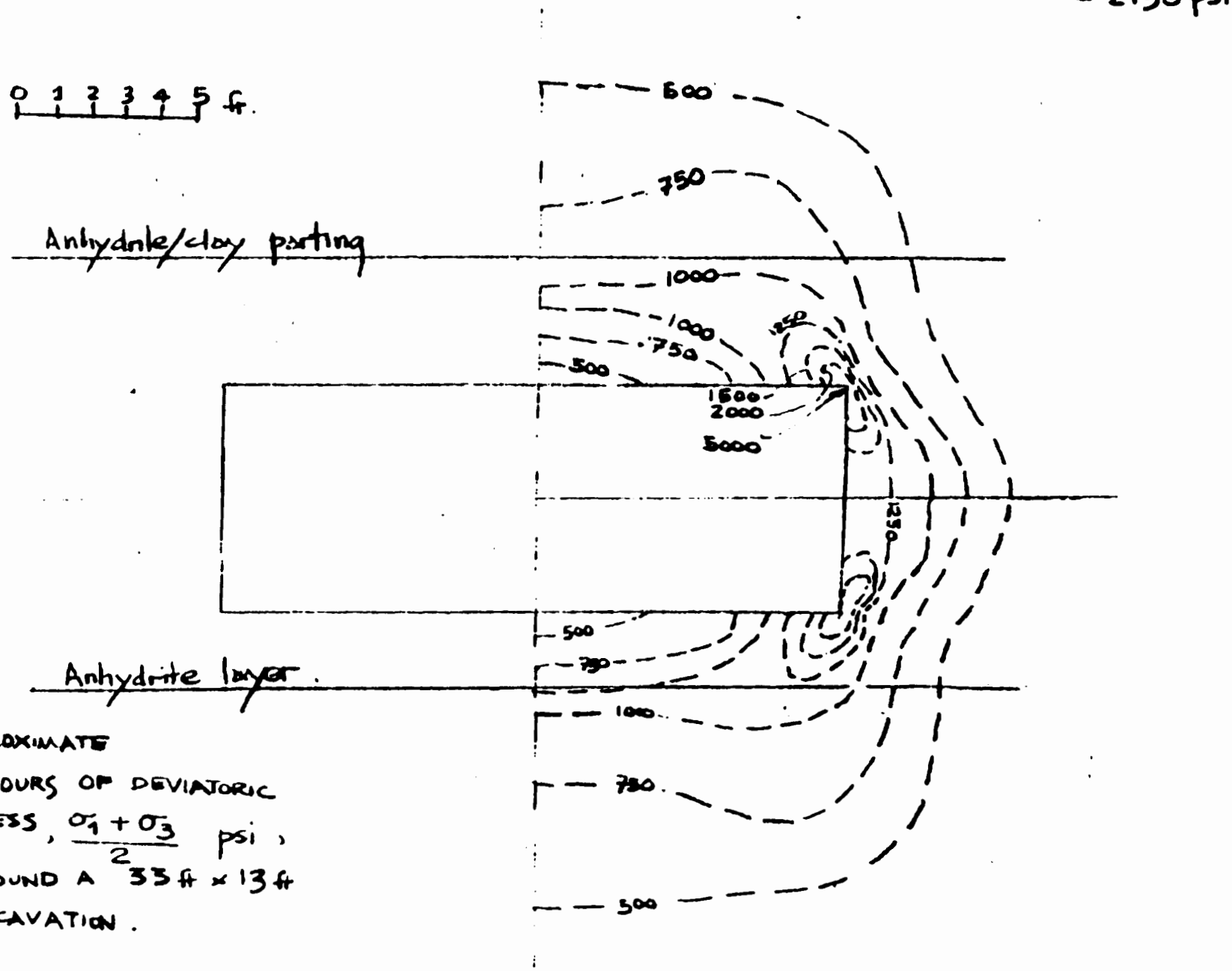


Fig 2. APPROXIMATE CONTOURS OF DEVIATORIC STRESS, $\frac{\sigma_1 + \sigma_3}{2}$ psi, AROUND A 33 ft × 13 ft EXCAVATION.

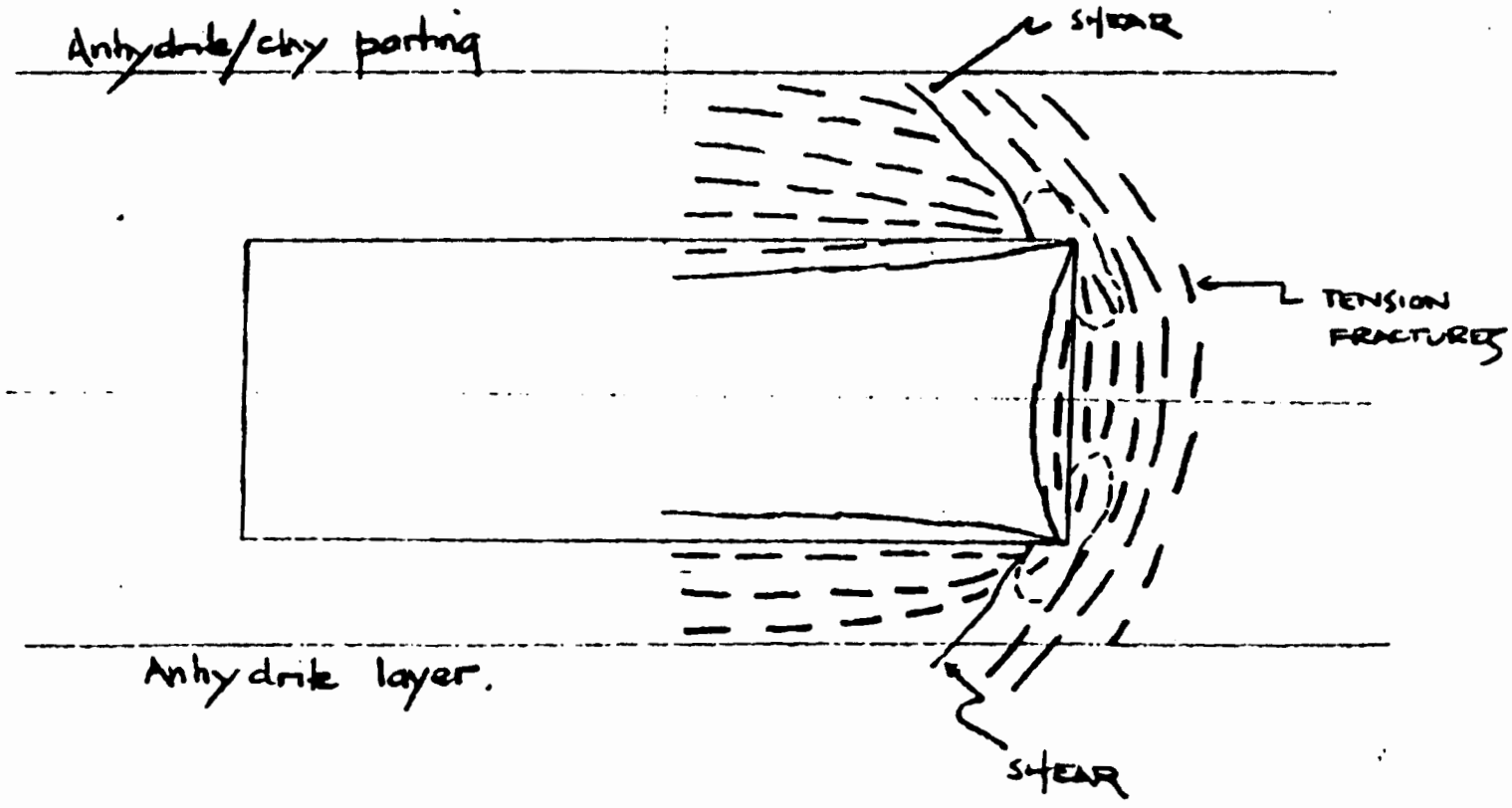


Figure 3. Proposed FRACTURE PATTERN AROUND A 33ft × 13 ft EXCAVATIO

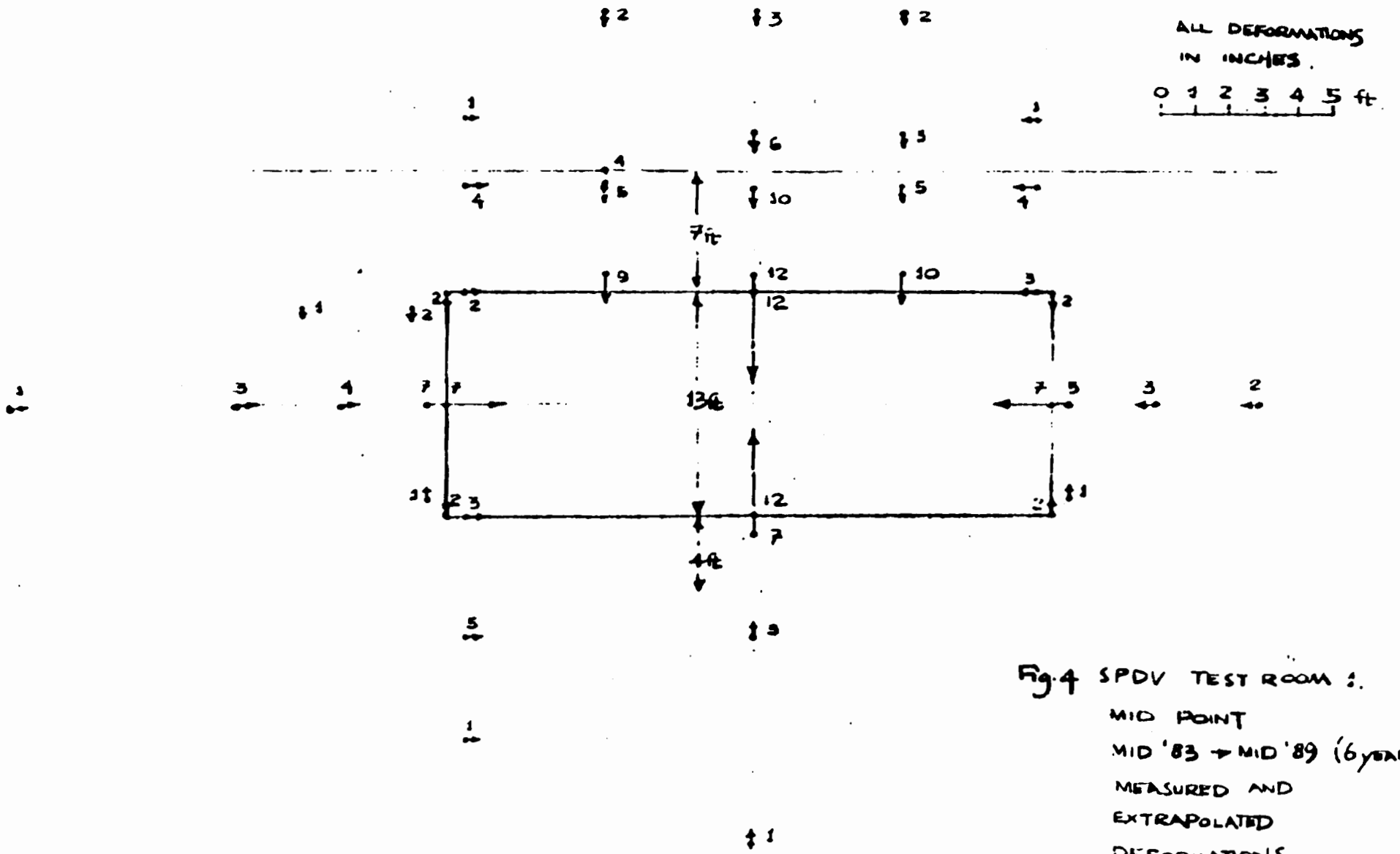


Fig.4 SPDV TEST ROOM :.
MID POINT
MID '83 → MID '89 (6 YEARS)
MEASURED AND
EXTRAPOLATED
DEFORMATIONS
LOOKING NORTH

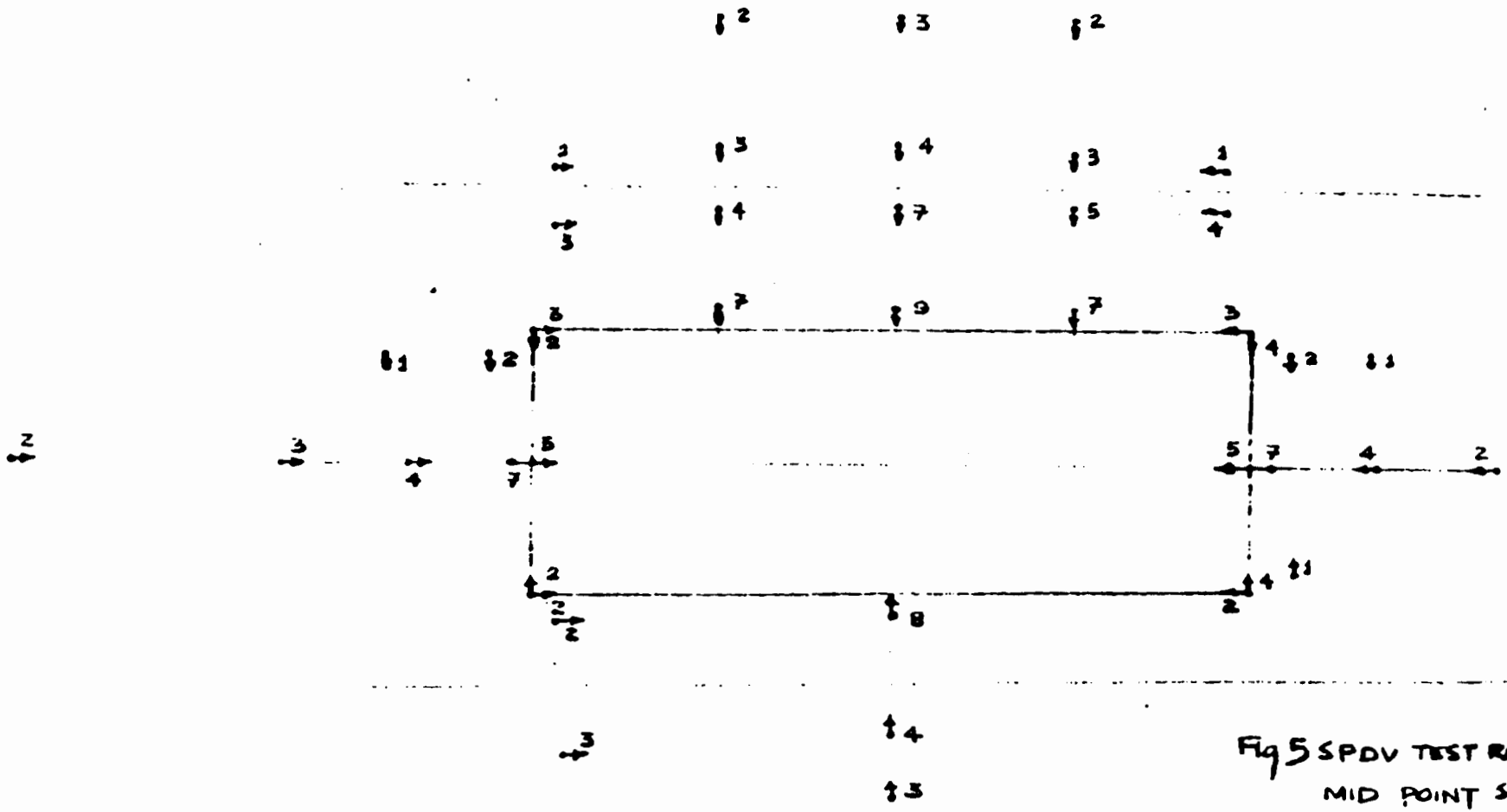


Fig 5 SPDV TEST ROOM 2
MID POINT SECTION
MID '83 → MID 89 - 6y5
MEASURED DEFORMATIONS
IN INCHES LOOKING N.

0 1 2 3 4 5 ft.

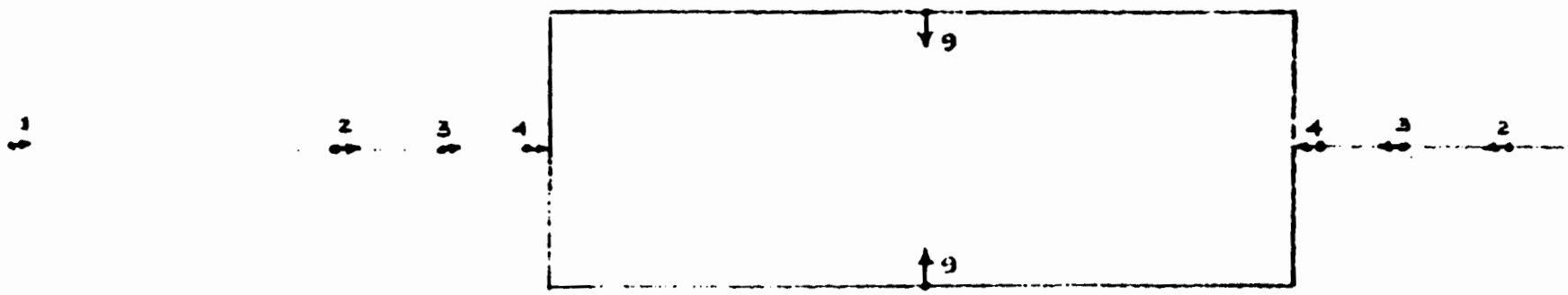
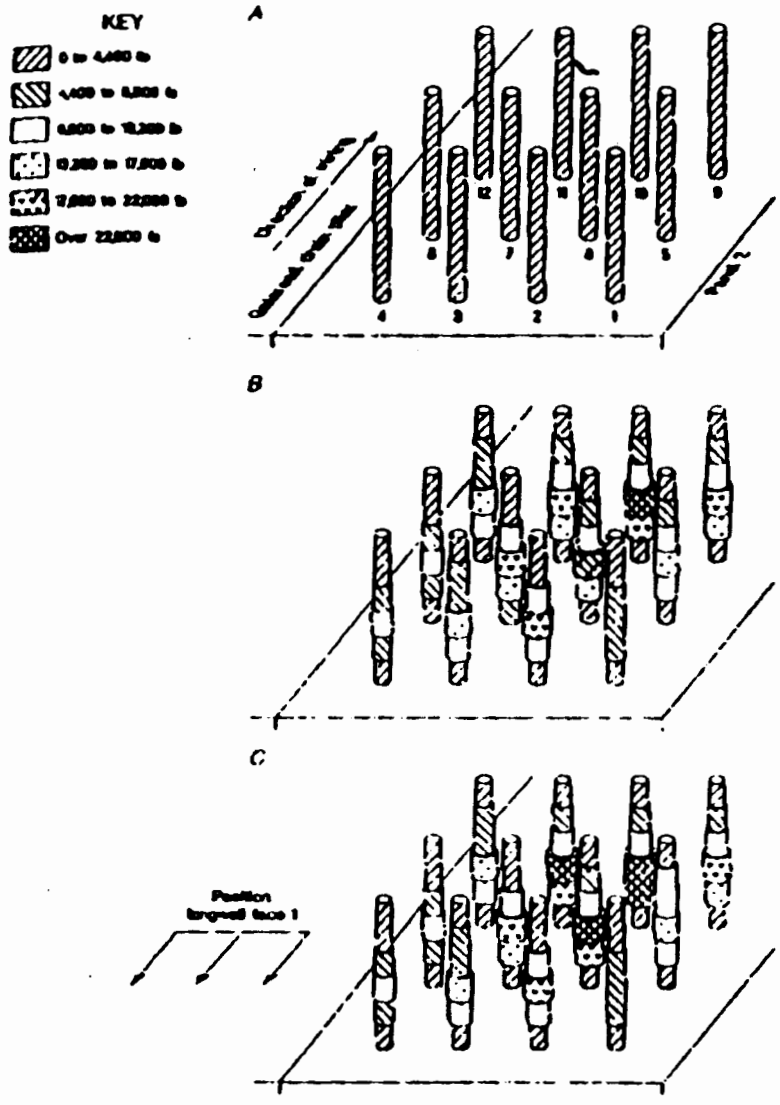


Fig. 6. WASTE PANEL 1 ROOM 1
MD POINT
JAN '86 – JAN 91 (5 YEARS)
MEASURED AND
EXTRAPOLATED DEFORMATIONS
LOOKING N.



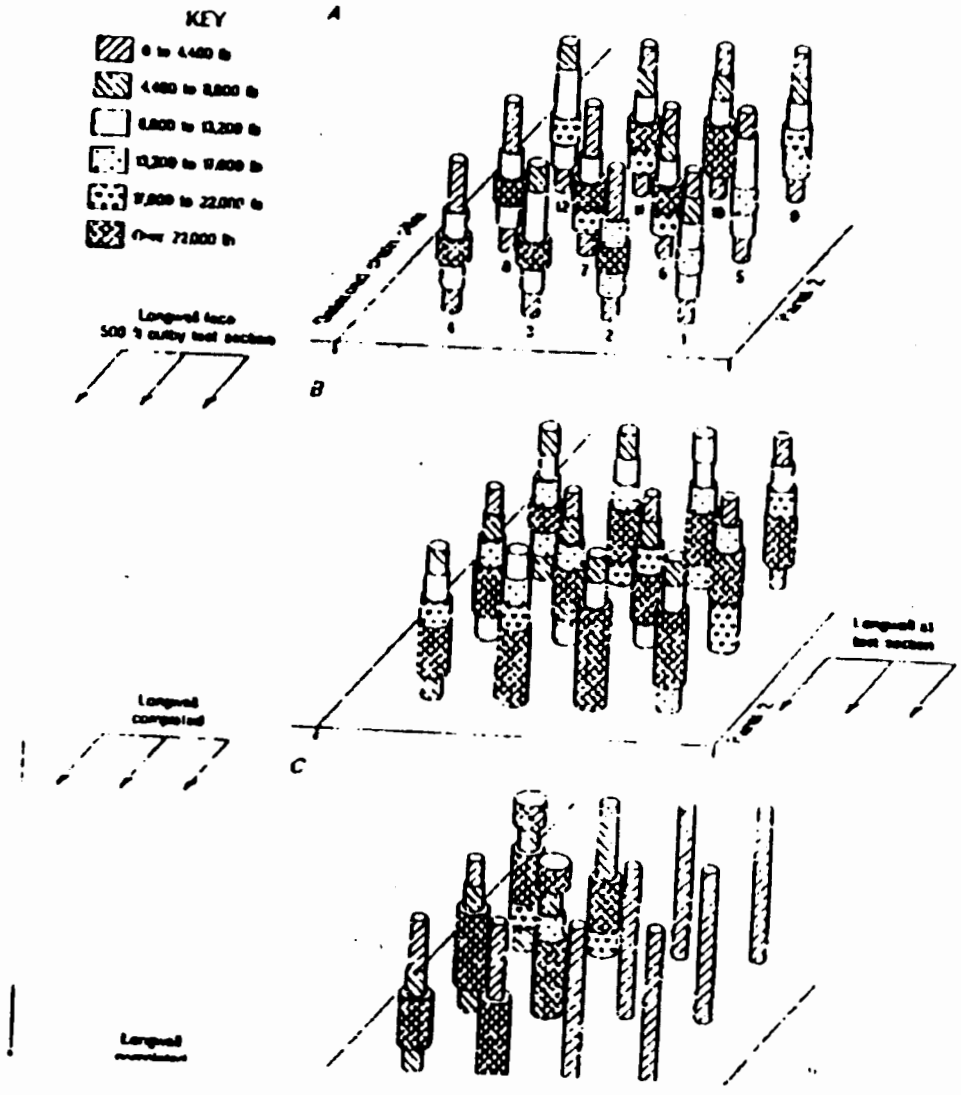


Figure 7 Circumferential load transfer and yield of resin grouted anchors (after Fuller and Jones, 1980)

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
11/19/91

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.

The following cases are 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.

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 I 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.

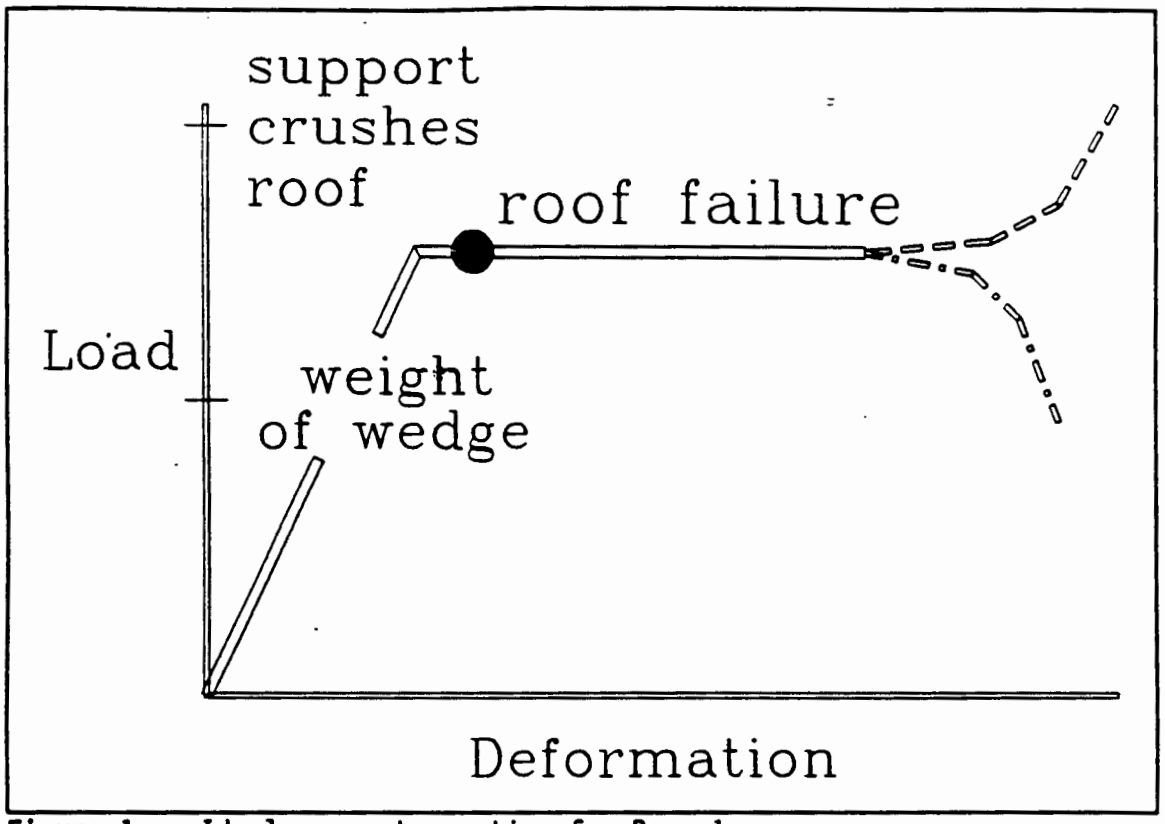


Figure 1. - Ideal support reaction for Room 1.

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

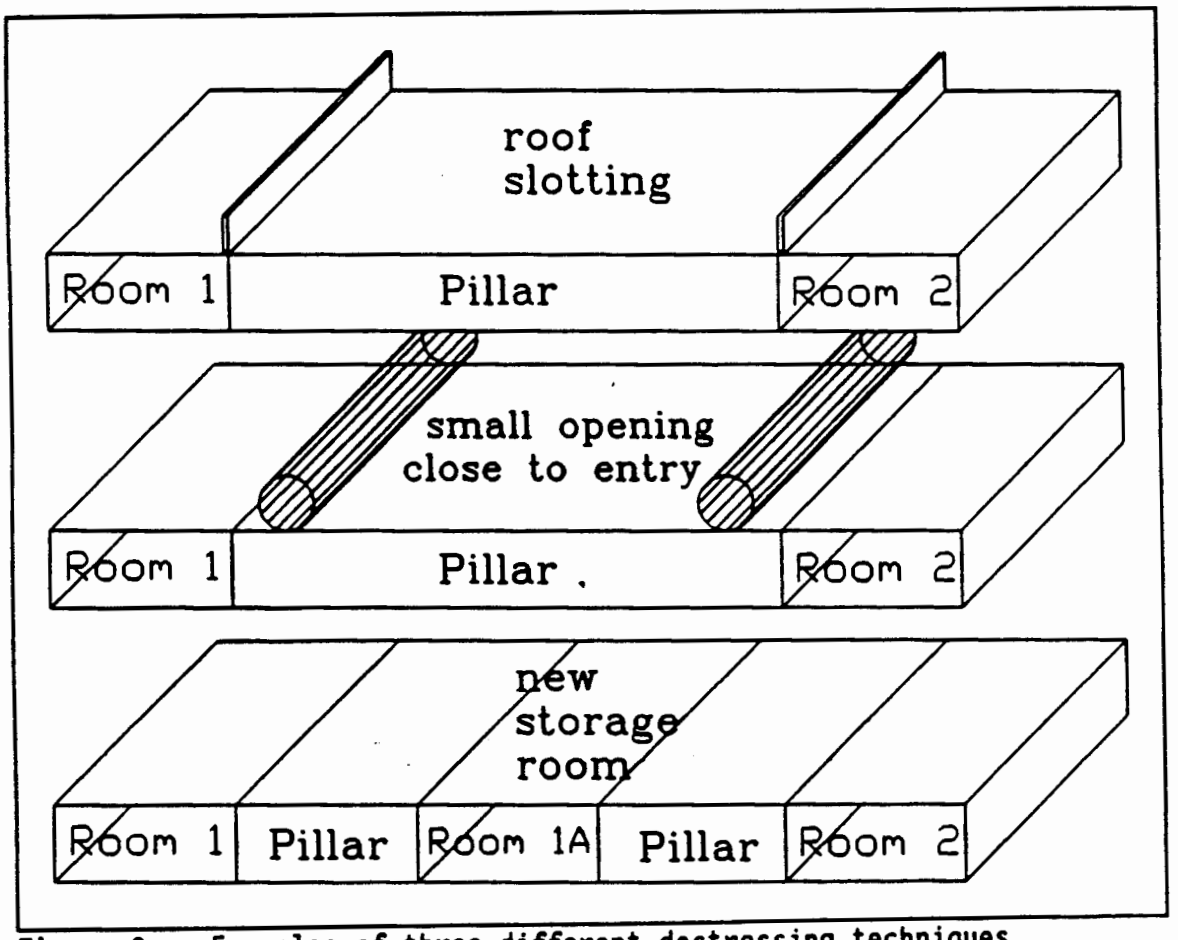


Figure 2. - Examples of three different destressing techniques.

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

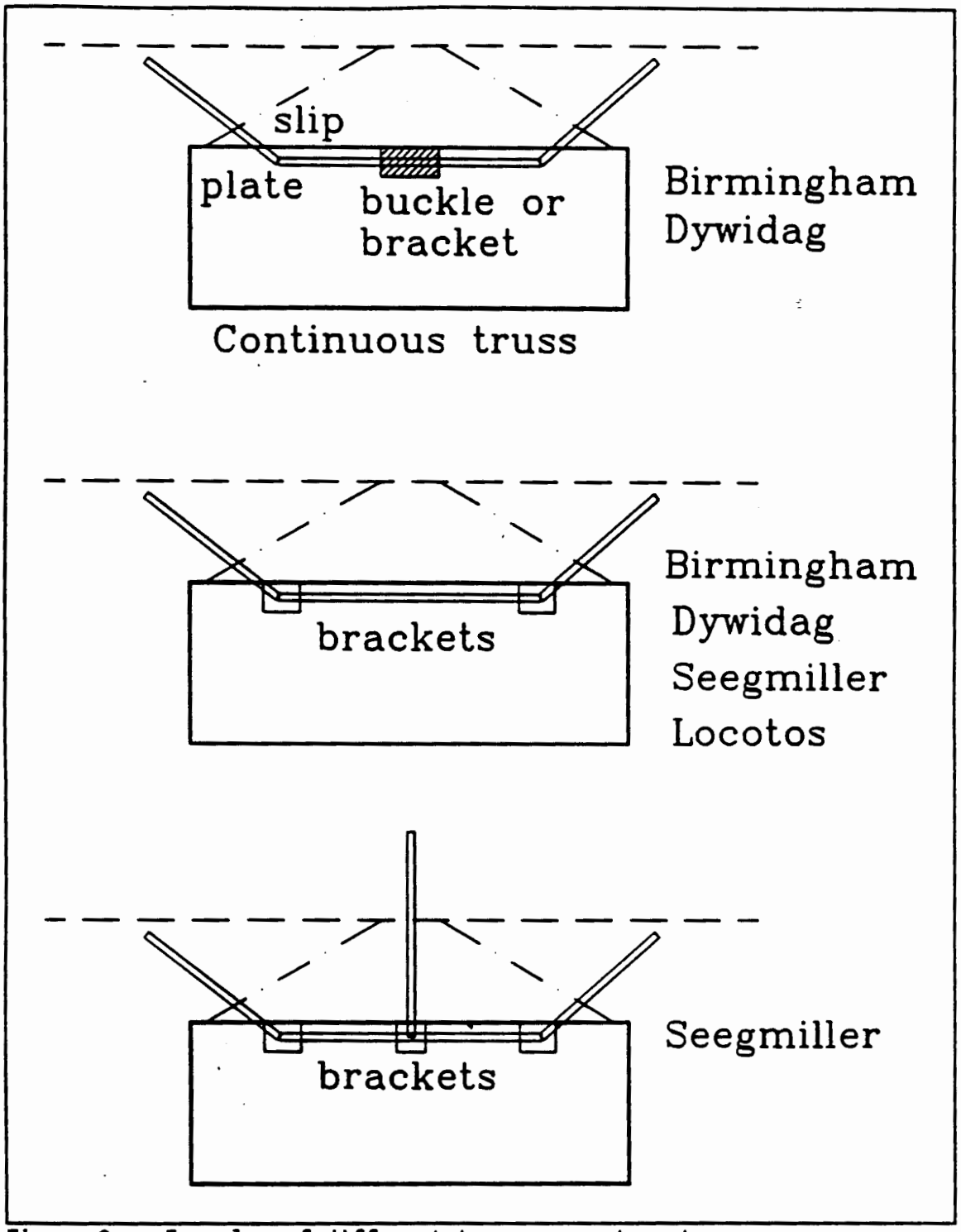


Figure 3. - Examples of different truss support systems.

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

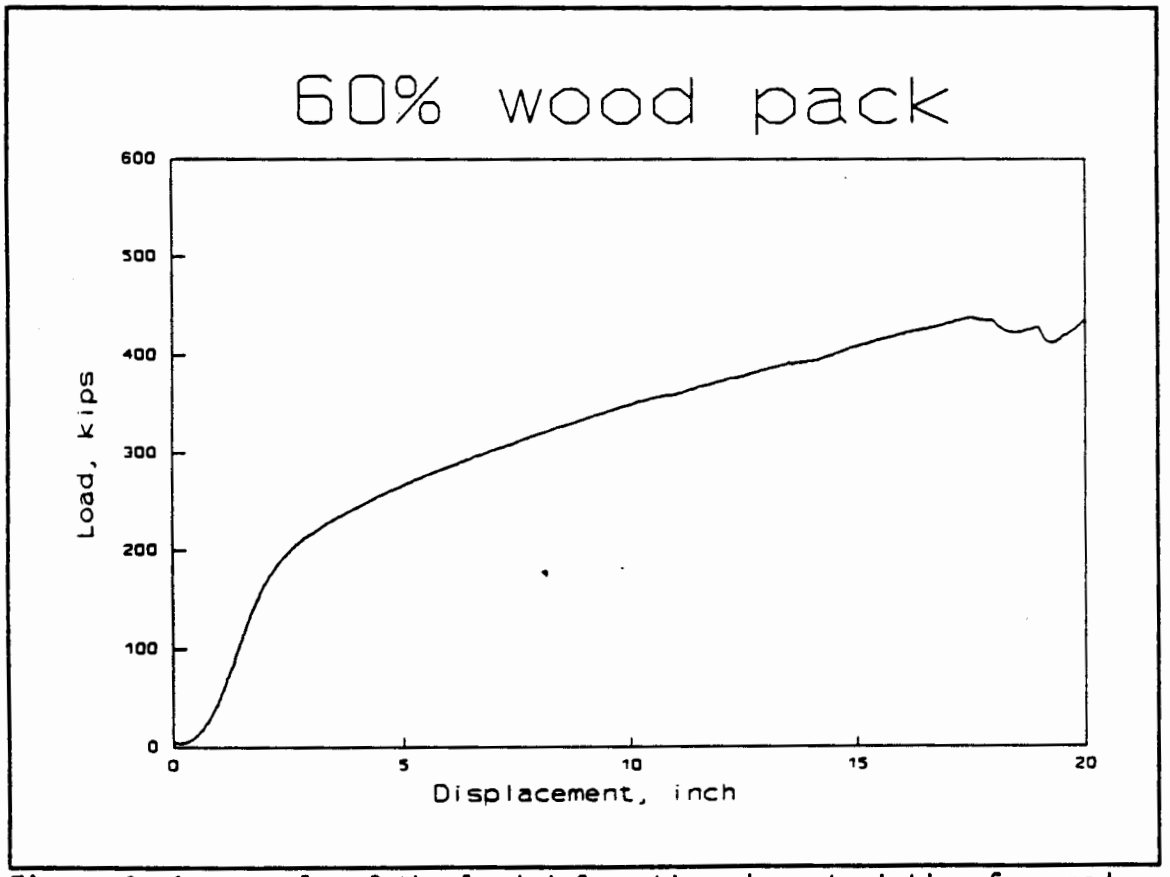


Figure 4. An example of the load-deformation characteristic of a wood crib.

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.

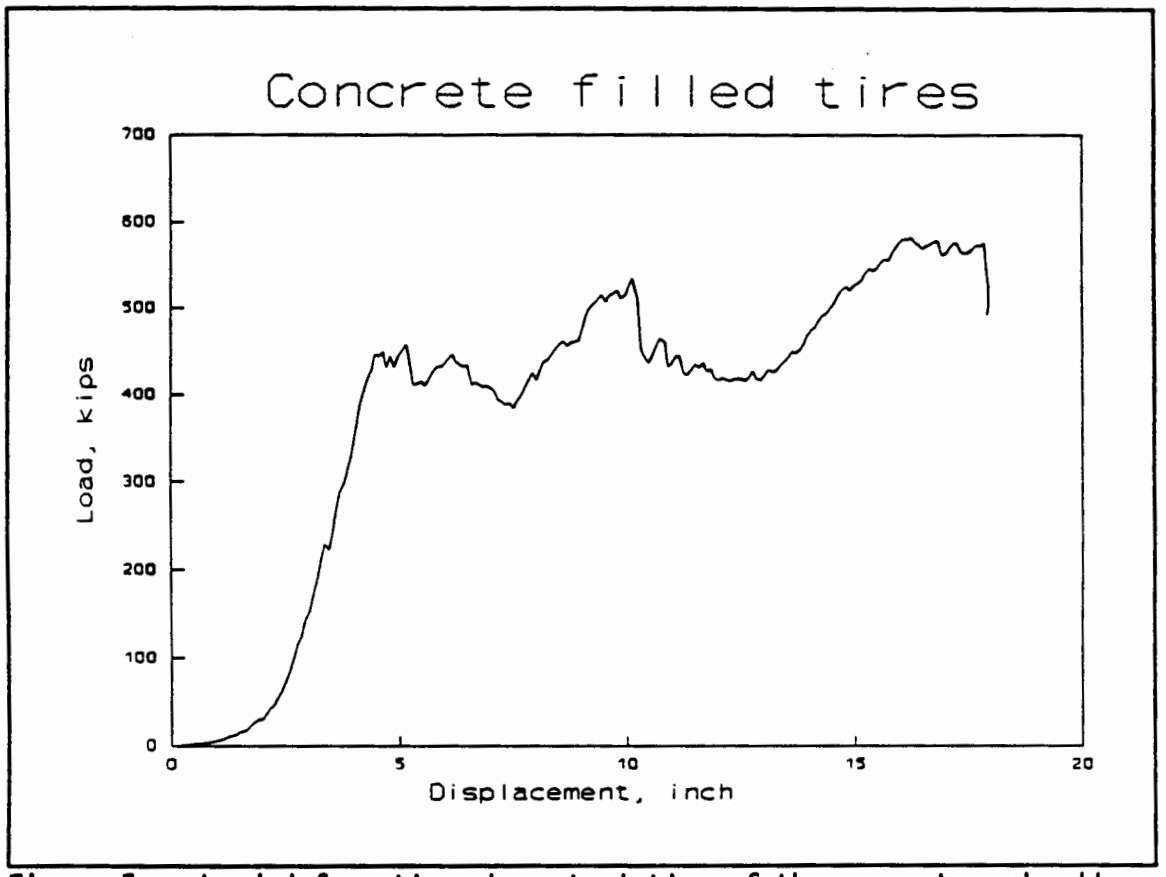


Figure 5. - Load deformation characteristics of the concrete and rubber pier.

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

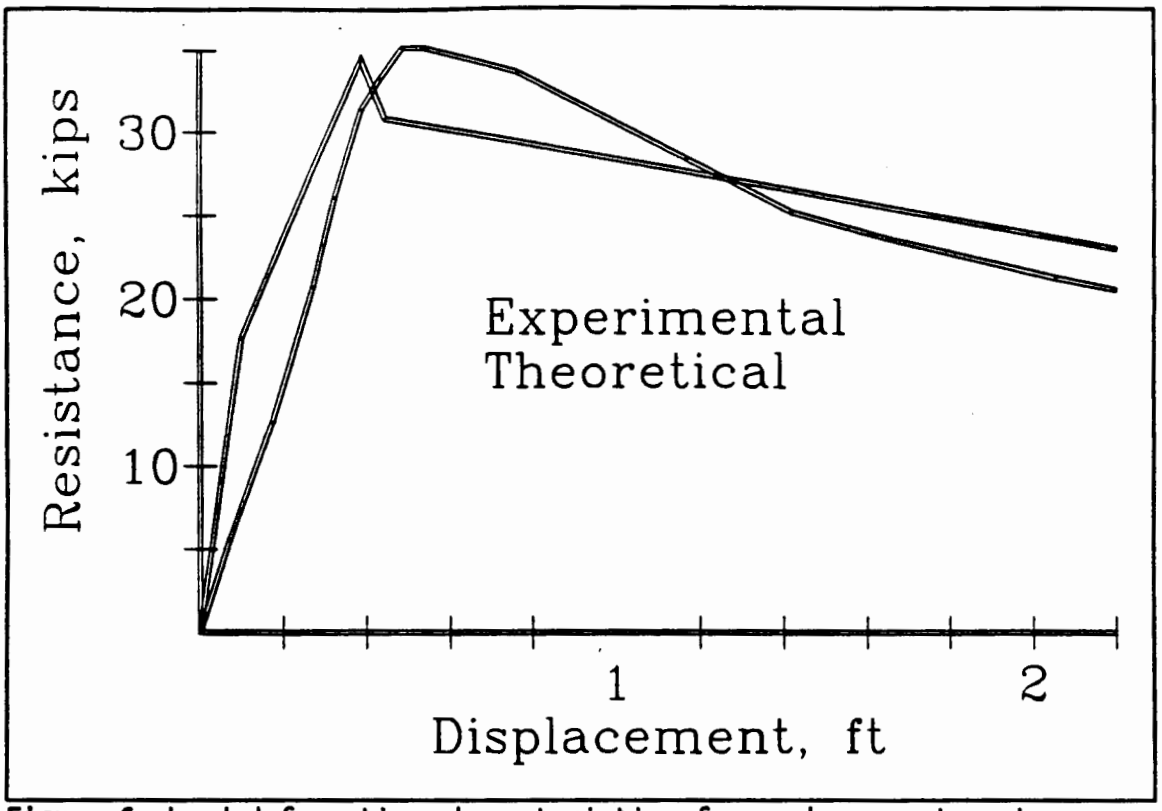


Figure 6. Load deformation characteristics for arch support system.

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 .

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

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

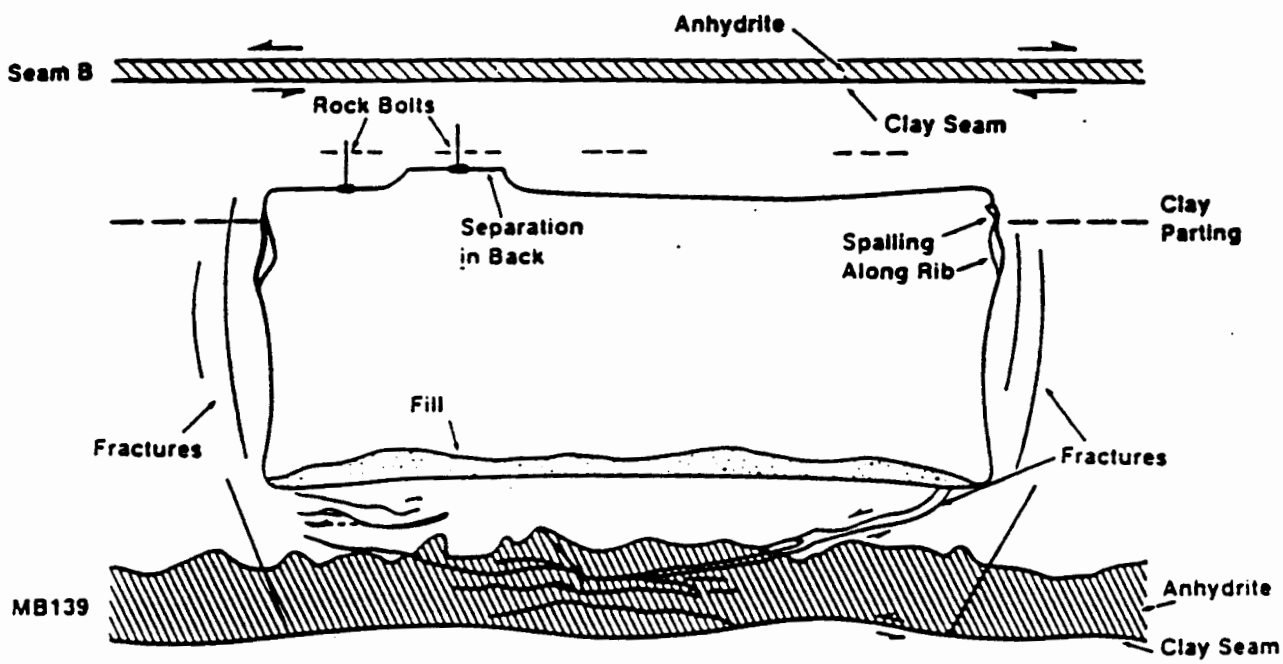
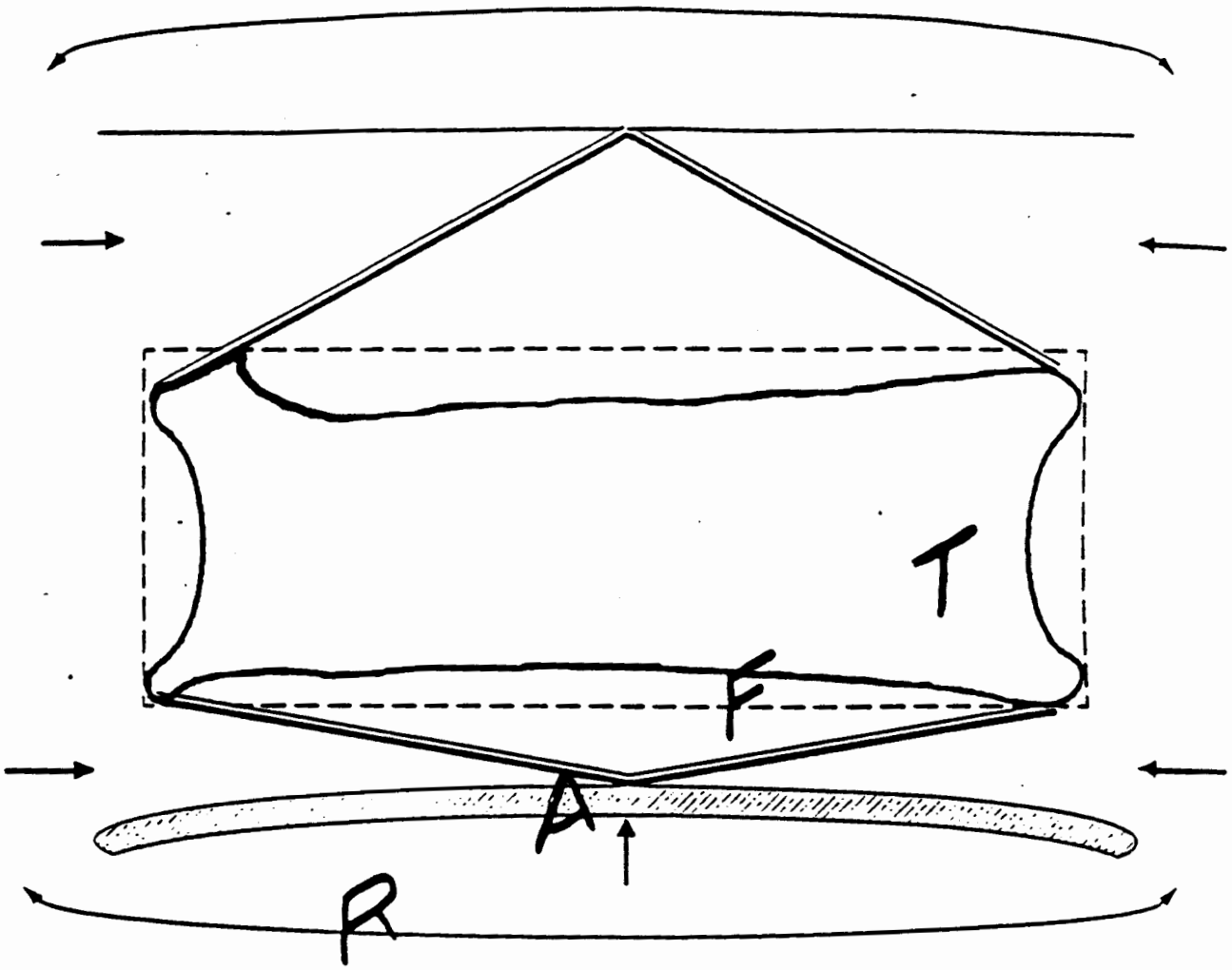


Fig. 1 Observed Discontinuous Behavior Around Test Rooms Up to 5 Years After Excavation (Stormont, 1990)



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)

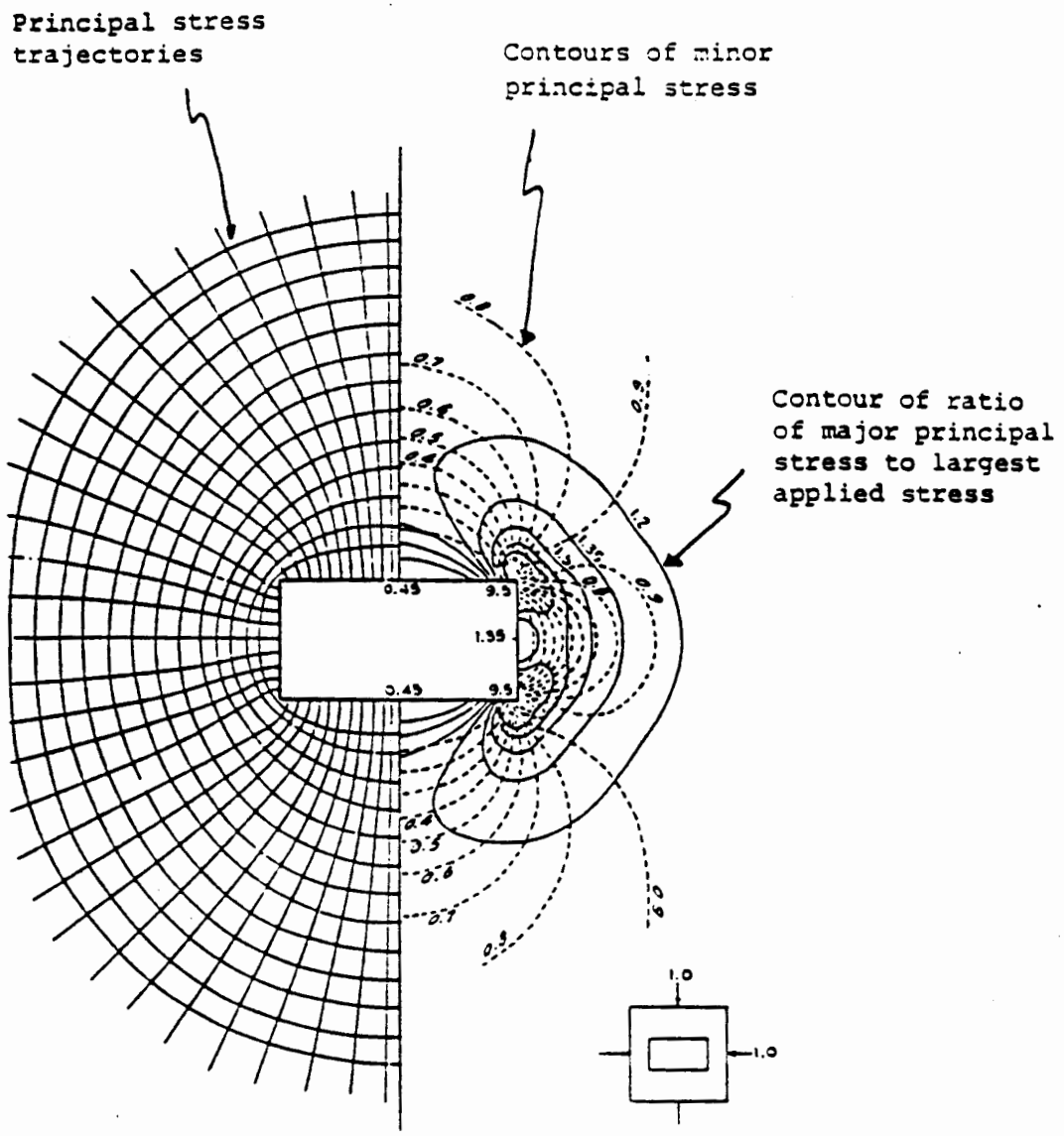


Fig. 3 Stress Distribution Around an Excavation With Width to Height Ratio of 2:1 in a Hydrostatic Stress Field (Hoek and Brown, 1980)

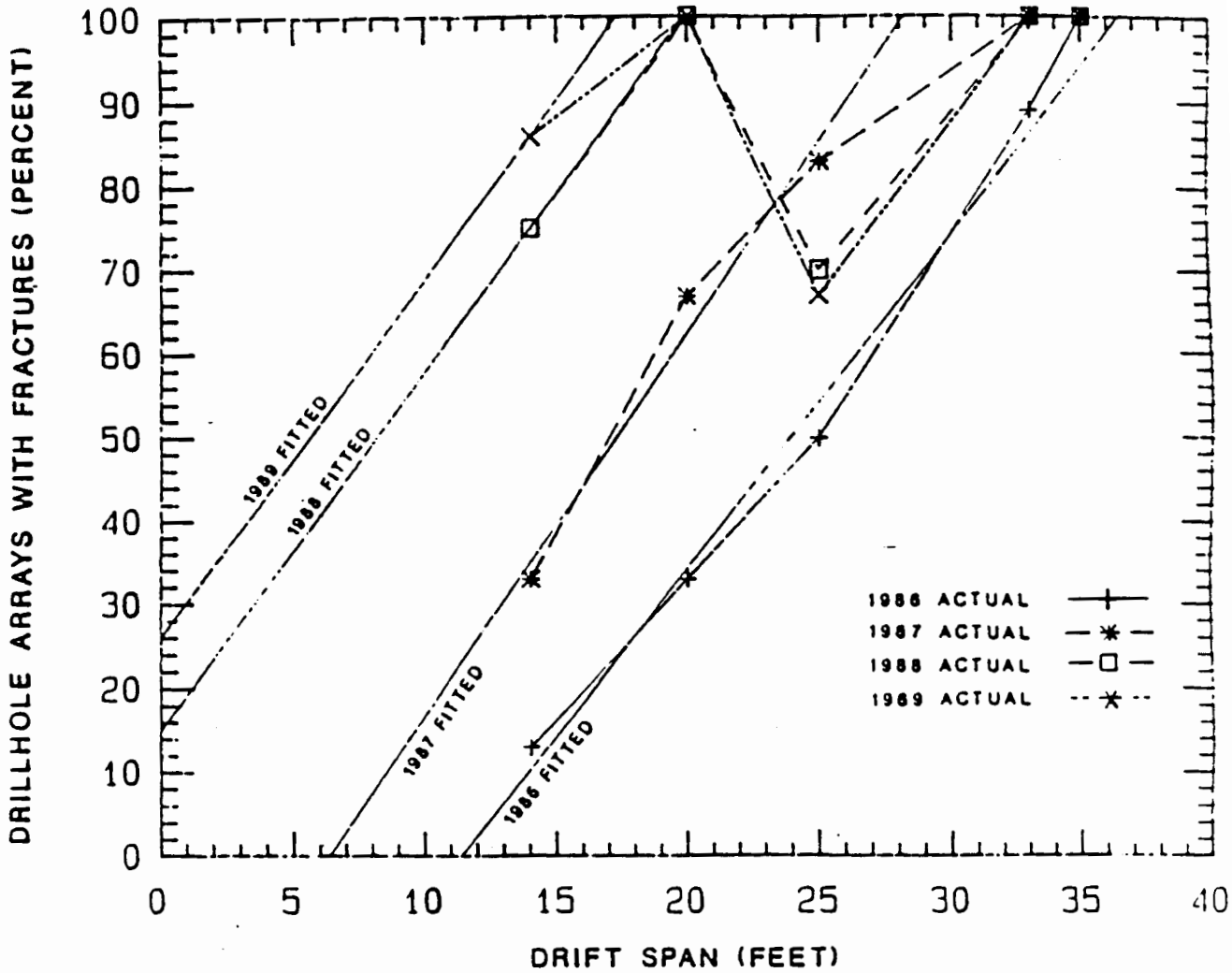


Fig. 4 Correlation of Drift Span to Degree of Fracture Development (unreferenced WIPP report)

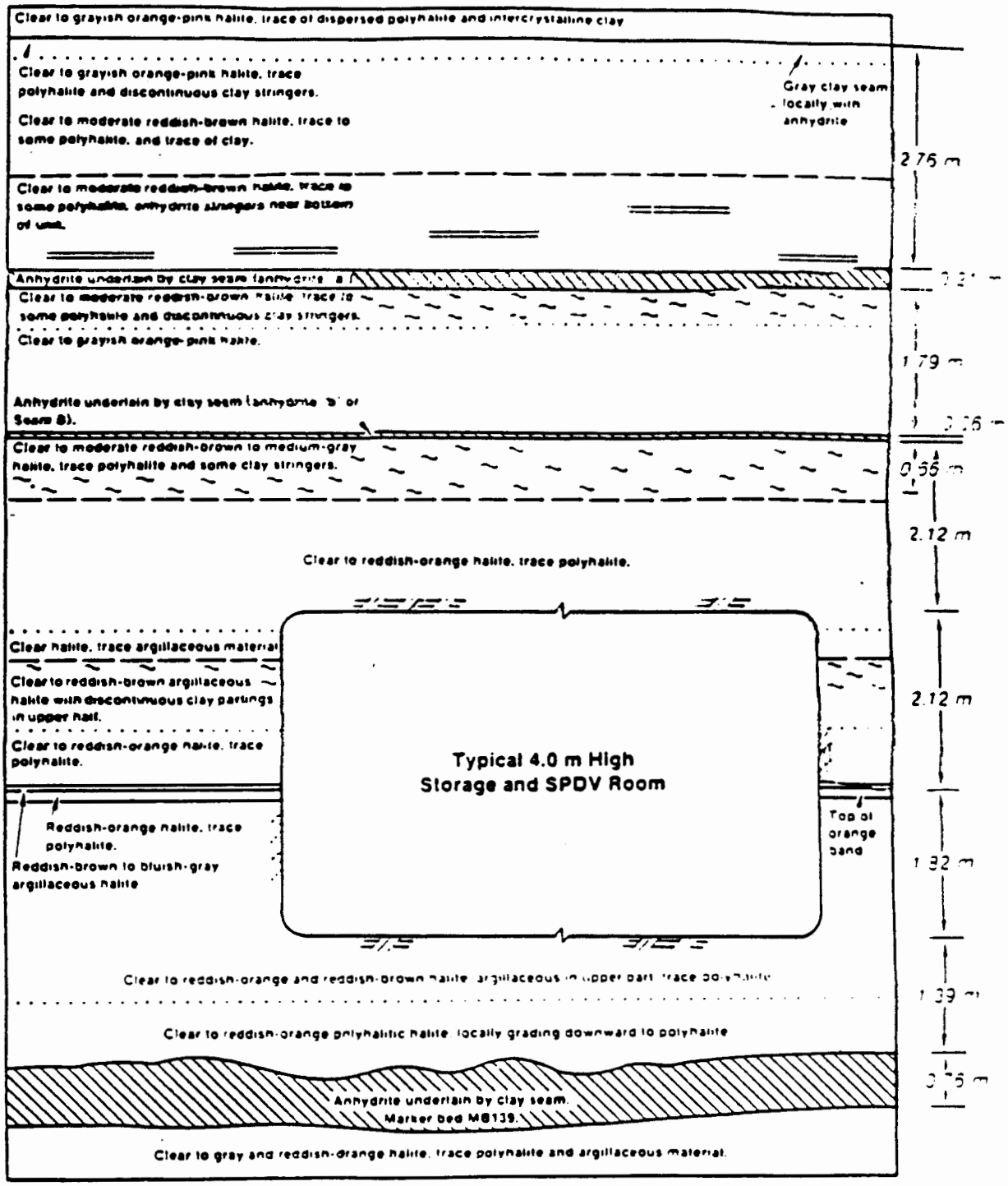


Fig. 5 Stratigraphy in the Vicinity of WIPP Excavations

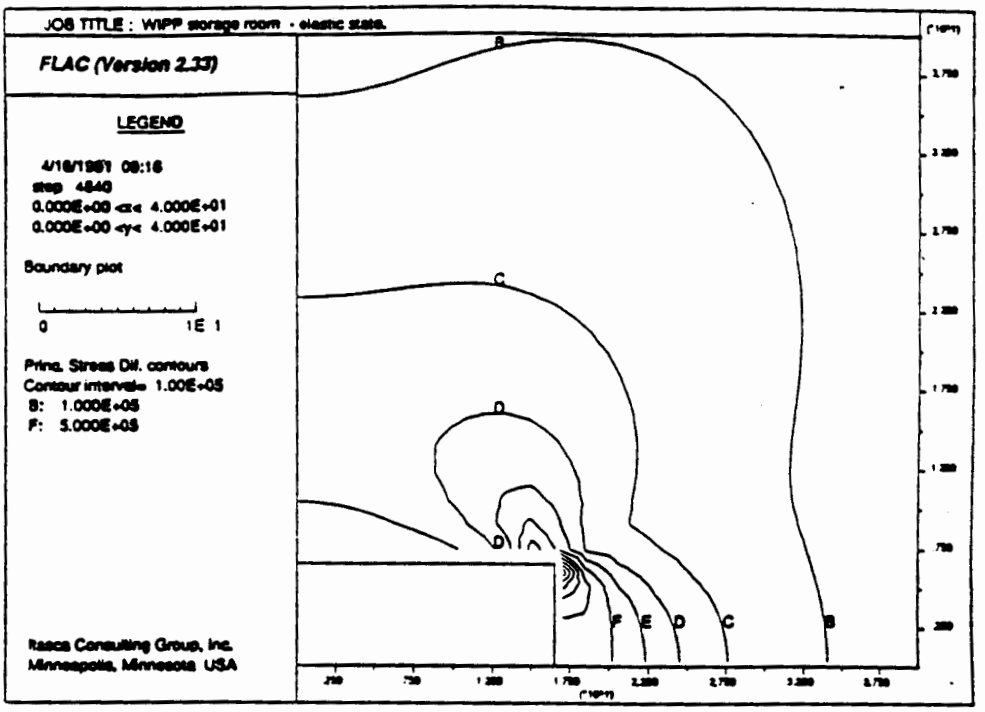


Fig. 6 Principal Stress Difference Contours, WIPP Storage Room — Elastic State

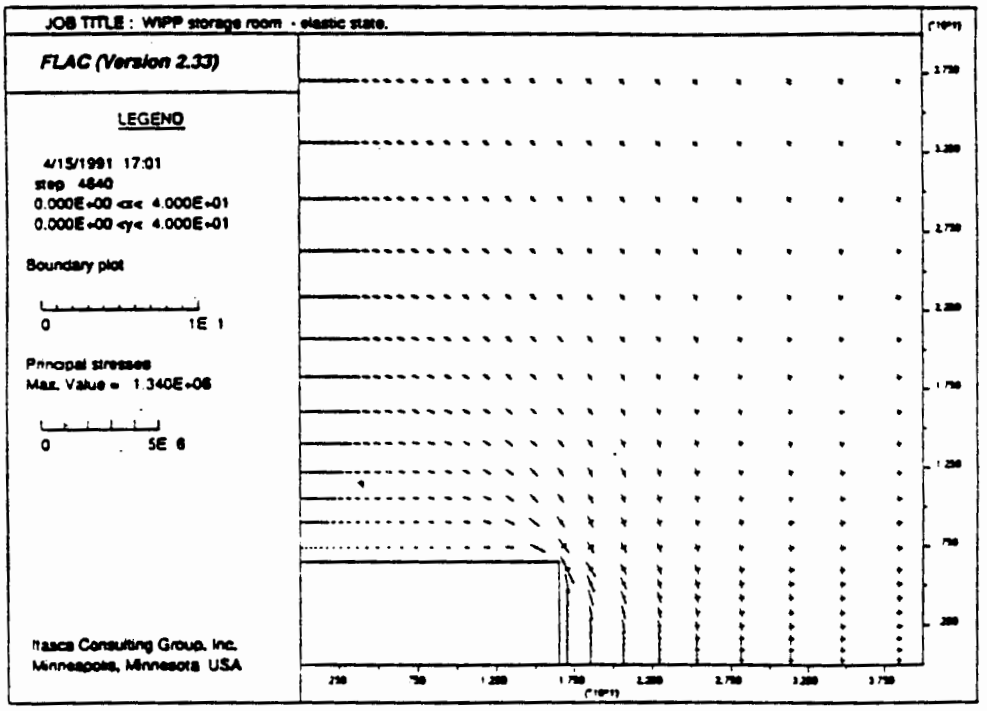


Fig. 7 Principal Stresses, WIPP Storage Room — Elastic State

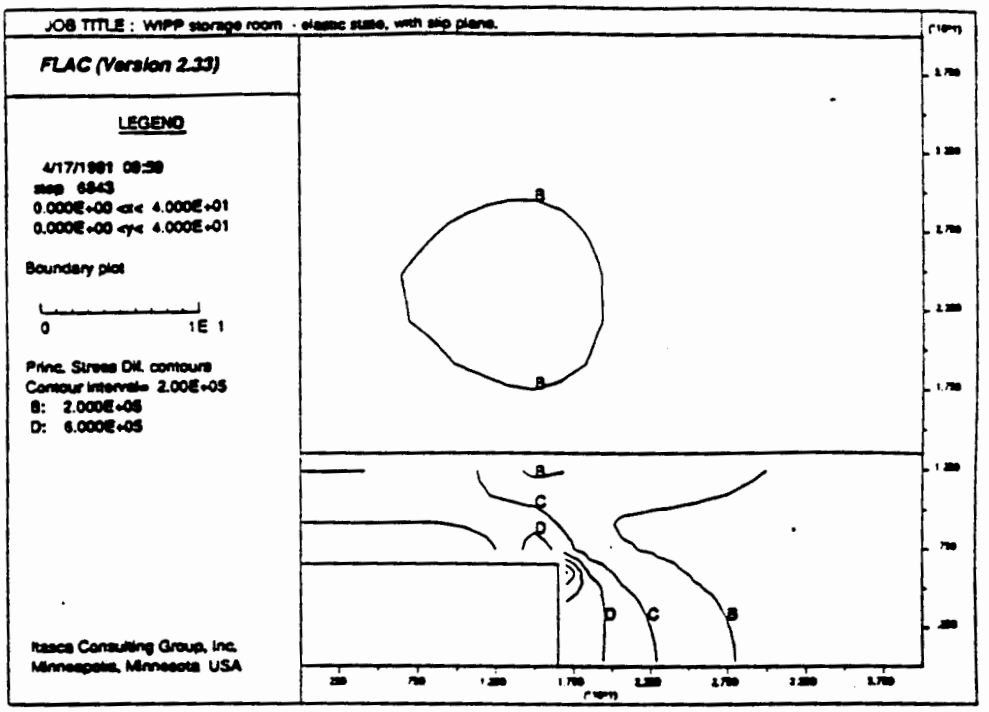


Fig. 8 Principal Stress Difference Contours, WIPP Storage Room — Elastic State With Slip Plane

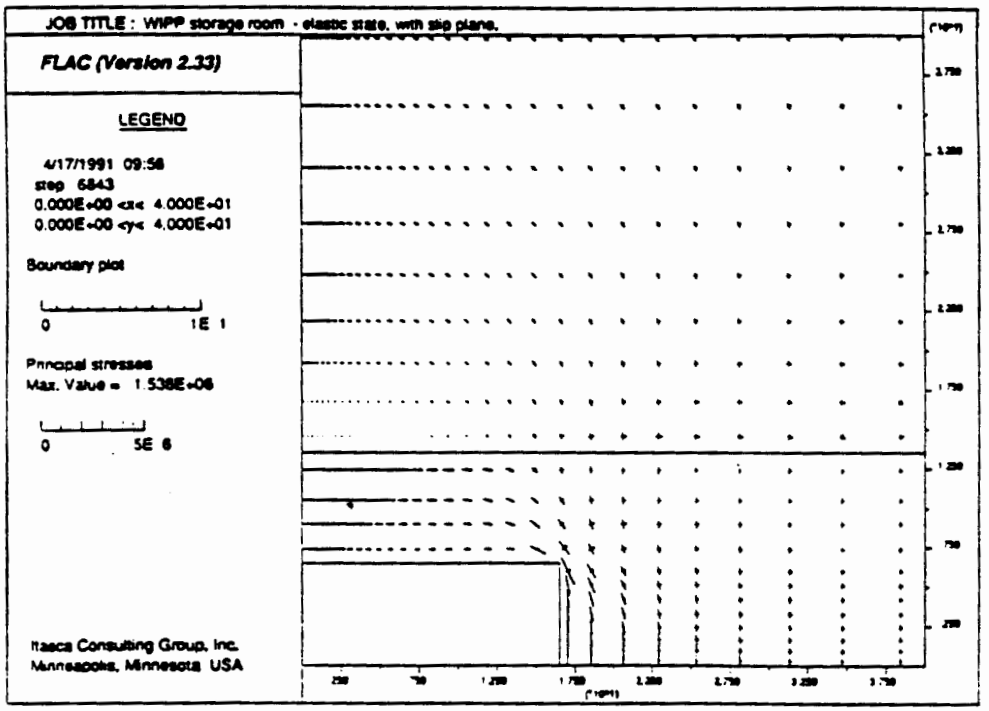


Fig. 9 Principal Stresses, WIPP Storage Room — Elastic State With Slip Plane

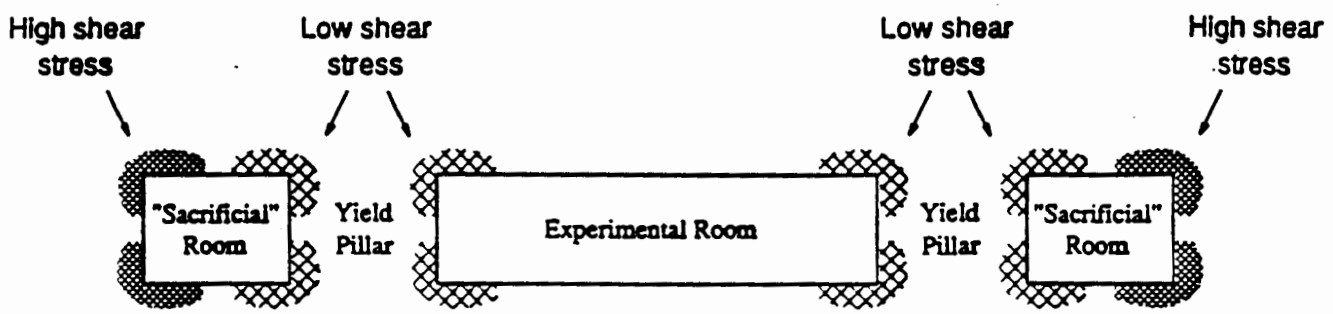


Fig. 10 Use of Nearly Sacrificial Excavation and Yield Pillars to Reduce Shear Stresses Around Experimental Rooms

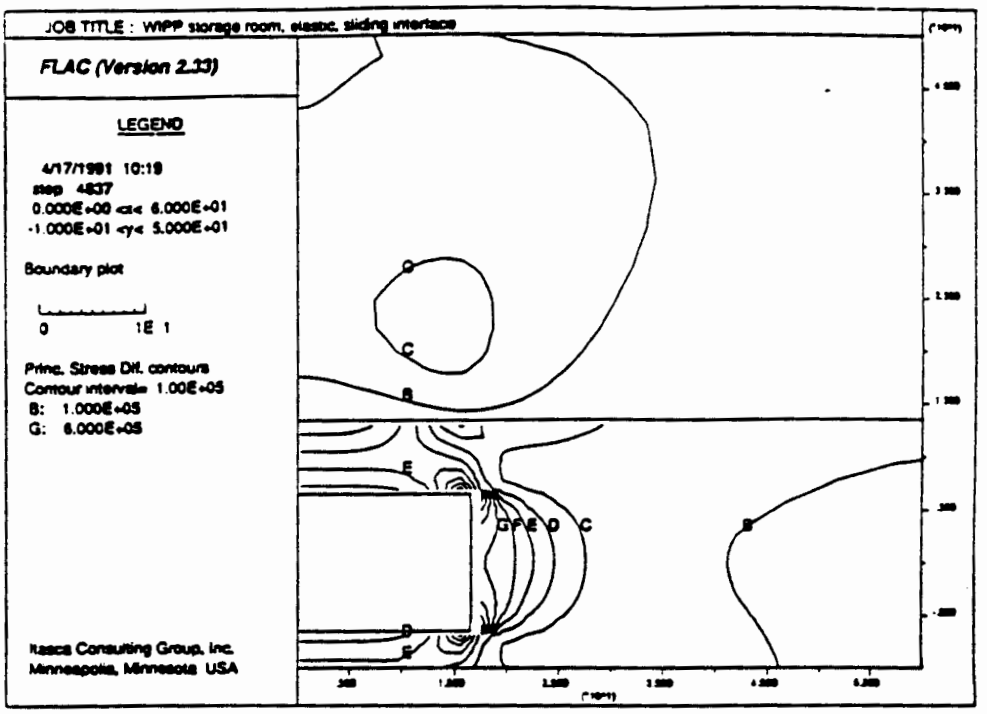


Fig. 11 Principal Stress Difference Contours, WIPP Storage Room — Elastic, Sliding Interface

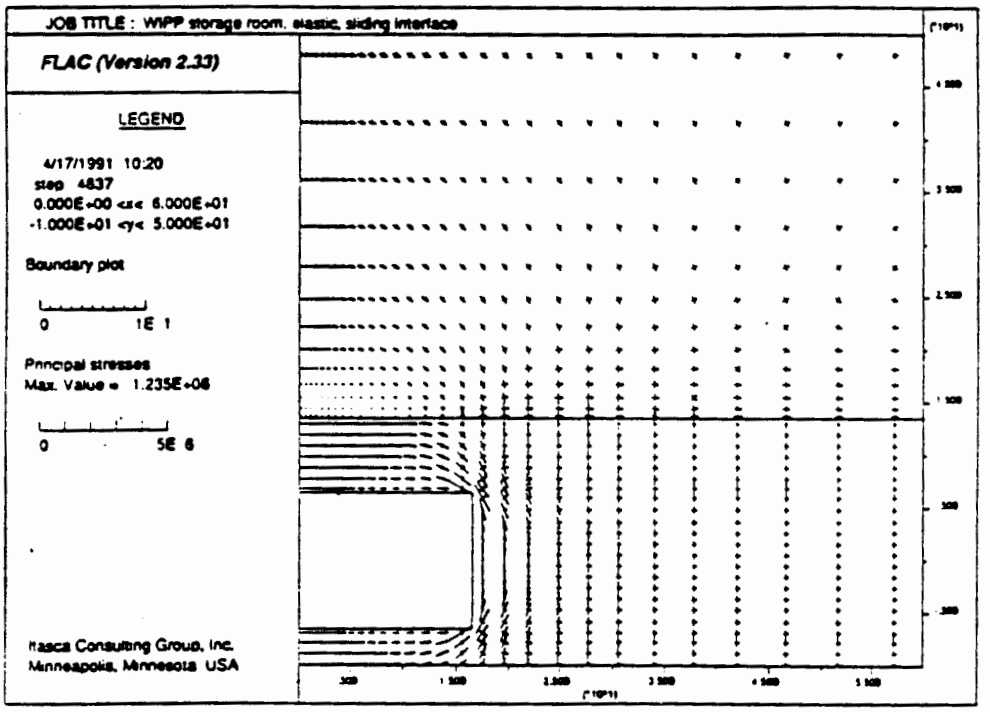


Fig. 12 Principal Stresses, WIPP Storage Room — Elastic, Sliding Interface

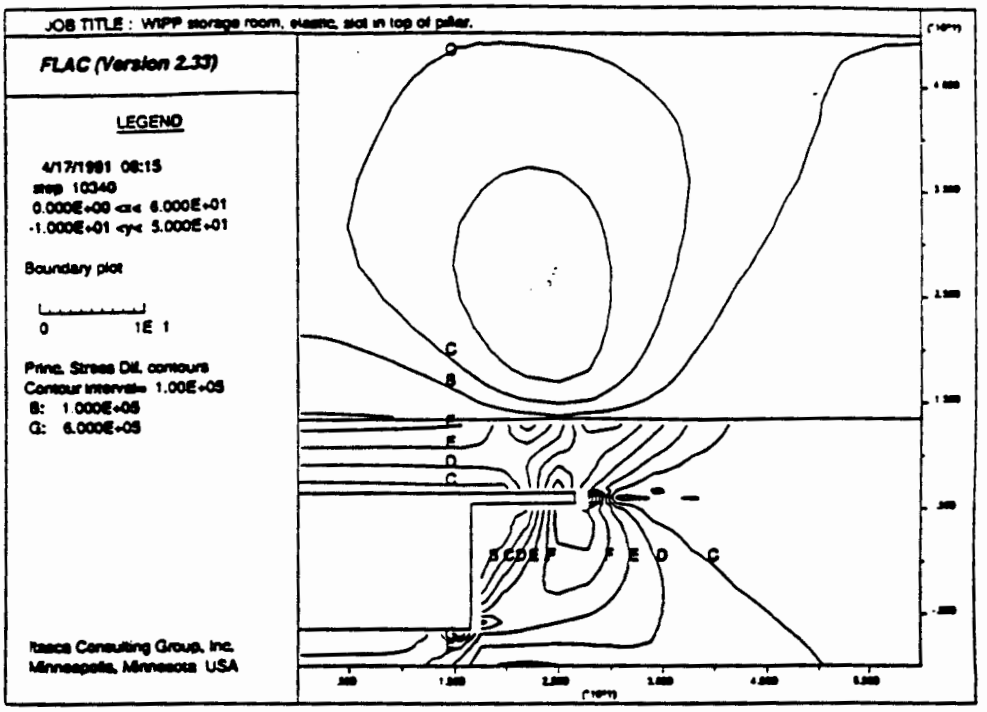


Fig. 13 Principal Stress Difference Contours, WIPP Storage Room — Elastic, Slot in Top of Pillar

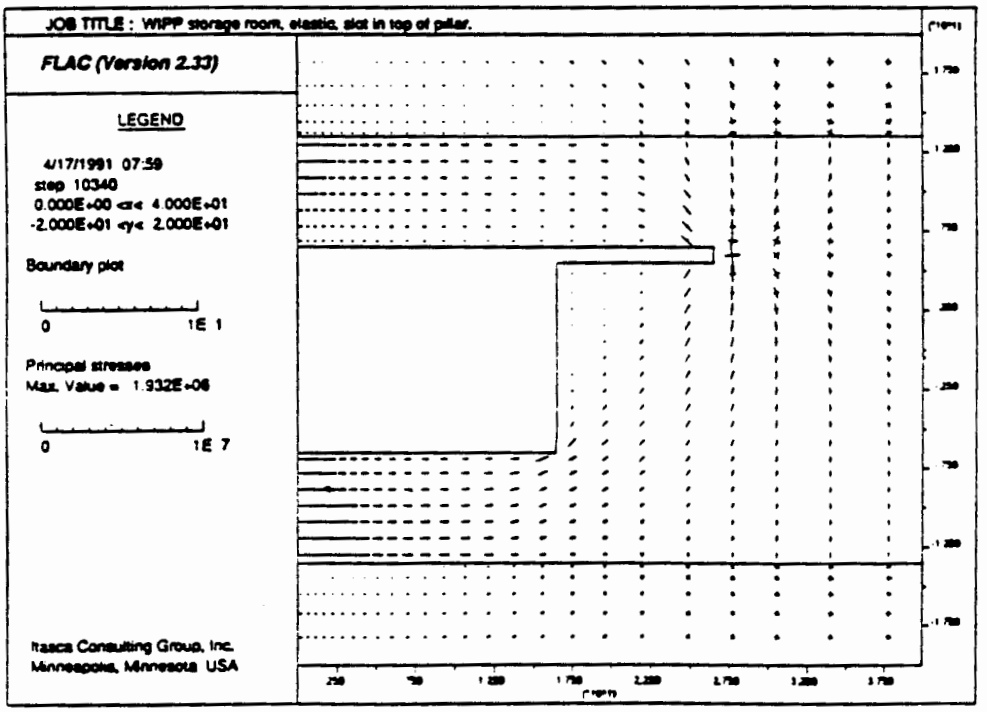


Fig. 14 Principal Stresses, WIPP Storage Room — Elastic, Slot in Top of Pillar

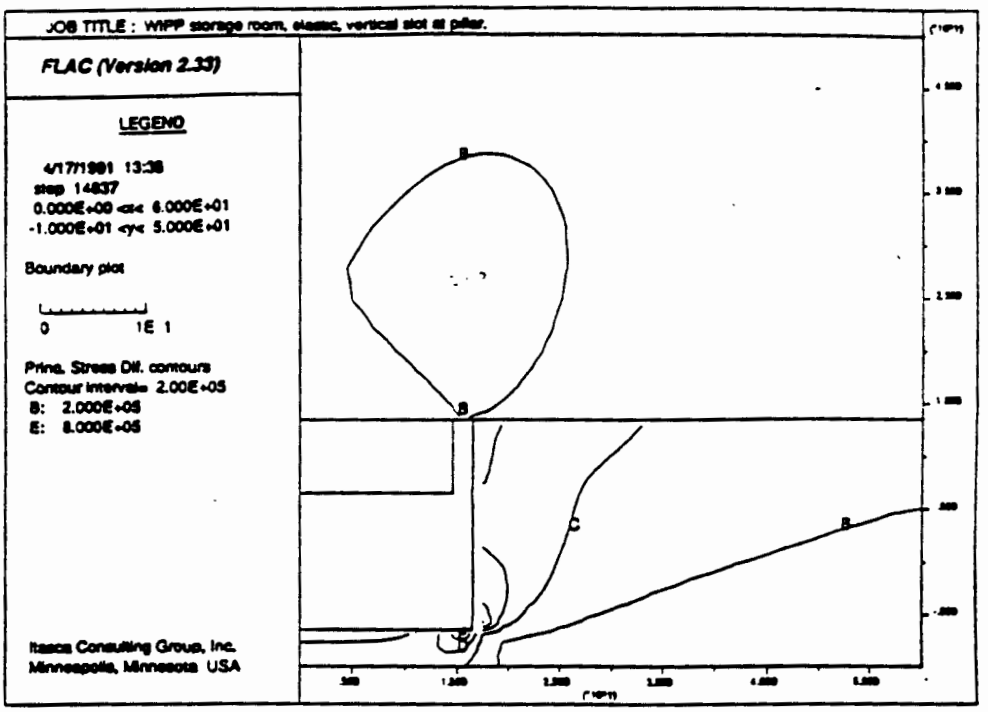


Fig. 15 Principal Stress Difference Contours, WIPP Storage Room — Elastic, Vertical Slot at Pillar

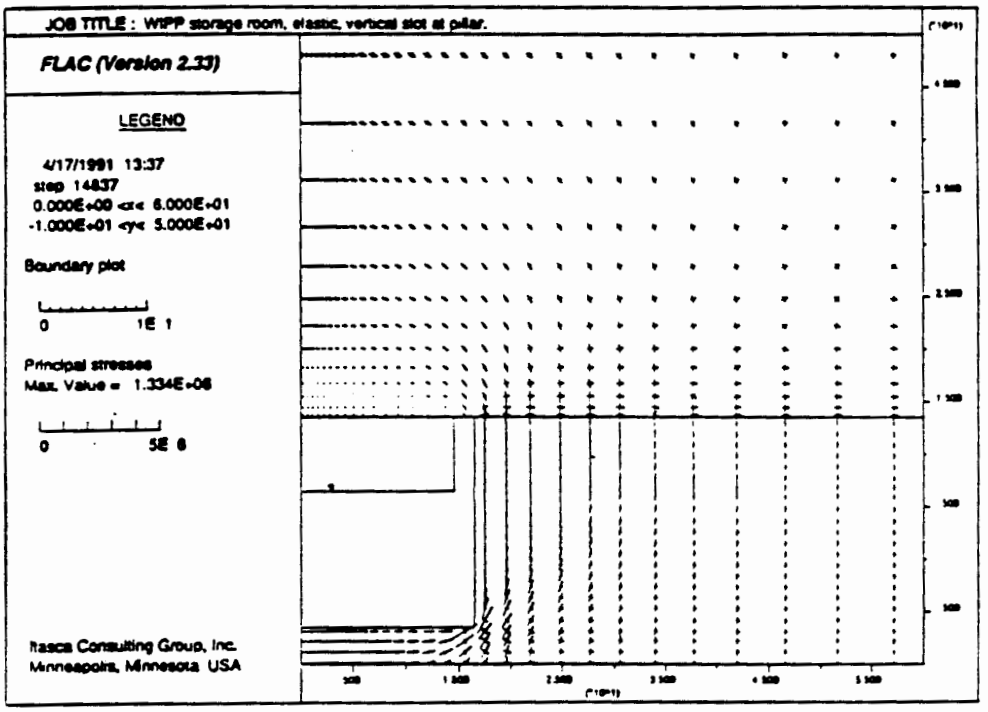


Fig. 16 Principal Stresses, WIPP Storage Room — Elastic, Vertical Slot at Pillar

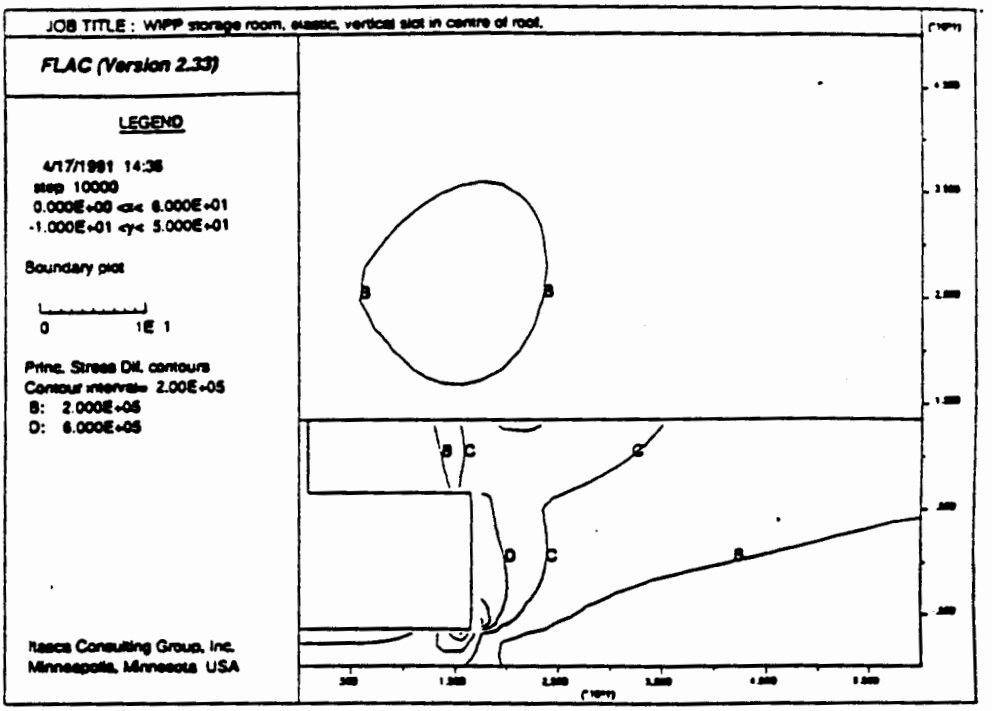


Fig. 17 Principal Stress Difference Contours, WIPP Storage Room — Elastic, Vertical Slot in Center of Roof

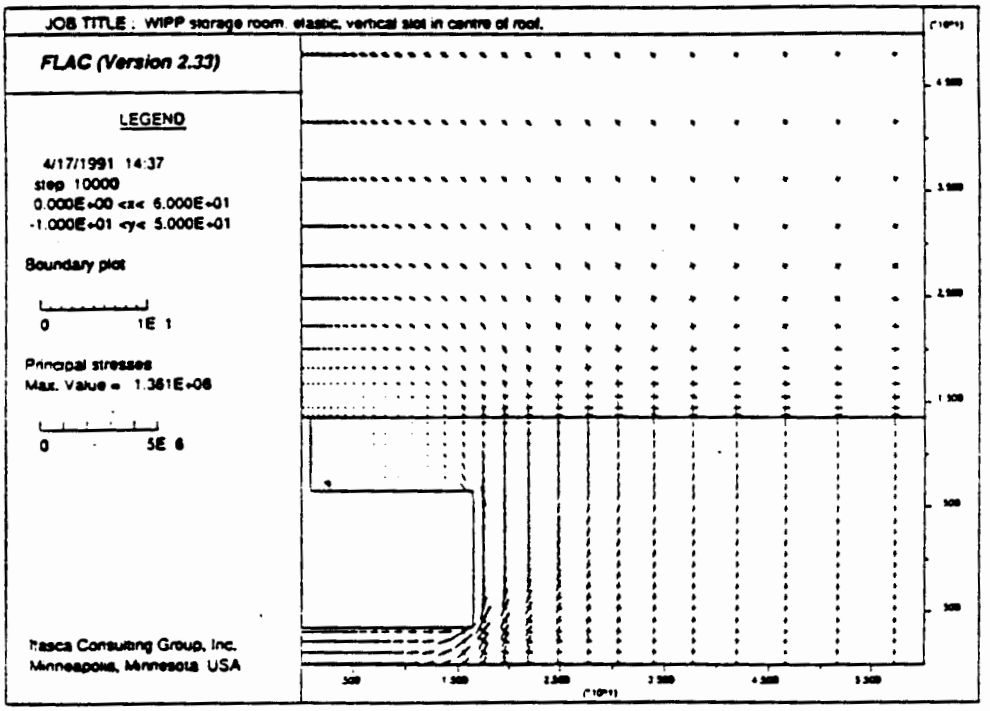


Fig. 18 Principal Stresses, WIPP Storage Room — Elastic, Vertical Slot in Center of Roof

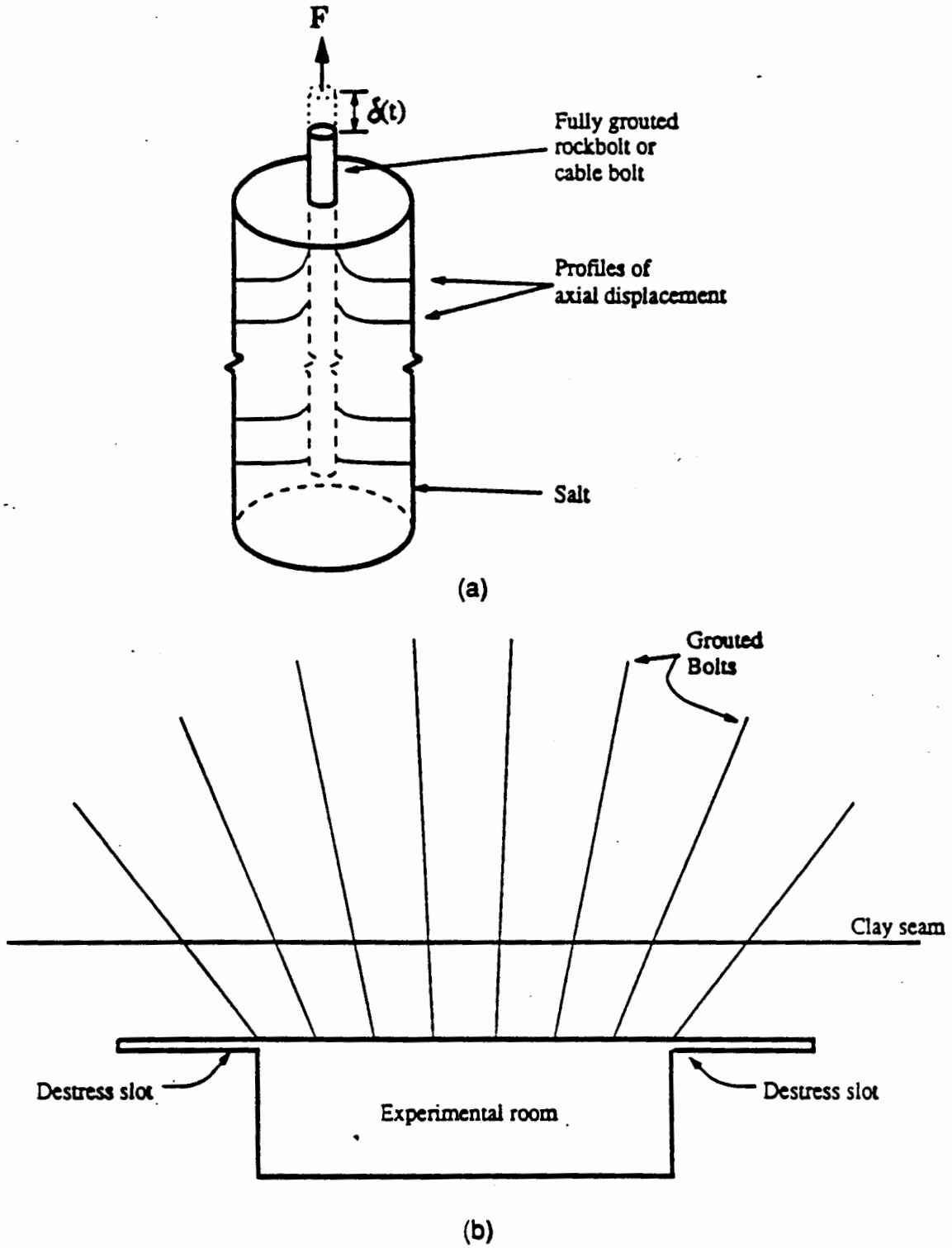


Fig. 19 Behavior of grouted bolts can be estimated by detailed modelling (a), and applying the resulting behavior to a model of the experimental room with bolts.

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APPENDIX I

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

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.

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.

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.

P. Mottahed

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.

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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

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 δH 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 δH.

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

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 - lb. 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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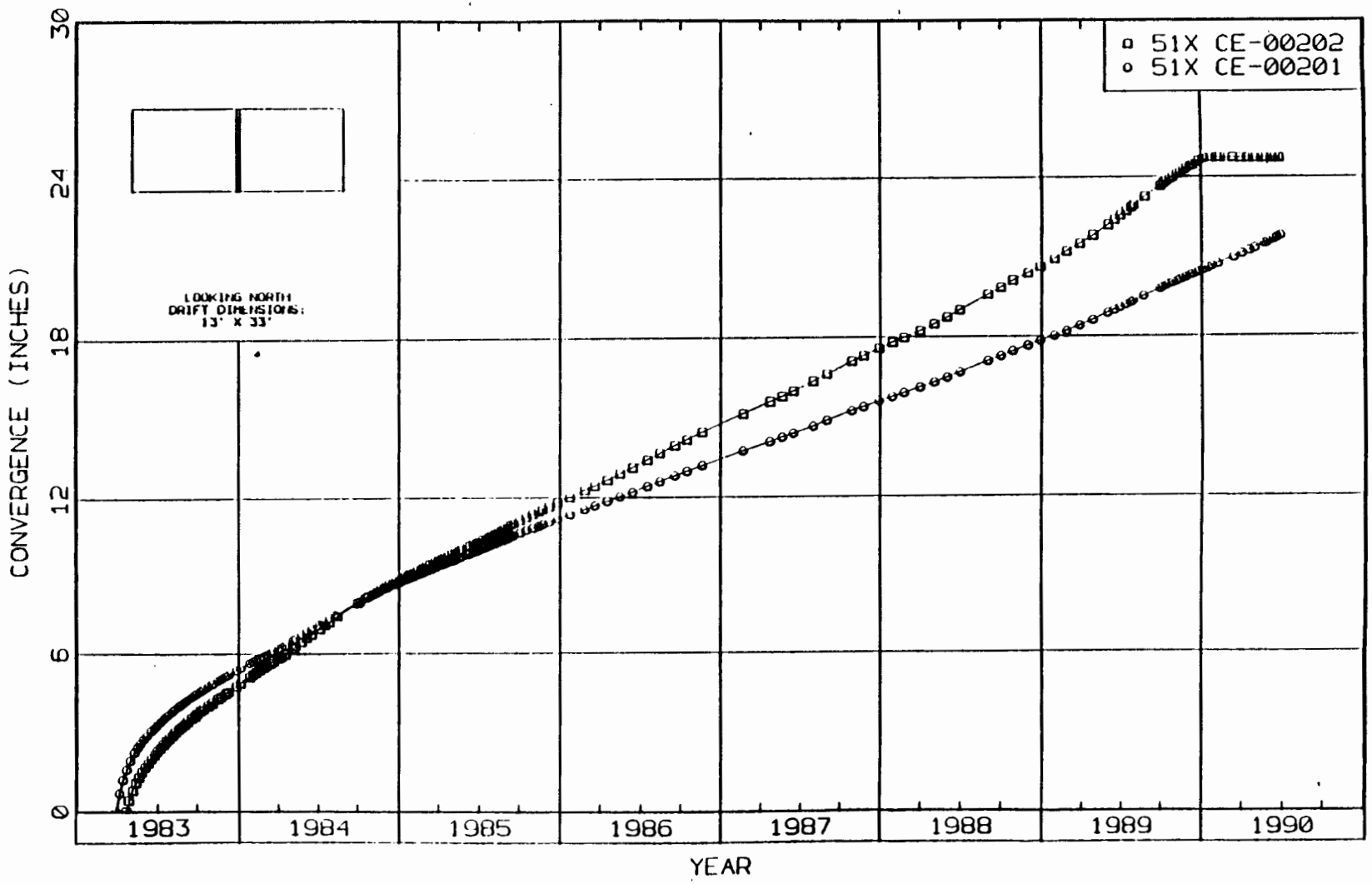
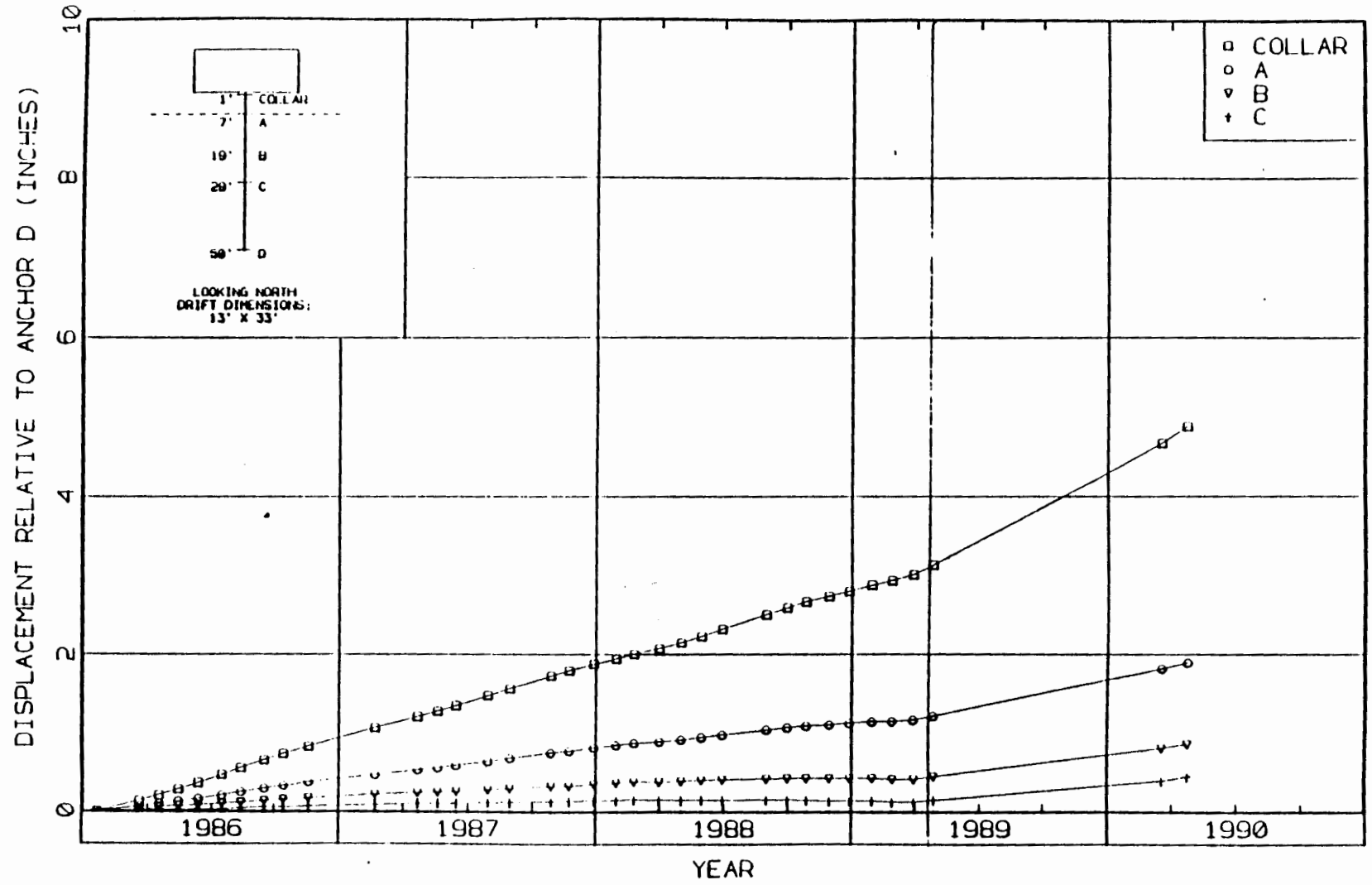


NOTES:
1. EXCAVATION DATE: TEST ROOM 1 CENTER APRIL 4,1983, TEST ROOM 2 CENTER MARCH 10,1983.

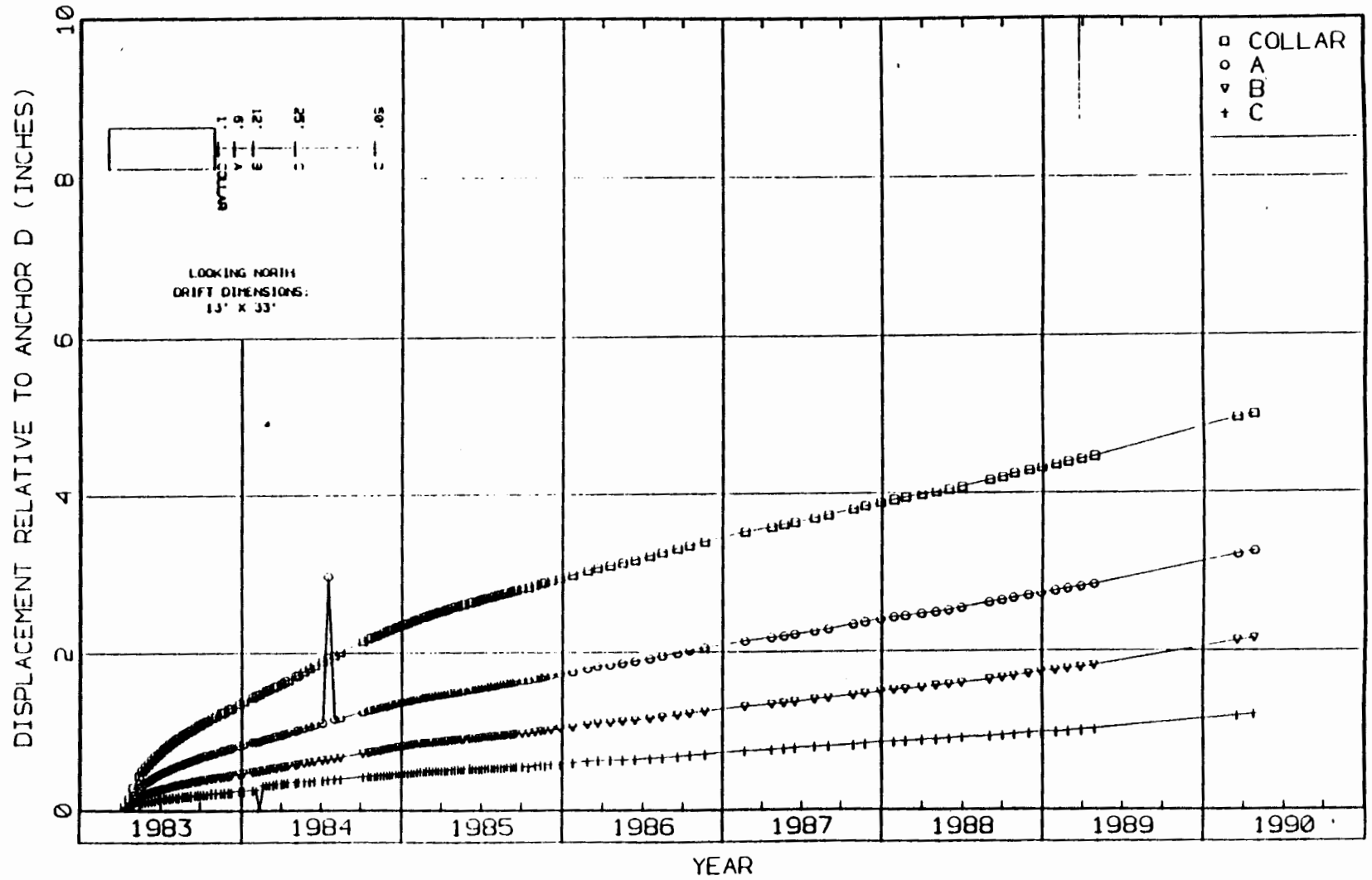
FIGURE 5-13
CONVERGENCE METERS
TEST ROOMS 1 & 2
CENTERPOINTS



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

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

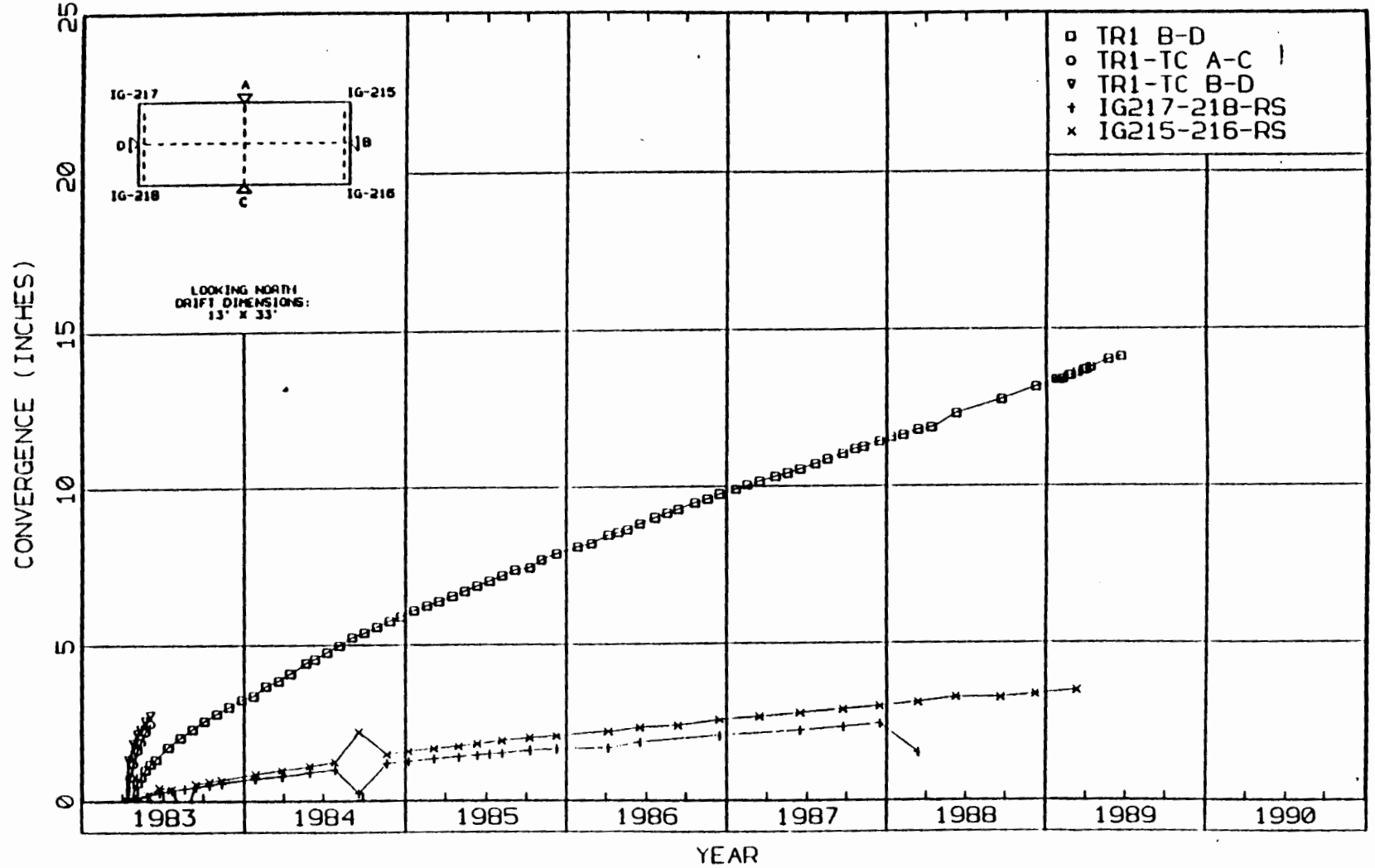
FIG 3



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

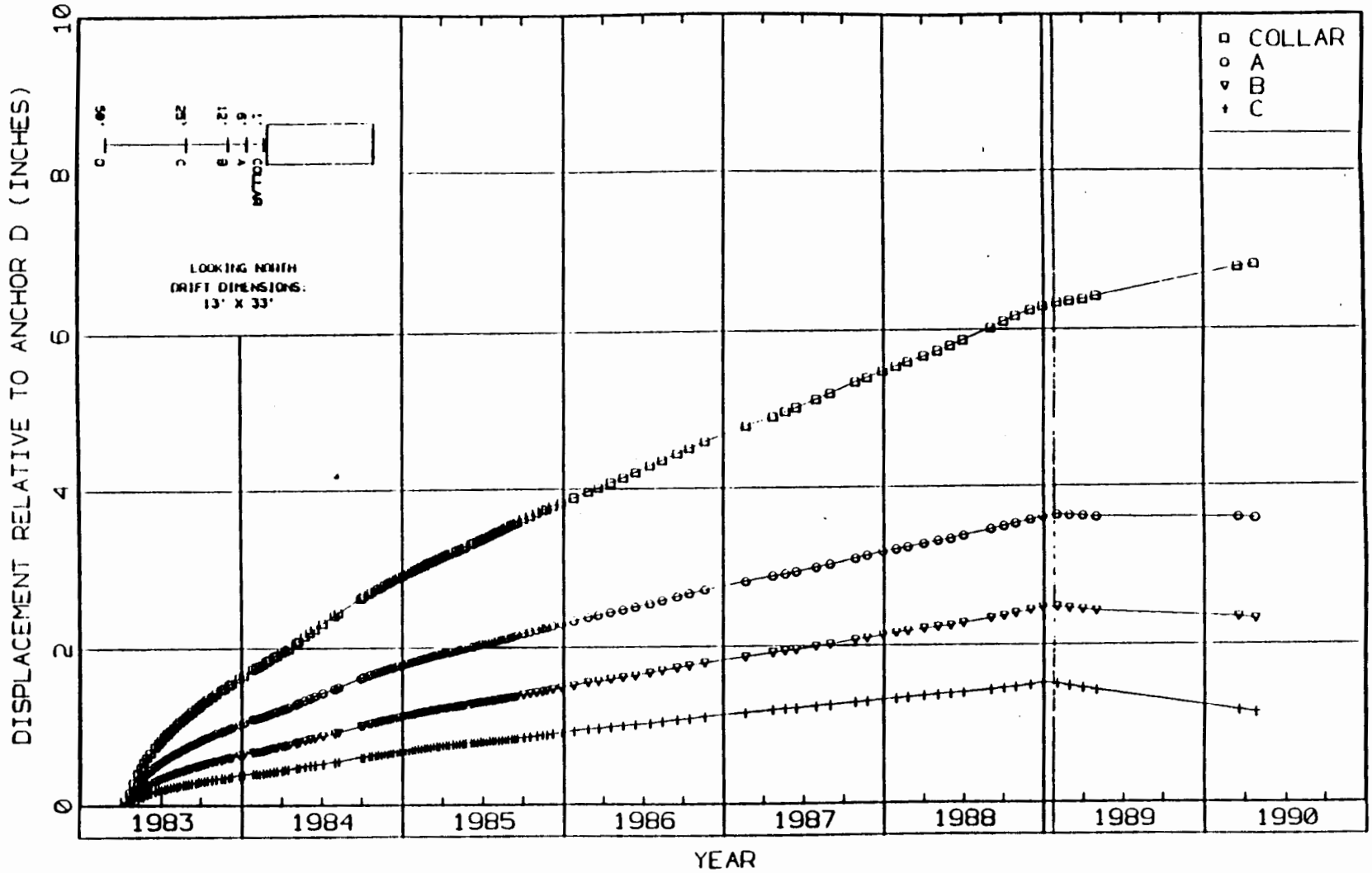
FIGURE 5-4
EXTENSOMETER 51X GE-00217
TEST ROOM 1-CENTER
EAST RIB

FIG 4.



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

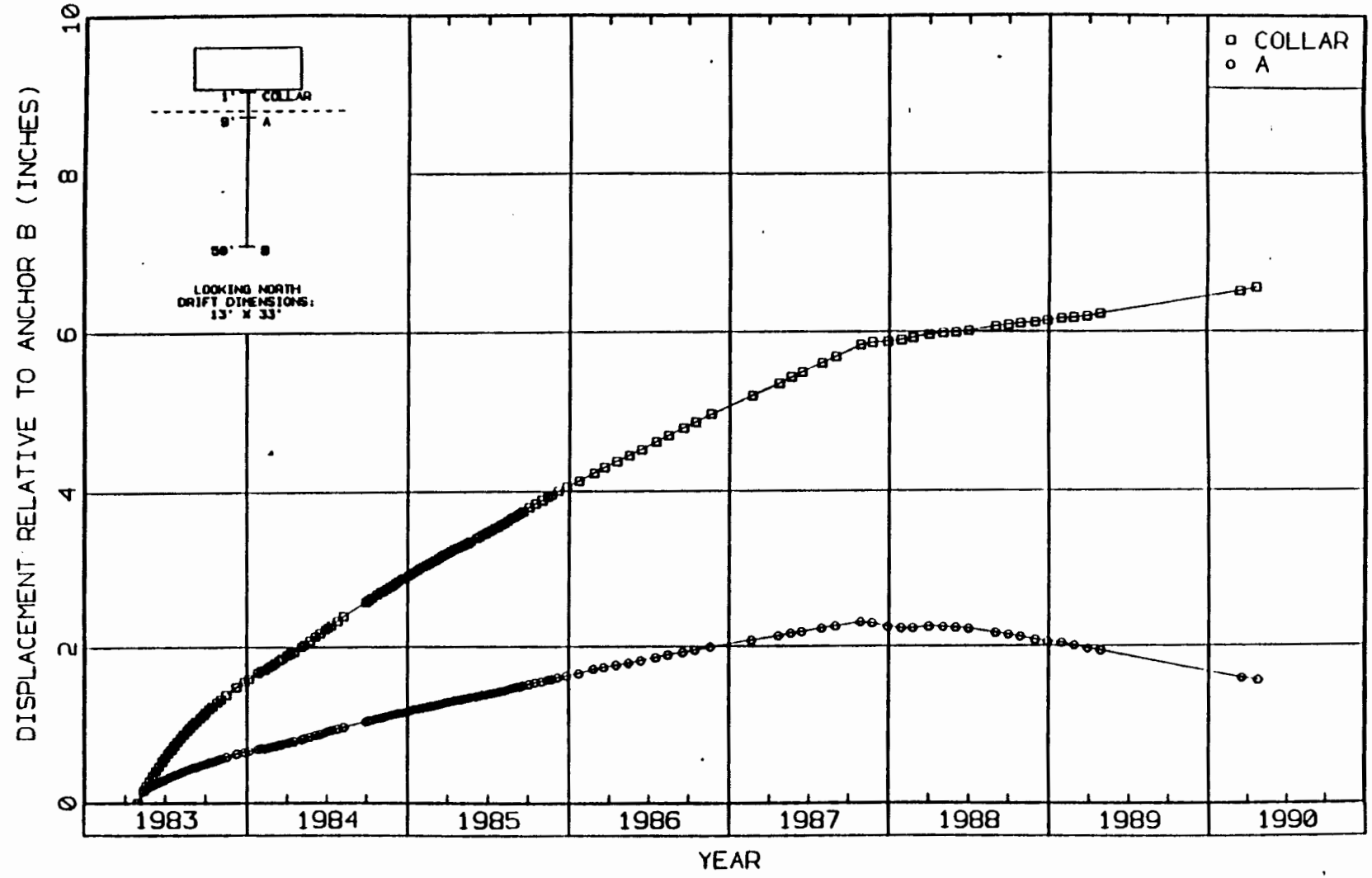
FIGURE 5-11
CONVERGENCE POINTS
TEST ROOM 1
ALL CHORDS



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 SPC/ 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 SFOV Room 1 rooffall. A similar situation seems to exist in the other SFOV 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.

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 5 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

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

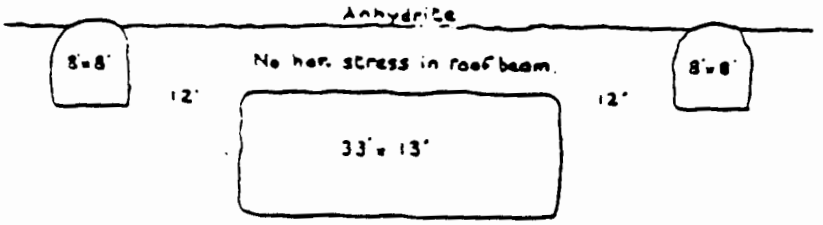
4.1. STRESS-RELIEVED ROOMS. The pillars between the storage rooms are 100ft wide. If we were to drive new rooms 33ft wide through the center of these pillars we would be leaving pillars between new and old rooms which would also be 33ft wide and about 14ft high. They would be marginally yielding pillars 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.

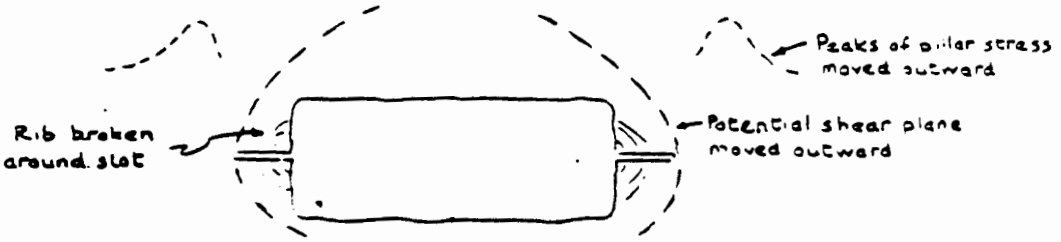
4.2. STRESS-RELIEVING TUNNELS. If we could drive small tunnels horizontally opposite the roof beam and perhaps opposite the floor beam, maybe with an Alpine-type miner, we could cut off the horizontal stresses. The new tunnels would be as small as possible, say 6x6 or 8x8ft, and separated from the rooms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

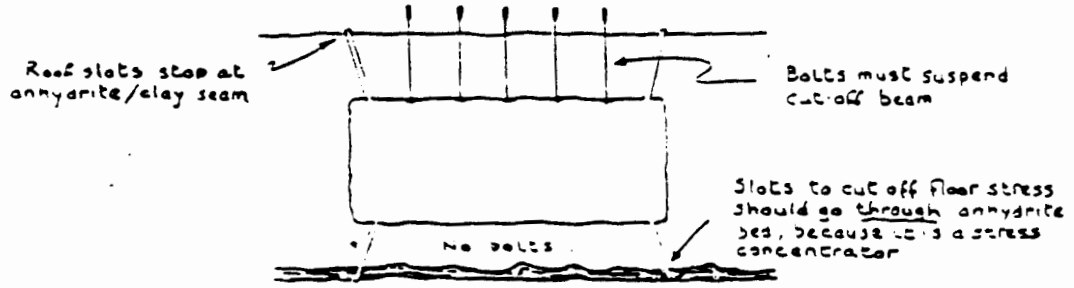
During and after the slotting operation the roofrock up to the anhydrite would have to supported entirely by roofbolts, in suspension, WITH NO LATERAL MOVEMENT. That means that we have to evaluate the bolts already installed, and perhaps install additional bolts. If we did have to I would probably recommend 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 USEM 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 13th, 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

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Dr R F Cook
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401 Canal
Carlsbad NM 88220

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 entire sag. 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 screeched or dried 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 not-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 wc

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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Waste Isolation Division
P.O. Box 2078
Carlsbad, New Mexico 88221

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 in the absence of bolts 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 <u>at this time</u>. 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:

- 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.

- 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.

- 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 Illerson
Panel Member

May 28, 1991
Date

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.

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.

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:

- o Limited maintenance without moving bins.
- o 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.

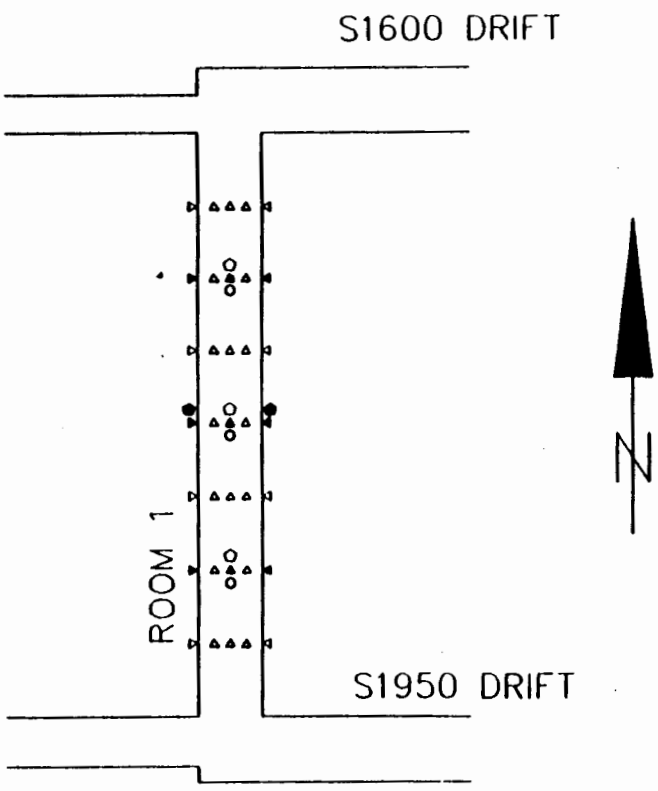
Proposed Instrumentation

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'

VIII. Parker C

DOE/WIPP-91 *9230

#655

DRAFT
09/10/91

WASTE ISOLATION PILOT PLANT

SUPPLEMENTARY ROOF SUPPORT SYSTEM

UNDERGROUND STORAGE AREA

ROOM 1, PANEL 1

RECEIVED
SEP 16 1991
ENVIRONMENTAL EVALUATION GROUP

WESTINGHOUSE ELECTRIC CORPORATION
WASTE ISOLATION DIVISION
CARLSBAD, NEW MEXICO
AUGUST 1991

ROOM 1, PANEL 1, SUPPLEMENTARY ROOF SUPPORT SYSTEM

EXTERNAL DESIGN REVIEW

Tuesday, September 17, 1991
6:45 a.m.
Meet at Stevens Motel Lobby
(1829 South Canal, Carlsbad, NM 88220)

AGENDA

7:00 a.m.	Depart for site after visiting Greene Street Office (Subhash Sethi, Chris Chmura, and Hamish Miller to escort the panel members).
8:15 a.m.	Security check-in.
9:00 a.m.	Safety briefing for underground tour (Support Building, Project Manager's Information Center (PMIC) room).
9:30 a.m.	Introductions and welcome.
10:00 a.m.	WIPP overview presentation.
10:30 a.m.	Surface tour.
11:30 a.m.	Lunch (cafeteria).
12:30 p.m.	Prepare for underground tour.
1:00 p.m.	Underground tour. A. Experimental area. B. Storage area.
3:30 p.m.	Return to surface
4:00 p.m.	Wrap-up meeting (Support Building, PMIC room).
4:30 p.m.	Depart for Stevens Motel.

ROOM 1, PANEL 1, SUPPLEMENTARY ROOF SUPPORT SYSTEM

EXTERNAL DESIGN REVIEW

Wednesday, September 18, 1991, through Friday, September 20, 1991
Motel Stevens
(1829 South Canal, Carlsbad, NM 88220)

Wednesday, September 18, 1991

8:00 a.m.	Presentation on Design Review Scope and Deliverables.
8:30 a.m.	Presentation on Geology and Rock Mechanics - Roy Cook.
9:15 a.m.	Break.
9:30 a.m.	Presentation on Design Requirements - Hamish Miller.
10:00 a.m.	Presentation on System Design and Installation - Chris Chmura.
10:45 a.m.	Break.
11:00 a.m.	Presentation on Monitoring and Instrumentation - Roy Cook.
	Additional presentations will be made, if required.
12:00 p.m.	Lunch.
1:00 p.m.	Panel discussions.
2:30 p.m.	Break.
2:45 p.m.	Panel discussions.
4:45 p.m.	Adjourn.

Thursday, September 19, 1991

8:00 a.m.	Panel discussions continued. Westinghouse provides responses to resolve issues raised by panel members.
	Finalize panel report and sign off.

Friday, September 20, 1991

8:00 a.m.	Panel meetings to continue, if required.

EEG

From: Engineering
WIN: 887-8182
Date: September 9, 1991
Subject: ROOM 1 PANEL 1 SUPPLEMENTARY ROOF SUPPORT SYSTEM - EXTERNAL DESIGN REVIEW 91-05

To: T. W. Halverson

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R. L. Wise, WPO

S. A. Orrell, SNL
T. M. Schultheis, SNL

The Design Verification Plan of August 19, 1991, (HA:91:5636) requires an External Design Review to be completed. The details of the External Design Review (No. 91-05) are given below:

1.0 EXTERNAL DESIGN REVIEW

1.1 <u>SCOPE</u>: The scope of this review will be to ensure that the roof support system shall perform to its designed function per the requirements established in Design Spec. 0087.

FORM 3504

9230

1.2 TYPE OF VERIFICATION: This will be a Formal Design Review.

1.3 STAGE OF VERIFICATION: This will be the Final Design Review by technically competent reviewers who are not a part of the WIPP Project.

1.4 REVIEWERS: The External Design Review Panel shall include:

Chairman	Dr. John Wilson	Chairman, Mining Engineering, University of Missouri, Rolla
Member	Dr. John Byrne	Golder Associates, Inc. Redmond, WA.
Member	Mr. Tony Iannacchione	U. S. Bureau of Mines, Pittsburgh Research Center, PA (was member of WIPP Geotechnical Panel)
Member	Dr. Parvis Mottahed	Head of Mining Technology Canada Center for Mineral & Energy Technology, Elliot Lake, Canada (was member of WIPP Geotechnical Panel)
Member	Mr. Gary Peterson	Engineering Manager, Cayuga Rock Salt Mine, Cargill Salt
Member	Mr. Robert Stahl	MSHA Safety & Health Technical Center, Denver, CO

Mr. M. R. Brown, Manager, Special Projects, WID Engineering, will act as the Secretary for Design Review Panel. His role is to support the chairman in the performance and documentation of the Design Review, including recording Design Review minutes and documenting action items, and assisting the Chairman in preparing the Design Review Report.

1.5 DATE AND PLACE OF REVIEW MEETING: Tuesday, Sept. 17, 1991 - WIPP site and underground visit

Wednesday, Sept. 18 through Friday, Sept. 20, 1991 - Design Presentation and Design Review, at Motel Stevens, Carlsbad, NM (887-2851).

CONCURRENCE:

S. C. Sethi, Manager
Mine Engineering

T. W. Halverson, Manager
Engineering

tw

HA:91:5657

ROOM 1, PANEL 1 SUPPLEMENTARY ROOF SUPPORT SYSTEM

EXTERNAL DESIGN REVIEW PANEL

Dr. John Byrne specializes in geotechnical engineering. He has over 16 years of experience in the civil, mining and waste disposal industries. Dr. Byrne's technical and managerial experience is broadly based and includes projects involving rock engineering (hydro, pumped storage and compressed air storage caverns; nuclear waste disposal facilities; tunnels; mine openings; rock slopes), soils engineering (foundations, tailings dams, water supply dams, tunnels, soil slopes, leach heaps, hazardous and municipal landfills, dynamic analysis), and off-shore engineering (oil platform foundations).

Mr. Tony Iannacchione is the supervisor of the Rock Mechanics Group at the U.S. Bureau of Mine, 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. Mr. Iannacchione served on the panel of geotechnical experts convened by Westinghouse and Department of Energy in April 1991 to evaluate the effective life of underground rooms in Panel 1 of the waste storage area.

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. Dr. Mottahed was a member of the panel of geotechnical experts convened by Westinghouse and Department of Energy in April 1991 to evaluate the effective life of the underground rooms in Panel 1 of the waste storage area.

Mr. Gary Peterson received a Bachelor of Science degree from Michigan Technological University in 1975. He has worked for 16 years at the Cayuga Rock Salt Mine for Cargill Salt. He developed a successful yielding pillar design at a mining depth of 2300 feet.

Dr. John Wilson is currently the chairman of the Mining Engineering at the University of Missouri-Rolla. He also is a Professor in this department. He received his PhD. in Mining Engineering from the University of the Witwatersrand, South Africa in 1971. His career began as a coal miner in England. He has since served in technical and management positions in production and service companies with operations in the United States, Europe, and Africa. His technical experience includes development and introduction of mechanized mining techniques in hardrock, application of mining techniques to hardrock and coal mines. He has also served in management capacity at operating mines and technical services companies. His projects have included economic analysis and feasibility, geo-engineering, hazardous waste, and mining generation start-up.

ROOM 1, PANEL 1 SUPPLEMENTARY ROOF SUPPORT SYSTEM

EXTERNAL DESIGN REVIEW PANEL CONT.....

Mr. Robert Stahl received a Bachelor's Degree from the University of Colorado in 1961. He has worked for the United States Bureau of Mines as a research geologist testing geophysical techniques for locating geologic structures in the earth's crust. Mr. Stahl is currently working with MSHA's Ground Support Division in Denver, Colorado.

DRAFT
09/10/91

WASTE ISOLATION PILOT PLANT

SUPPLEMENTARY ROOF SUPPORT SYSTEM

UNDERGROUND STORAGE AREA

ROOM 1, PANEL 1

WESTINGHOUSE ELECTRIC CORPORATION
WASTE ISOLATION DIVISION
CARLSBAD, NEW MEXICO
AUGUST 1991

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54-D-003-W2	Supplementary Roof Support Room 1, Panel 1, *Steel Set Locations and Details*
54-D-003-W3	Supplementary Roof Support Room 1, Panel 1, *Steel Set Fabrication*
54-D-003-W4	Supplementary Roof Support Room 1, Panel 1, *Room 1 Cross-section*
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B	Destructive Tests
C	Support System Design * Anchor Design * Chanel Design
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1.0 EXECUTIVE SUMMARY

WIPP is designed to provide a full-scale facility to demonstrate the technical and operational principles for permanent isolation of defense-generated transuranic waste. It is also designed to provide a facility in which studies and experiments can be conducted.

Dry Bin-Scale Tests are being planned as a portion of the WIPP Test Phase Performance Assessment Program described in the WIPP Test Phase Plan: Performance Assessment (DOE 1990 b). These Tests are anticipated to be conducted for a period of up to seven years. Room 1 of Panel 1 of the Underground Storage Area is to be used as the location of the Bin Scale Tests to investigate the generation of gas from the waste that is proposed to be stored at the WIPP in the near future.

The original design for the waste storage rooms in Panel 1 provided for a limited period of time during which to mine the openings and to emplace waste. Room 1 was scheduled to be filled in fewer than five years before being sealed. Initially mined to rough dimensions in 1986, Room 1 was later mined to finished dimensions in 1988. Information obtained from the Site and Preliminary Design Validation (SPDV) program indicates that the rooms in Panel 1 should remain stable without ground support and that creep closure would not adversely affect equipment clearances during at least five years following excavation.

The demonstration phase was later deferred and an experimental program including Bin Scale Tests was added for Panel 1. Delays in the test schedule have revised the date for first waste receipt. Therefore, based on the timing and scope of the test phase, an additional seven years of useful life may be required to complete the tests in Room 1, Panel 1.

To assess the long term stability of Panel 1, a panel of geotechnical experts was convened in April, 1991. The final report of the panel was issued on June 5, 1991. The panel agreed that the WIPP geotechnical monitoring program as used in the SPDV Test Rooms is adequate to provide early warning of deteriorating conditions in Panel 1. The panel reviewed the design and stability of the rooms in Panel 1 and concluded that these rooms could be expected to provide a useful life of at least seven years from the time of excavation (up to 11 years with a decreasing level of confidence) with routine maintenance (DOE, 1991). However, the panel also agreed that ground support measures could be used that would allow the Bin Scale Tests to be carried to completion. The test period as currently defined is up to seven years, thus requiring a room life of up to 12 years from when the room was mined. The following options or their combinations recommended by the Expert Panel have been evaluated to extend the life of Room 1 of Panel 1 and to provide added confidence in its ability to support the test program:

- Relying on the currently installed rock bolt system and upgrading, if necessary, based on the results of the geomechanical monitoring program.
- A ground support system using resin anchored rock bolts.
- Interlaced grout anchored wire cables and wire mesh to control rock falls.
- Cutting slots in the back and/or floor to relieve the lateral stresses.

- Yielding support system such as timber cribs or steel yielding supports.
- Roof truss system.
- Mine new rooms.

In order to extend the life of Room 1, Panel 1, a ground support system needs to consider the past history of Room 1, the on-going deformations in the room, and the potential roof failure mode. Also, the support system must be designed to accommodate the bins and test equipment, including forklift access for bin installation and subsequent monitoring activities.

To be acceptable, the ground support system must:

- Be capable of fully supporting the anticipated roof wedge such as that produced in SPDV Room 1.
- Be capable of yielding in a manner which would accommodate the future closure and deformation of the roof rock.
- Accommodate the bin scale equipment, including forklifts and ancillary equipment.
- Extend the life of Room 1 to allow completion of the experiments, for an additional period of up to seven years (from July 1991).

The initial roof support concept developed for Room 1 of Panel 1 involved timber "crib sets" with interconnected steel beams. After further analysis, timber crib supports were abandoned in favor of yieldable roof supports which would provide more uniform roof support. These supports consisted of resin anchored steel rock bolts and steel cross beams, with yielding steel columns as commonly used in the coal mining industry. More importantly, the rock bolts could be continuously monitored using load cells and adjusted to accommodate further room creep.

As the design process proceeded, it became clear that the majority of the load would be carried by the rock bolts. The yielding columns were therefore eliminated. The steel beam was modified from an initial I beam configuration to an inverted channel, thus eliminating the complex attachment plate structure needed for the I beam.

The final roof support design contained in this document consists of 8.23m (27 feet) long 15 x 40 steel channel support sets installed laterally across Room 1 on 2.44m (8 feet) to 3.05m (10 foot) centers. Each channel set is divided into three nine foot long segments which are bolted together in place using connecting plates. Each support set is secured by eleven 3.96m (13 feet) long Dywidag steel tendons (anchor bolts) that are resin anchored in relatively stable ground above the Anhydrite "b" clay horizon. The channel support anchor bolts are designed so that their loads can be monitored and adjusted to accommodate continuing roof deformation. To allow for differential lateral deformations, each tendon is located in an oversized .076m (3 inch) diameter hole which extends from the hole collar to the Anhydrite "b" clay horizon.

ANHYDRITE "

ROOF SUPPORT SYSTEM DESIGN AND IMPLEMENTATION

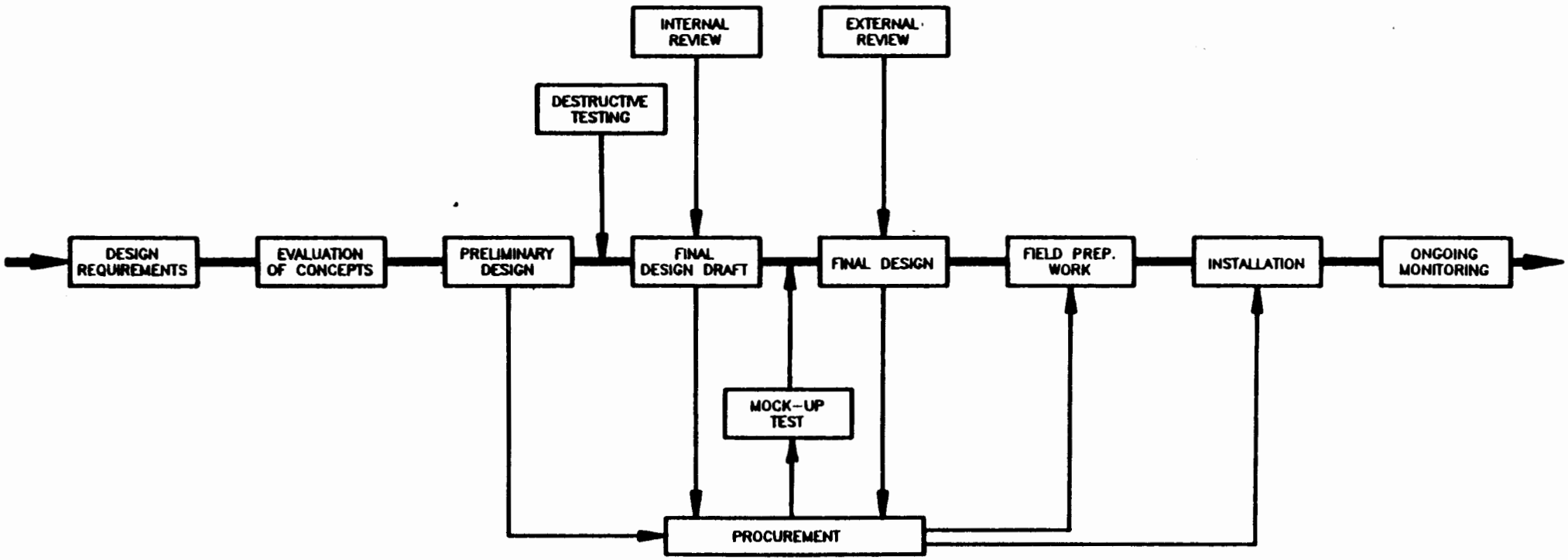


FIGURE 2.0-1 Room 1, Panel 1

The area between the channel support sets is covered by a network of steel wire lacing cables underneath a mat of steel welded wire mesh and expanded metal. This mat is held in place by the channel support sections. Its function is to contain loose rock in between the channel support sets.

Chainlink wire mesh pinned to the ribs (sidewalls), is provided to contain any minor spalling down to approximately 2.13m (7 feet) above the floor.

A conservative approach has been used throughout the design process. Areas where this has been done include the following:

- A minimum .76m (3 foot) grouted bolt length has been used where tests have shown 18 inches to be sufficient.
- The manufacturer's minimum yield load has been used for bolt design - tests give results 22-28% higher.
- The support effect of the existing 3.04m (10 feet) mechanically anchored rockbolts and the meshing and lacing has been disregarded.
- The wedge-shaped salt beam has inherent strength which has been disregarded.

As designed, the supplementary roof support system incorporates the four acceptance criteria stated above as well as five out of the seven Expert Panel recommendations. The support system can also be installed concurrently with bin operations. Figure 1.0-1 provides an isometric view of the support system for Room 1, Panel 1.

The geomechanical monitoring system represents an integral part of the roof support system design. The monitoring system is designed to monitor loads on each rock bolt, measure continuing creep and deformation in and around the room, identify stress loads on the rock and deflections of the steel channel supports.

The monitoring system allows for adjustment of loads in the rock bolts to accommodate room creep and to provide early indication of any unusual closure activity.

The test bins, within the standard waste boxes, are stacked two high along the ribs of the test room. The spacing is sufficient to allow personnel access between the bins for ground support installation, inspection, and routine ground control maintenance tasks.

In addition to the monitoring program, a testing program was implemented to confirm the validity of rock anchor calculations and installation procedures. The testing program included destructive testing of rock anchors and a mock-up installation of a portion of the entire system.

The WIPP is committed to safely providing long term roof support.

2.0 INTRODUCTION

WIPP is designed to provide a full-scale facility to demonstrate the technical and operational principles for permanent isolation of defense-generated transuranic waste. It is also designed to provide a facility in which studies and experiments can be conducted.

Bin Scale Tests are being planned as part of the WIPP Test Phase Performance Assessment Program described in the WIPP Test Phase Plan: Performance Assessment (DOE 1990 b). These Tests are anticipated to be conducted over a period of up to seven years.

Room 1 of Panel 1 of the Underground Storage Area is to be used as the location of the Bin-Scale Tests to investigate the generation of gas from the waste that is proposed to be stored at the WIPP in the near future.

The original design for the waste storage rooms in Panel 1 provided for a limited period of time during which to mine the openings and to emplace waste. Room 1 was initially mined to rough dimensions in 1986. Information obtained from the Site and Preliminary Design Validation (SPDV) program showed that the rooms would remain stable without ground support and that creep closure would not adversely affect equipment clearances during at least five years following excavation.

The demonstration phase was later deferred and an experimental program including Bin Scale Tests was added for Panel 1. Delays in the test schedule have revised the date for first waste receipt. Therefore, based on the timing and scope of the test phase, up to seven years of useful life are required to complete the tests in Room 1, Panel 1. This document presents the design for a supplementary roof support system for Room 1 of Panel 1 of the Underground Storage Area. System design and its implementation process is presented in Figure 2.0-1.

3.0 DESIGN REQUIREMENTS

3.1 DESIGN CRITERIA/CONSIDERATIONS

The support system must be designed to accommodate the following criteria:

1) Provide a suitable supplementary roof support system to ensure that the Bin Scale Tests conducted in Room 1, Panel 1, of the Underground Storage Area will not be interrupted during the seven year period starting July 1991.

2) The basic design parameters are determined by geotechnical considerations such as the age of Room 1, existing and future ground deformations in and around Room 1, and the prevailing stratigraphy and stress conditions.

3) The support system takes cognizance of the recommendations of the Geotechnical Expert Panel.

4) The support system takes cognizance of Design Spec. No. D-0087.

3.2 DESIGN SPECIFICATIONS

Design specifications are contained in Document entitled "Design Spec., No. D-0087, Supplementary Roof Support for Room 1 of Panel 1."

3.3 DESIGN BASES

The Supplementary Roof Support System for Panel 1, Room 1, is a yieldable type support that consists of evenly spaced sets of 15 x 40 inverted steel channel sections supported by eleven rock anchors.

The design for the Supplementary Roof Support System for Room 1, Panel 1, is based on the following:

3.3.1 GENERAL

- The support system is able to be installed concurrently with bin operations.
- Safe access is provided for a minimum of seven years from July 1991.
- A minimum access height of 3.45m (11 feet, 4 inches), is provided after seven years.
- Support installation procedures take into account working within RMA boundaries.

- Corrosion is a non-impactive factor for the duration of the system installation, based on experience gained at the WIPP and in the potash basin mines.

- Because of accessibility limitations and RMA requirements during the testing program, only the center portion of Room 1 located between the ventilation bulkheads has been considered in this supplementary roof support design. Roof control for the remainder of the room will be addressed elsewhere at a later date.

3.3.2 ROCK MECHANICS

- The zone of rock between Anhydrite "b" and Anhydrite "a" is sufficiently stable to provide a good anchoring base for the support system rock anchors.

- Horizontal and vertical virgin stresses are equal at the repository horizon of 655m (2,150 feet).

- The geology and stratigraphy at Room 1, Panel 1, are similar to those in the SPDV Test Room area.

- Observations and measurements from the SPDV Test Rooms will be used as the bases for describing the deformation mechanisms occurring in Room 1, Panel 1.

- Creep deformations arise from differential stresses created as a result of excavating an opening of the given shape.

- Low-angle shear fractures will occur in the immediate roof rock, and once these have formed, roof movements into the excavation are increasingly associated with gravity rather than salt creep.

- The supplementary support system accommodates past and future room deformations.

- The roof failure mode is that of a detaching wedge, triangular in section, 10m (33 feet) wide and 2.13m (7 feet) high at the center.

- The density of the immediate roof rock above Room 1, Panel 1, is 2,160 kg/m^3 (135 pound per $foot^3$) for all calculations.

- The rock anchor holes have a 7.6mm (3 inch) reamed-out section below the grouted portion that will be sufficient to prevent shearing of the tendons that may arise from differential lateral deformations that might take place in the roof rock.

- The roof expansion between the anchor horizon and hole collar is assumed to be 38mm (1.5 inch) per year.

- The maximum lateral differential deformation below Anhydrite "b" is assumed to be 12mm (0.5 inch).

3.3.3 SYSTEM DESIGN

- The system of rock anchors consist of resin grouted rock anchors, grade 60 steel, anchored above Anhydrite "b" horizon.
- Minimum resin bond length between anchor and salt is 0.91m (3 feet).
- The steel channel set assembly would act as a surface plate system that:
  - is capable of accommodating the design load
  - is capable of accommodating monitoring devices that would allow continuous monitoring of bolt load
  - allows detensioning of anchor loads as and when required
  - assists in distributing the load between bolts
  - is capable of supporting the lacing and meshing.
- Each rock bolt anchor extends downwards through the channel section plate for a distance of 18 inches to provide for a downward adjustment life of seven years. This accommodates the expected 38mm (1.5 inches) per year of roof expansion as well as the bearing plates and load cell assembly. If required, couplings will provide for additional adjustment.
- Rock spalling in between sets is controlled by a system of wire mesh and lacing.
- Floor maintenance will be carried out as and when required.
- The transverse 16 mm (5/8 inch) diameter wire lacing ropes will be adjustable.
- Rib spalling that may occur is contained by a wire mesh system that extends down to a height of approximately 2.1m (7 feet) above the floor.
- No stability problems are expected from fracturing of the ribs, based on experience gained at the WIPP since the opening of the project.

3.3.4 INSTALLATION AND MAINTENANCE

- The bolt loads are readjusted when the load on a bolt reaches 1.1 times the design load.

- Existing 3.04m (10 foot) rockbolts may be removed in order to facilitate installation of the supplemental rockbolt supports.

- Existing instrumentation fixtures, installed cables, and piping will be relocated to the sides of the room to avoid damage during system installation and maintenance.

- Rock anchor holes will be drilled vertically with a tolerance of ± 2 degrees measured in such a way that the ends of the holes will not be closer than 0.5m (20 inches).

- Drilling tolerance for the depth of the hole is ± 25mm (1 inch).

3.3.5 TESTING AND MONITORING

- A complete full-scale mock-up test will be carried out in Room 2. This will have at least five channel sets.

- Quality control and creep tests will be carried out on each bolt. The test load will be taken to 1.33 times the maximum design load.

- The monitored data from Room 1 will be evaluated on an ongoing basis.

- The design loads for the rockbolts and associated anchoring system have been confirmed by the destructive tests that have been conducted.

- The geomechanical monitoring system is designed as an integral part of the support system and will:

  - monitor the load on every rockbolt

  - measure ongoing creep and deformation

  - allow an assessment of the length of room life that might be obtained beyond seven years.

4.0 SYSTEM DESIGN

4.1. GEOLOGY AND ROCK MECHANICS

4.1.1 ROCK MECHANICS

Much of the understanding regarding the performance of excavations in salt at the WIPP has been gained from observations taken in the Site and Preliminary Design Validation (SPDV) Test Rooms. The case study presented by the roof fall in SPDV Test Room 1, together with numerical modelling results, provides the information for defining the size and shape of the rock wedge that must be supported by roof support system. This is assumed to have a triangular cross-section as shown in Figure 4.0-1.

The virgin in-situ stresses are one of the basic determining factors governing the rate of deformation in and around the mined opening.

The initial stress state at the repository horizon is established from Heims Rule for weak rocks (Hoek and Brown, 1980). This rule establishes the vertical stress as dependent on the depth of overburden and its average density, and the horizontal stresses to be equal to the vertical stress. Taking the average density for the overburden at the WIPP site as 2130 kg/m^3 (144 $pound/foot^3$), the initial stresses at the repository horizon are about 2000 psi.

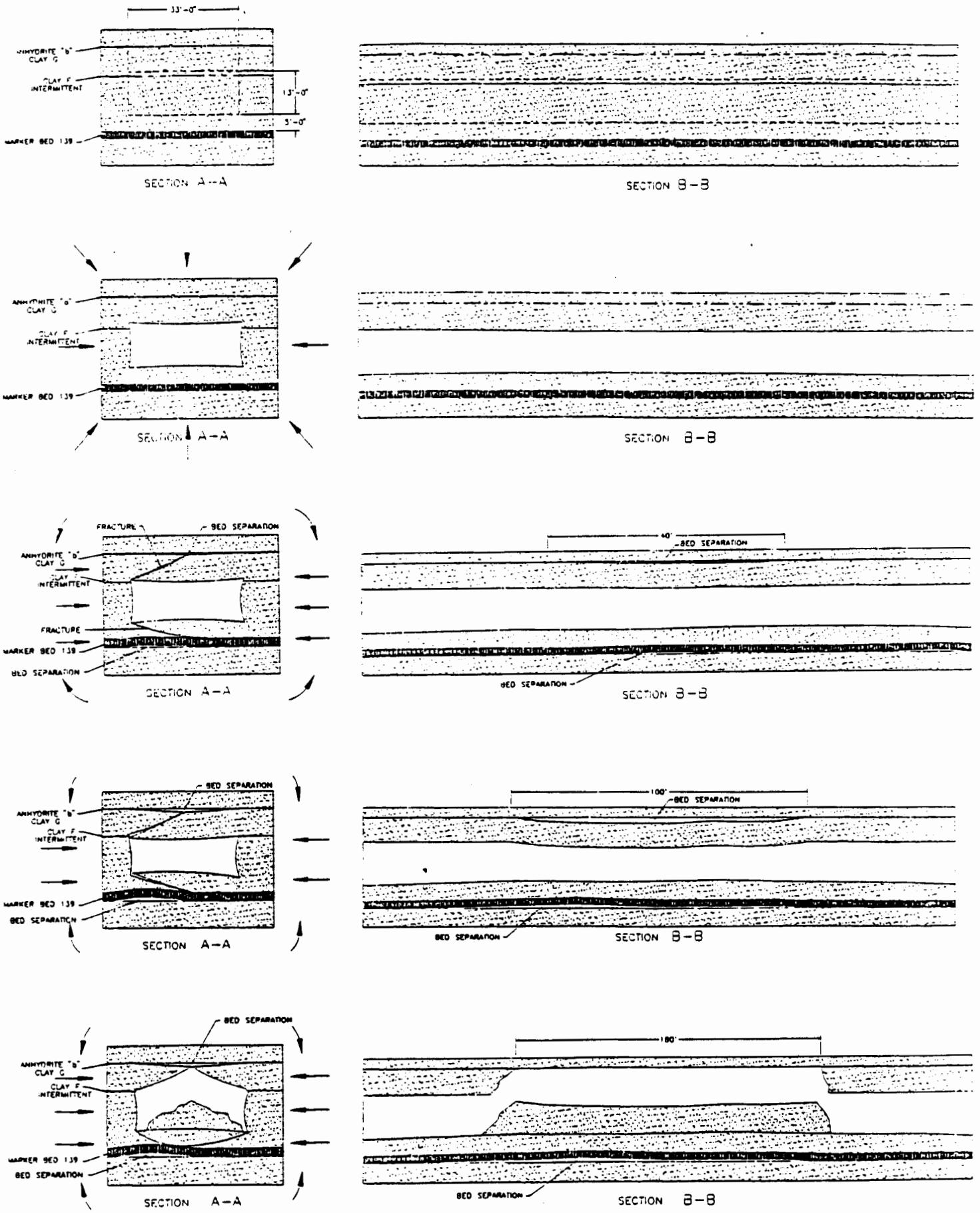
When a room is excavated in salt, the local virgin in-situ stress field is disturbed. The immediate initial response of the rock is to set up stress as if it were in an elastic rock, the so-called "time zero" response. Differential stresses are created around the excavation and it is these stress differences that drive the subsequent creep deformations that result in closure of the room.

With time, the stresses close to the excavation are relieved by creep of the salt into the excavation. Shear stresses develop at the strata interfaces due to the differences in the mechanical properties of the different rock types and lead to slippage at these contacts and eventually to bed separation. The presence of these strata interfaces further leads to the concentration of lateral stresses in the roof and floor beams leading ultimately to the development of low angle shear fractures. Once the shear fractures have developed, roof movements in an excavation are increasingly associated more with gravity effects than with salt creep. At this stage there are two processes at work in the strata above the excavation. These are:

- Creep of the salt - Salt creep is still occurring in the competent salt above the rock wedge and above the ribs.

- Kinematic movement of the immediate rock due to gravity - The rock wedge, if it is unsupported, will move down under its own weight. Figure 4.0-1.

SUPPLEMENTARY ROOF SUPPORT SYSTEM
UNDERGROUND STORAGE AREA
PANEL 1, ROOM 1



8918:0606

Figure 4.0-1 Kinematic Movement of the Rock Wedge

SUPPLEMENTARY ROOF SUPPORT SYSTEM
UNDERGROUND STORAGE AREA
PANEL 1, ROOM 1

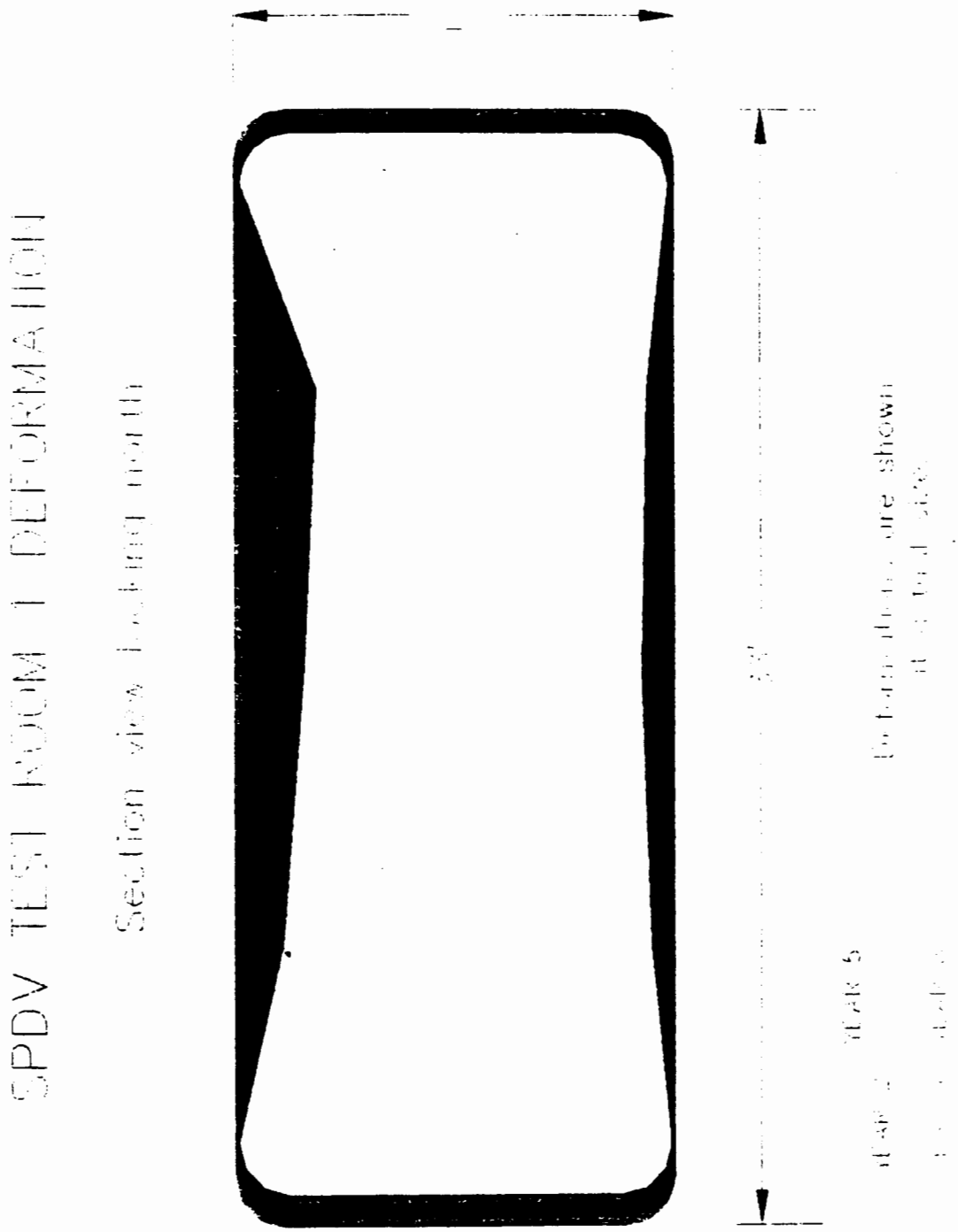
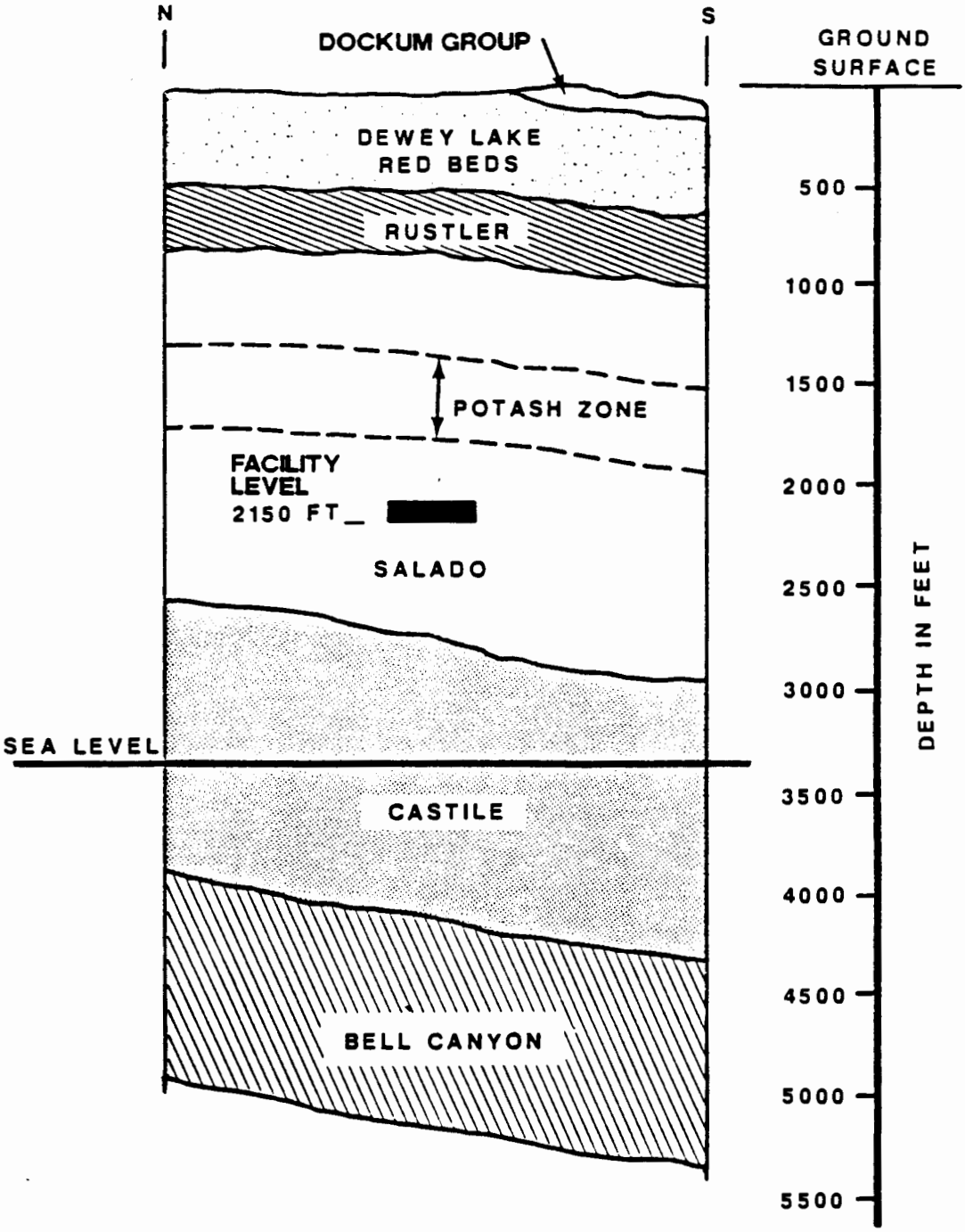


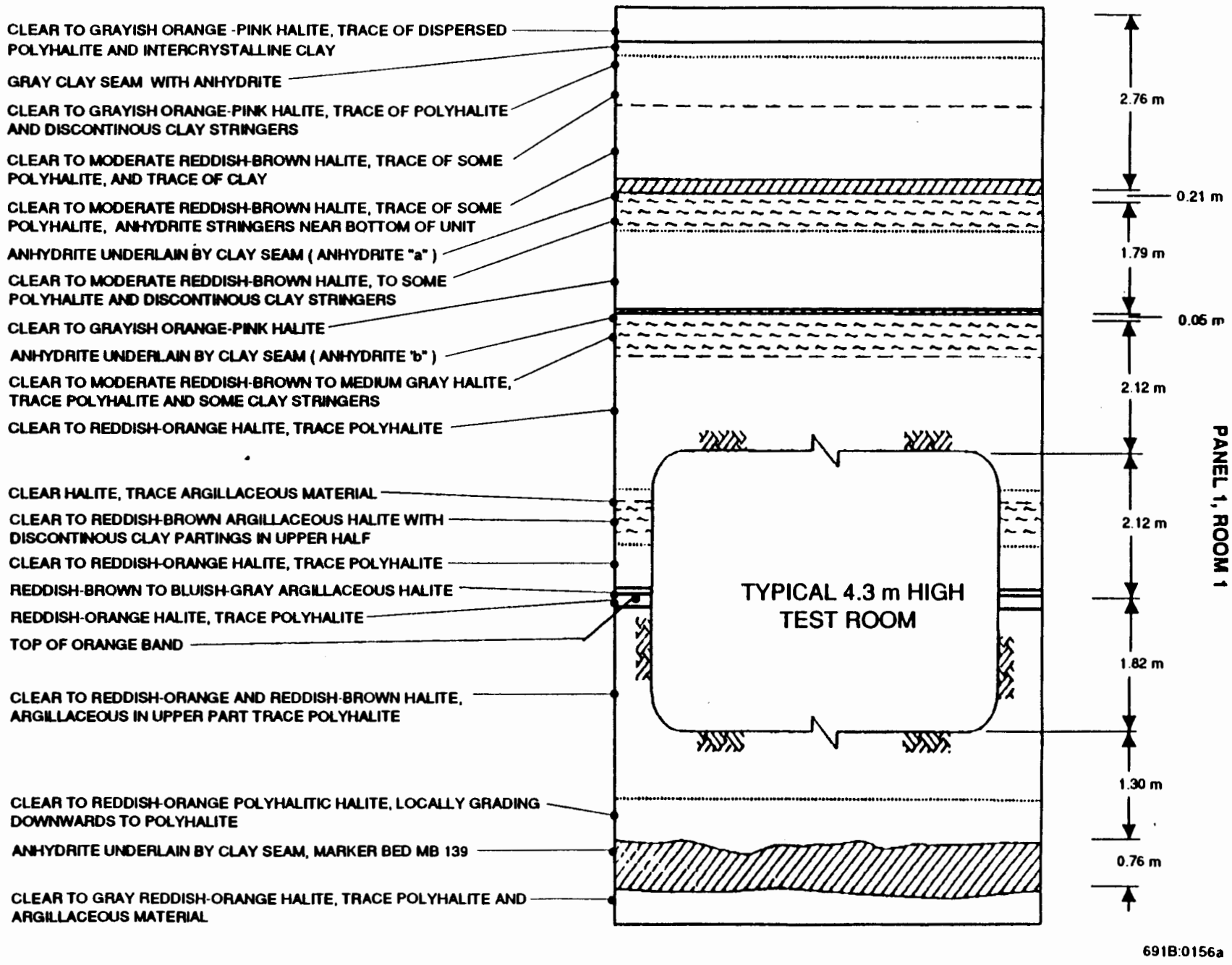
Figure 4.0-2 SPDV Test Room 1 Deformation

SUPPLEMENTARY ROOF SUPPORT SYSTEM
UNDERGROUND STORAGE AREA
PANEL 1, ROOM 1



191B:0136a

Figure 4.0-3 Generalized Site Stratigraphy



SUPPLEMENTARY ROOF SUPPORT SYSTEM
UNDERGROUND STORAGE AREA
PANEL 1, ROOM 1

Figure 4.0-4 Stratigraphy of the WIPP Repository Horizon

Inclinometer measurements, in vertical and horizontal boreholes, give the horizontal and vertical deflections of these boreholes in the rooms roof and ribs respectively. The effect of the 2.13m (7 foot) clay seam can clearly be seen as a large relative horizontal difference in the movement of the salt immediately below the clay seam.

Extensometers located in the roof, ribs and floor measure the extensions at different distances along the lengths of their boreholes. Closure meters measure the closure of the rooms, either between roof and floor or rib to rib. A composite picture of how the rock is deforming and moving into the excavated room is obtained when inclinometer, extensometer, and closure measurements are put together. The Expert Panel unanimously agreed that the mechanism of deformation at Room 1, Panel 1 would be very similar to that experienced at the SPDV Test Rooms. For further detailed discussion and information on Geology and Rock Mechanics at WIPP, refer to Appendix A.

4.1.2 STRATIGRAPHY

The proposed underground storage facility is located 655m (2,150 feet) below the surface in bedded salt of the Permian Salado Formation. This formation consists primarily of halite, argillaceous halite, minor anhydrite, and minor polyhalitic units. Over 365m (1,200 feet) of impermeable evaporitic deposits separate the facility horizon from the first overlying sedimentary rocks and 620m (2,034 feet) of evaporites lie below the facility horizon and provide a barrier to Permian limestones and sandstones. Figure 4.0-3.

The facility horizon lies within a 12m (39.4 feet) thick unit consisting of halite, argillaceous halite, and polyhalitic halite (Figure 4.0-4). A thin, 0.3m (.98 feet) to 0.5m (1.64 feet) thick layer consisting of anhydrite and polyhalite, and identified as Marker Bed 139 lies about 1.5m (4.92 feet) below the floor level. Anhydrite beds (less than 10mm [.03 feet] thick), called anhydrite "a" and "b" occur about 4m (13.12 feet) and 2m (6.56 feet) above the roof. Thin clay seams called Clay G and Clay H are associated with the bottom of these beds. In addition, an intermittent thin clay layer identified as Clay F is found in the immediate roof of excavations.

The anhydrite and clay layers have a significant impact on the mechanical performance of excavations. The clay layers provide interfaces along which slip can occur whereas the thick layers can provide a stiff anhydrite band within the strata sequence that does not deform plastically with time. For further detailed discussion and information on Geology at the WIPP, refer to Appendix A.

4.1.3 VERTICAL MOVEMENTS

The vertical movement that the roof support must accommodate are a combination of salt creep, dilation of the salt due to fracture development, and gravity effects once fractures have formed.

The total roof expansion that has to be accommodated has been taken as 38mm (1.5 inches) per year at midspan.

4.1.4 LATERAL MOVEMENTS

Lateral displacements occur at strata interfaces and within the immediate roof beam where discrete fractures have formed. These lateral differential displacements have been observed up to 15m (50 feet) into the roof at strata changes particularly the clay/salt contacts. The largest shifts are found at the clay/salt contact below the Anhydrite "b" layer. Lateral shifts can also be expected within the immediate roof beam where fractures form. The support system has been designed to accommodate a lateral shift of 12mm (.5 inch) per year and bed separation of 25mm (1 inch) per year at the clay/salt contact below the Anhydrite "b" layer once the bond at the interface is disrupted.

4.1.5 ANCHOR HORIZON

The zone in which rock anchors were to be installed had to satisfy three main criteria:

- It had to be relatively stable, in that the creep deformations occurring in it should be low.
- The effect of installing anchors in it should not induce loading conditions that would increase the creep deformations significantly or reduce the overall strata stability at that horizon.
- Penetration of roof stratigraphy should be kept to a minimum.

The reasons for going into the zone above Anhydrite "b" can be summarized as follows:

- Rock mechanics data from extensometers, inclinometers and borehole surveys have shown that the zone above Anhydrite "b" is relatively stable. This was in part due to the fact that the well-defined clay seam associated with Anhydrite "b" served to concentrate differential stresses in the immediate roof rock below Anhydrite "b". The reason for this phenomenon arose from the inability of the clay seam to sustain shear stress.
- Large numbers of 3.04 m (10 feet) mechanically anchored rock bolts had been installed at a fairly precisely defined horizon, some .91m (3 feet) above Anhydrite "b". After more than two years of installation, during which time the rock bolts were known to have developed load, there was no evidence from rock mechanics measurements of roof strata deformations that this concentration of anchoring load was causing any separation to occur.
- Sandia have a requirement that Anhydrite "a" should not be penetrated if this can be avoided.

- No separation has been observed to date across the Anhydrite "a" layer.

- The difficulties involved in drilling, reaming, and installing the grout anchored rockbolts increase rapidly with depth. In order to meet the required design requirements with a high degree of confidence the anchor horizon chosen is that above Anhydrite "b".

4.2 CONCEPTUAL DESIGN

Room 1 of Panel 1 is currently five years old and must remain accessible for an additional seven years in order to support the Bin Scale Testing program without interruption. In order to extend the life of Room 1, a supplementary roof support system has been developed to minimize the possibility of any roof fall during testing.

On February 4, 1991, a substantial section of roof fell in Room 1 of the SPDV area. Geotechanical instrumentation had indicated accelerated room closure rates for some time, and the roof fall had been anticipated for several months. At that time, the room had been open for almost eight years. The four SPDV rooms had been mined to exactly the same dimensions as the waste storage (and disposal) rooms 91.4m (300 feet) long, 10.05m (33 feet) wide, 3.96m (13 feet) high in Panel 1, in order to simulate, monitor, and study their behavior in response to lithostatic (overburden) pressure over time. No ground support, such as rock bolts, had been installed.

In response to the roof fall in SPDV Room 1 and to assess the long-term stability of Panel 1, Westinghouse convened a panel of geotechnical experts in April 1991. The final report of this panel was released on June 5, 1991. The expert panel agreed that the WIPP geotechnical monitoring program as used in the SPDV Test Rooms is an adequate tool for giving early warning of deteriorating conditions in Panel 1. Based on collected geotechnical monitoring data, panel members concluded that 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 3.04m (10 feet) long mechanically anchored rock bolts. Mining of Room 1, Panel 1, began during the second half of 1986. Therefore, as of July 1991, the remaining life of Room 1 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. In order to carry out the Bin Scale Tests, a solution to the support problem had to be found to extend the required life of Panel 1 for up to seven years. The expert panel suggested alternative actions which included use of the following:

- The use of full column resin or resin anchor bolts;

- Grout anchored cable with lacing and wire mesh;

- Slotting and/or relief entries;

- Yielding support;

- Rely on currently installed support and upgrade when necessary;

- Roof trusses;
- Mining new rooms.

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.

The WIPP project has evaluated the support systems suggested by the Geotechnical Expert Panel.

The initial evaluations looked at support systems (wooden cribs, wooden cribs with steel beams) that could be installed within the room and would provide a passive support as the rock deforms into the room. These systems were eventually abandoned because they interfered with the functional use of the room largely as a result of the physical size of the supports. They limited the number of bins that could be placed in the rooms and more importantly, the support could not be placed where it was most needed (i.e., midspan where the largest loads are developing) without eliminating equipment access to the bin locations.

Another form, a yielding support system, was then considered. This eliminated any need for bin removal and provided a more uniform support of the roof strata. The yielding system consisted of deep grout anchored rock bolts supporting a steel cross beam with supplemental support being provided by yielding steel columns. The beam anchor bolts were designed so that their loads could be continually monitored and adjusted to accommodate room deformation by lowering the beam. As the design process became more detailed, it became clear that the major share of the load was carried by the beam support bolts; the yielding columns were in fact unnecessary and were therefore eliminated from the design. The beam itself was also modified from an initial I-beam to a more structurally convenient inverted channel section. This eliminated complex attachment plate structures needed for the I-beam suspension system. Any roof rock spalling between the steel sets is contained by a network of steel wire rope lacing underneath a mat of steel meshing and expanded metal.

As designed, roof support for Room 1, Panel 1, is providing the following:

- Progressive support of the detaching triangular wedge of roof rock as it develops.
- Containment of the detaching wedge of roof rock and safe control of the rate at which the detaching section moves downward based on the creep rate produced by the roof strata above.
- Accommodation of lateral movements in the roof strata above.

Throughout the design process, a conservative approach was used wherever possible. The design is largely based on the room deformations and subsequent roof fall that took place in SPDV Test Room 1, which is seen as a worst case scenario. Previous resin grout anchor tests have shown that an eighteen inch bond length of grout would be sufficient, whereas a minimum .91m (3 feet) bond length is used in this design. The rock bolt design is based on the manufacturer's minimum yield strength of

209,000N (47.4 kips), whereas the destructive tests carried out in Room 2, Panel 2 gave actual minimum yield strengths 22% to 26% higher. A continuous channel section beam is used where individual plates would have been sufficient. Very little support capability has been assigned to the meshing and lacing, whereas it is certain that this will be capable of a considerable load carrying capacity. No strength has been assigned to the wedge-shaped rock salt beam formed as a result of roof fracture formation. It will have an inherent strength, this effect being enhanced by the existing 3.04m (10 foot) mechanically anchored rockbolts as well as the steel meshing and lacing. The existing system of 3.04m (10 foot) mechanically anchored rockbolts, installed on a 1.2m (4 foot) by 1.5m (5 foot) spacing have considerable load bearing capability which has been disregarded in the design.

The net effect of all the above factors when added together means that the design is very conservative, thus reducing the risk of potential failures.

4.3 DESIGN DESCRIPTION

The yielding roof support system for Room 1 of Panel 1 is designed to contain and support the detaching load while allowing it to be lowered. The system is not designed in any way to prevent the creep of rock into the room. The roof support consists of 27 steel channel support sets installed laterally across the room on 2.37m (7.8 feet) to 3.04m (10 foot) centers. The actual location of the channel sets will be determined in the field during their installation based on the location of the existing roof bolts which are installed on a 1.2m (4 foot) by 1.5m (5 foot) pattern. This is to minimize interference with the existing roof bolts.

Each support set is secured by 11 Dwyidag steel tendons (anchor bolts) that are 4.0m (13 feet) long. The resin grouted anchor bolts are anchored in between the Anhydrite "a" and the Anhydrite "b" horizons. The channel support anchor bolts are designed so the load of each bolt can be monitored and adjusted to accommodate continuous roof deformation. System adjustment is accomplished by keeping the tension on each anchor bolt within the design limits which are calculated to support the detaching load. Once the tension in the anchor bolt reaches the design limits, the bolt load is then relieved. Each anchor bolt extends .46m (18 inches) below the roof to accommodate downward movement of the roof due to creep. Roof strata extension measurements have shown that the anchor bolts will have to accommodate approximately 38mm (1.5 inches) of movement per year. To allow for differences in lateral deformations, each tendon is located in an oversized .08m (3 inch) diameter hole extending from the hole collar to the Anhydrite "b" clay horizon.

The roof area between the channel sets is covered by a network of steel wire lacing cables underneath a mat of steel wire mesh and expanded metal. This mat is held in place by the channel support sets. Its function is to contain any rock spalling in between the channel support sets.

4.3.1 STEEL CHANNELS

Each roof support set consists of a structural steel channel placed with the web flat against the roof, running across the room (rib to rib) and 11 rock anchor bolts. The set is designed to accommodate the triangularly distributed wedge load, which has a weight of 8,000N (1,800 pounds) per foot near the ribs and 45,150N (10,150 pounds) per foot in the middle of the room. Eleven rock anchor bolts are required to support the channel.

The support channel is a 15 x 40 channel section 8.2m (27 feet) long. Each channel set consists of three 2.7m (9 foot) long sections connected together with four .19m (7.5 inch) by .08m (3 inch) splice plates. Two of these splice plates are welded to each end of the section center. The other channel is then connected by a .02m (.625 inch) bolt passing through the un-welded end of the plate and through the flange. The plates are placed on the outside of each flange and have a .04m (1.5 inch) long slot in which the bolt passes through; this allows for a small horizontal movement in each of the sections without affecting its performance. The channel has been divided in to three equal sections for ease of transport and installation.

The rock anchors are fastened through the centerline of each channel. The anchors are spaced every .61m (2 feet) in the middle of the room and every .76m (2.5 feet) or .91m (3 feet) near the ribs. This accommodates the distribution of the load which is greater in the middle of the room than near the ribs. Each rock anchor passes through a .04m (1.5 inch) diameter hole in the channel which allows for a .006m (.25 inch) tolerance in placing the anchors in the salt.

Each rockbolt will be tested to ensure its quality of installation. These tests are described in section 6.1.2.

Detailed channel support calculations are given in Appendix "C".

4.3.2 ROCKBOLTS

The rock anchors that support the roof load are the most critical element of the design. The rockbolts are 4.0m (13 feet) long threaded No. 8 Dywidag rods with a 209,000N (47,400 pound) minimum yield strength. The rock anchors are inserted 3.5m (11.5 feet) into the roof and anchored between Anhydrite "a" and Anhydrite "b" horizons. The rock anchors are anchored by resin which bonds at least .91m (3 feet) of the bolt to the salt using one .84m (33 inch) long resin cartridge. Each rockbolt extends .46m (18 inches) below the roof to allow the back to expand downwards while supporting the load.

Since the pattern of the detaching load is uneven, the design tension in each rockbolt is different, ranging from 44,400N (10,000 pounds) for the bolts near the rib to 85,400N (19,200 pounds) for the bolts in the middle of the room. Each anchor is to be monitored to ensure that the nuts are relieved before the tension rises above the designed limit. Further details regarding the design of the rockbolts are provided in Appendix "C".

4.3.3 WIRE MESH AND LACING

A mat of wire mesh and lacing is installed between the channel sets. The primary function of the mesh and lacing is to keep small pieces of salt from falling down from the roof. The lacing is made of .02m (.625 inch) diameter wire ropes placed not more than .91m (3 feet) apart in both the longitudinal and transverse directions. Above this, a layer of .10 x .10m (4 inch x 4 inch) welded wire mesh is placed. And lastly, a 10 gauge small aperture expanded metal is placed against the salt.

The transverse wire lacing ropes are appoximately 9.1m (30 feet) long and extend from rib to rib. Each rope is anchored in place by 2.4m (8 feet) resin anchored rockbolt. For ease of channel installation, the wire mesh and expanded metal are temporarily attached to the existing 3.0m (10 foot) roof bolts and/or Hilti bolts.

4.3.4 RIB SUPPORT

The fracturing which is occurring in the ribs along potential failure planes, is similar to those in the roof. However, based on experience elsewhere at the WIPP, this fracturing is not expected to result in any serious stability problems over the anticipated working life of Room 1. Small scale flaking and peeling off of small pieces of rock will take place, these being expected to have a nuisance value as well as posing something of a threat to installed piping and cabling.

Wire mesh is therefore extended from the end of the channels in the roof, around the corners, and down the rib to a height of approximately 2.13m (7 feet) above floor level. This extra wire meshing is pinned to the wall with 1.2m (4 feet) mechanical rock bolts. The mesh and bolts do not prevent the flaking from occurring but contain it and allow subsequent removal and maintenance. This is standard practice in the WIPP underground to accommodate rib spalling.

4.4 DESIGN CALCULATIONS

Detailed design calculation for the Room 1 of Panel 1 roof support system are included in Appendix "C".

5.0 DESIGN IMPLEMENTATION

5.1 SURVEYING AND MARKING

The locations of currently installed rock bolts, and instrumentation equipment that may interfere with support installation has been accurately surveyed and is shown on a map of Room 1 (Drawing No. 54-D-003-W1 and 54-D-003-W2). These drawings will be used to locate the positions of the channel support sets and their associated rock anchors. Roof profiles will be determined at each channel support set location. Once the channel set locations have been plotted on the room map, they will be transferred underground and marked on the roof and ribs of the room. Individual rock anchor holes will then be accurately marked to determine precise locations of the anchor bolts. Experience has shown that a minimum accuracy of $\pm$ .063m (.25 inch) will be achieved while marking the positions of the boreholes.

5.2 RELOCATION OF INSTRUMENTATION, CABLES AND PIPING

Any damage to the existing instrumentation installations consisting of electric cables, gas piping, distribution boxes and lighting fixtures must be avoided during installation of the support system. For that reason, the suspended cabling will be moved to the side of the room and attached to the rib, and the piping will be lowered. Relocation of these fixtures will not affect the testing and monitoring program. Conveyor belting suspended from the rib will also be used to provide additional protection to the existing equipment.

5.3 SYSTEM INSTALLATION

5.3.1 ROCK DRILLING

5.3.1.1 STEEL CHANNEL ANCHOR BOLTS

Each channel set has been designed for 11 bolt anchor holes which are to be installed vertically. It is required that all bolt anchor holes are drilled to a specified depth of 3.55m (11.5 feet), measured from the hole collar. An allowable vertical alignment tolerance of 2 degrees and a depth tolerance of .025m (1 inch) are expected. Deviations of anchor bolt holes are to be addressed on a hole-by-hole basis so that the distance apart at hole ends will not be less than 20 inches. In order to achieve the specified vertical tolerance, collaring alignment holes are required. The holes shall be drilled as accurately as possible to vertical and to a minimum depth of .076m (3 inches). After completing the collar alignment drilling, a .035m (1.375 inch) diameter pilot holes of the correct length will be drilled. These will then be reamed to .076m (3 inch) diameter to a depth of approximately 2.13m (7 feet). To ensure that there is an acceptable annulus in the anchorage zone, all .035m (1.375 inch) $\pm$0.030 diameter finishing bits will be properly gauged. Drawing No. 54-D-003-W2 provides details of the drilled hole.

5.3.1.2 CABLE LACING ANCHORS

The support system requires that lacing across the room width and length be terminated with resin grouted rock anchors and be provided with means for tensioning the cables. Lacing cable termination anchors will consist of a truss plate fastened to the roof by a .016m (5/8 inch) diameter and 2.4m (8 feet) long resin grouted bar. An adjustable eyebolt will pass through a truss plate and the lacing cable will be terminated through this eyebolt as shown on Drawing No. 54-5-003-W5. The termination anchor holes will be drilled at 45° angle to a depth of 2.4m (8 feet) with drilling accuracy of $\pm$.025m (1 inch). Drilling detail for lacing holes anchors are shown on Drawing No. 54-5-0003-W4.

5.3.1.3 RIB SUPPORT ANCHORS

The rib anchors are installed to hold the chainlink wire mesh against the rib from the top corner down to approximately 2.13m (7 feet) above the floor. The chainlink meshing is installed to support small pieces of loose or broken salt. Chainlink meshing is currently used at the WIPP as a standard practice for rib maintenance underground. The rib anchors are standard 1.22m (4 feet) long mechanical anchors. Rib support anchor holes are drilled on approximately 1.52m (5 foot) pattern and are 1.22m (4 feet) deep to accommodate the anchors.

5.3.1.4 RECOVERY OPERATIONS

In the event that drilling tolerances or anchorage capacities are not met, the hole depth will be extended an additional .91m (3 feet). The anchor will be reinstalled at this depth.

5.3.2 INSTALLATION SEQUENCE

5.3.2.1 STEEL ANCHOR BOLTS

Installation of the roof support system in Room 1, Panel 1, will commence with drilling of the steel channel anchor holes and installation of Dywidag anchor bolts.

Correct installation of the Dywidag anchors is critical to the whole support system. The minimum bond length of .91m (3 feet) is required in order to maintain design load capacity of the support system. This has been confirmed by the destructive tests described in Appendix B of this report. A minimum of .457m (18 inches) of the anchor bolt measured from the hole collar will protrude from the mouth of the drill hole. Both the Dywidag tendons and the resin will be installed according to the manufacturers specifications. Following anchor installation, each bolt will be quality checked to confirm its anchorage capacity. Details regarding quality control testing are included in Section 6.1.2.

5.3.2.2 WIRE MESH AND LACING

The wire mesh and lacing will be installed after testing of the anchor bolts is completed and advanced enough to provide adequate working space for wire mesh hanging operations. This would allow for simultaneous installation of wire mesh, cable lacing, and drilling of anchor holes. The wire mesh system consists of two layers, a layer of .10m x .10m (4 inch x 4 inch) welded wire mesh and another layer of expanded metal which is installed directly against the salt and above the welded wire mesh layer.

Both layers are attached to the existing 3.04m (10 foot) rock bolt plates in order to provide temporary suspension until the lacing and channel supports are finally installed. Drawing No. 54-D-003-W5 shows the installation detail of the meshing and lacing. The wire lacing rope will be doubled back through one of the eyebolts attached to the anchor plate, and the free end clamped with three crosby clamps. The loose end will then be passed through the opposite eyebolt and snug tensioned by means of a come-along and clamped with three crosby clamps. The transverse lacing will be installed before the longitudinal ropes.

5.3.2.3 STEEL CHANNEL SETS

The .381m (15 inch) by 177.92N (40 pound) channel steel set will be installed in three 2.74m (9 foot) long sections with the flanges down. Each section will be joined in place by the splice plates located along the flanges. An .356m (14 inch) wide by 2.743m (9 foot) long by .0191m (.75 inch) thick treated plywood gasket will be placed on top of the channel with .038m (1.5 inch) holes drilled to coincide with the hole pattern of the channel. The plywood gasket will first be fitted over the anchor bolt ends, thus forcing them to be correctly aligned before installation of the steel channel is attempted. The steel channel sections will be installed next, by passing the protruding Dywidag anchor ends through the .0381m (1.5 inch) pre-drilled holes. The final step will be the installation of the fastening nut assemblies, together with their associated plates and load cells. Drawing No. 54-D-003-W3 shows the channel support details. A setting load of 4448.22N (1000 pound) will be applied to the anchors. Steel spacers will be used to ensure contact at the rock anchor positions.

5.3.2.4 RIB SUPPORT

Installation of the rib support system will commence with drilling of the rib anchor holes followed by hanging of the

chainlink mesh together with anchor installation. Approved current WIPP installation procedures for rib bolting will be followed.

Since the rib support system is intended for protection against small scale flaking and peeling of small rock which are of nuisance value, its installation sequence will depend more on convenience than on a rigid schedule.

6.0 TESTING AND MONITORING

The most important component of the roof support system described herein is the rock anchor system. After each Dywidag rock anchor is grouted, both in the mock-up test as well as during actual Room 1 installations, the rock anchors will be subjected to performance tests. The testing will be carried out using as a guideline the specifications laid down in "Recommendations For Prestressed Rock And Soil Anchors", 1989, produced by the Post-Tensioning Institute of Phoenix, Arizona.

Monitoring of the loads on the rock anchors will be done during the test phase as well as prior to the actual installations. The information will be used to determine when and by how much the loads should be adjusted in order to keep pace with the deformation of the salt rock into the room. In addition to monitoring rock anchor loads, deformations in and around the room as well as deflections of the supporting channels will also be monitored. These measurements will enable a clear picture of the room stability to be obtained.

6.1 TESTING

6.1.1 MOCK-UP TEST

A mock-up test will be performed in Room 2 of Panel 1 and will include installation of five complete channel sets. The objectives of the mock-up tests are as follows:

- Provide information necessary to evaluate existing equipment;
- Establish practical and safe installation procedures;
- Install and test monitoring equipment;
- Check the performance of the overall system as well as individual components;
- Establish procedures for rock anchor performance tests, and ensure that personnel are proficient in the use of these tests.

6.1.2 QUALITY CONTROL TESTING

The purpose of Quality Control testing is to ensure that every rock anchor installed is capable of handling at least 1.1 times the maximum design load. Because of the large number of bolts (approximately 300) to be installed, these tests have to be done as quickly as possible.

After a rock anchor has been installed for a minimum of 8 hours, it will be loaded to 26 kips (1.1 times the design load). The loading arrangement is shown in Drawing No. 54-D-003-W2. The load will be measured by the load cell and conveniently displayed during loading. After reaching 26 kips, the nut will be tightened and the loading ram removed. The load will be continually monitored for a minimum of 1 hour to check whether there is any loss of load due to creep. If there is a load loss, the load will be reapplied and monitored. This will be repeated until all "slack" in the

system has been removed, or the rock anchor installation is deemed to be unsatisfactory. Recovery operations as detailed in Section 5.3.1.4 may then be instituted. The above procedure may be modified based on field experience.

6.1.3 DESTRUCTIVE TESTS

The main component of the support system described in this design document is the rock anchor system.

The primary emphasis of the anchorage system testing program is to guard against the most probable modes of movement that may lead to its failure. The importance of field validation tests of the anchorage system has been recognized as essential to the success of the whole support system.

The rock anchor system was tested by loading correctly installed bolts to destruction. These destructive tests determined that the anchorage capacity of the No. 8 Dywidag rockbolts tested in Room 2 of Panel 1, are equal or greater than guaranteed manufacturers specification of (47.4 KIPS) yield load or (71.1 KIPS) of ultimate load used for support system design. Details of the tests and the results obtained are given in Appendix B.

6.2 MONITORING

Since the support system will be adjusted during its operational life, it is essential to ensure that the loads on the anchors do not exceed the working loads specified by the design. The two main parts of the monitoring program will be observations of room performance and of support performance.

Room stability will be determined from data that will establish the rock mechanics performance of the excavations in terms of room closure, rock deformations in and around rooms, the development of fractures and bed separation at strata interfaces.

The support system performance will be determined from tests that will provide input data from field tests for the design, from tests to prove quality during installation, and from a program that will monitor loads that develop in the rock anchors and on the lacing during the working life of the support. The evaluation of the rock mechanics data characterizing room performance and of the support performance data will establish the effectiveness of the support system. A description of the Geomechanical Monitoring Program including specifications for the instruments is given in Appendix "D".

6.2.1 GEOTECHNICAL MONITORING

Geomechanical instrumentation can adequately establish the performance of excavations at the WIPP and provide adequate warning of deteriorating conditions. This has been demonstrated by the early warnings provided in SPDV Test Room 1 prior to its roof fall, and was confirmed by the views expressed by the Geotechnical Expert Panel convened to establish an estimate of the life of Panel 1 (US DOE, 1991 91-023). The geomechanical

instrumentation for Room 1, Panel 1, has been upgraded based on a monitoring program presented to the Geotechnical Expert Panel (US DOE, 1991 91-023). The basis of the revised monitoring program is:

- The measurement of deformations across the Anhydrite "a" and "b" layers in the roof in order to assess the development of bed separations at these strata interfaces.

- The measurement of room closure in order to assess the development of closure rates that exceed bounding levels and to establish the development of assymmetric room closure that may be an indication of fracture development along one rib and rotation of the roof slab. These measurements will be made by convergence measurements of roof/floor and wall/wall closure.

- The observation of conditions in the roof in observation boreholes in order to establish the extent of fracturing and bed separation.

- The measurement of lateral deformations within the pillars to establish the competency of the pillars. These measurements will be made by means of borehole extensometers.

6.2.2 SUPPORT SYSTEM MONITORING

Monitoring of the support system under working conditions in the field is an integral element to ensure its successful performance. The monitoring program consists of measurement of:

- The load that develops in each rock anchor. This provides the basis for adjusting the tension on the anchor so that the load build-up does not exceed the design limits of 1.1 times the design load while accommodating the continued movements of the salt. The load will be measured by means of load cells located at the anchor nut.

- The load that develops on the lacing and mesh. This will be evaluated over selected lengths of the room. The load will be measured by means of hydraulic flat jacks located at cross-over points of the lacing.

- The extension of the cables due to the development of the load due to the detaching wedge. The cable deformation will be measured by means of a calibrated standard length and dial indicator.

- The deflection of the channel support arising from the action of the rock anchor supports and the load transferred by the lacing and meshing. The deflection will be measured by precise surveying techniques.

The purpose of the monitoring of lacing and meshing loads and extensions is to gather information for later analysis. Initially, the monitoring will be carried out daily but this frequency will be adjusted as data becomes

available on the load changes with time. The measurement frequency will be based on observing load changes equivalent to 2 percent of maximum working load. The measurements of the loads will be compared with the criteria presented in Appendix "D" to establish when loads in the rock anchors must be adjusted, and the extent of that adjustment:

It should be noted that these criteria are preliminary. Field tests and analytical computations will be performed in order to more effectively define these criteria and the method of load adjustment that will be based on them.

The criteria and the adjustment to the loads will be reviewed as data becomes available and may be changed to be more effective. The process by which these factors become adjusted will require approval by the manager of Engineering for the Managing and Operating Contractor with concurrence from the managers of Operations and Safety.

6.2.3 INSTRUMENTATION

Geomechanical Instrumentation installed in Room 1 of Panel 1 will include:

- Beam support rock bolt load cells - Rock bolt load cells will be used to monitor the axial loading on the rock bolts. Loads on the cells are measured by means of resistance strain gages bonded to the cell in a full bridge configuration. The load cells are capable of monitoring loads of up to 444,800N (50 tons) with an approximate instrument sensitivity of 88.9N (20 pounds). In order to maximize the adjustment range of the support system, a low profile type cell will be used.

- Pressure cells or flat jacks - will be used to monitor loading on the cable lacing and mesh as a result of creep displacement. Pressure cells will be constructed of stainless steel and be capable of monitoring the range from 0 to 70 MPa (0 to 10,000 psi) with sensitivity of 0.4 per cent.

- Extensometers - Five borehole extensometers are installed in Panel 1, Room 1, to monitor rock mass deformation adjacent to the excavation. Three extensometers are installed in the roof to monitor possible bed separation withih the roof beam. These extensometers are installed along the centerline in the middle at approximately 1/4 length locations of the room. Horizontal extensometers are installed in each wall at the center of the room.

- Convergence points - Convergence measurements will be taken from installed convergence points throughout the room. These measurements are used to determine the amount and rate of closure at selected points. Monitoring of horizontal and vertical convergence will allow for a comparison of the performance of the support system in response to the actual room closure.

- Level survey - Changes in elevation will be monitored at selected locations along the support beams and the rock surface. The elevation surveys will identify areas of differential movement which, in addition to the results from installed geomechanical instrumentation, will establish the response of the system to creep closure.

Due to the large number of instruments, a data acquisition system will be installed. This system will be capable of monitoring up to 330 resistance strain gaged rock bolt load cells. The data loggers will be incorporated into the Geomechanical Instrumentation System which will allow for timely monitoring and reporting.

7.0 ADDITIONAL REQUIREMENTS

Quality of installation and monitoring is of key importance to the successful performance of the support system.

- In order to provide continuity to all the activities and assure future system performance a Project Control Group shall be assembled to oversee all installation and monitoring activities. This group shall include representatives from Mine Engineering, Geotechnical Engineering, Mine Operations, Quality Control, Safety and an outside project consultant.

- Collected monitoring data shall be routinely reviewed and results compared with design parameters.

- Periodic progress reveiws shall be conducted to evaluate system performance and to define possible changes to the system not forseen during design stage.

- Possible further testing might be required to define performance of individual components.

- Further steps shall be taken to develop and validate a mathematical or numerical model which would more easily describe the behavior of the support system.