App. D

ENTERED

DOE/WIPP 91-057
Revision 0

WIPP Library

Copied incorrectly
correct copy inserted
06/03/92
keep this a copy of
what was replaced
(see back)

Waste Isolation Pilot Plant Supplementary Roof Support System Underground Storage Area Panel 1, Room 1

October 1991

Waste Isolation Pilot Plant

911001
1091W:00002

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.

DOE CONTRACT NUMBER: DE-AC04-86AL31950

DISCLAIMER

This document was prepared as an account of work sponsored by an agency of the United States Government. Neither the United States Government nor any agency thereof, nor any of their employees, makes any warranty, express or implied, or assumes any legal liability or responsibility for the accuracy, completeness, or usefulness of any information, apparatus, product or process disclosed, or represents that its use would not infringe privately owned rights. References herein to any specific commercial product, process, or service by trade name, trademark, manufacturer, or otherwise, does not necessarily constitute or imply its endorsement, recommendation, or favoring by the United States Government or any agency thereof. The views and opinions of authors expressed herein do not necessarily state or reflect those of the United States Government or any agency thereof.

This document has been reproduced directly from the best possible copy. It is available to DOE and DOE contractors at the following address:

Office of Scientific and Technical Information
P.O. Box 62
Oak Ridge, TN 37831

Prices available from (615) 576-8401; FTS 626-8401

Available to the public from the
National Technical Information Service
U.S. Department of Commerce
5285 Port Royal Road
Springfield, VA 22161

791B:3942

WASTE ISOLATION PILOT PLANT
SUPPLEMENTARY ROOF SUPPORT SYSTEM
UNDERGROUND STORAGE AREA
PANEL 1, ROOM 1

Table of Contents

Section	Title	Page
1.0	EXECUTIVE SUMMARY	1-1
2.0	INTRODUCTION	2-1
3.0	DESIGN REQUIREMENTS	3-1
	3.1 Design Criteria/Considerations	3-1
	3.2 Design Specifications	3-1
	3.3 Design Bases	3-1
	3.3.1 General	3-1
	3.3.2 Rock Mechanics	3-2
	3.3.3 System Design	3-2
	3.3.4 Installation and Maintenance	3-3
	3.3.5 Testing and Monitoring	3-3
4.0	SYSTEM DESIGN	4-1
	4.1 Geology and Rock Mechanics	4-1
	4.1.1 Rock Mechanics	4-1
	4.1.2 Stratigraphy	4-3
	4.1.3 Vertical Movements	4-3
	4.1.4 Lateral Movements	4-3
	4.1.5 Anchor Horizon	4-6
	4.2 Design Evaluation	4-6
	4.3 Design Description	4-8
	4.3.1 Steel Channels	4-9
	4.3.2 Rockbolts	4-10
	4.3.3 Wire Mesh and Lacing	4-10
	4.3.4 Rib Support	4-10
	4.4 Design Calculations	4-10
5.0	DESIGN IMPLEMENTATION	5-1
	5.1 Surveying and Marking	5-1
	5.2 Relocation of Instrumentation, Cables and Piping	5-1
	5.3 System Installation	5-1
	5.3.1 Rock Drilling	5-1
	5.3.2 Installation Sequence	5-2

50

WASTE ISOLATION PILOT PLANT
SUPPLEMENTARY ROOF SUPPORT SYSTEM
UNDERGROUND STORAGE AREA
PANEL 1, ROOM 1

Table of Contents
(Continued)

Section	Title	Page
6.0	TESTING AND MONITORING	6-1
	6.1 Testing	6-1
	6.1.1 Mock-up Test	6-1
	6.1.2 Quality Control Testing	6-1
	6.1.3 Destructive Tests	6-2
	6.2 Monitoring	6-2
	6.2.1 Geotechnical Monitoring	6-2
	6.2.2 Support System Monitoring	6-3
	6.2.3 Instrumentation	6-4
7.0	ADDITIONAL REQUIREMENTS	7-1

WASTE ISOLATION PILOT PLANT
SUPPLEMENTARY ROOF SUPPORT SYSTEM
UNDERGROUND STORAGE AREA
PANEL 1, ROOM 1

Table of Contents
(Continued)

List of Appendices

Appendix	Title
A	Geology and Rock Mechanics
B	Destructive Tests
C	Support System Design
D	Geomechanical Monitoring Program

WASTE ISOLATION PILOT PLANT
SUPPLEMENTARY ROOF SUPPORT SYSTEM
UNDERGROUND STORAGE AREA
PANEL 1, ROOM 1

Table of Contents
(Continued)

List of Figures

Figure	Title	Page
1-1	Isometric View Room 1, Panel 1, Supplementary Roof Support System	1-4
2-1	Room 1, Panel 1, Roof Support System Design and Implementation	2-2
4-1	Kinematic Movement of the Rock Wedge	4-2
4-2	Generalized Site Stratigraphy	4-4
4-3	Stratigraphy of the WIPP Repository Horizon	4-5

WASTE ISOLATION PILOT PLANT
SUPPLEMENTARY ROOF SUPPORT SYSTEM
UNDERGROUND STORAGE AREA
PANEL 1, ROOM 1

Table of Contents
(Continued)

List of Drawing

Drawing Number	Title
54-D-003-W1	Supplementary Roof Support Room 1, Panel 1, General Arrangement
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
54-D-003-W5	Supplementary Roof Support Room 1, Panel 1, Wire Mesh and Lacing Detail

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.

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 1-1 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.

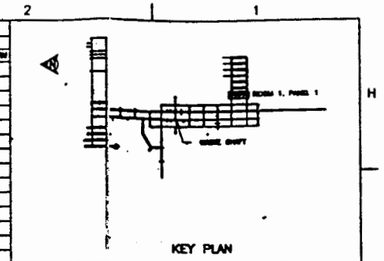
The WIPP is committed to safely providing long term roof support.

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:

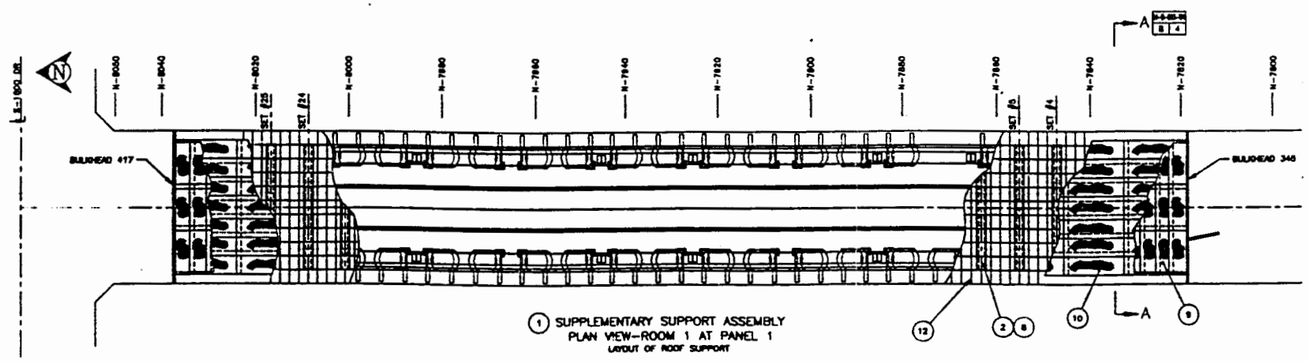
A. RYERSON OR APPROVED EQUAL
B. McMASTER-CARR OR APPROVED EQUAL
C. JOHNMAR CORP. OR APPROVED EQUAL
D. DYNIDAG OR APPROVED EQUAL
E. CELTITE OR APPROVED EQUAL
F. TEXAS LUMBER OR APPROVED EQUAL
G. RAYNELL OR APPROVED EQUAL
H. ACME FENCE OR APPROVED EQUAL



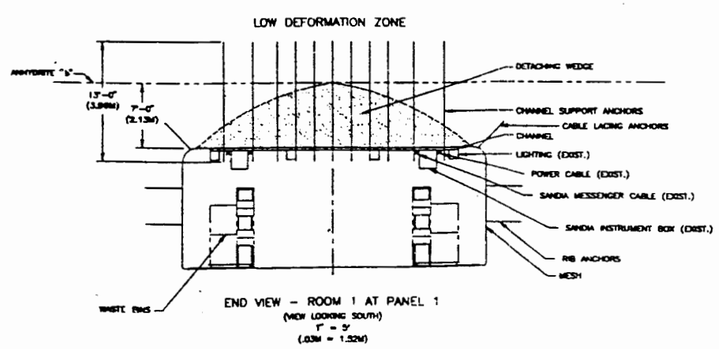
KEY PLAN

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



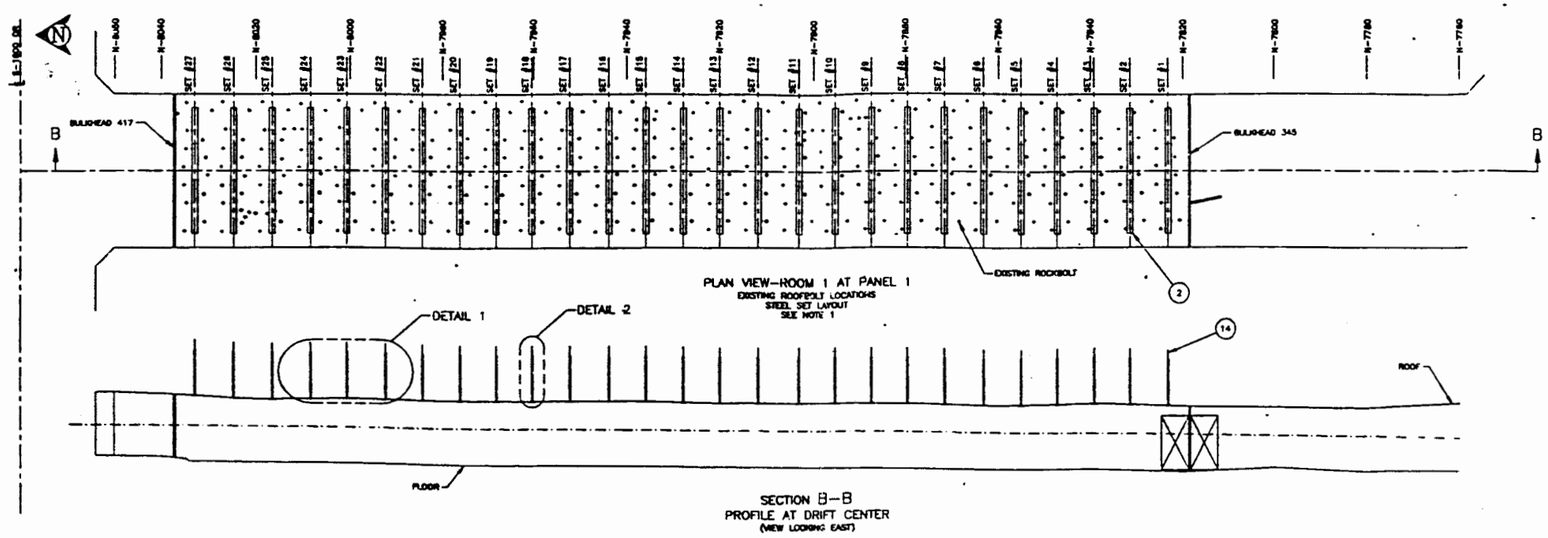
END VIEW - ROOM 1 AT PANEL 1
(VIEW LOOKING SOUTH)
1" = 5'
(.03M = 1.52M)

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

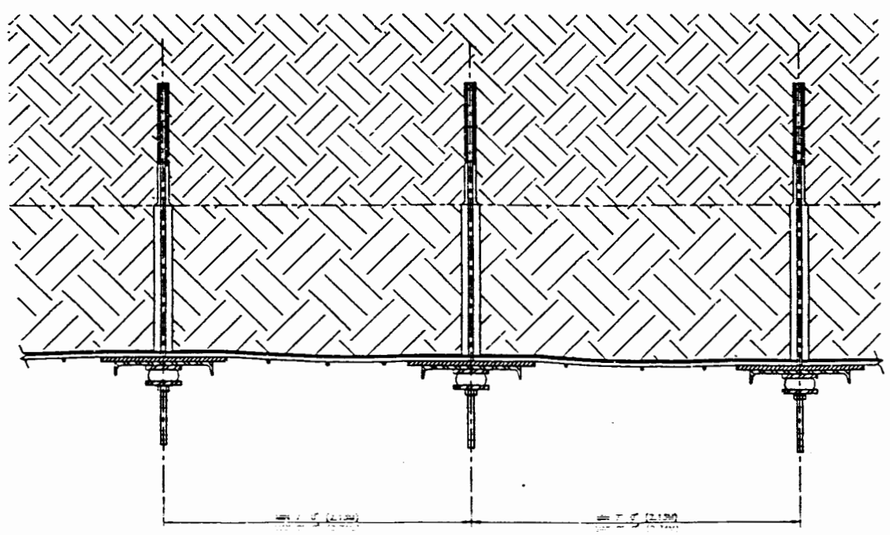
SUPPLEMENTARY ROOF SUPPORT
ROOM 1, PANEL 1
GENERAL ARRANGEMENT

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

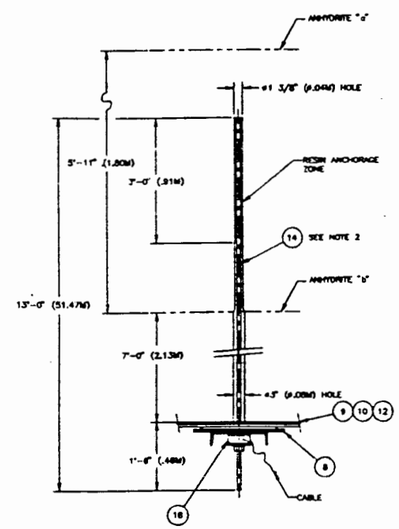


PLAN VIEW–ROOM 1 AT PANEL 1
EXISTING ROOFBOLT LOCATIONS
STEEL SET LAYOUT
SEE NOTE 1

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.

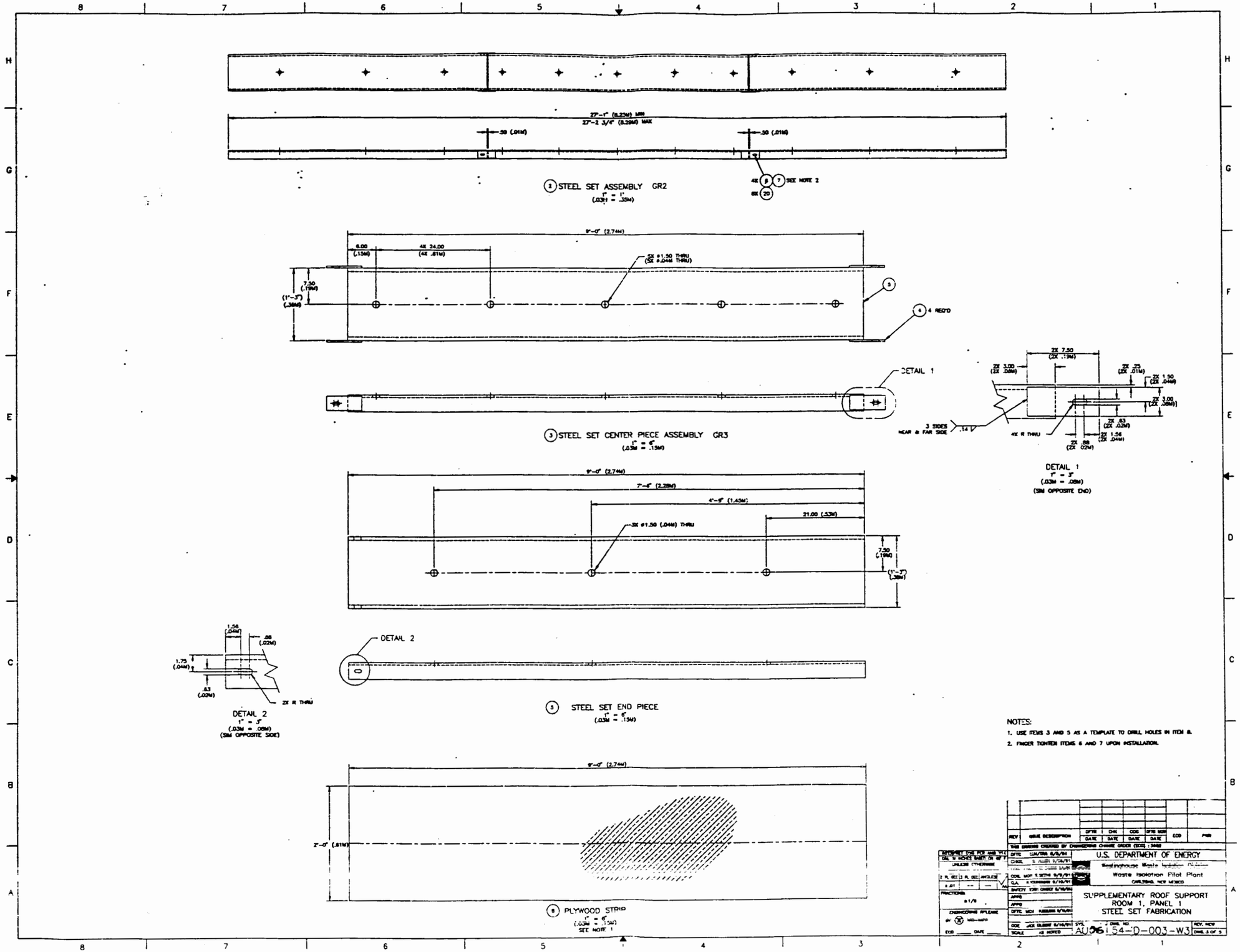
REV.	ISSUE DESCRIPTION	DFTR DATE	CHK DATE	CDS DATE	DFTR MGR DATE	ECO	PWR

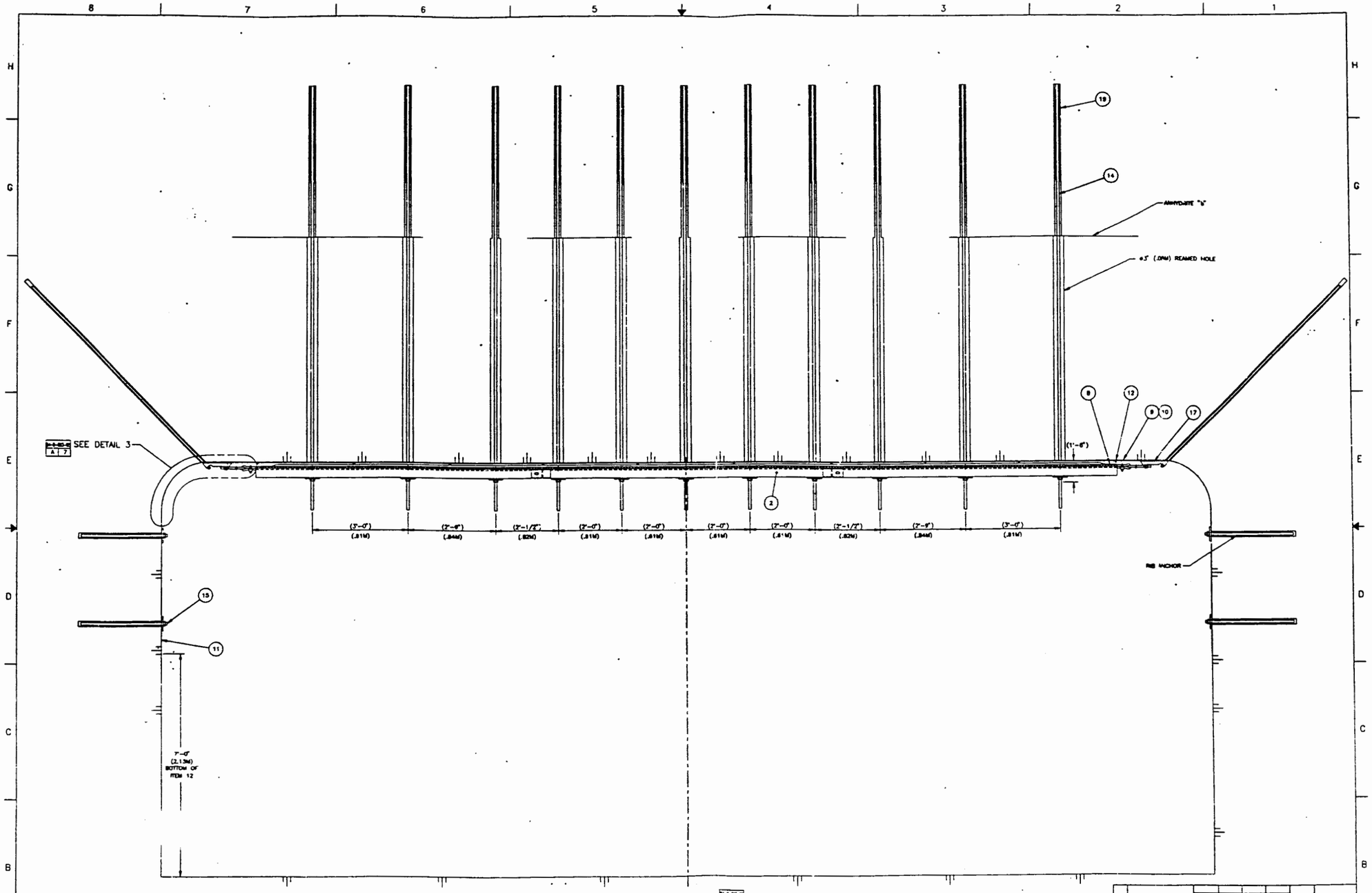
THIS DRAWING CREATED BY ENGINEERING CHANGE ORDER (ECO) : 5492

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"

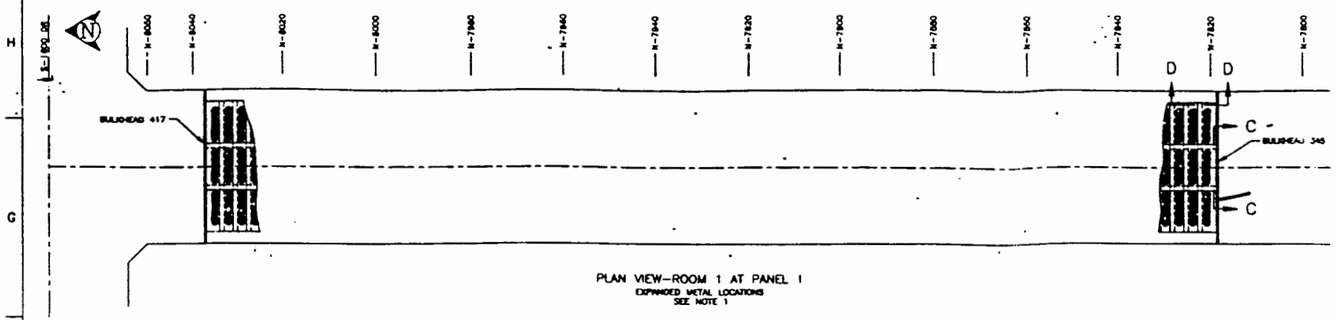
CHANGE/FINAL RELEASE
BY (W) WIO-SSMP
ECO ____ DATE ____

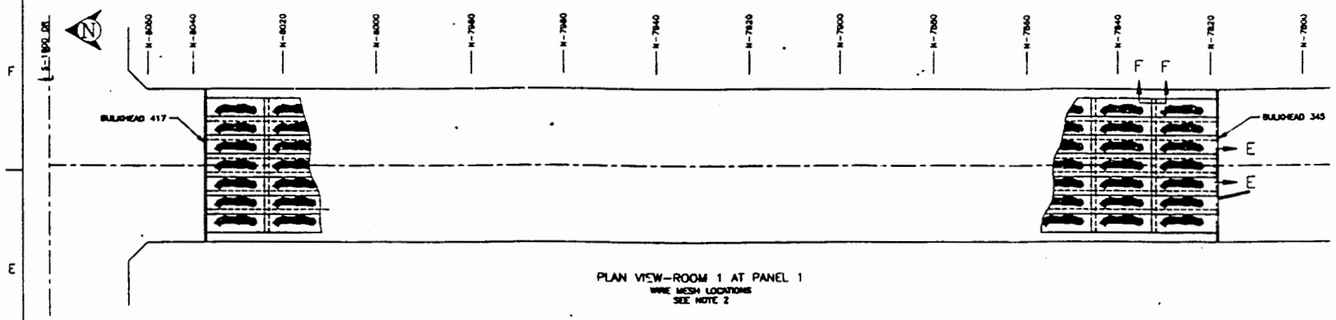
APPR
APPR
DFTG MGR
CDC MGR 9/10/91
SCALE

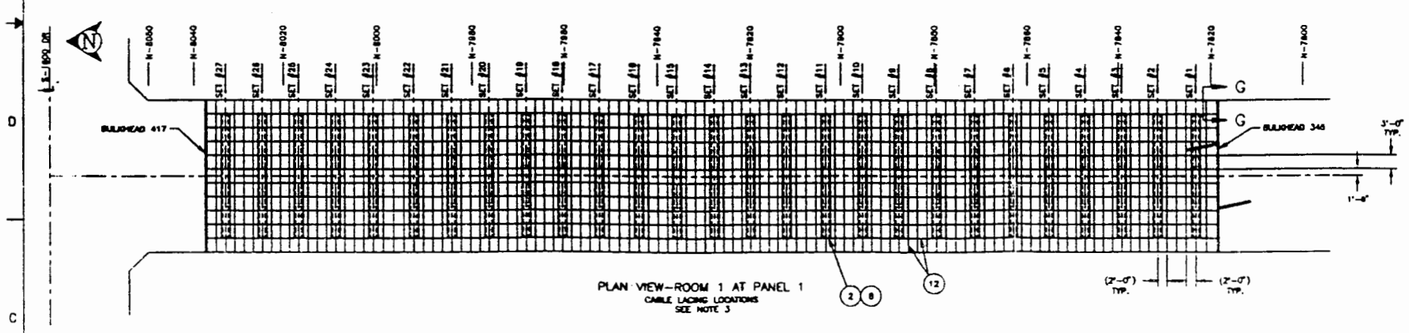
U.S. DEPARTMENT OF ENERGY
Westinghouse Waste Isolation Division
Waste Isolation Pilot Plant
CARLSBAD, NEW MEXICO

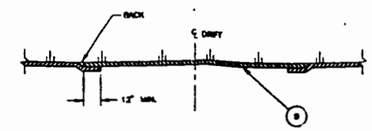
SUPPLEMENTARY ROOF SUPPORT
ROOM 1, PANEL 1
ROOM 1 CROSS SECTION

SYS AUC6 | DWG NO 54-D-003-W4 | REV NEW | DWG # OF #

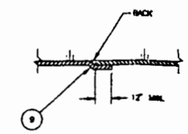




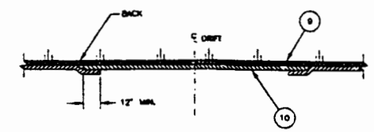




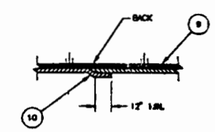
SECTION C-C
EXPANDED METAL OVERLAP
N.T.S.
SEE NOTE 4



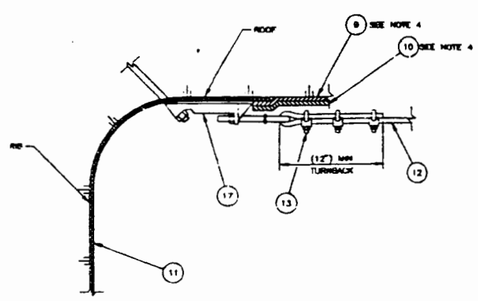
SECTION D-D
EXPANDED METAL OVERLAP
N.T.S.
SEE NOTE 4



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

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

TABLE OF CONTENTS

1.0 Introduction

1.1 Background

1.2 Design Methodology

2.0 Stratigraphy of Repository Horizon

3.0 Properties of Rocks at Repository Horizon

4.0 In Situ Stress Regime

5.0 Structural Response of Excavated Room

5.1 Roof Deformation

5.2 Rock Deformations About Room

5.3 Stresses Around Room

6.0 Performance Requirements for Support System

6.1 Performance

6.2 Geotechnical Design Basis for the Roof Support System for Room 1, Panel 1

6.2.1 Rockload To Be Supported

6.2.2 Vertical Movements

6.2.3 Lateral Movements

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

591L51698

SEP-26-91 THU 13:50 P.04

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.

SENT BY:Xerox Telecopier 7020 ; 9-26-91 ;12:58PM ; 5058858418→ 00000000;# 8

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.

REFERENCES

Baar C. A., 1977 "Applied Salt-Rock Mechanics 1, The In Situ Behavior of Salt Rocks", Elsevier Scientific Publishing Company

British Standards Institute, 1989 "British Standard Code of Practise For Ground Anchorages" BS8081: 1989

Cook, R. F. and W. M. Roggenthen, 1991, "Fracturing Around Excavations in Salt at the WIPP", U.S. Symposium on Rock Mechanics, Oklahoma City

Department of the Army Corps of Engineers, 1980, "Rock Reinforcement"

Greenwald, H. P. and H. C. Howarth, 1938, "Compression Tests of Roof-salt Slabs by Potash Salt Pillars", Report of Investigations, U. S. Bureau of Mines RI 3386
Habenicht H, 1983, "The anchoring effects - Our Present Knowledge and its Short Comings -A Keynote Lecture", Proceedings of the International Symposium on Rock Bolting, Abisko, Sweden edited by O. Stephansson, Balkema

Hansen F.D., "T Quasi-Static Compression and Creep Behavior of Bedded Salt from Southeastern New Mexico," SAND79-7045, prepared by Sandia National Laboratories.

Hansen F.d. and K.d. Mellegard, 1979, "Creep Behavior of Bedded Salt from Southeastern New Mexico at Elevated Temperature," SAND79-7030, prepared by Sandia National Laboratories.

Herrmann W., W.r. Wawesik, and H.S. Lawson, 1980, "Analysis of Steady State Creep of Southeastern New Mexico Bedded Salt," SAND80-0558, prepared by Sandia National Laboratories.

Herrmann W., W.R. Wawesik, and H.S. Lawson, 1980, "Creep Curves and Fitting Parameters for Southeastern New Mexico Bedded Salt," SAND80-0087, prepared by Sandia National Laboratories.

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

Kreig R.D., 1983, "Reference Stratigraphy and Rock Properties for the Waste Isolation Pilot Plant (WIPP) Project," SAND83-1908, prepared by Sandia National Laboratories.

Littlejohn G.S. and D.A. Bruce, 1975, "Rock Anchors State of the Art Part 1: Design", Ground Engineering.

Miller, H. D. S., 1991 Personal Communication

Molecke M. A., 1990, "Test Plan: WIPP Bin-Scale CH TRU Waste Tests", prepared by Sandia National Laboratories for the U.S. Department of Energy

Panek L., 1962, "The Effect of Suspension in Bolting Bedded Mine Roof", Report of Investigations, U.S. Bureau of Mines RI 6138

Post-Tensioning Institute, 1986 "Recommendations for Prestressed Rock and Soil Anchors

U.S. Department of Energy, 1986, The Design Validation Final Report prepared by Bechtel

U.S. Department of Energy, 1990, "The Geotechnical Field Data and Analysis Report, July 1988 to June 1989", DOE/WIPP 90-

U.S. Department of Energy, 1991c, "Geologic Mapping of the Air Intake Shaft at the Waste Isolation Pilot Plant", DOE/WIPP 90-051

U.S. Department of Energy, 1991, "The Geotechnical Field Data and Analysis Report July 1989 to June 1990", DOE/WIPP 91

U.S. Department of Energy, 1991 "Report of the Geotechnical Panel on the Effective Life of Rooms in Panel 1", DOE/WIPP 91-023

U.S. Department of Energy, 1989 "The Geotechnical Field Data and Analysis Report, July 1987 to June 1988" prepared by Westinghouse Electric Corporation, Waste Isolation Division

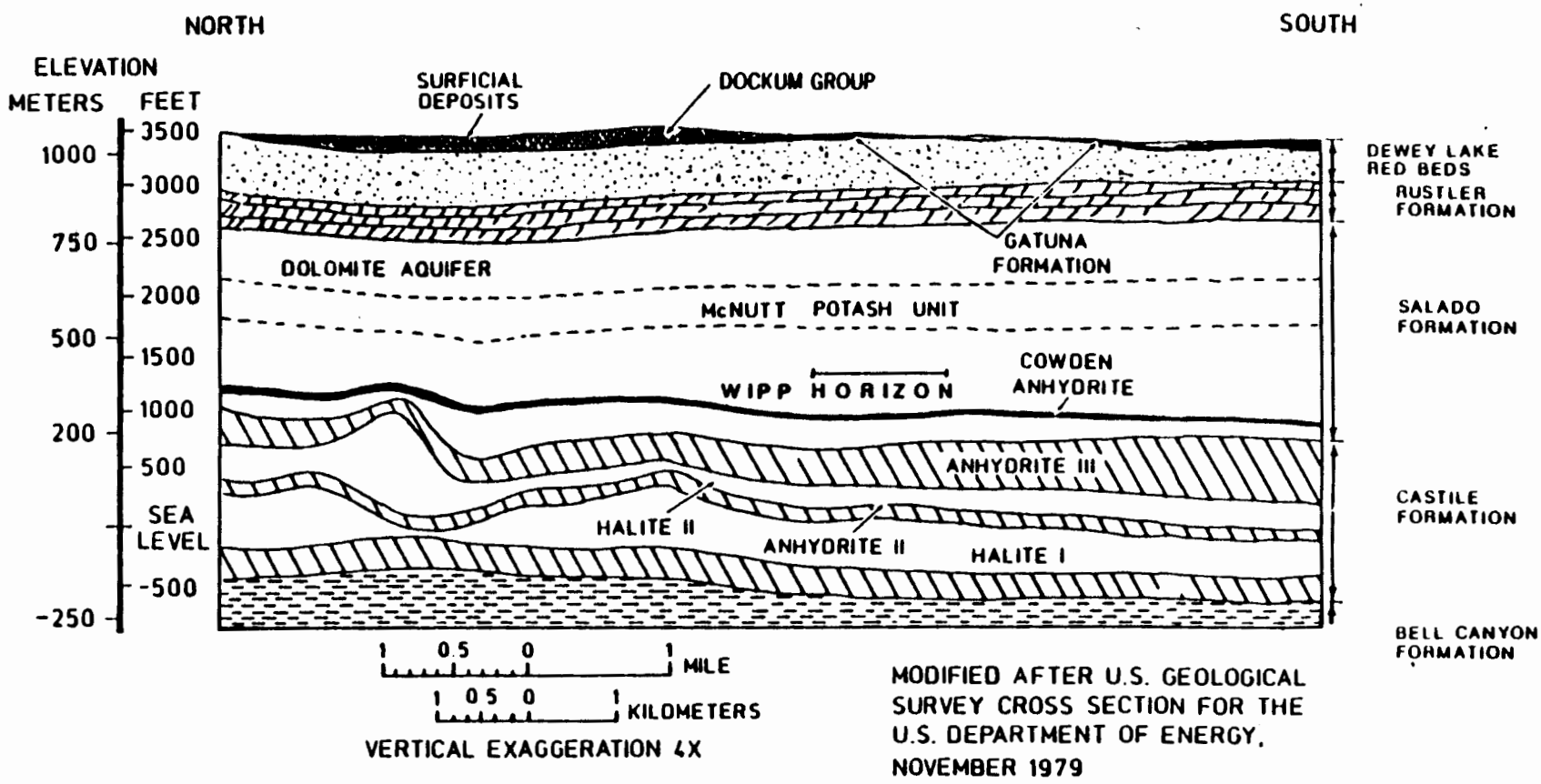


Figure 2.1 WIPP Site Stratigraphy

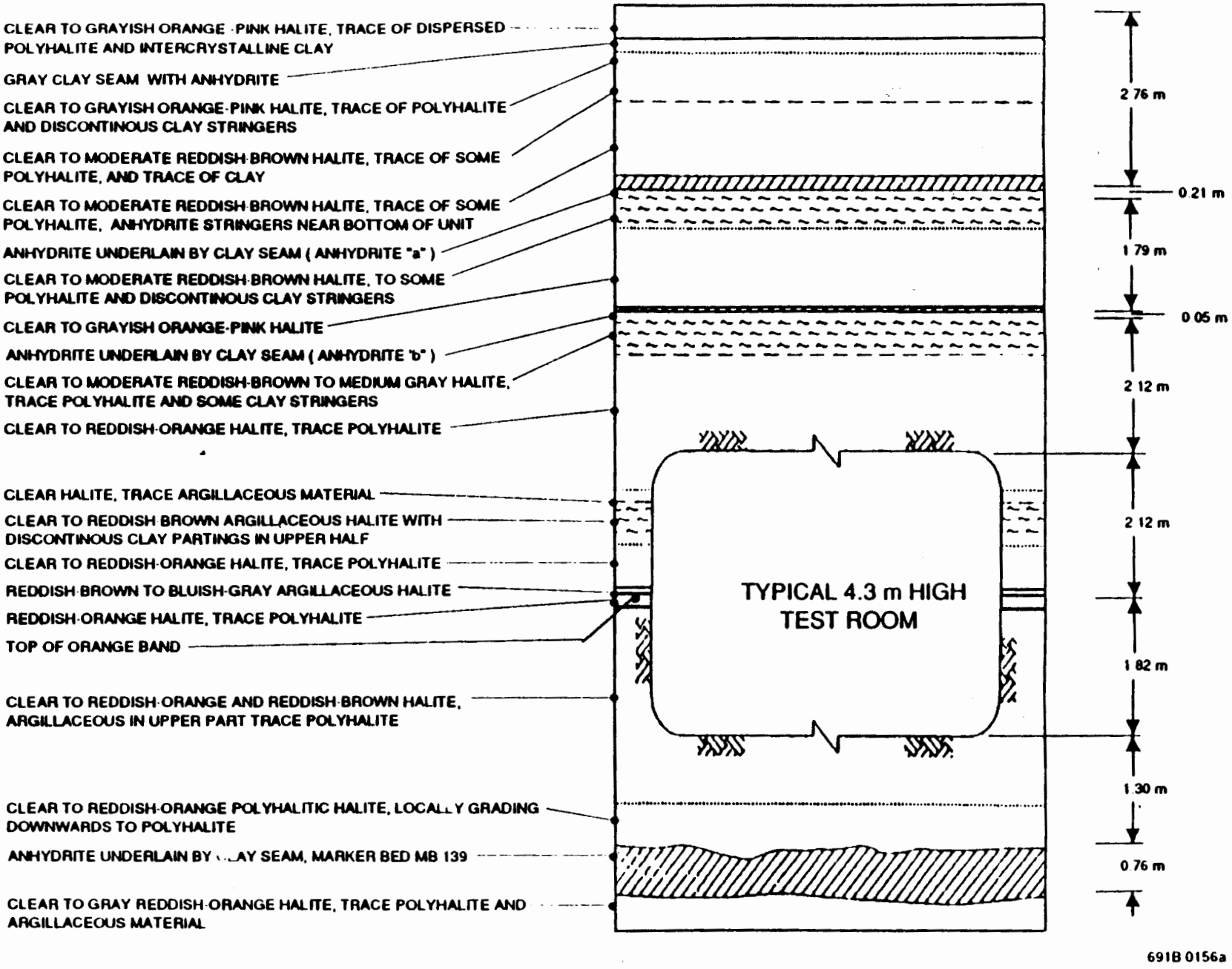


Figure 2.2 Stratigraphy of the WIPP Repository Horizon

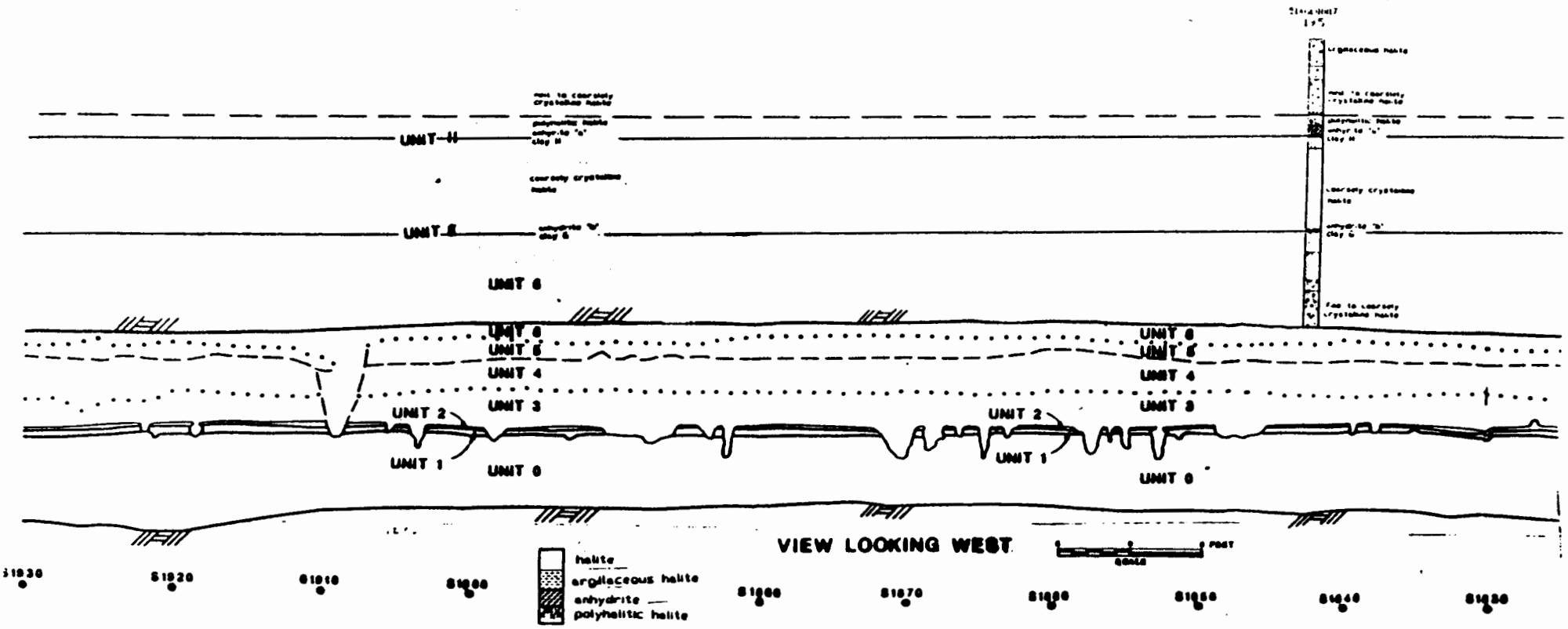


Figure 2.3a PANEL 1 - ROOM 1

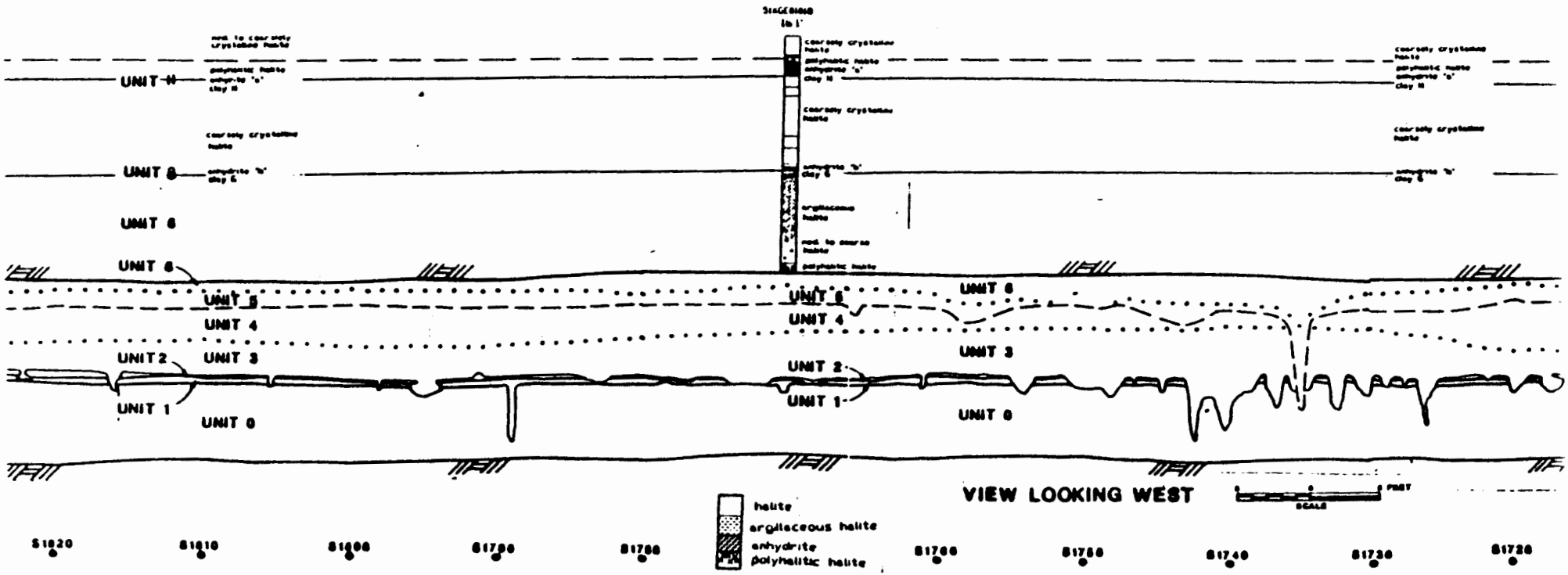


Figure 2.3b PANEL 1 - ROOM 1

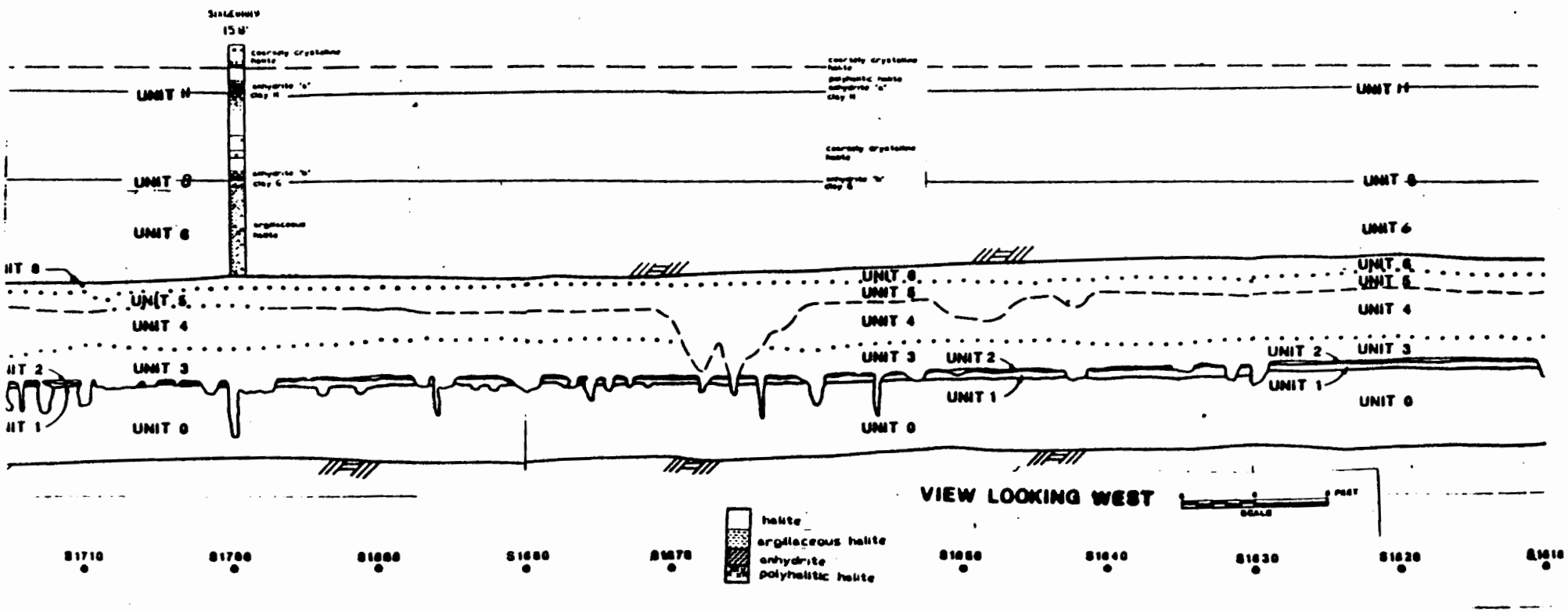
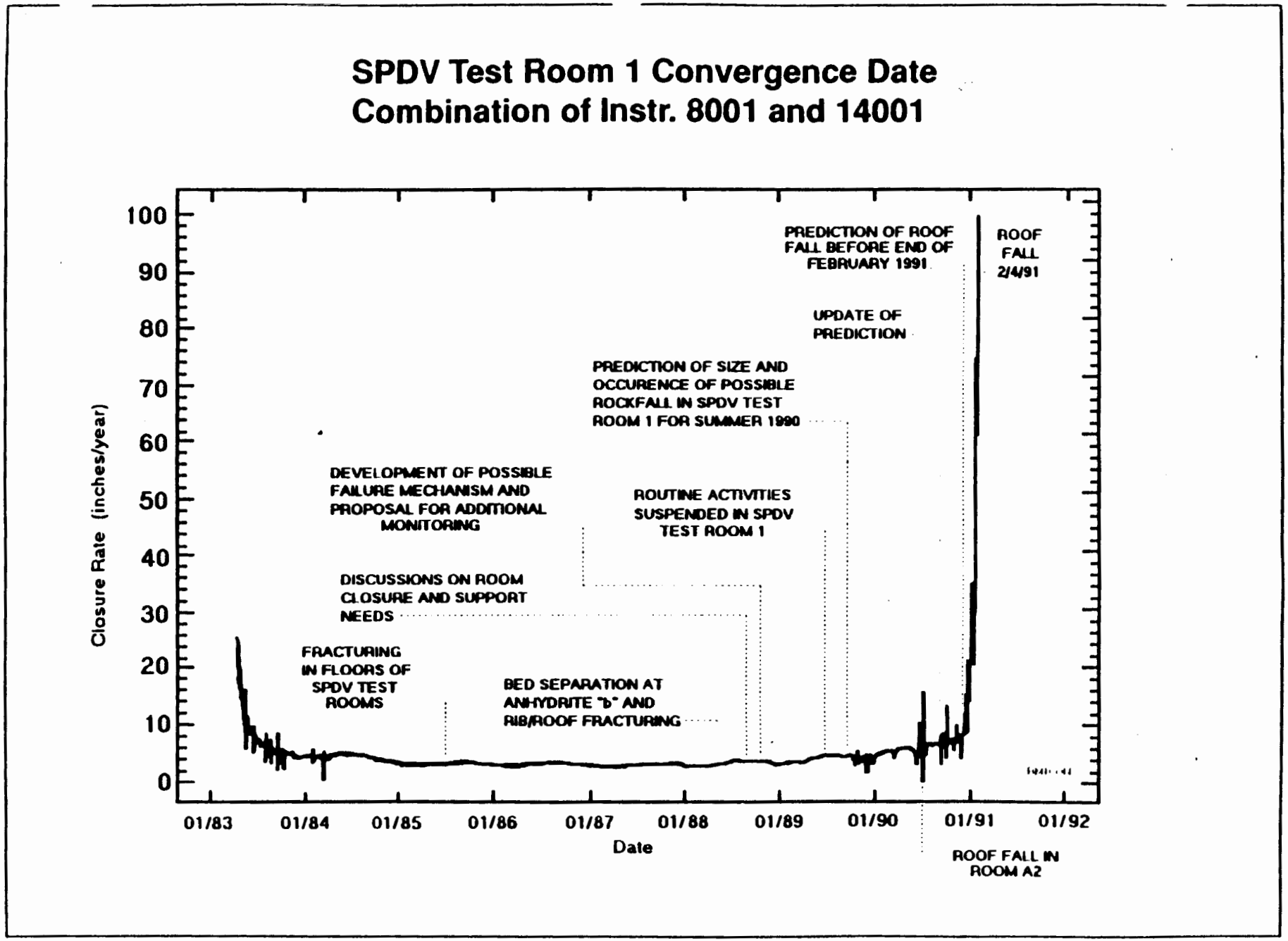
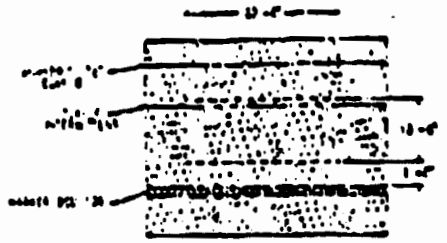


Figure 2.3c PANEL 1 - ROOM 1

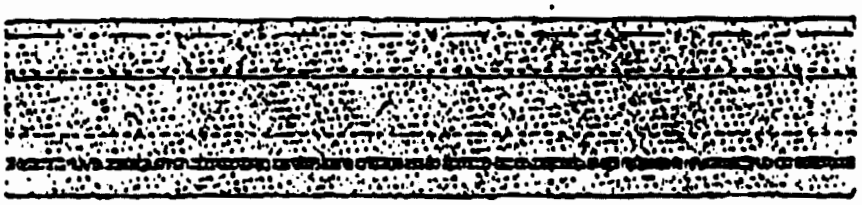


8918-0438

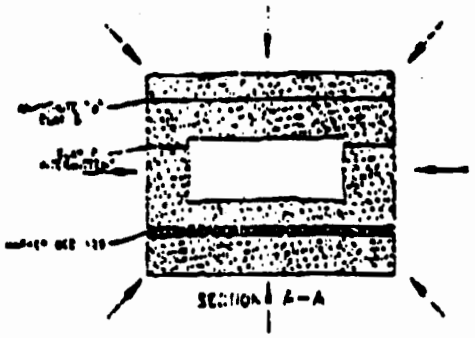
Figure 5.1



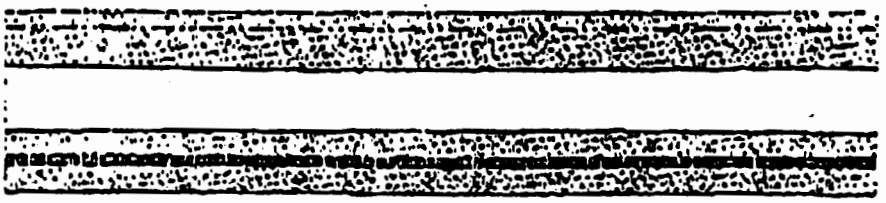
SECTION A-A



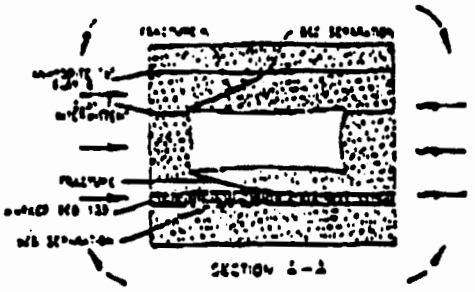
SECTION B-B



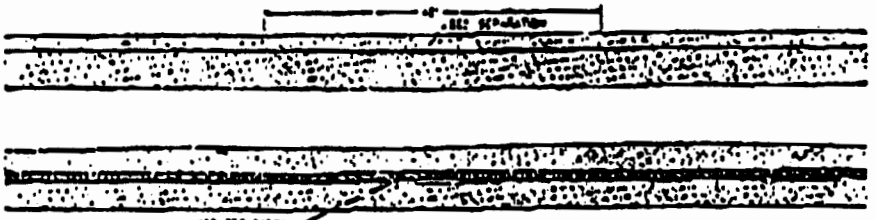
SECTION A-A



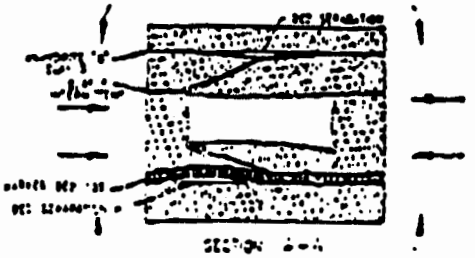
SECTION B-B



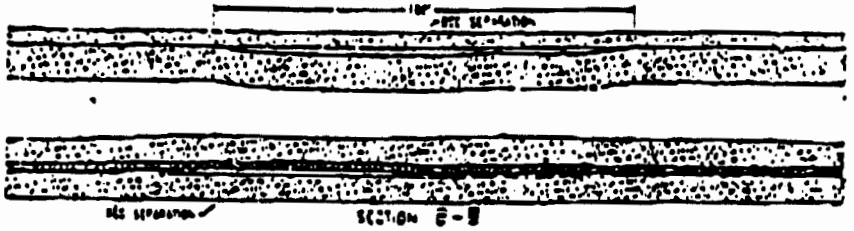
SECTION A-A



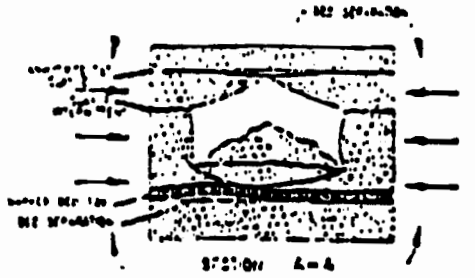
SECTION B-B



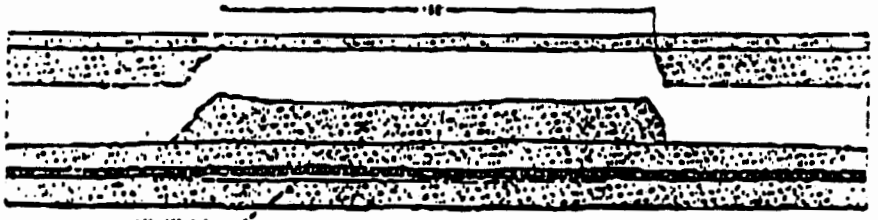
SECTION A-A



SECTION B-B



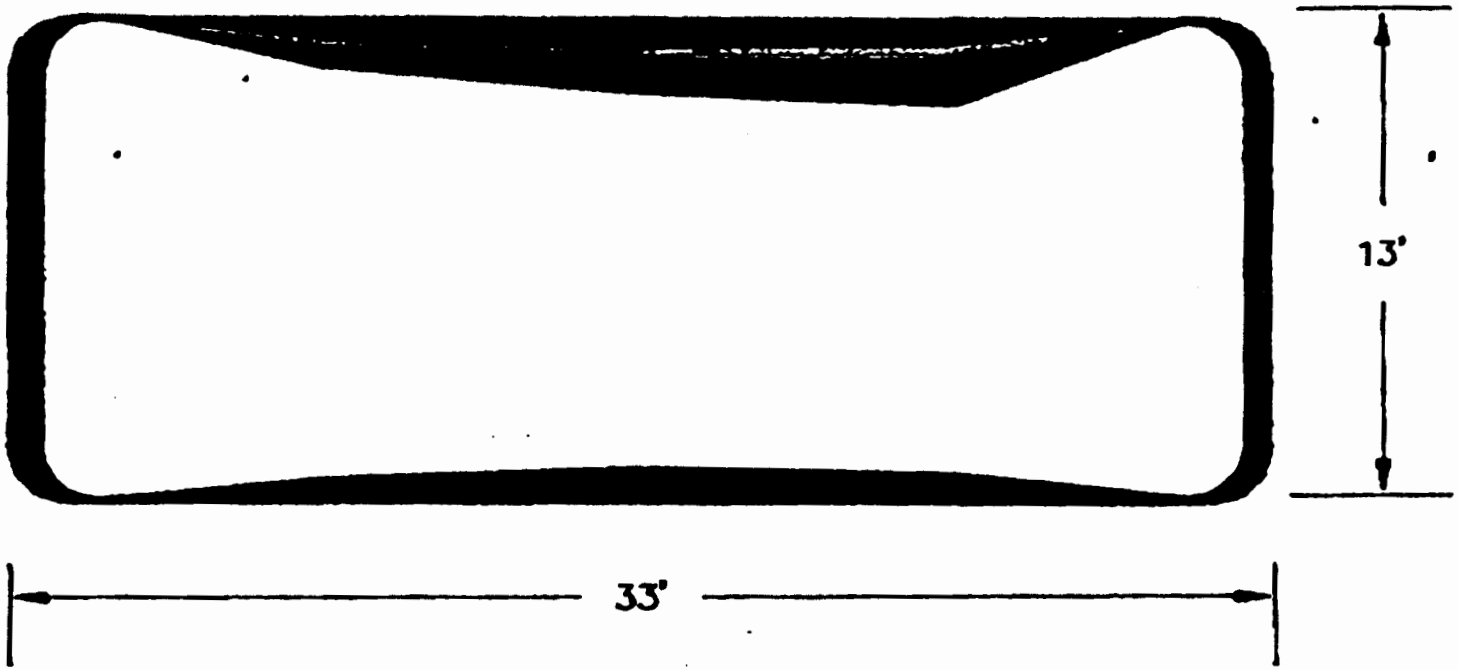
SECTION A-A



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

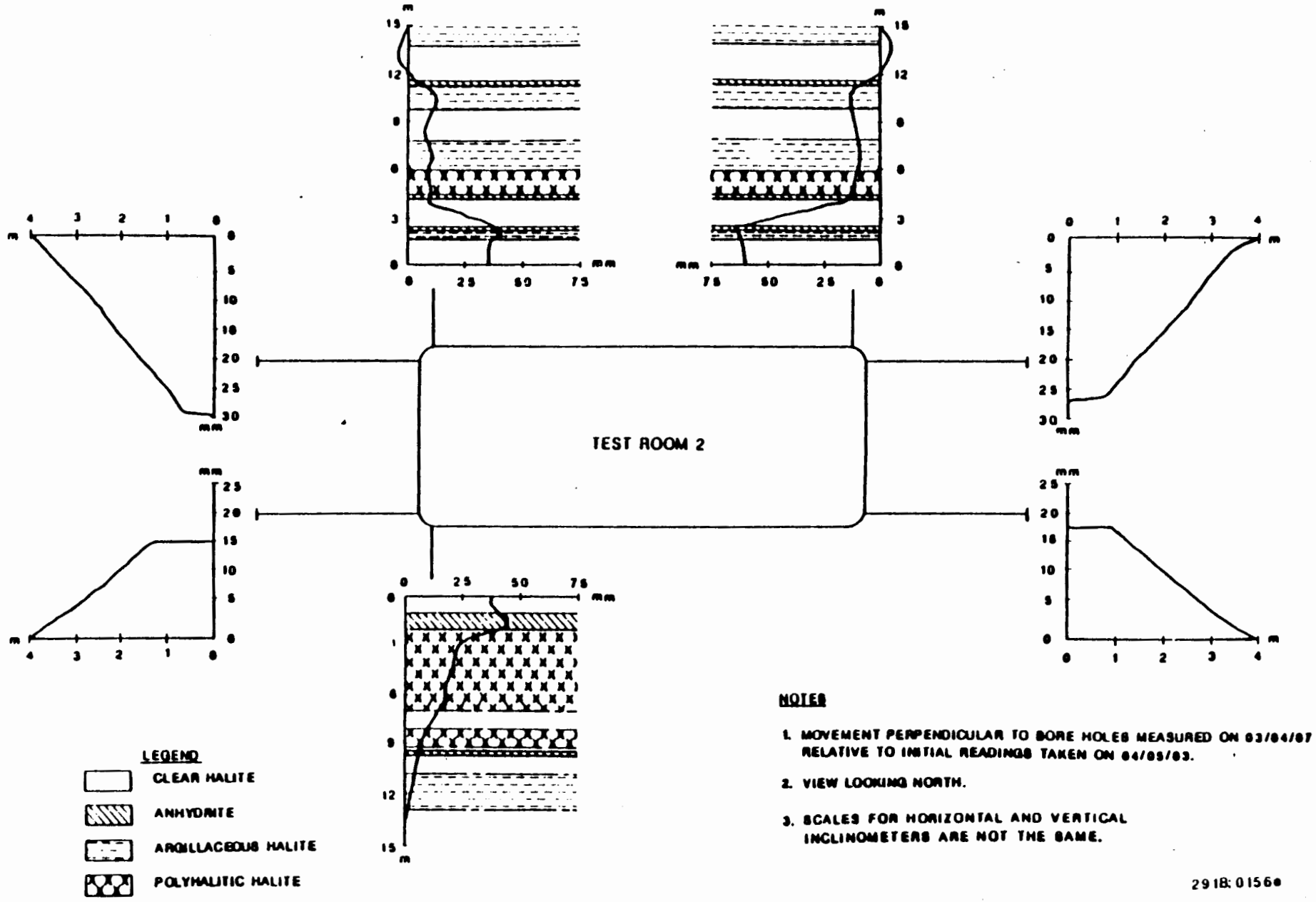


Figure 5.4 Inclinometer Data from SPDV Test Room 2

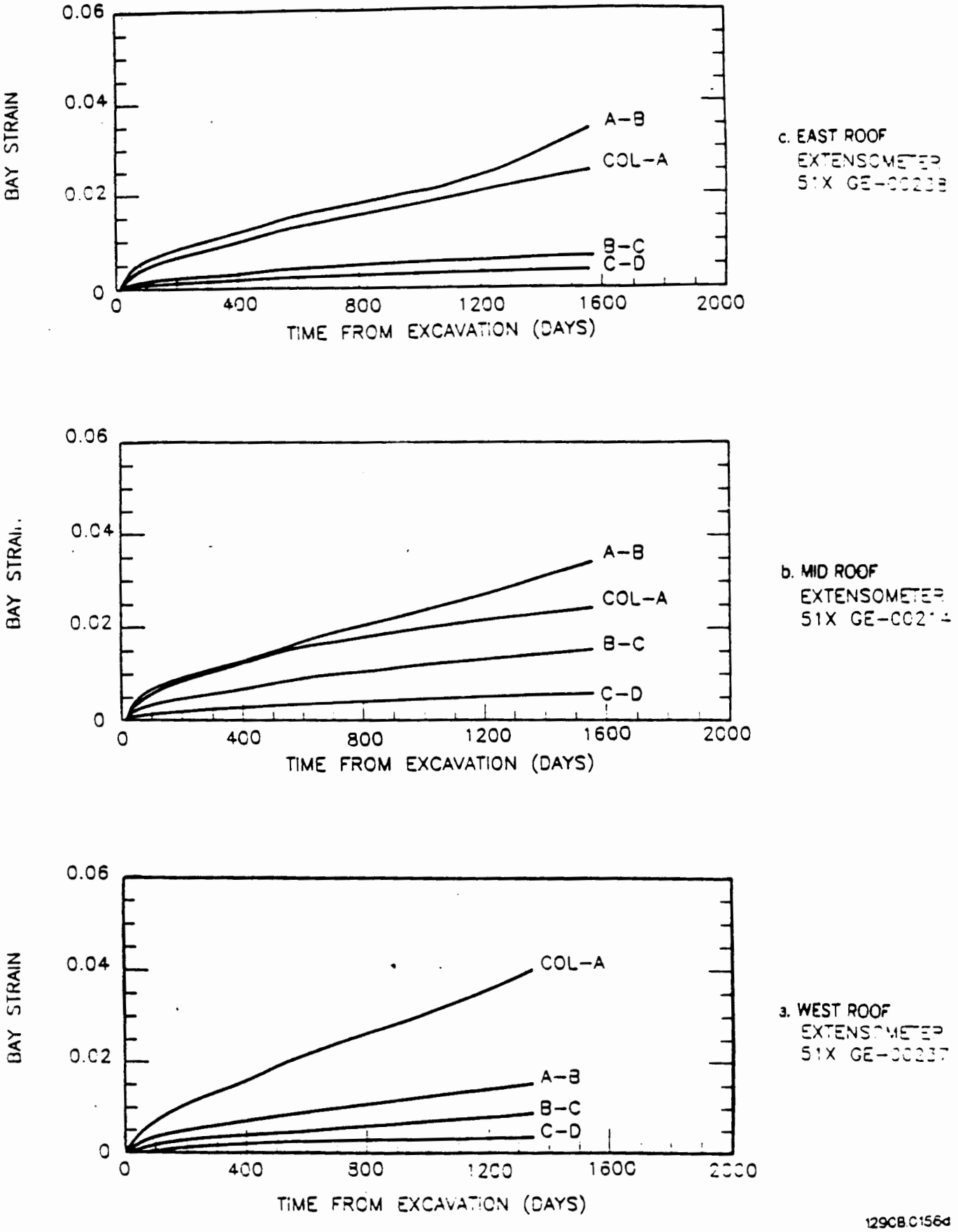
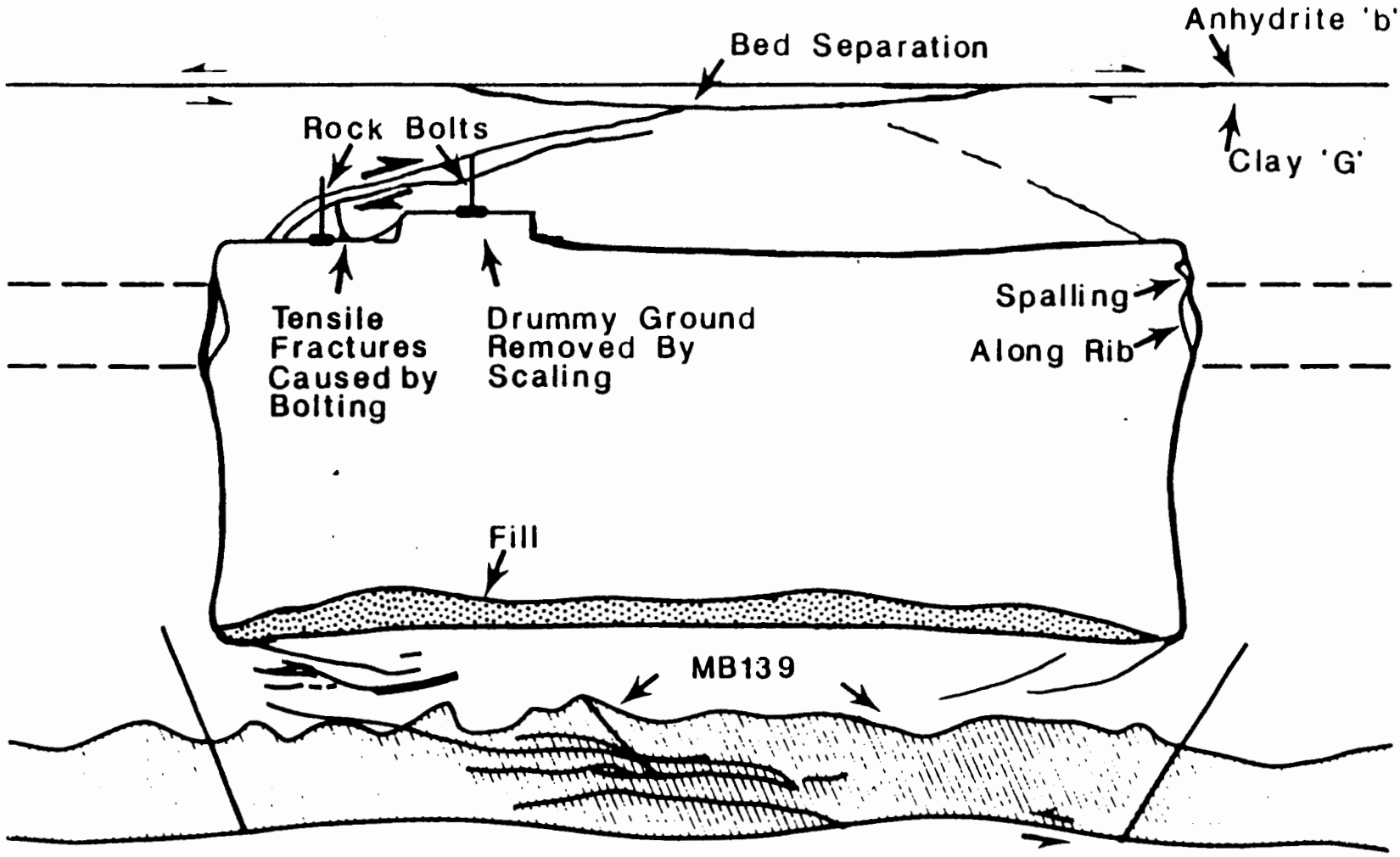


Figure 5.5 Extensometer Data from SPDV Test Rooms



291B: 0156b

Figure 5.6 Patterns of Fracturing (Idealized excavation effects modified from Borns & Stormont, 1987)

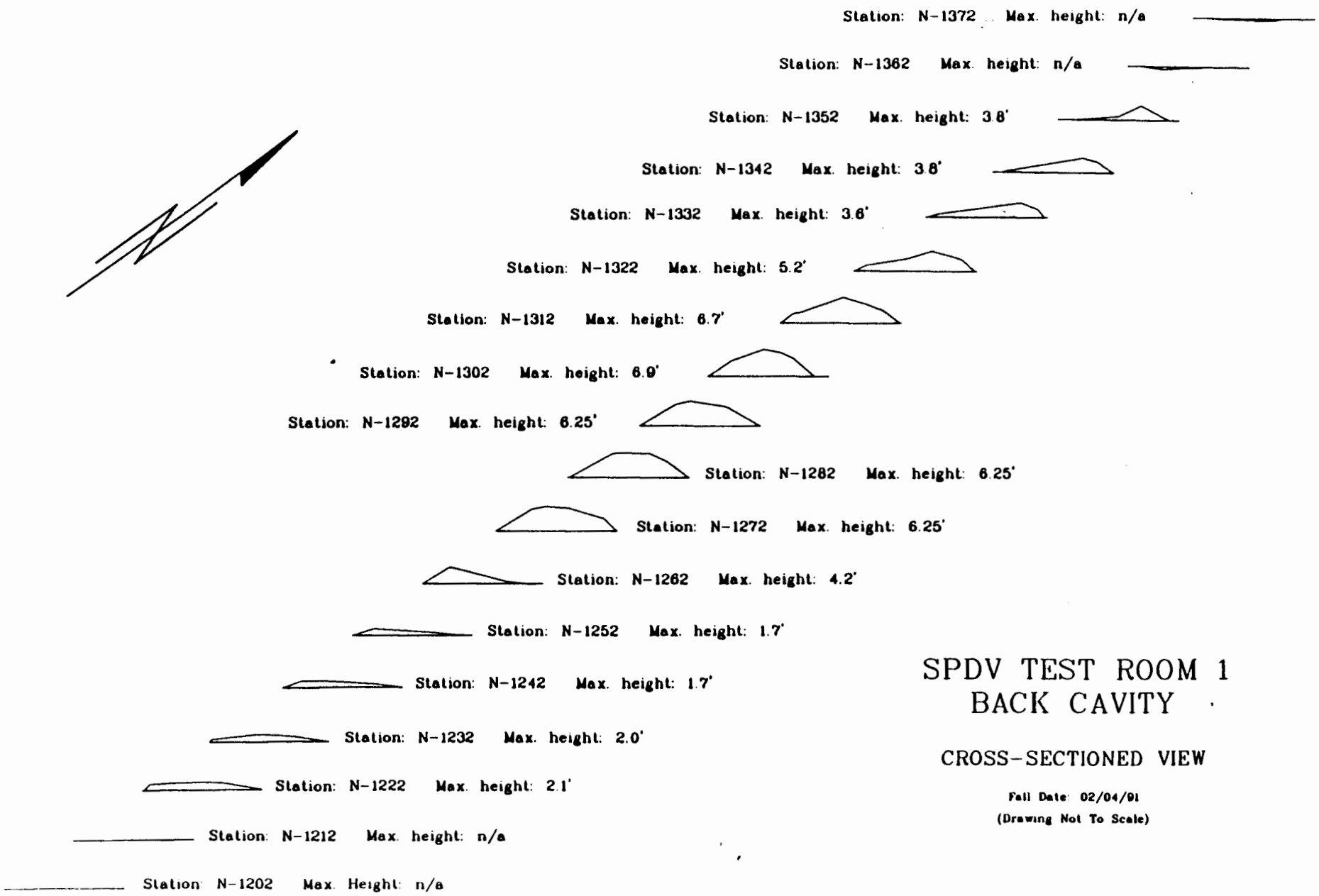

Station: N-1372 Max. height: n/a
Station: N-1362 Max. height: n/a
Station: N-1352 Max. height: 3.8'
Station: N-1342 Max. height: 3.8'
Station: N-1332 Max. height: 3.6'
Station: N-1322 Max. height: 5.2'
Station: N-1312 Max. height: 6.7'
Station: N-1302 Max. height: 6.9'
Station: N-1292 Max. height: 6.25'
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

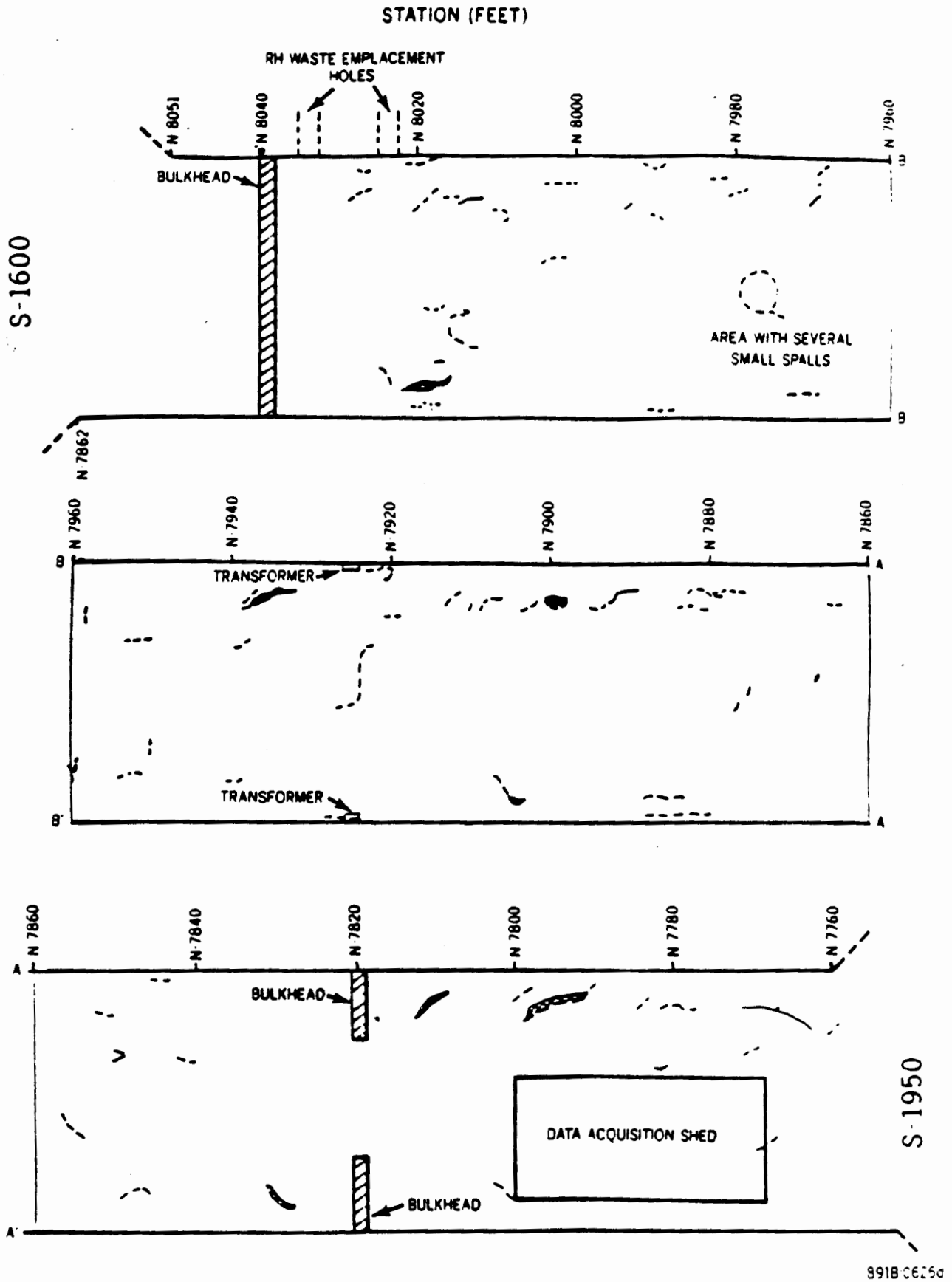
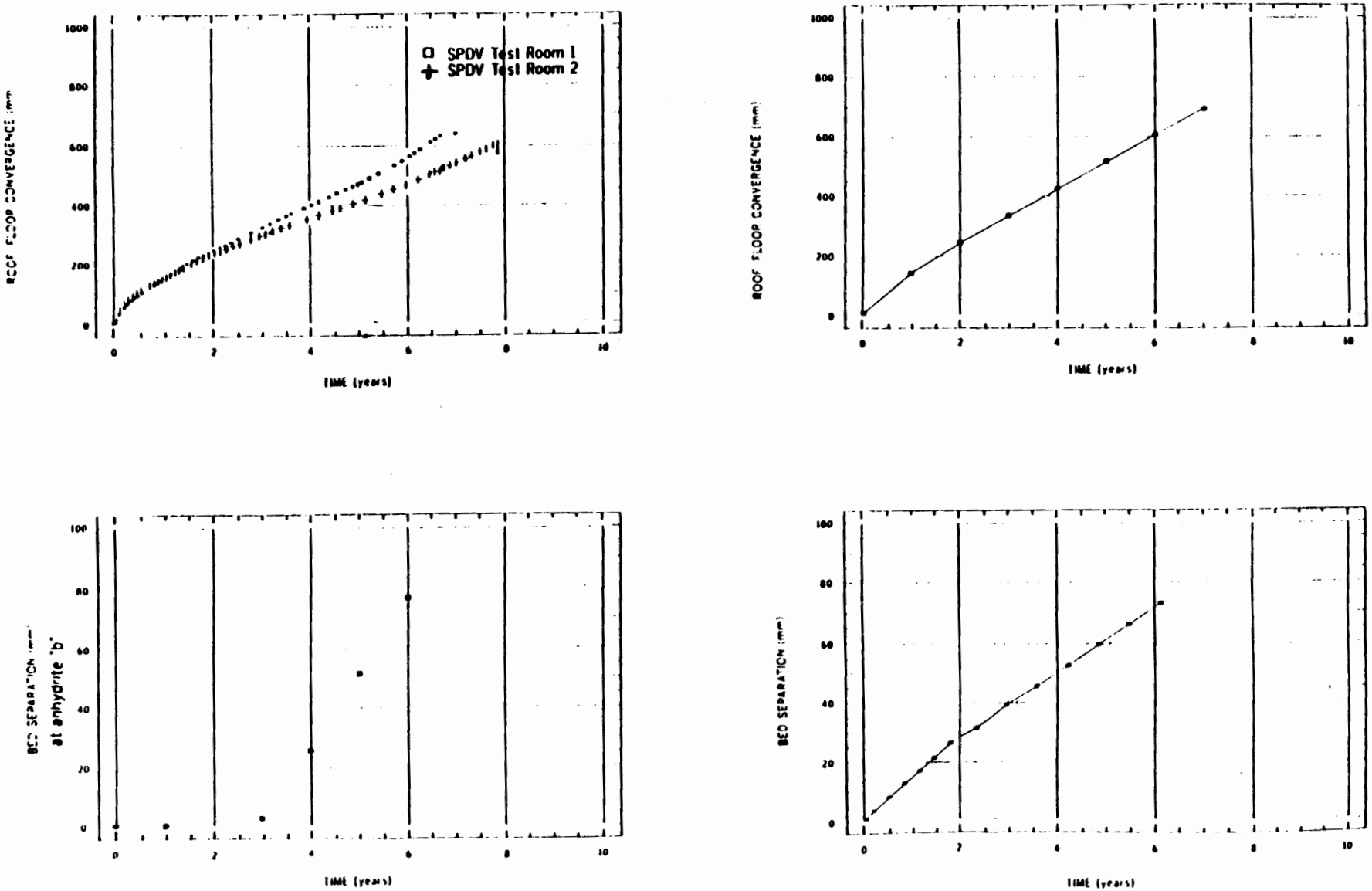


Figure 5.8 Back Fractures, Panel 1, Room 1 mapped on 2/12/91

Figure 5.9

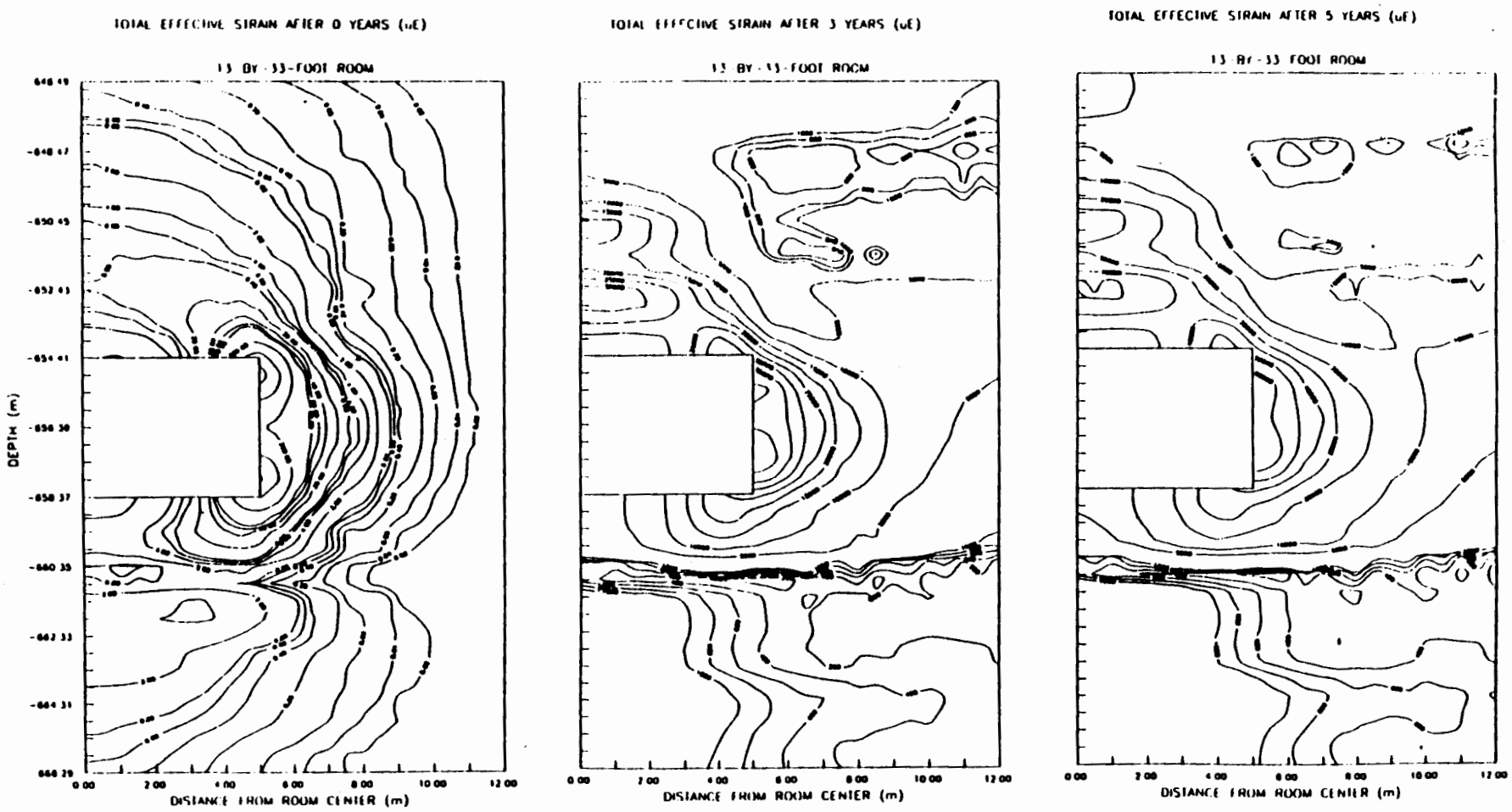
FIELD MEASUREMENTS

MODEL RESULTS



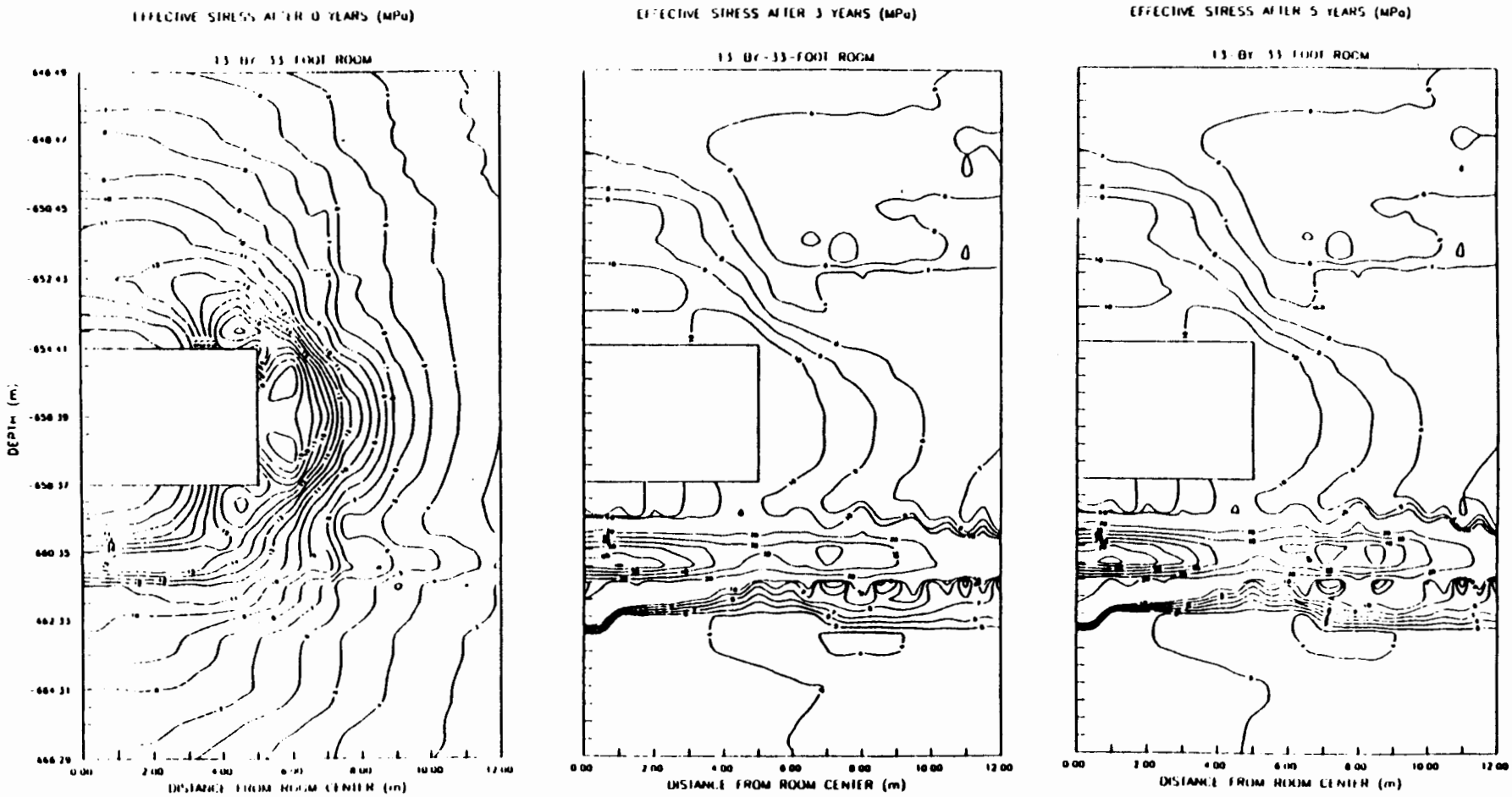
8918-0635

Figu. 5.10



891:0606e

Figure



89L0606c

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.

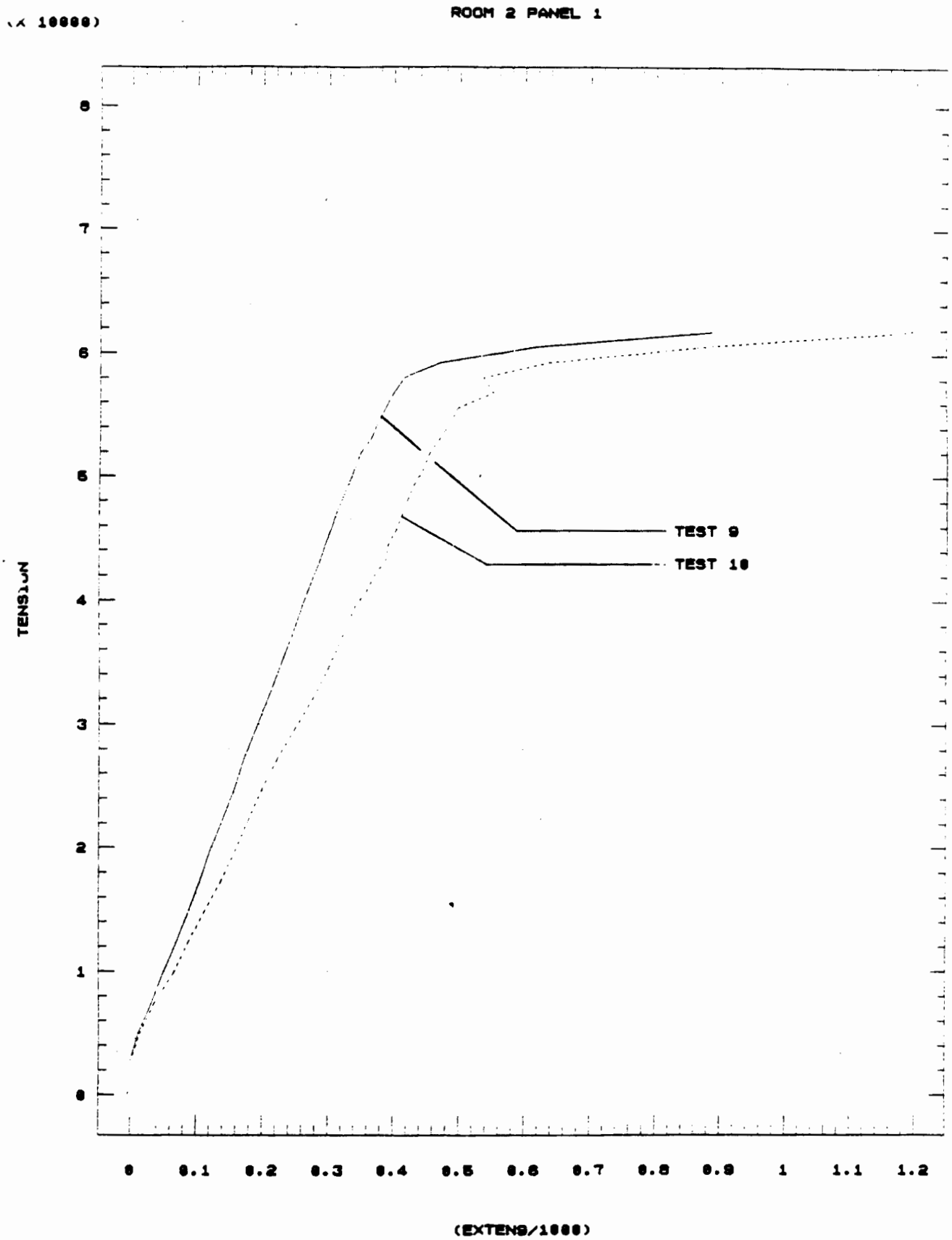
The results thus indicate that the mode of failure is that of bolt yield and not failure of the resin anchor bond.

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

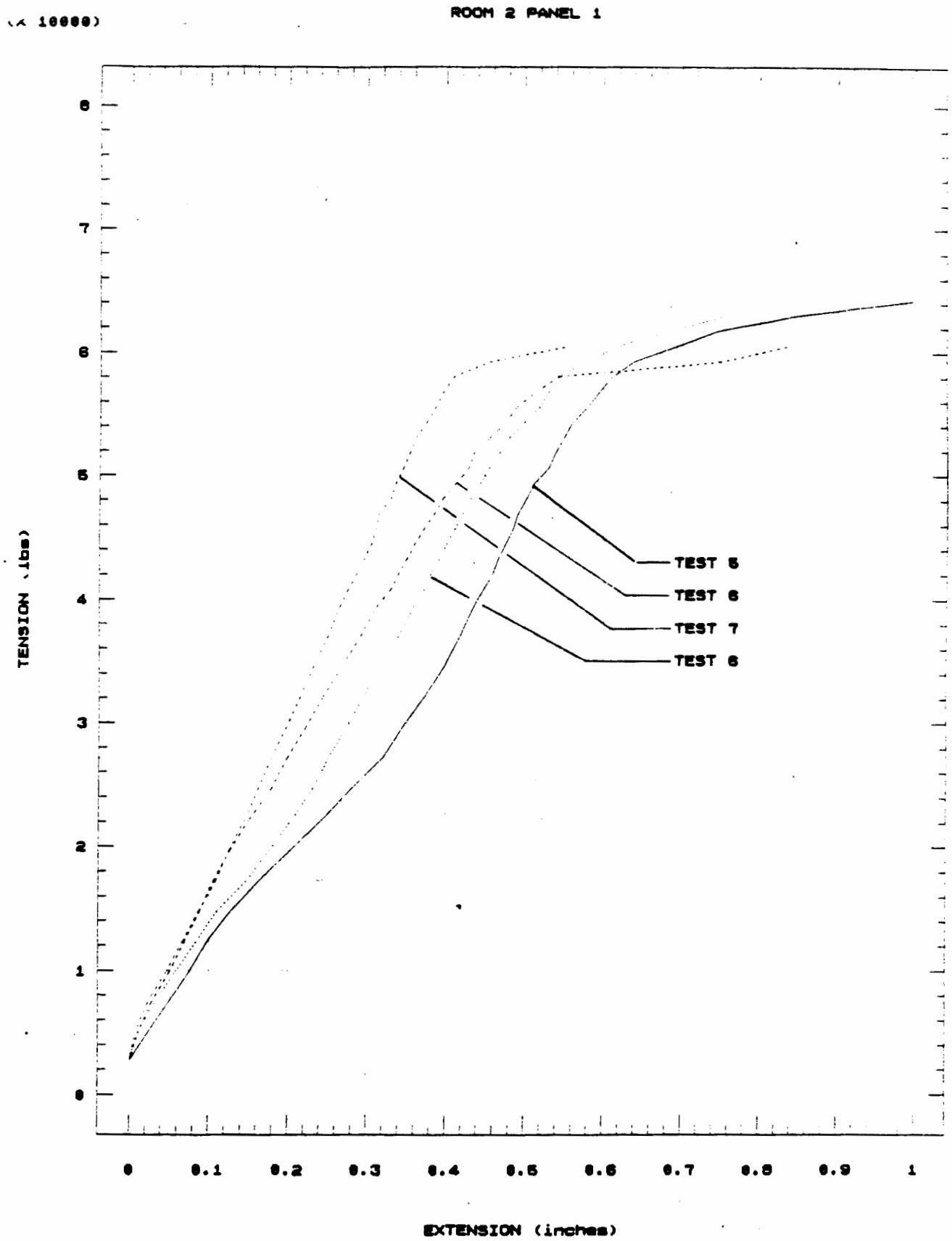
ROOM 2 PANEL 1



2 AND 3 DAYS CURE

BOLT TENSION VERSUS EXTENSION

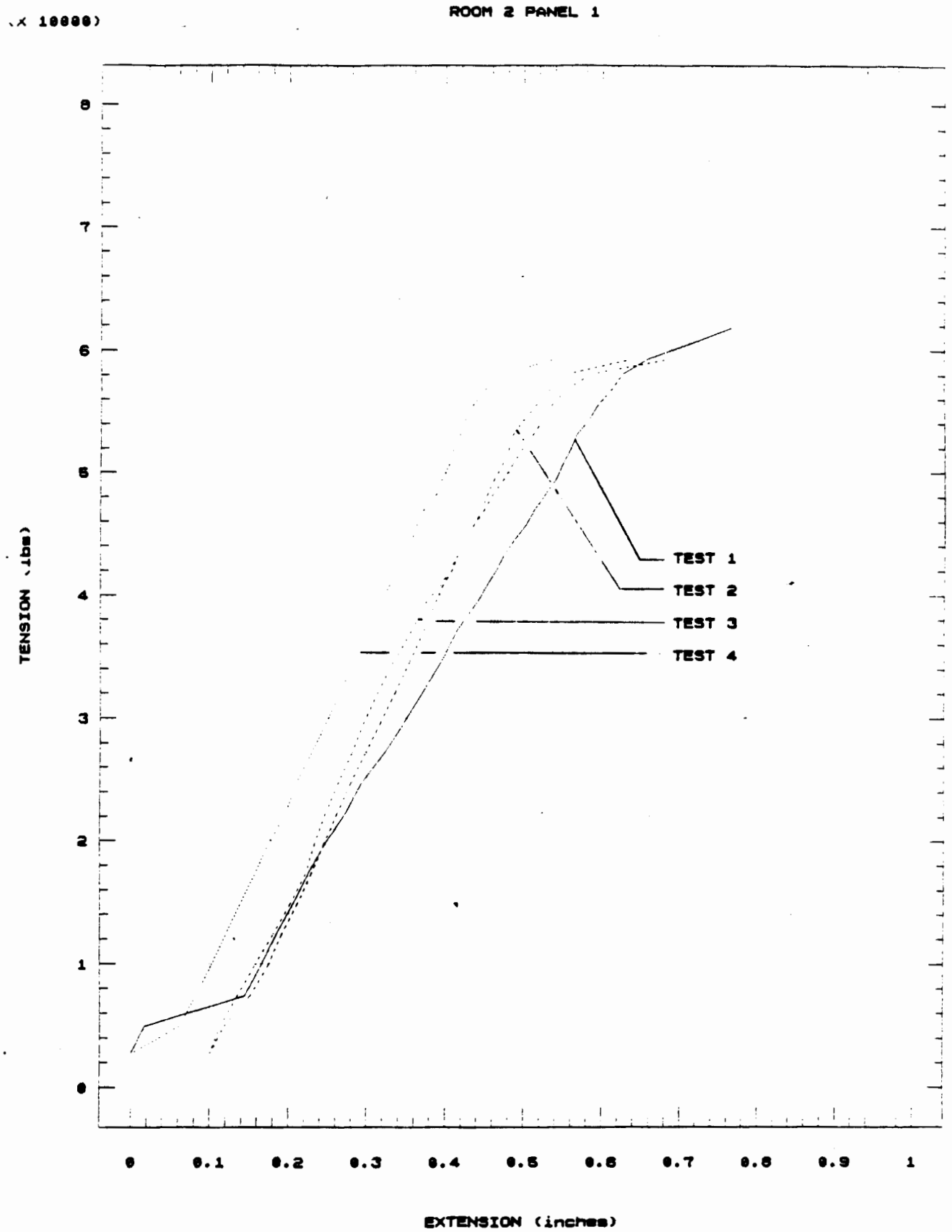
ROOM 2 PANEL 1



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

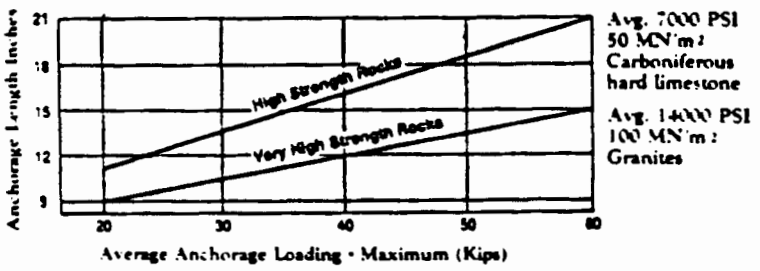
Lokset® cartridges can be applied in underwater anchorage applications over 24 inches in length, in both concrete and rock. *Consult CELTITE® TECHNIK for your specific job requirements.*

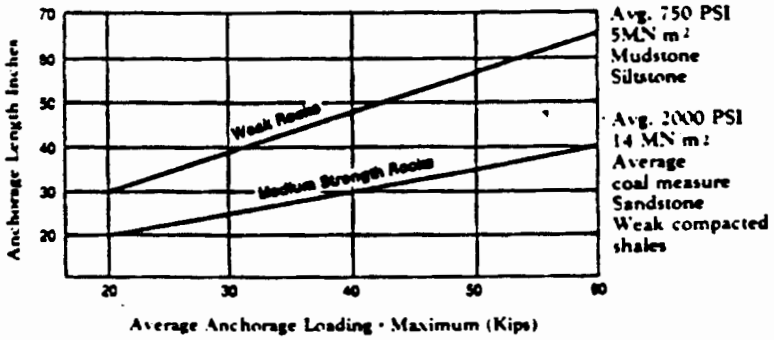
APPLICATION EQUIPMENT

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.

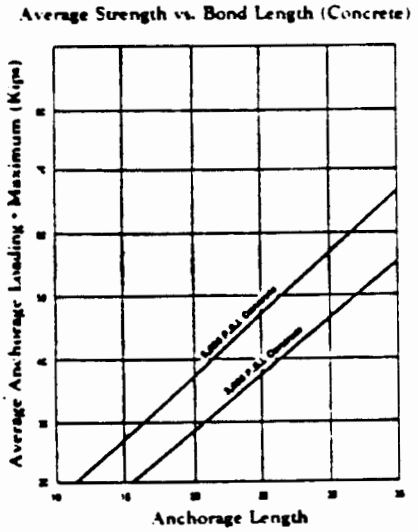
RESIN ANCHORAGE CHARTS

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

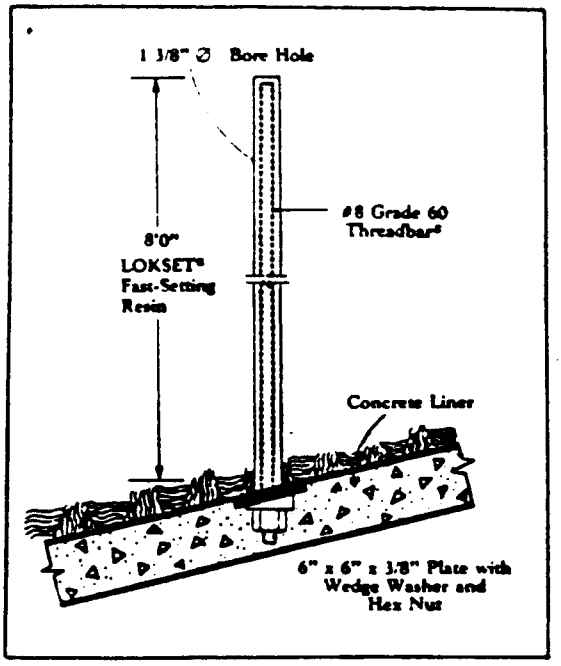
Lokset® cartridges should be stored in a cool, well-ventilated and dry area away from direct sunlight. High temperature conditions can reduce shelf life. Cartridges stored in extreme temperatures should be "normalized" at 50-70°F for at least two days prior to use to provide the expected gel time.

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

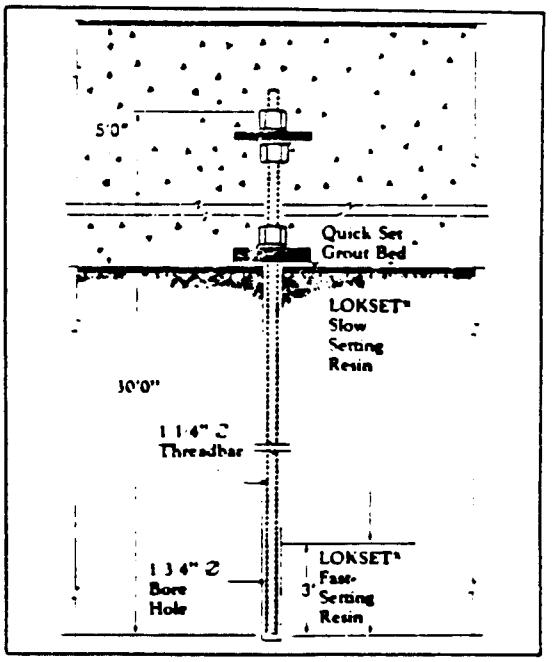
SAFE HANDLING PROCEDURES

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: _____

_____

_____

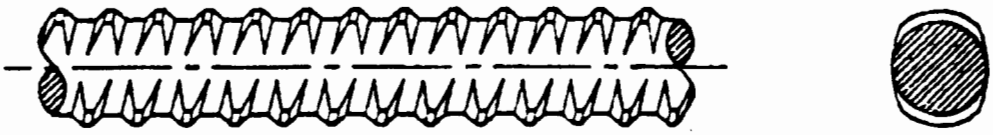
_____

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

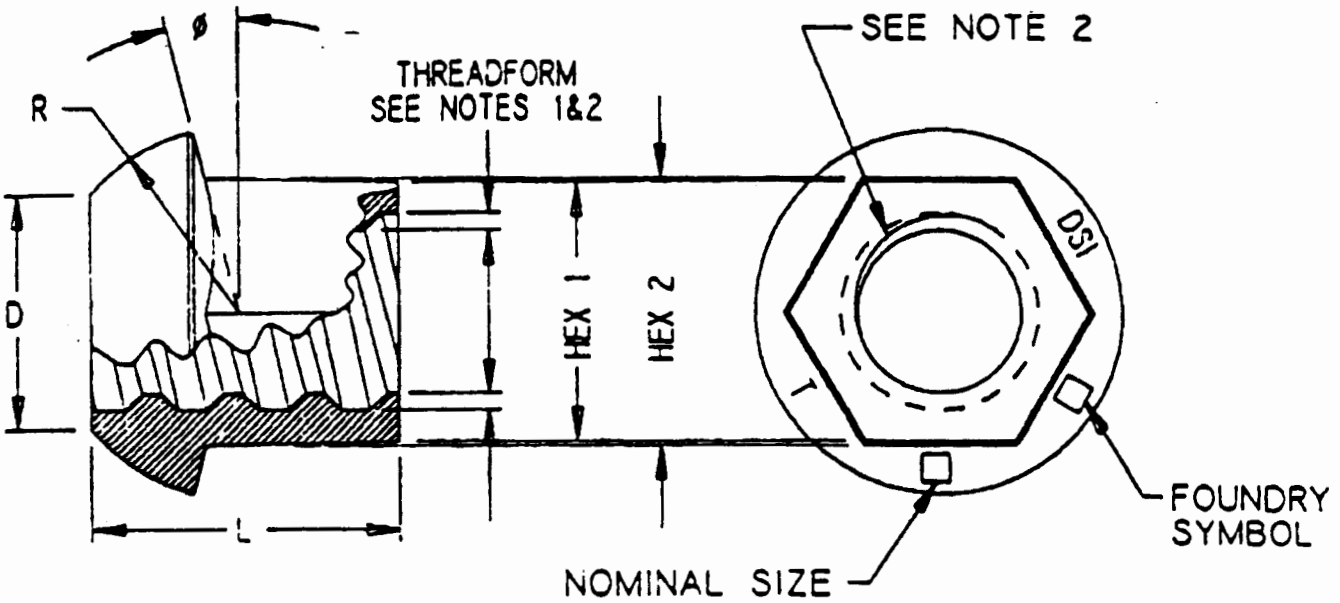
THIS _____ DAY _____

_____
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

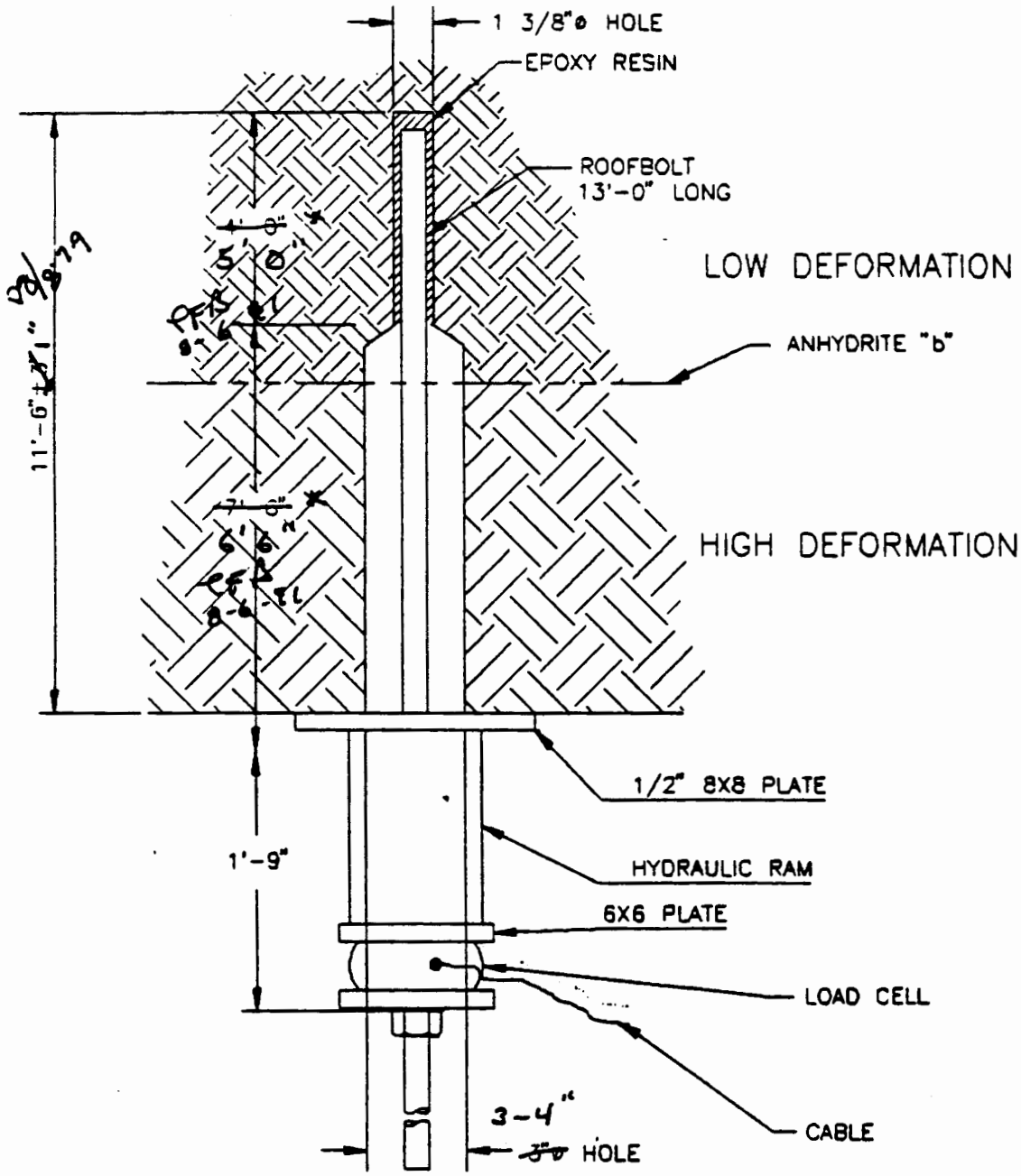
SCALE 1:1 FOR #7

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.

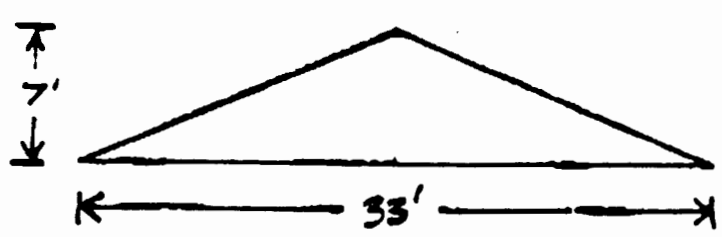
ASSUMPTIONS:

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
PROJECT	AUTHOR C. FRANKE 9/5/91	CHK'D BY JF Tyborski 9/9/91	CHK'D BY DATE
SUB	CALC NO	FILE NO	GROUP

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.

---

[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.

The American Society for Testing and Materials takes no position respecting the validity of any patent rights asserted in connection with any item mentioned in this standard. Users of this standard are expressly advised that determination of the validity of any such patent rights, and the risk of infringement of such rights, are entirely their own responsibility.

This standard is subject to revision at any time by the responsible technical committee and must be reviewed every five years and if not revised, either reapproved or withdrawn. Your comments are invited either for revision of this standard or for additional standards and should be addressed to ASTM Headquarters. Your comments will receive careful consideration at a meeting of the responsible technical committee, which you may attend. If you feel that your comments have not received a fair hearing you should make your views known to the ASTM Committee on Standards, 1916 Race St., Philadelphia, PA 19103.

WESTINGHOUSE WASTE ISOLATION DIVISION

TITLE			PAGE
FACTOR OF SAFETY - ANCHOR BOND			1 OF 3
PROJECT	AUTHOR / DATE	CHK'D BY / DATE	CHK'D BY / DATE
	C. FRANCKE 9/8/91	J.R. Tyburski 9/9/91	
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.

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

TITLE			PAGE
FACTOR OF SAFETY - ANCHOR BOND			2 OF 3
PROJECT	AUTHOR / DATE	CHK'D BY / DATE	CHK'D BY / DATE
	C. FRANCKE 4/8/91	JRTyburski 4/8/91	
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:

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

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

WESTINGHOUSE WASTE ISOLATION DIVISION

TITLE				PAGE
FACTOR OF SAFETY - ANCHOR BOND				3 OF 3
PROJECT	AUTHOR / DATE	CHK'D BY / DATE	CHK'D BY	DATE
	C. FRANCKE 9/5/91	J R Tyburski 9/6/91		
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.

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

WESTINGHOUSE FORM 48238A

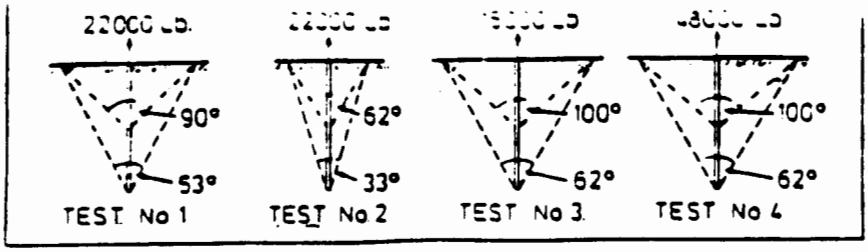


Fig. 3 Possible failure modes based on test results at Trinity Clear Creek
(after Saliman and Schaefer, 1968)

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-

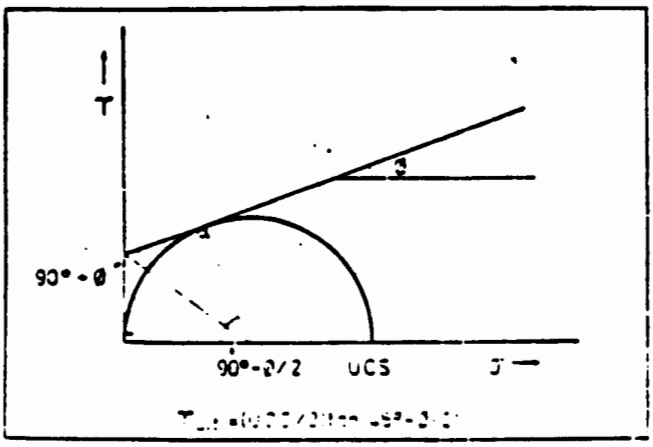


Fig. 4 Relationship between shear stress and uniaxial compressive strength

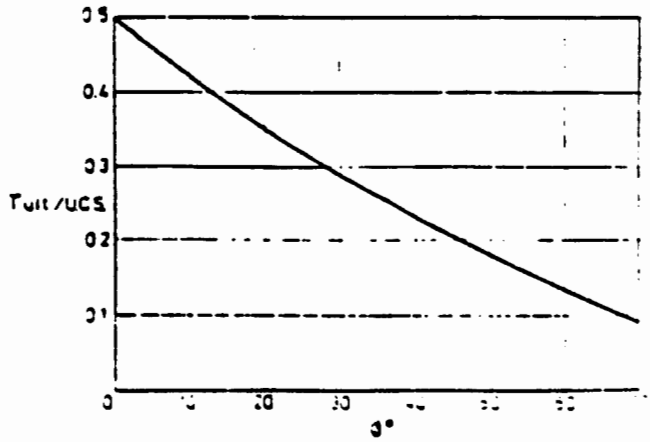


Fig. 5 Effect of Ø on τult/UCS ratio

[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

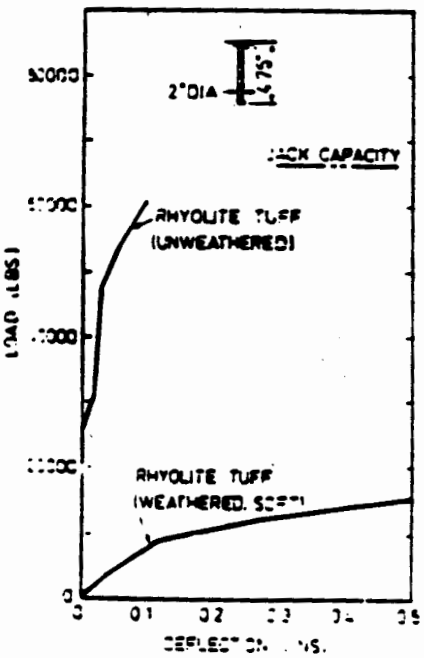


Fig 5 Effect of weathering at Currecanti Midway Transmission Line

(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)

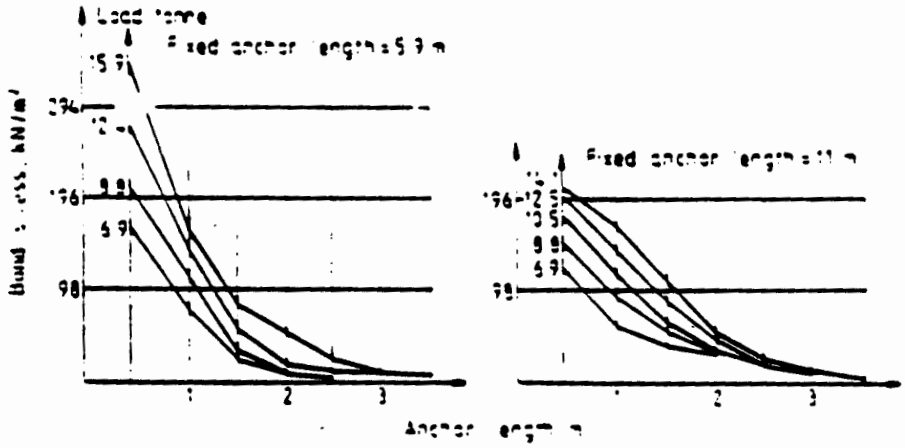


Figure 56. Distribution of bond along fixed anchor length (after Berardi 1967)

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

TITLE			PAGE
FACTOR OF SAFETY - WEDGE PULLOUT			1 OF 4
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.

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

WESTINGHOUSE FORM 40120A

WESTINGHOUSE WASTE ISOLATION DIVISION

TITLE				PAGE
FACTOR OF SAFETY - WEDGE PULLOUT				2 OF 34
PROJECT	AUTHOR / DATE	CHK'D BY / DATE	CHK'D BY / DATE	
	C. FRANCKE 7/26/91	JR Tyler 9/4/91		
BU	CALC NO	FILE NO	GROUP	

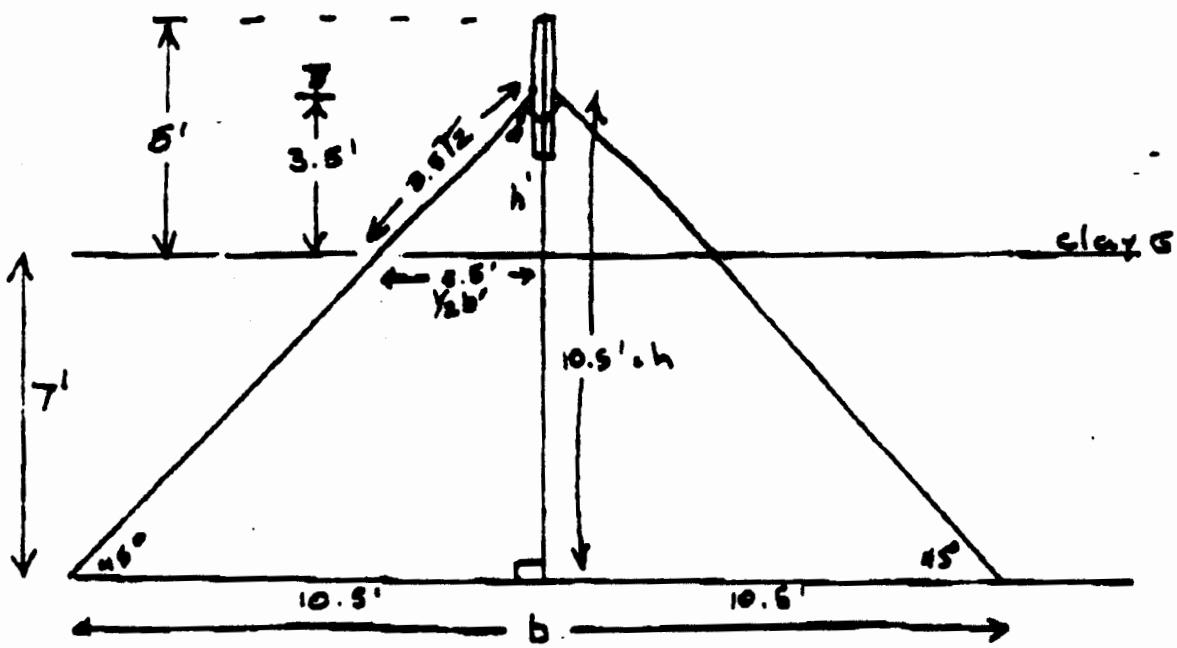


FIG. 1

W = WIDTH = 33'

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

WESTINGHOUSE FORM 40280A

WESTINGHOUSE WASTE ISOLATION DIVISION

TITLE				PAGE
FACTOR OF SAFETY - WEDGE PULLOUT				3 OF 4
PROJECT	AUTHOR / DATE	CHK'D BY / DATE	CHK'D BY	DATE
	C. FRANCKE 9/5/91	J.R. Tyburski 9/9/91		
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.

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

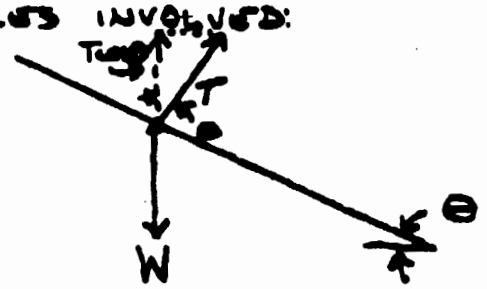
$$W = 491\ kip = 2.18\ MN$$

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:



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

TITLE				PAGE
FACTOR OF SAFETY - WEDGE PULLOUT				44 OF 44
PREPARED	AUTHOR C. FRANCKE DATE 9/5/91	CHK'D. BY J R Tyburski DATE 9/9/91	CHK'D. BY	DATE
SU	CALC NO.	FILE NO	GROUP	

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

APP NO	REV DATE	DIVISION	DATE	CHK'D BY	DATE	CHK'D BY	DATE

"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

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

TITLE U/G. Room 1 – Roof Support System			PAGE 3 OF
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

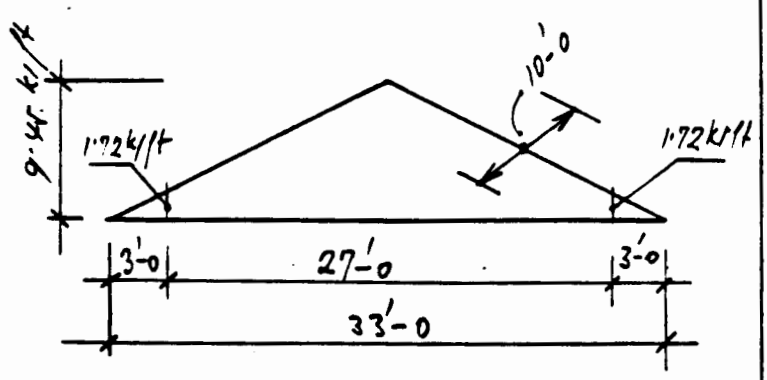
$$P_W = A \times F_t = 0.79 \times 36.0 = \underline{28.4^k}$$

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 $q = \dfrac{2 \times 156.0}{33.0} = \underline{9.45\ k/ft}$

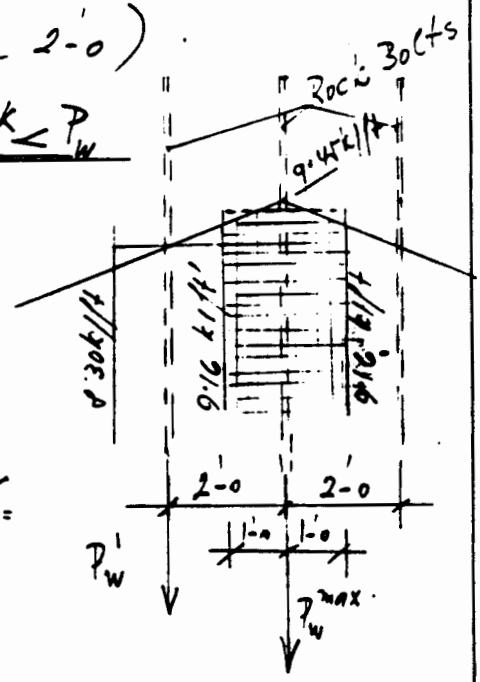


REV. NO.	REV. DATE	AUTHOR	DATE	CHK'D. BY	DATE	CHK'D. BY	DATE

TITLE U/G. Room 1 - Roof Support System			PAGE 4 OF 7
PROJECT	AUTHOR JMK DATE 08/29/91	CHK'D. BY HS DATE 9/6/91	CHK'D. BY DATE
S.O.	CALC. NO. 6-91-JMK	FILE NO.	GROUP H & C.S.

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

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



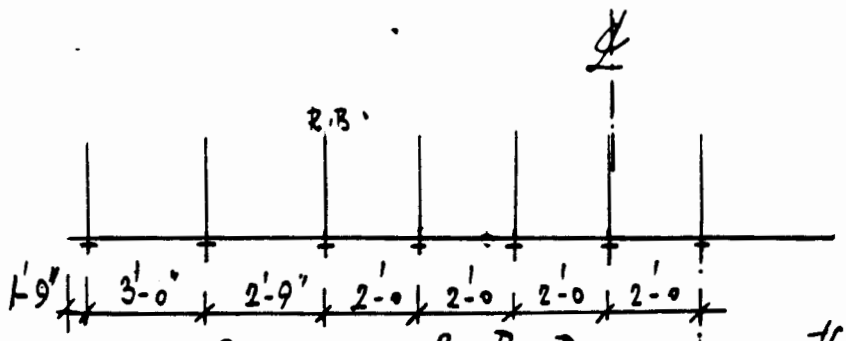
Adjacent Rock Bolt:

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

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

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

TITLE U/G. Room 1 – Roof Support System			PAGE 5 OF 9
PROJECT	AUTHOR JMK 8/29/91	CHK'D. BY HS 9/6/91	CHK'D. BY DATE
S.O.	CALC. NO. G-91-JMK	FILE NO.	GROUP M&C.E.

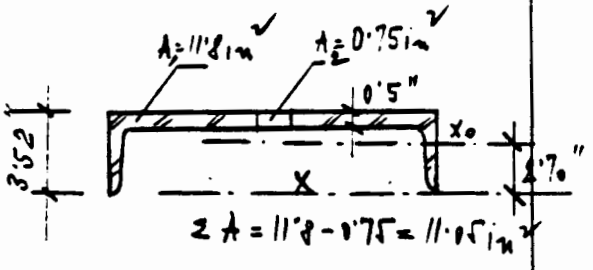
Check of stress in the channel from bending. ([15x40; $S = 3.37 in^3$).

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:

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

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

$M_x = 29.88 in^3$

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

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

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

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

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

TITLE U/G. Room 1 – Roof Supporting System			PAGE 6 OF ?
PROJECT	AUTHOR JMK DATE 08/30/91	CHK'D BY HS DATE 9/6/91	CHK'D. BY DATE
S.O.	CALC. NO. 6-91-JMK	FILE NO.	GROUP M. & C.E.

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 $2\ 5\frac{1}{2} \times 5\frac{1}{2}$)

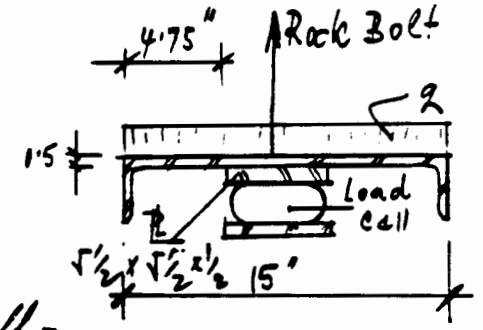
Bending of the [15 perpendicular to the length of the beam @ support

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

$\doteq 78.9\ psi$

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

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



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

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

TITLE U/G. Room 1 – Roof Supporting System			PAGE 7 OF 7
PROJECT	AUTHOR JMK – 08/30/91	CHK'D. BY HS 9/6/91	CHK'D. BY DATE
S.O.	CALC. NO. 6-91-JMK	FILE NO.	GROUP M.&C.E

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)

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

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

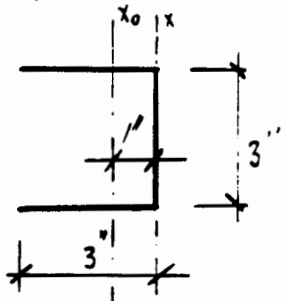
REV. NO.	REV. DATE	AUTHOR	DATE	CHK'D. BY	DATE	CHK'D. BY	DATE

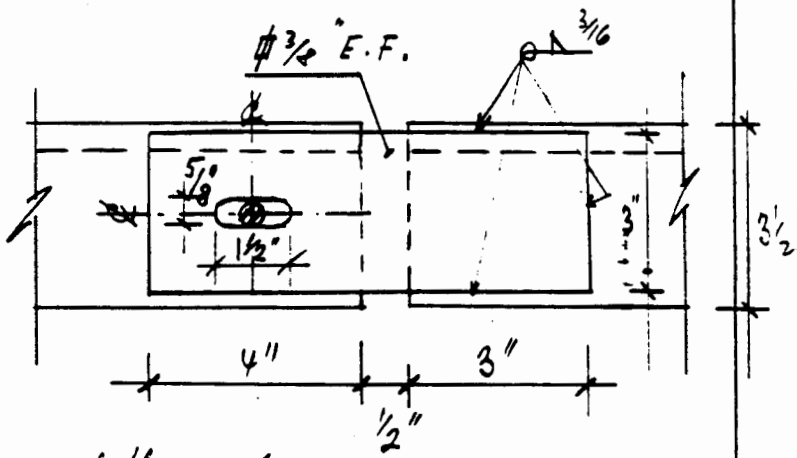
TITLE			PAGE 8 OF
U/G Room-1; Panel-1; Roof Support System			
PROJECT	AUTHOR TWX DATE 8/30/91	CHK'D. BY HS DATE 9/6/91	CHK'D. BY DATE
S.O.	CALC. NO. 6-91-TWX	FILE NO.	GROUP

Design of splice:

- assumed as a flexible pin, allowing small horizontal movement

C.G. of the weld:





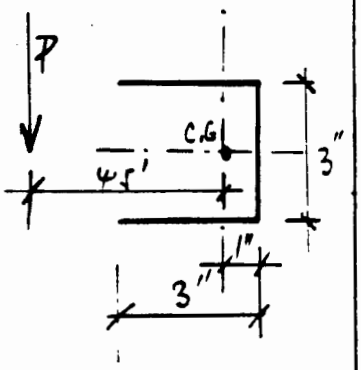
$S_{M_x} = 2(3 \times 1.5) = 9 in$

$L = 3 \times 3 = 9 in$

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

Design of the weld:

See table XXIII, page 4-90 AISC.



$k = 1.0 \quad a = 1.5$

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

$C = 0.775$

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

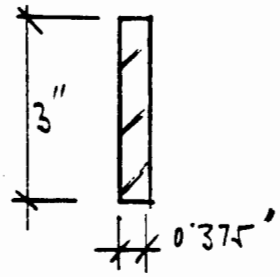
Use 3/16 weld

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

TITLE U/G Room 1, Panel-1; Roof Supp. System			PAGE 9 OF 2
PROJECT	AUTHOR MK 08/30 91	CHK'D. BY HS 9/6/91 DATE	CHK'D. BY DATE
S.O.	CALC. NO. 6-91-MK	FILE NO.	GROUP M. & C.E.

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.

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

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.

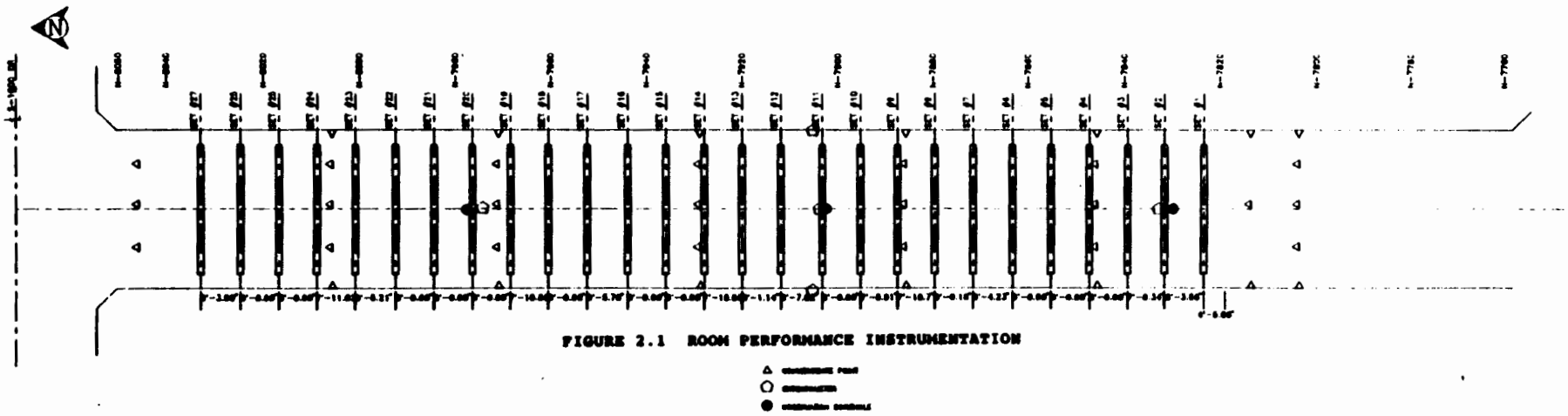


FIGURE 2.1 ROOM PERFORMANCE INSTRUMENTATION

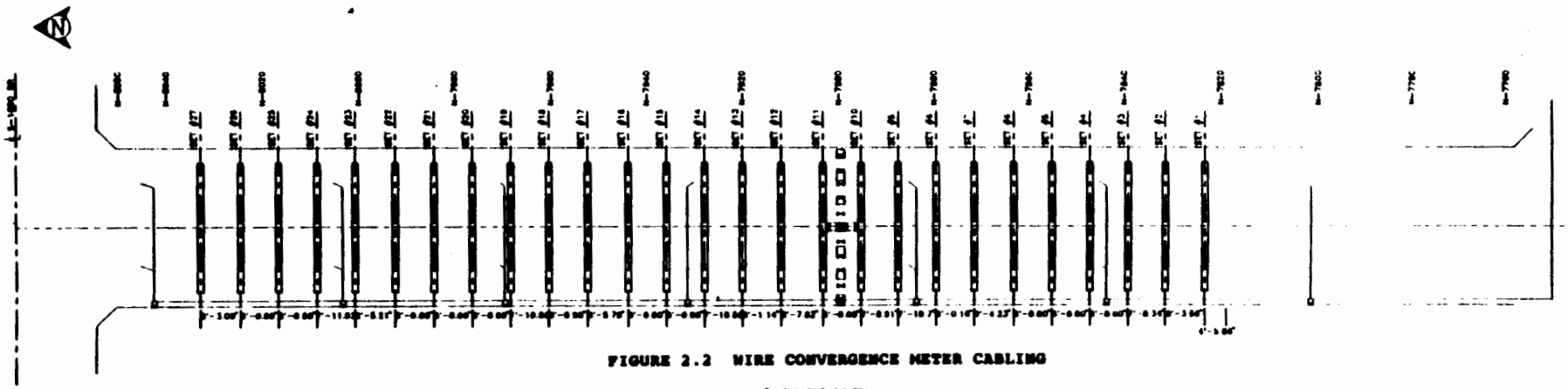


FIGURE 2.2 WIRE CONVERGENCE METER CABLING

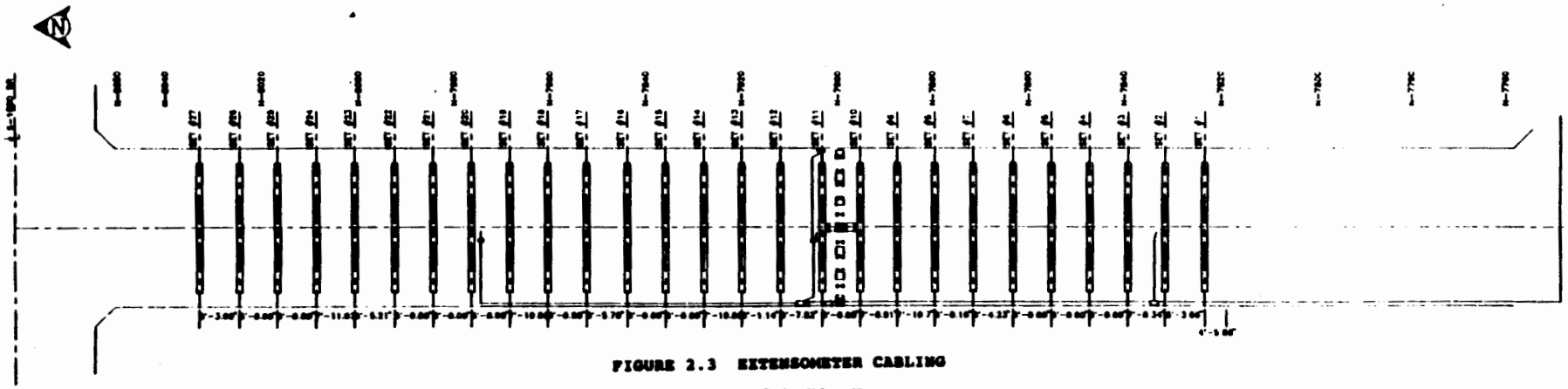
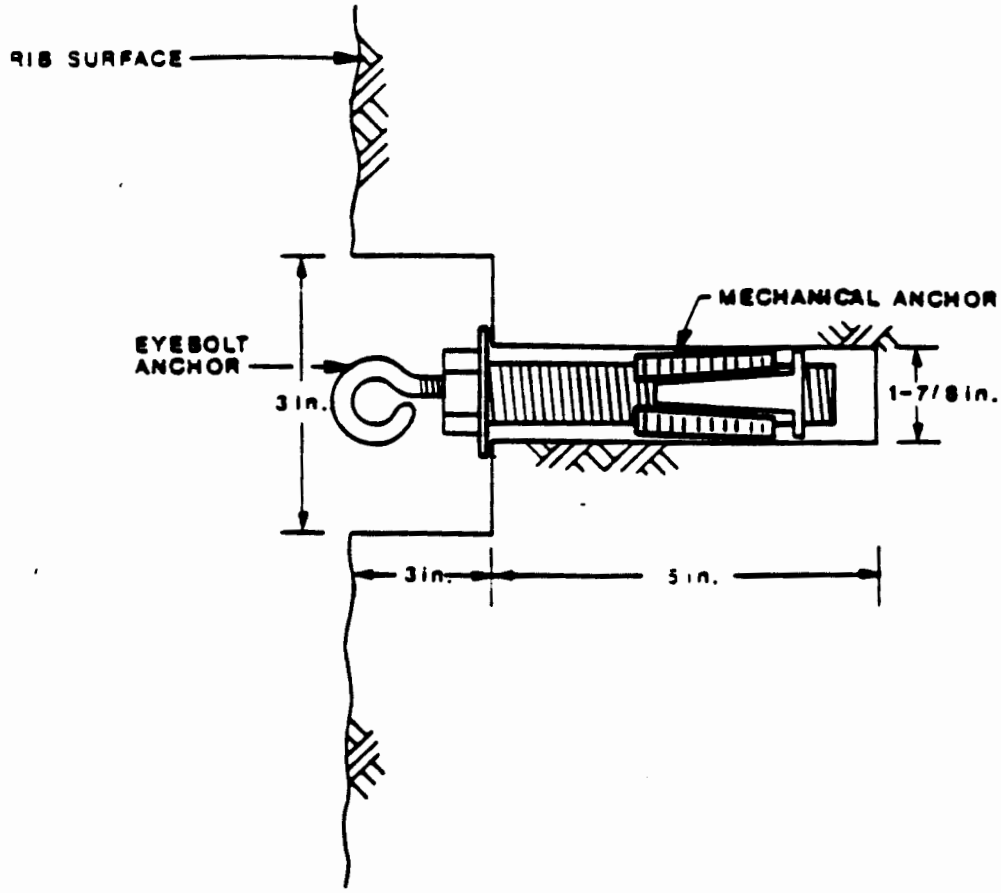


FIGURE 2.3 EXTENSOMETER CABLING

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

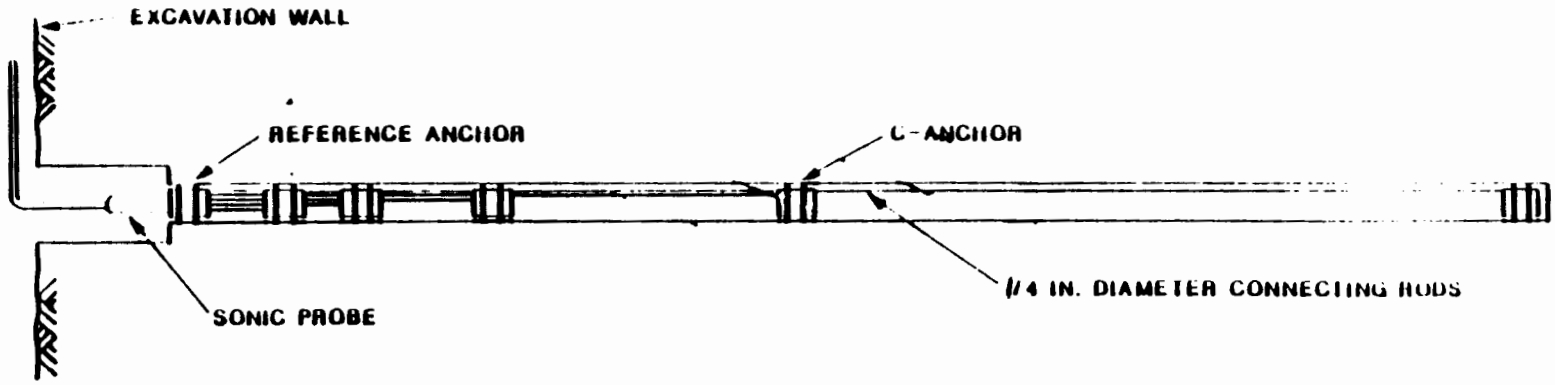


FIGURE 2.5 TYPICAL ROD-TYPE EXTENSOMETER INSTALLATION

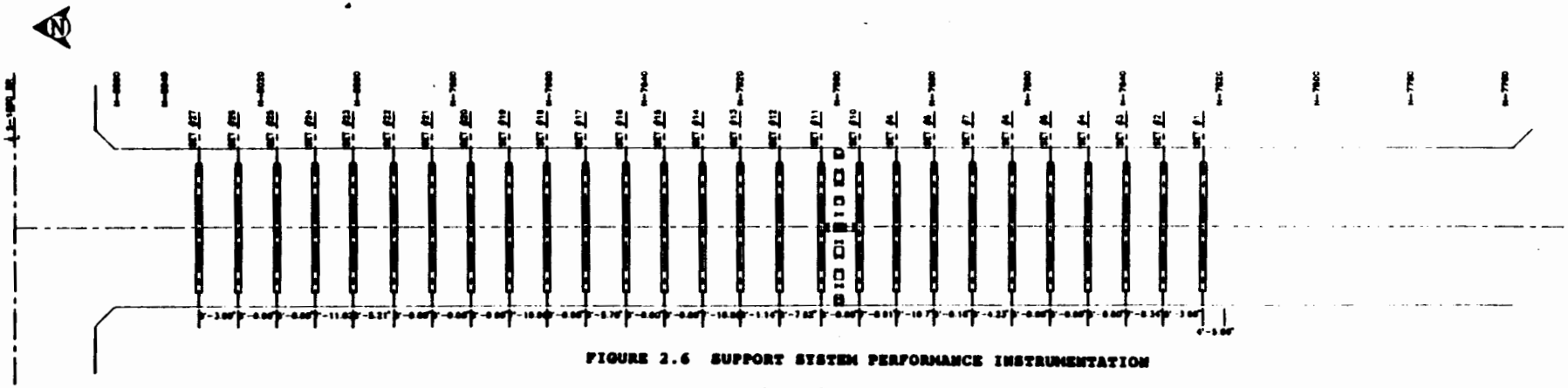


FIGURE 2.6 SUPPORT SYSTEM PERFORMANCE INSTRUMENTATION

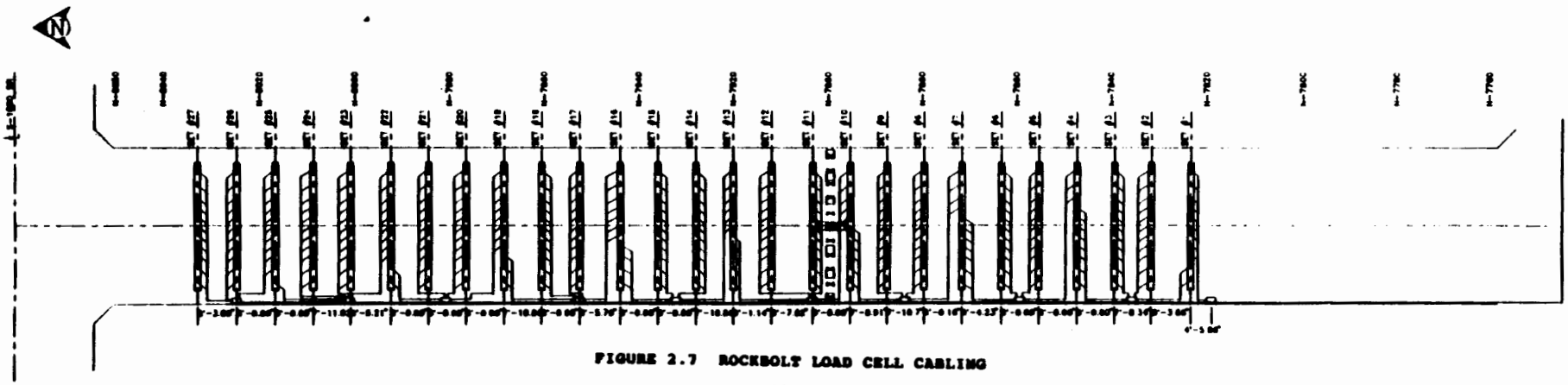


FIGURE 2.7 ROCKBOLT LOAD CELL CABLING

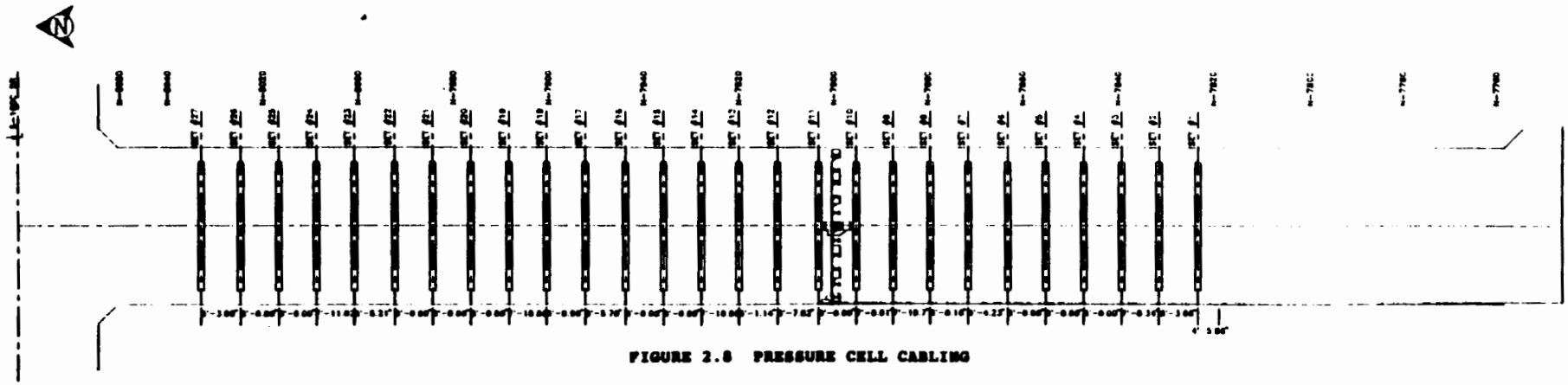


FIGURE 2.8 PRESSURE CELL CABLING

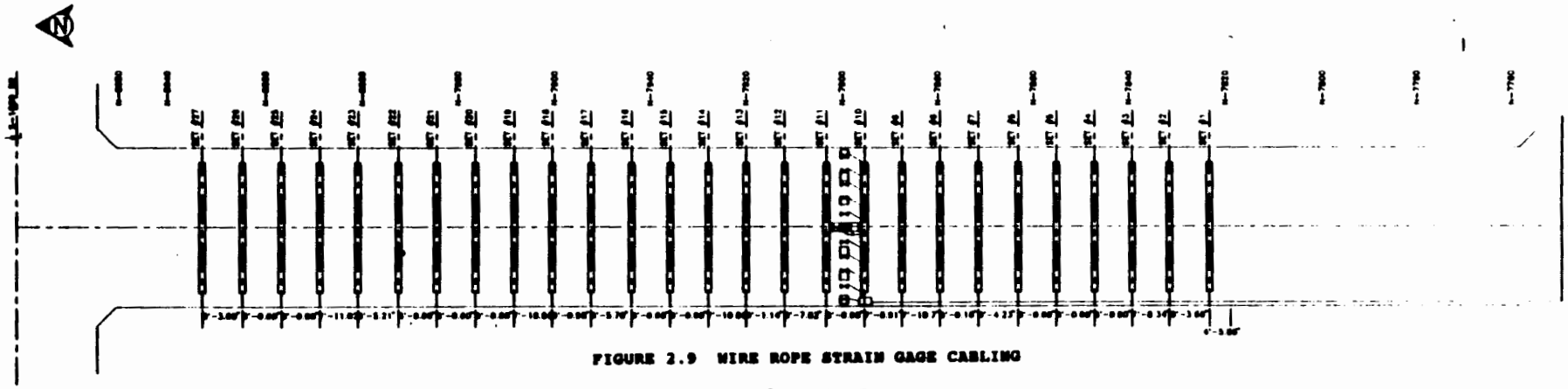


FIGURE 2.9 WIRE ROPE STRAIN GAGE CABLING

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

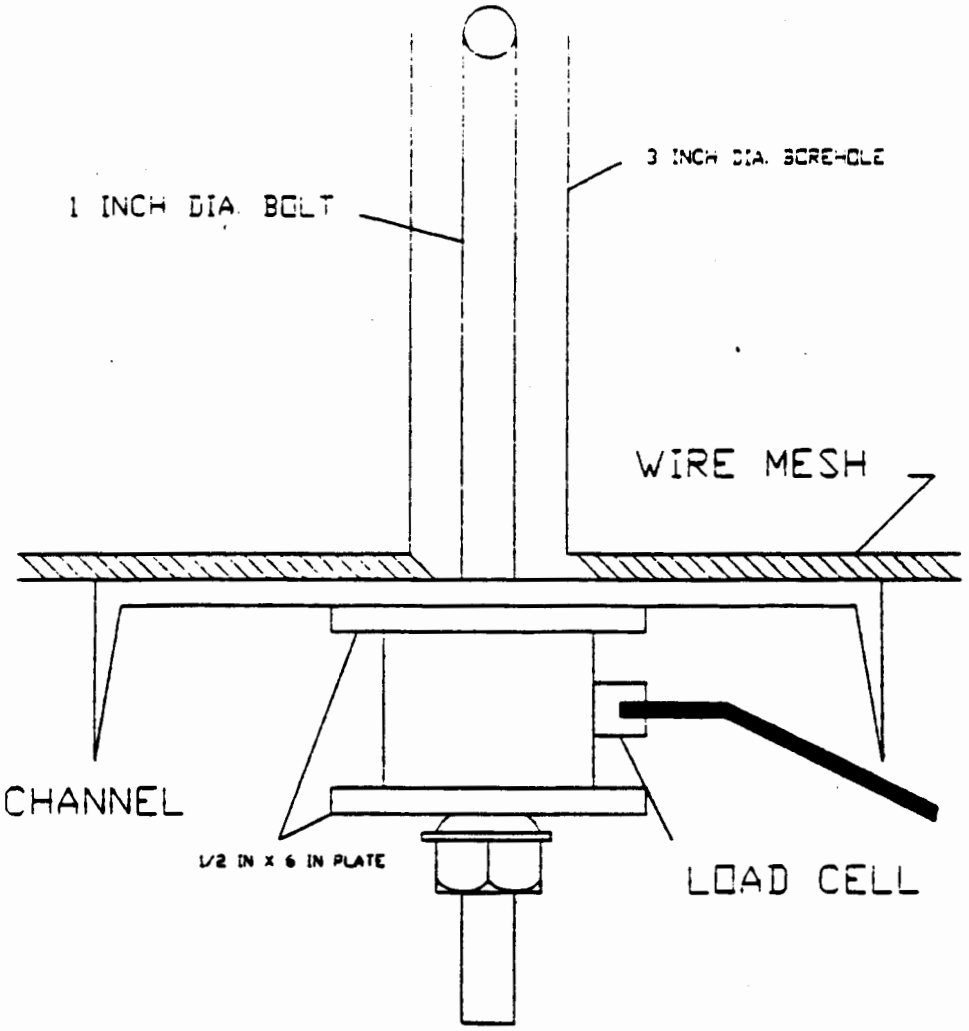


FIGURE 2.10 TYPICAL LOAD CELL INSTALLATION

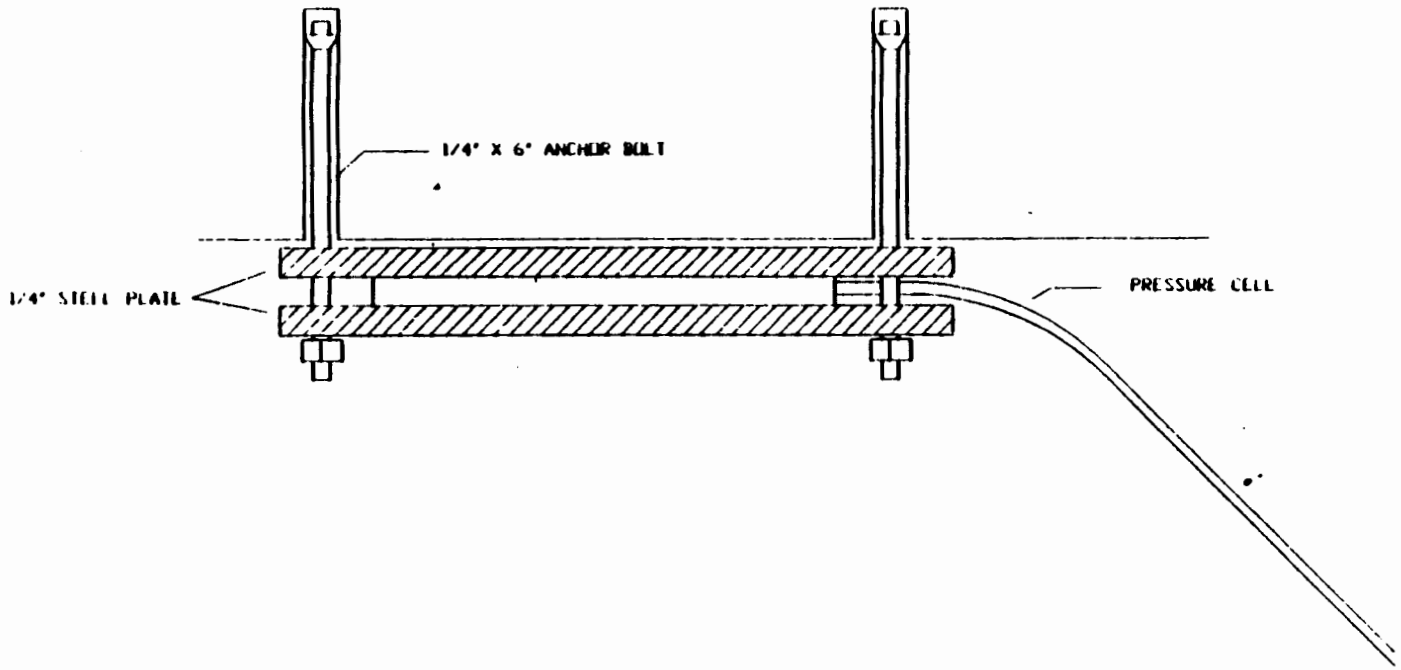


FIGURE 2.11 TYPICAL PRESSURE CELL INSTALLATION

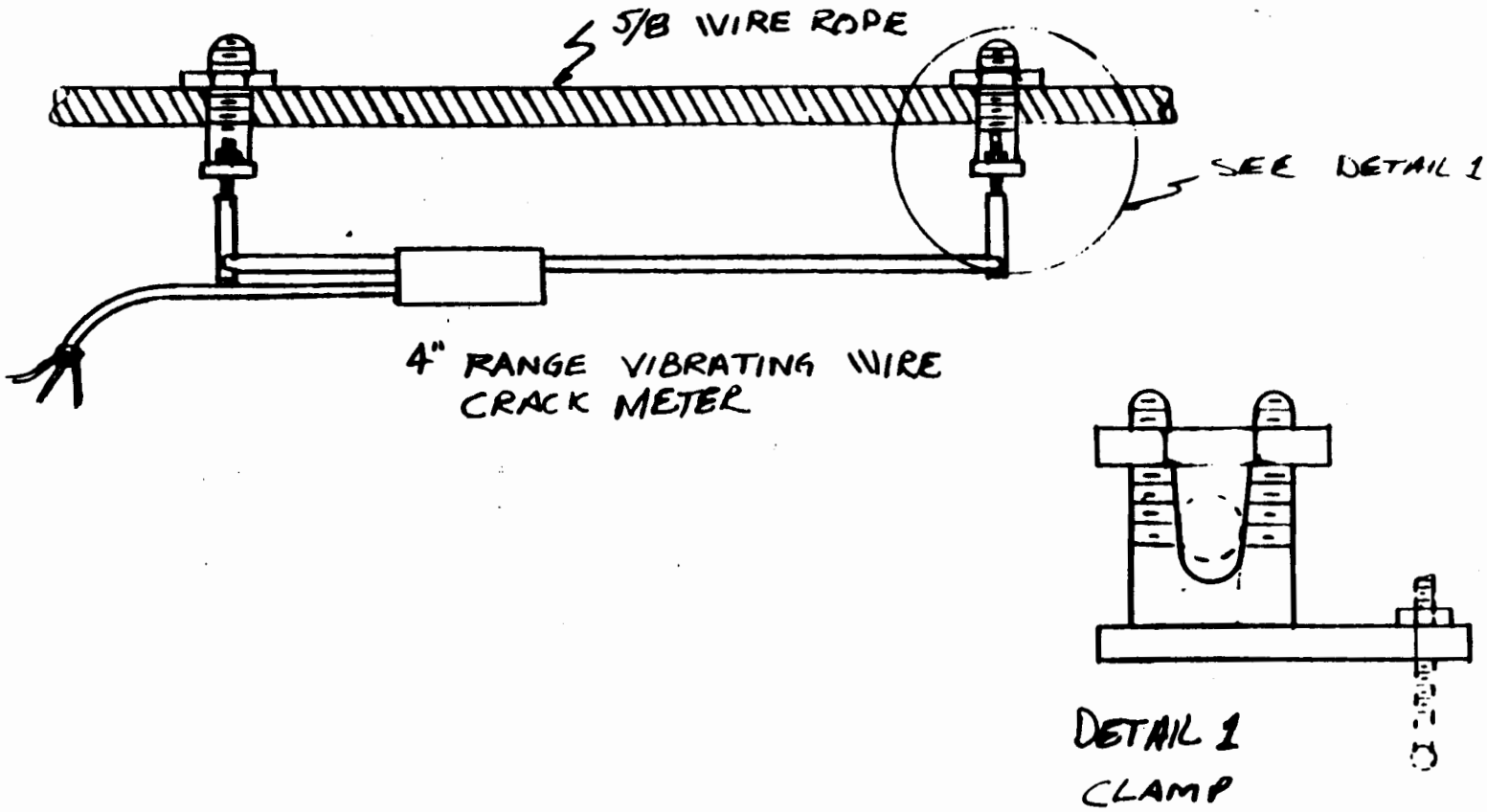
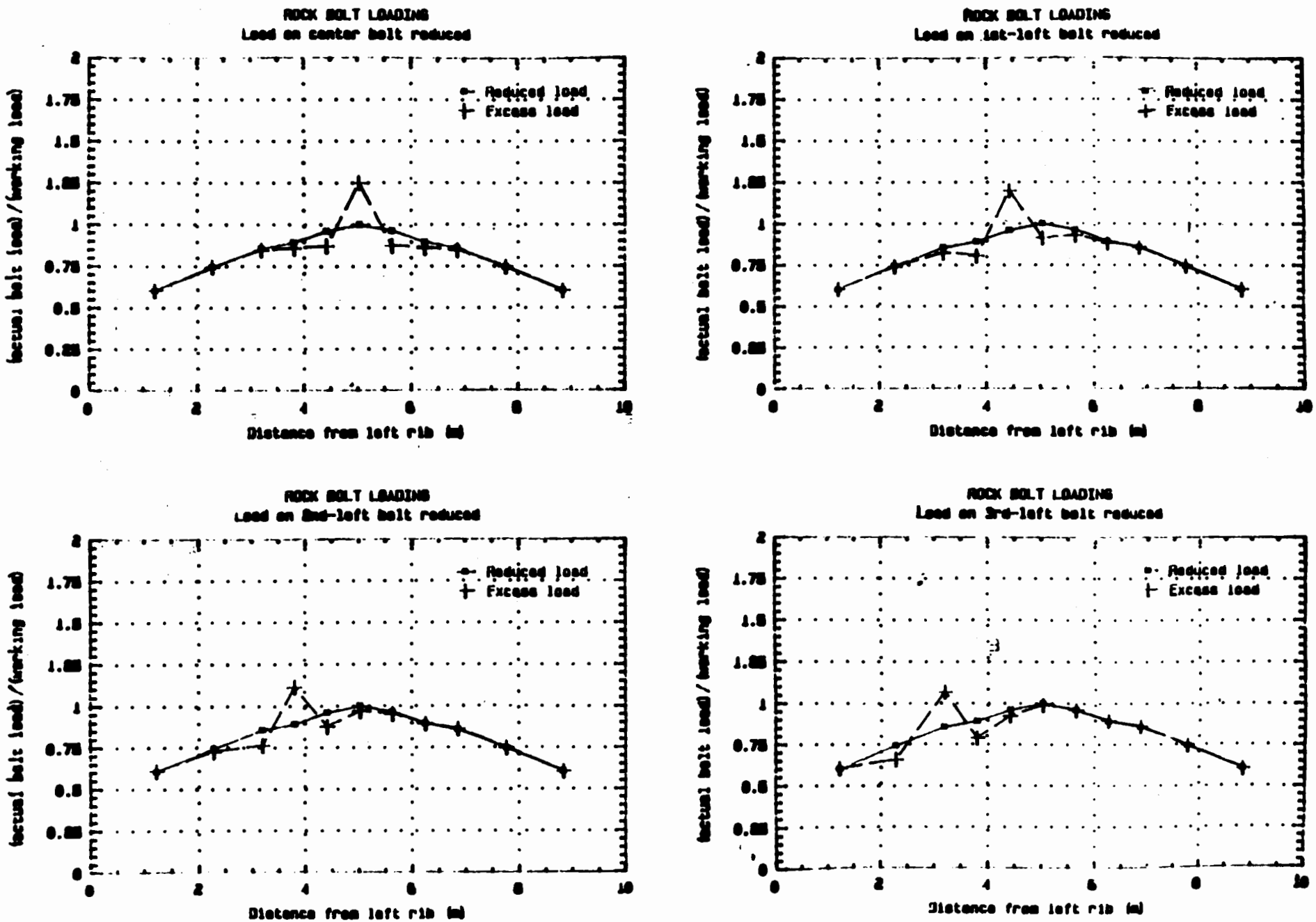


FIGURE 2.12 TYPICAL CRACKMETER INSTALLATION

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

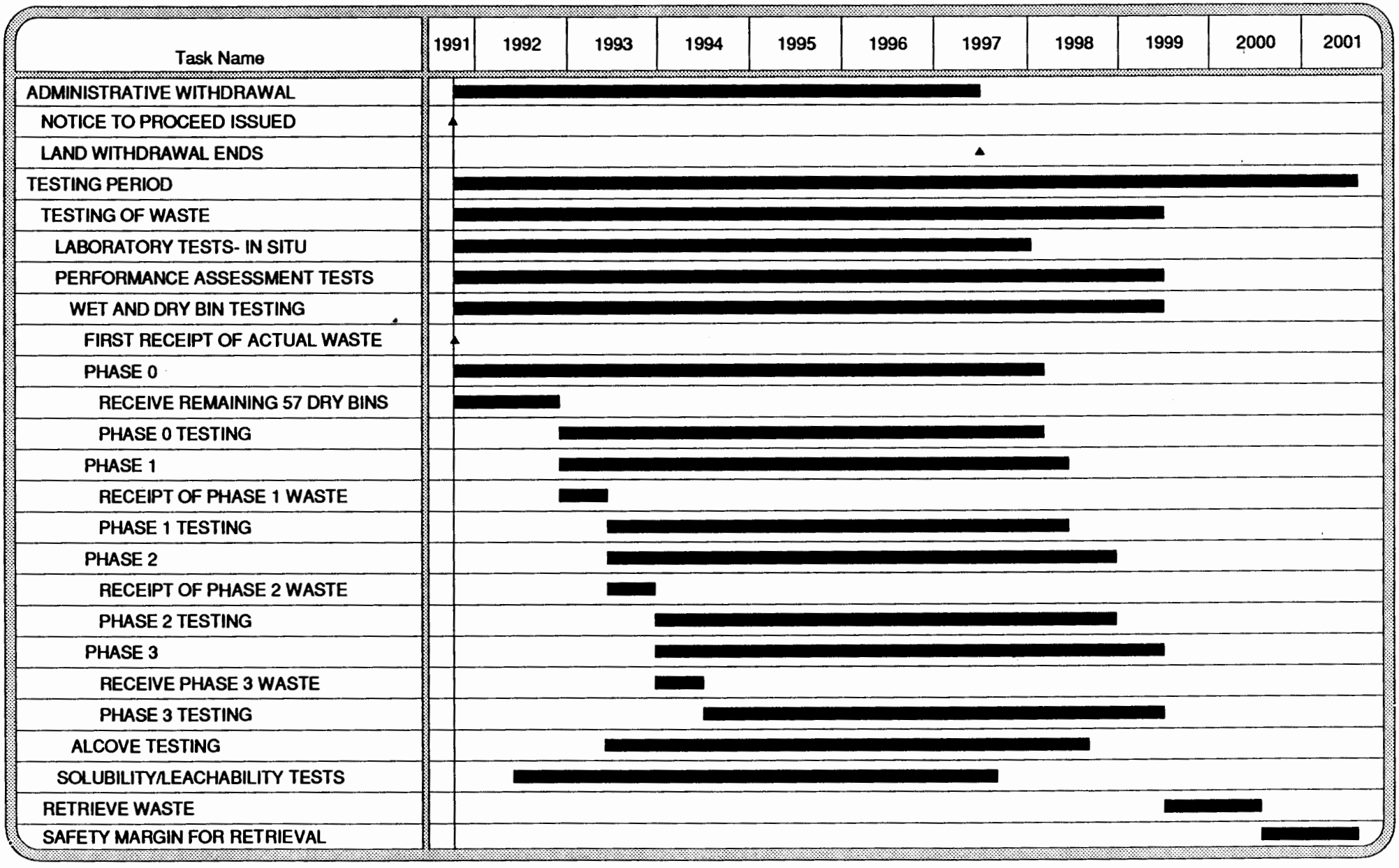
Marcus A. Wiley

X. Wiley 1

WIPP PROJECT SCHEDULE OF ACTIVITIES

WILEY ENGINEERING, INC.

OCTOBER 5, 1991

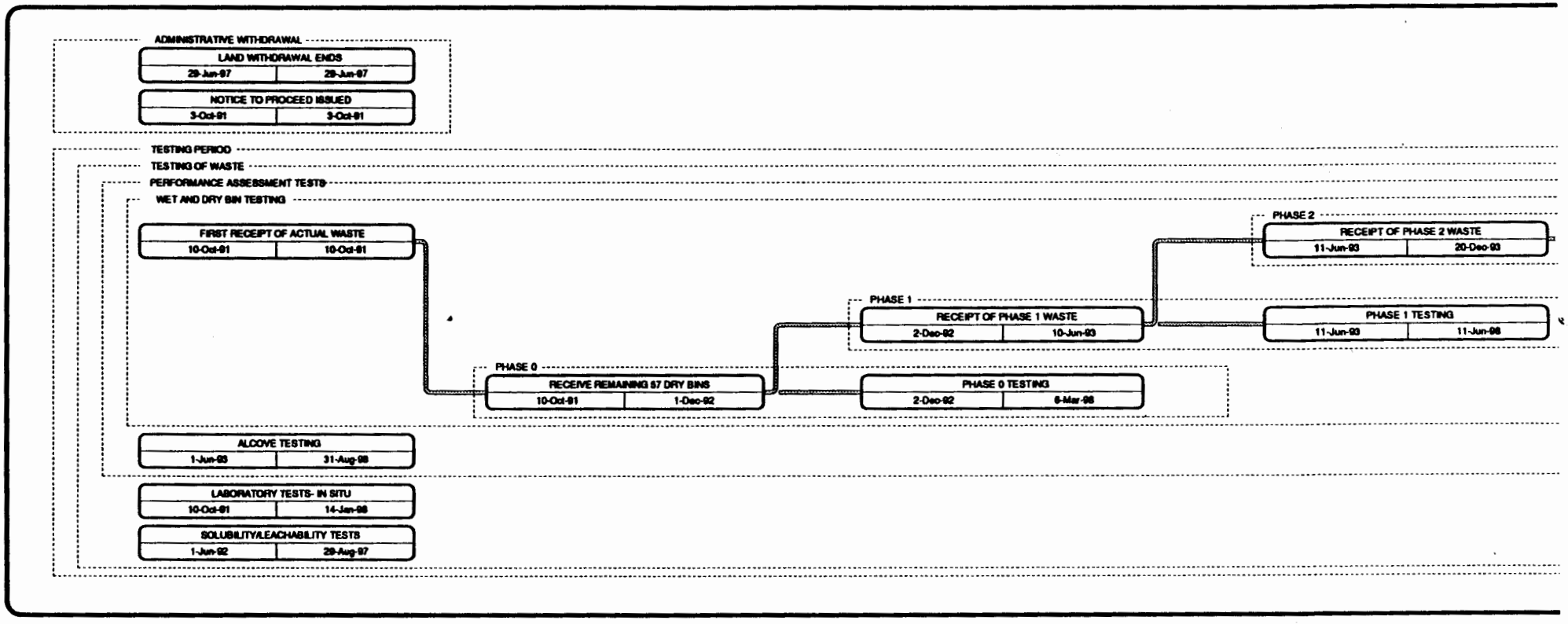


MARCUS A. WILEY AFFIDAVIT

EXHIBIT 1

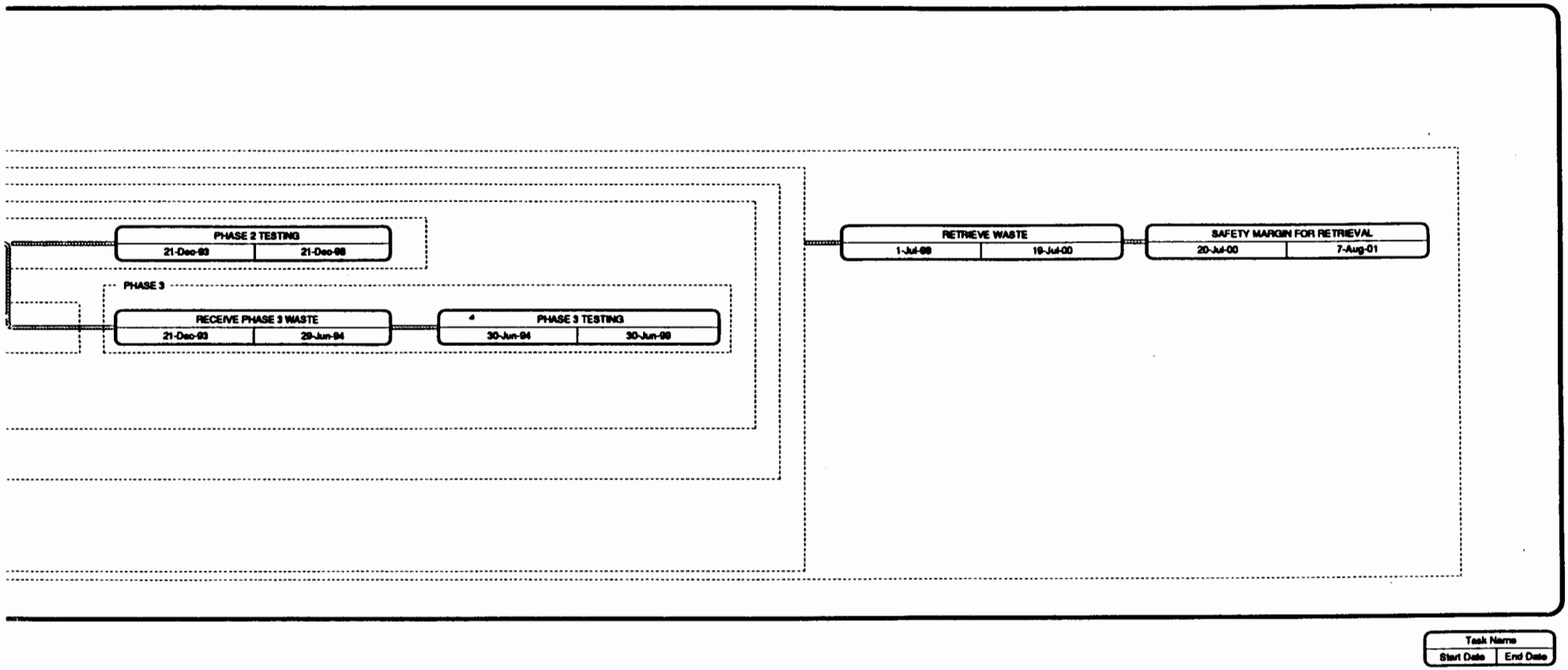
X. Wiley 2

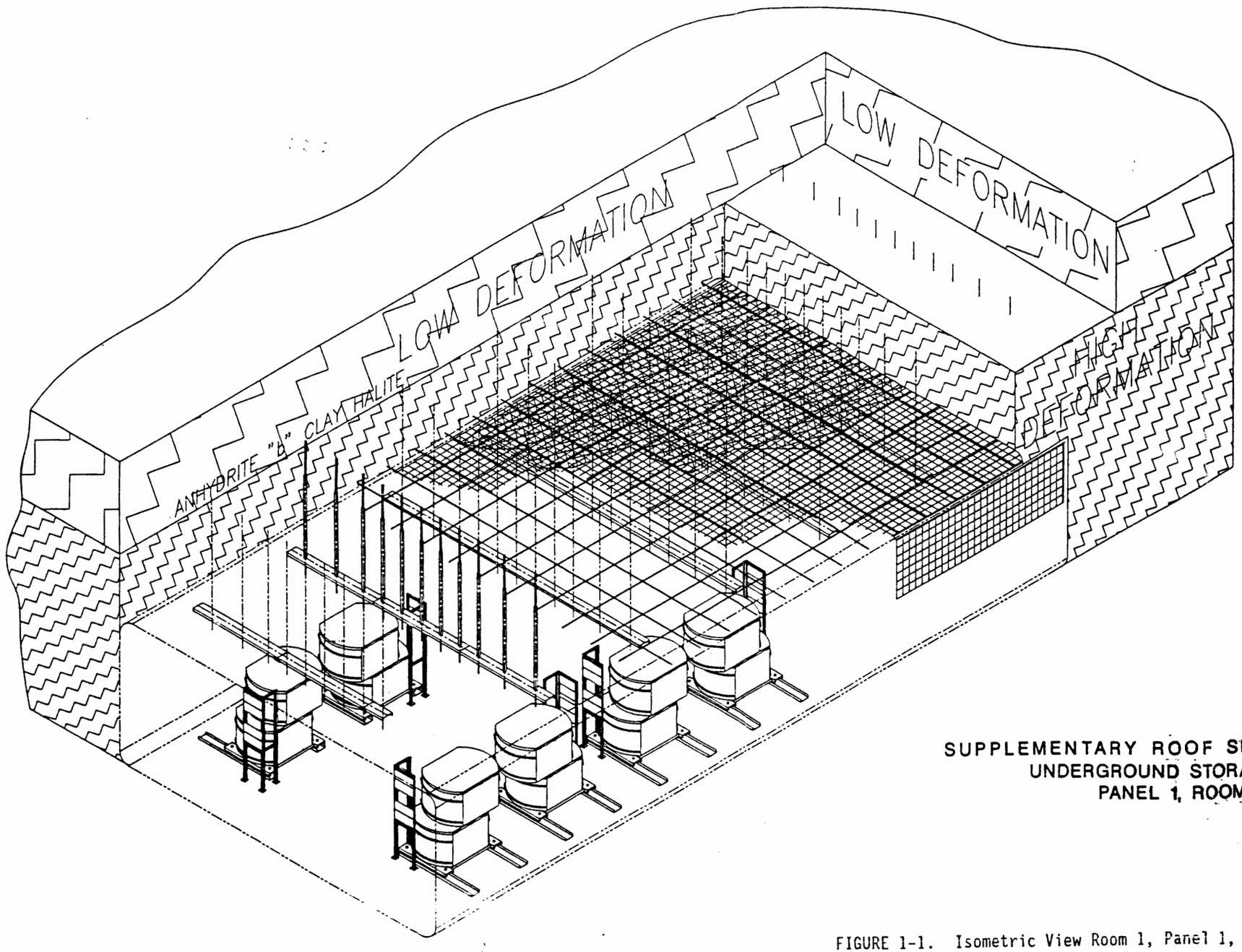
WIPP PROJEC
WILEY ENGIN
OCTOBE



MARCUS A. WILEY AFFIDAVIT
EXHIBIT 2

CT PERT CHART
INEERING, INC.
ER 5, 1991





SUPPLEMENTARY ROOF SUPPORT SYSTEM
UNDERGROUND STORAGE AREA
PANEL 1, ROOM 1

FIGURE 1-1. Isometric View Room 1, Panel 1, Supplementary Roof Support System

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 2-1.

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

- 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 $\pm$ 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 $\pm$ 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.

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 4-1.

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 2160 kg/m^3 (135 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 4-1.

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 4-2.

The facility horizon lies within a 12m (39.4 feet) thick unit consisting of halite, argillaceous halite, and polyhalitic halite (Figure 4-3). 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.

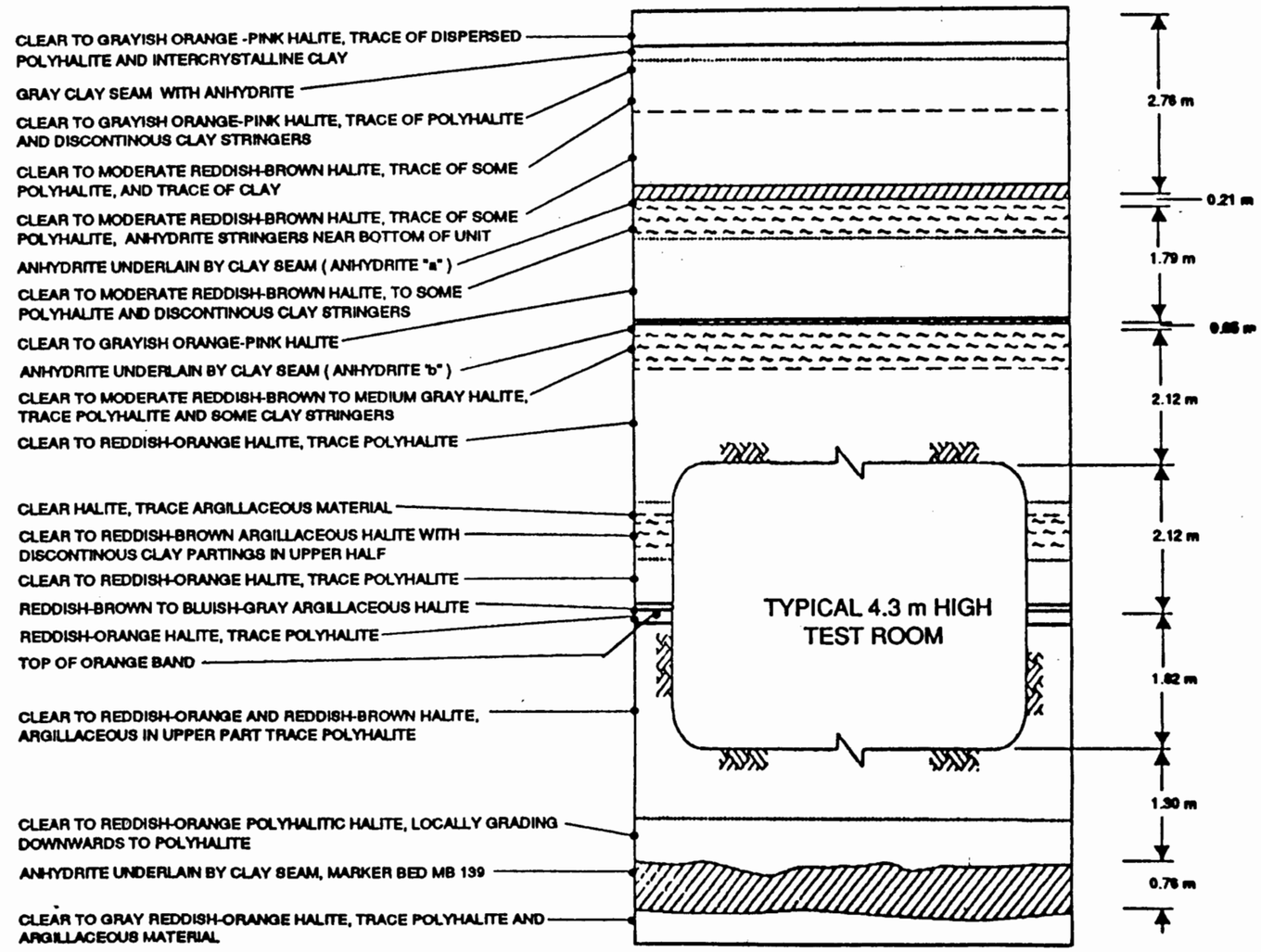


FIGURE 4-3. Stratigraphy of the WIPP Repository Horizon

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

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.

In order to cater for the possibility of shear and bending at Anhydrite "b" exceeding these limits, a set of five test anchors and four observation holes will be installed in Room 2 as a part of a mock-up. Periodic proof testing of the test anchors would be conducted when observations indicate that shearing and bending of the anchors are occurring.

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. Eleven rock anchor bolts are required to support the channel. In the unlikely event that the roof detaches as a rectangular shaped section, the anchor capacities are sufficient to cater for this extra load.

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. 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".

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 ± .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) ±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

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.

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

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 within 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.

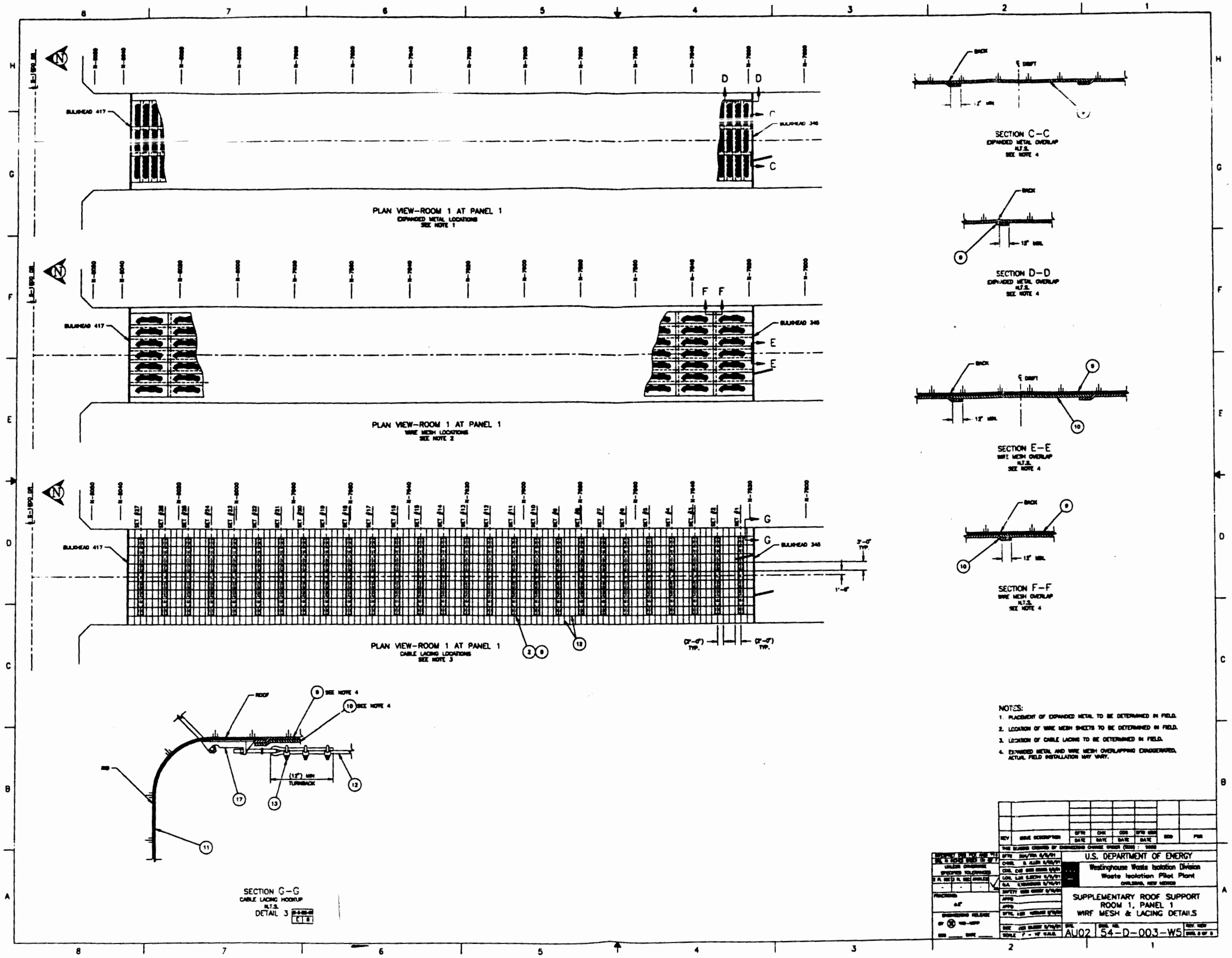
- Level survey - Changes in elevation will be monitored at selected locations along the support beams and the rock surface. The elevation surveys will identify areas of differential movement which, in addition to the results from installed geomechanical instrumentation, will establish the response of 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.

WASTE ISOLATION PILOT PLANT
SUPPLEMENTARY ROOF SUPPORT SYSTEM

DRAWINGS

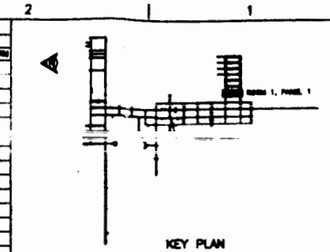
Westinghouse Electric Corporation
Waste Isolation Division
Carlsbad, New Mexico
October 1991


PLAN VIEW-ROOM 1 AT PANEL 1
EXPANDED METAL LOCATIONS
SEE NOTE 1
PLAN VIEW-ROOM 1 AT PANEL 1
WIRE MESH LOCATIONS
SEE NOTE 2
PLAN VIEW-ROOM 1 AT PANEL 1
CABLE LACING LOCATIONS
SEE NOTE 3
BULKHEAD 417
BULKHEAD 346
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
CABLE LACING HOOKUP
N.T.S.
DETAIL 3
ROOF
SEE NOTE 4
(12") MIN TURNBACK
BACK
12" MIN.
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.
U.S. DEPARTMENT OF ENERGY
Westinghouse Waste Isolation Division
Waste Isolation Pilot Plant
CARLSBAD, NEW MEXICO
SUPPLEMENTARY ROOF SUPPORT
ROOM 1, PANEL 1
WIRE MESH & LACING DETAILS
AU02
54-D-003-WS

LIST OF MATERIAL

QTY REQ SH1	QTY REQ SH2	QTY REQ SH3	ITEM NO.	ORDERING INFORMATION	MFR	DRAWING NO. OR PART NO.	MATERIAL SPECIFICATION MAIL	STD	NUMBER	COMP	FORM
		-	1	ENHANCED ROOF SUPPORT ASSEMBLY		54-D-003-0-01					
	-	AR	2	STEEL SET ASSEMBLY		54-D-003-0-02					
-	1		3	STEEL SET CENTER PIECE ASSEMBLY		54-D-003-0-03					
4			4	BAR, FLAT .38 THK STK	A		CS	ASTM	A36		
1	3		5	CHANNEL, C15X40	A		CS	ASTM	A36		
	4		6	BOLT, .63-11 UNC-2A X 2.00 LG, HEX HEAD	B	812076788	CS	ASTM	[illegible]		
	4		7	NUT, .63-11 UNC-2B, HEX HEAD	B	904734035	CS	ASTM	A563		
		AR	8	PLYWOOD, .75 THK STK 4'X10' SHT	F		CDX				
		AR	9	METAL, EXPANDED 10 GA. 4'X10' SHT	G		CS				
		AR	10	WIRE, MESH .25 W/4X4 OPENINGS 3'X14' SHT	G		CS				
		AR	11	FENCING, CHAINLINK 9 GA.	H		GAL.	ASTM	A392		
		AR	12	CABLE, 0.63 6X25 IWRC	B	34687T13					
		AR	13	CLAMP, CABLE	B	34687T10					
		AR	14	ROCKBOLT, 1.3' 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	3213490					
	8		20	WASHER, FLAT .63	B	918664035	CS	ASTM	F436		

KEY PLAN

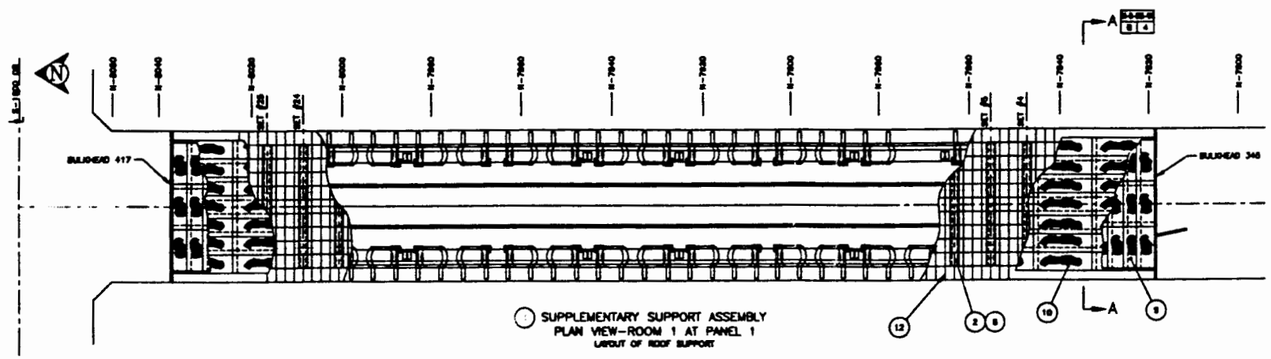


REFERENCE DRAWINGS

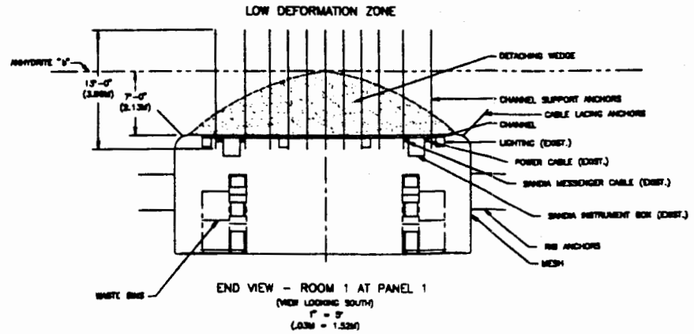
SM-SCALE TESTING PANEL 1 ROOM 1 POWER AND LIGHTING	53-J-053-W
UNDERGROUND WORK PANEL 1-PROFILES ALONG ROOM 1 AND S1600 DR	54-D-055-W
SM SCALE TESTS VOC MONITORING AND HELIUM STATION DETAILS	53-L-053-W
SM-SCALE TESTS ROOM 1 LAYOUT	53-D-051-W

SUPPLIERS:

A. RIVERSON OR APPROVED EQUAL
B. McMASTER-CARR OR APPROVED EQUAL
C. JOHNNAR CORP. OR APPROVED EQUAL
D. DYWIDAG OR APPROVED EQUAL
E. CELTITE OR APPROVED EQUAL
F. TEXAS LUMBER OR APPROVED EQUAL
G. RAWHELL OR APPROVED EQUAL
H. ACME FENCE OR APPROVED EQUAL



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



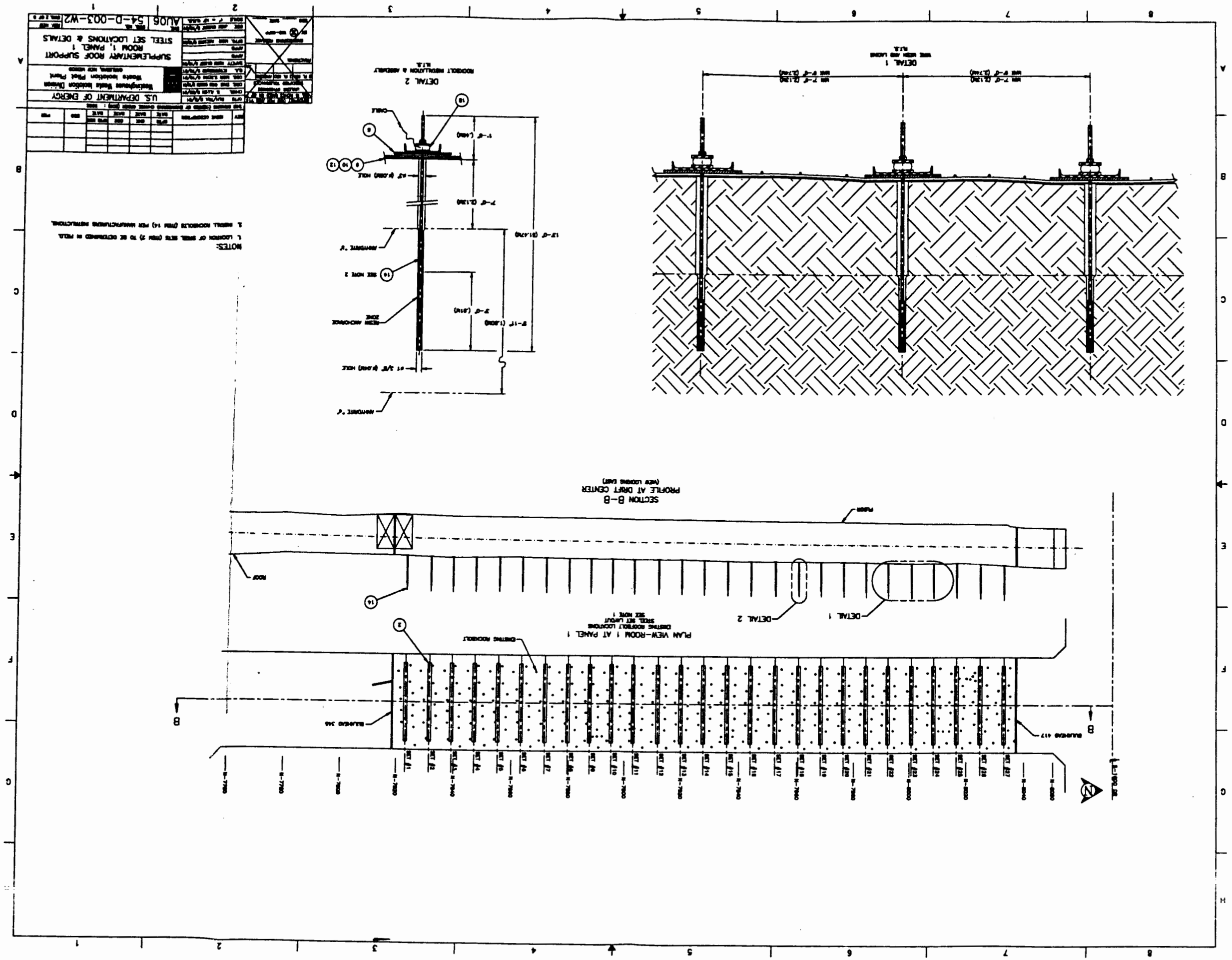
END VIEW - ROOM 1 AT PANEL 1
(VIEW LOOKING SOUTH)
1" = 5'
(.03M = 1.52M)

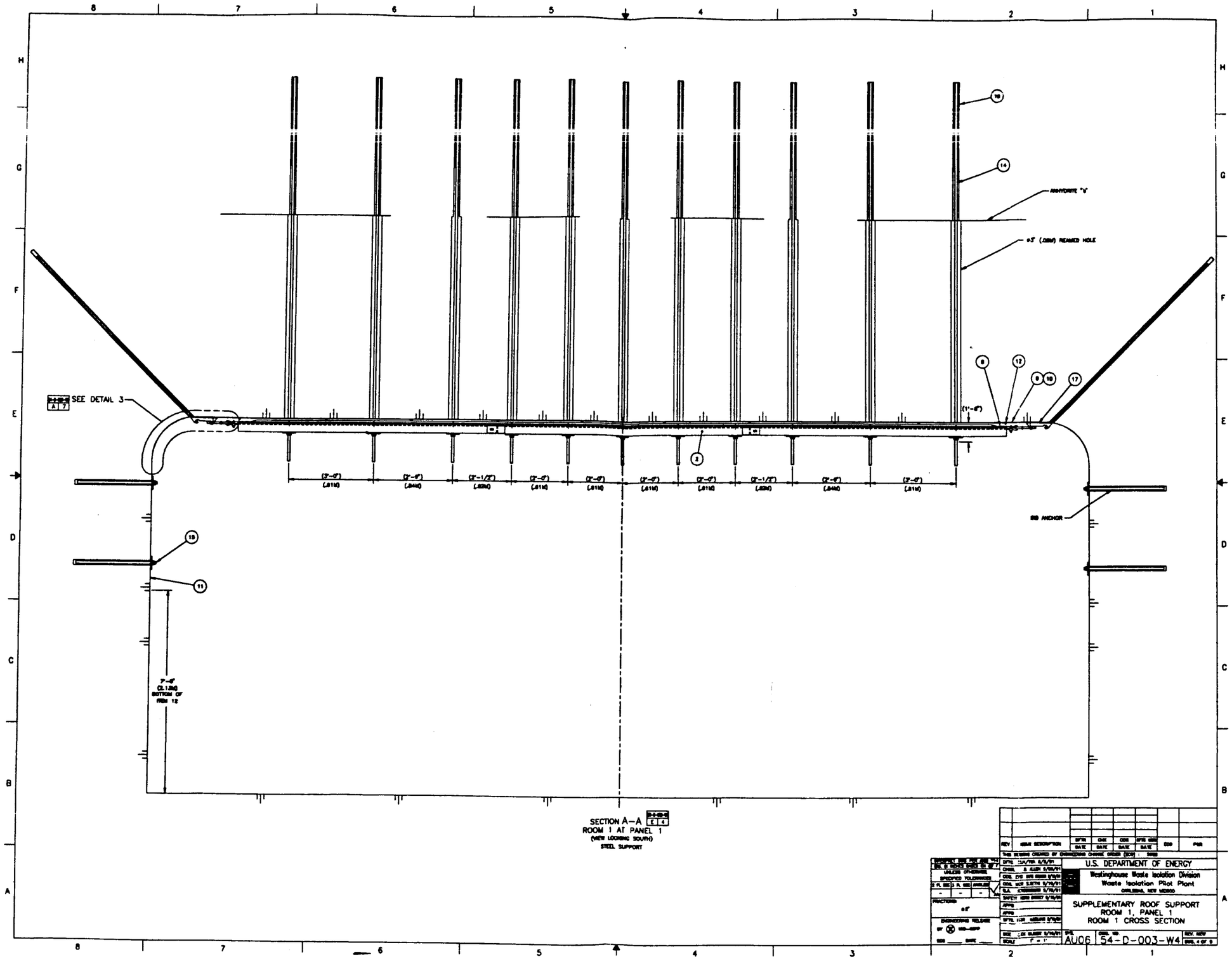
U.S. DEPARTMENT OF ENERGY
Westinghouse Waste Isolation Division
Waste Isolation Pilot Plant
CARLSBAD, NEW MEXICO

SUPPLEMENTARY ROOF SUPPORT
ROOM 1, PANEL 1
GENERAL ARRANGEMENT

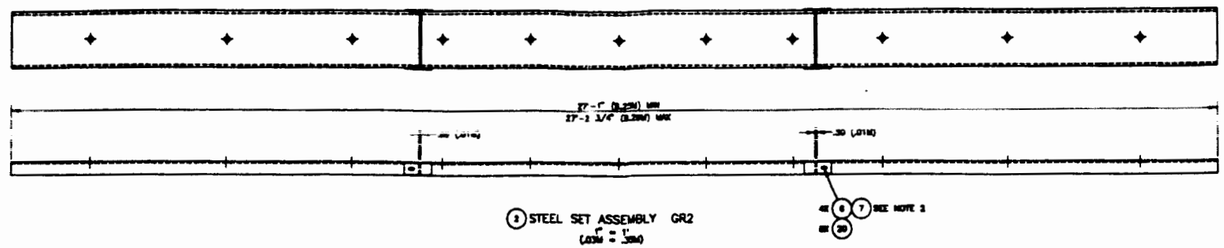
AU06 | 54-D-003-W1

2

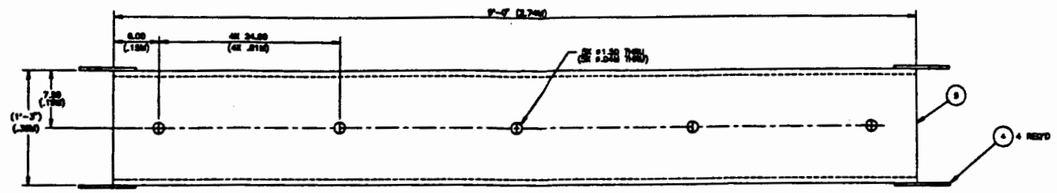

PLAN VIEW-ROOM 1 AT PANEL 1
EXISTING ROOFBOLT LOCATIONS
STEEL SET LAYOUT
SEE NOTE 1
EXISTING ROOFBOLT
DETAIL 1
DETAIL 2
ROOF
PANEL
SECTION B-B
PROFILE AT DRIFT CENTER
(VIEW LOOKING EAST)
DETAIL 1
WIRE MESH AND LOCKS
N.T.S.
DETAIL 2
ROOFBOLT INSTALLATION & ANCHORAGE
N.T.S.
ANCHORAGE "A"
ANCHORAGE "B"
NOTES:
1. LOCATION OF SMALL SETS (ITEM 2) TO BE DETERMINED IN FIELD.
2. INSTALL ROOFBOLTS (ITEM 14) PER MANUFACTURERS INSTRUCTIONS.
U.S. DEPARTMENT OF ENERGY
Yucca Mountain Site Characterization Project
Waste Isolation Pilot Plant
SUPPLEMENTARY ROOF SUPPORT
ROOM 1, PANEL 1
STEEL SET LOCATIONS & DETAILS
AU06 54-D-003-W2

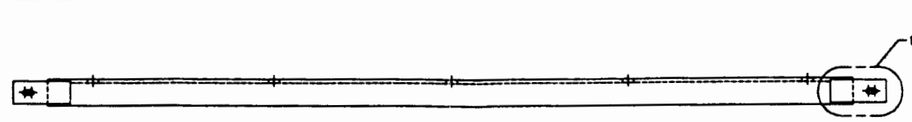

SEE DETAIL 3
ANHYDRITE "A"
RIB ANCHOR
7'-6" (2.13M) BOTTOM OF RIBS 12
SECTION A-A
ROOM 1 AT PANEL 1
(VIEW LOOKING SOUTH)
STEEL SUPPORT
U.S. DEPARTMENT OF ENERGY
Westinghouse Waste Isolation Division
Waste Isolation Pilot Plant
CARLSBAD, NEW MEXICO
SUPPLEMENTARY ROOF SUPPORT
ROOM 1, PANEL 1
ROOM 1 CROSS SECTION
AU06
54-D-003-W4

4

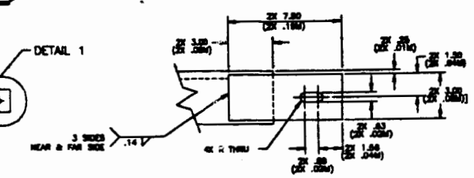


③ STEEL SET ASSEMBLY GR2
1" = 1'
(.03M = .30M)

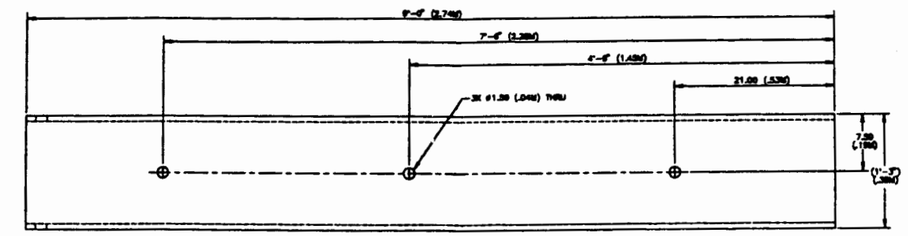


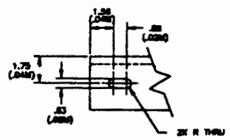


③ STEEL SET CENTER PIECE ASSEMBLY GR3
1" = 6"
(.03M = .15M)

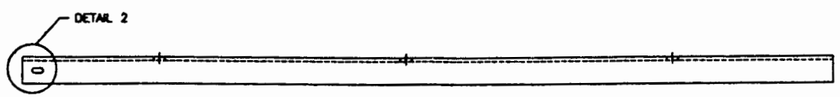


DETAIL 1
1" = 3"
(.03M = .08M)
(SIM OPPOSITE END)





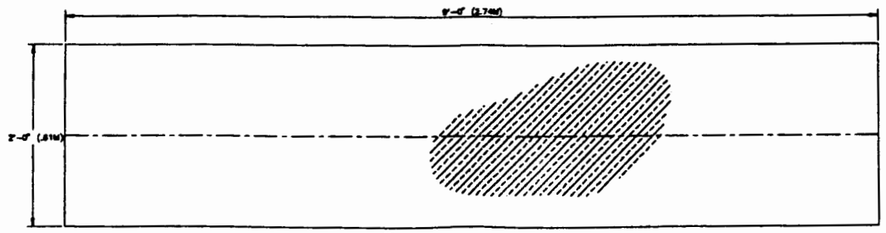
DETAIL 2
1" = 3"
(.03M = .08M)
(SIM OPPOSITE SIDE)



⑨ STEEL SET END PIECE
1" = 6"
(.03M = .15M)

NOTES:

1. USE ITEMS 3 AND 5 AS A TEMPLATE TO DRILL HOLES IN ITEM 6.
2. FINGER TIGHTEN ITEMS 6 AND 7 UPON INSTALLATION.



⑧ PLYWOOD STRIP
1" = 6"
(.03M = .15M)
SEE NOTE 1

REV	WORK DESCRIPTION	DFTR DATE	CHK DATE	CDD DATE	APR DATE	BOD	PMR

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

SIZE AU02 | DWG. NO. 54-D-003-W3 | REV. NONE | SHEET 2 OF 5

5

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 - _____ f

Test Date - _____ Time - _____ Torque - _____ f
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

ATT
WP
Pag

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~~ DX/8.13.91 ROOM ℄ APPROX. 4-6 FT FROM 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 11:30 dm COMPLETED AT
FOR CERTIFICATION.

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

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

ROCK BOLT CERTIFICATION TEST
DATA SHEET

Mine WIPP Rock HALITE Rock Bolt No. PTOB1391-2
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 - GRADE 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/l

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 258886 238A
Dial Indicator - AMES 4122 CD-0040 Calibrated - DUE 10-4-91
Pressure Gauge - WSG 4886 PSI #001 Calibrated - 08-06-91

PSI X100 Pump Pressure	LBS Bolt Tension	Extensometer Reading	Displacement	Remarks
2	2470	0.000		
4	4940	0.025		
6	7410	0.049		
8	9880	0.075		
10	12350	0.098		
12	14820	0.128		
14	17290	0.164		
16	19760	0.204		
18	22230	0.248		
20	24700	0.291		
22	27170	0.320		
24	29640	0.345		
26	32110	0.373		
28	34580	0.398		
30	37050	0.418		
32	39520	0.435		
33	40755	0.447		
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	0.579		
46	56810	0.595		
47	58045	0.611		REMOVE DIAL INDICATOR
48	59280	0.635		BLEED TO 4700 PSI
49		0.652		BLEED TO 4000 PSI 2 min
~~50~~	~~0~~	~~0.746~~		
51		0.846		
52				BLEED TO 4800 PSI
53		0.996		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 HRS AFTER INSTALLATION
Tested By - D. Ballmann Approved By - P.B. [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. BULK

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. P108 1391-3
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 - GR60 Length - 13'-0" Diameter - #8

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

Resin Manufacturer - CEMTITE Model - H30-0204 Diameter - 1¼"
Length - 12" No. Cartridges - 3

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

Test Date - Time - 2:00 PM Torque - N/A ~~ft/lb~~ 08/15/91
Test Jack - RC-60-CH 9.5A Test Pump - RC-60 & CH95 SN 238 236
Dial Indicator - AMES CD-0040 Calibrated - DUE 10-4-91
Pressure Gauge - US GAGE DSI #001 Calibrated - 05-06-91

PSI 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	0.085		
14	17290	0.106		
16	19760	0.127		
18	22230	0.151		
20	24700	0.179		
22	27170	0.199		
24	29640	0.222		
26	32110	0.244		
28	34580	0.268		
30	37050	0.292		
32	39,520	0.313		
33	40,755	0.325		
34	41990	0.335		
35	43225	0.350		
36	44460	0.360		
37	45695	0.372		
38	46930	0.385		
39	48165	0.398		Bled TO 3800 PSI
40	49400	0.413		
41	50635	0.429		
42	51870	0.439		
43	53105	0.456		
44	54340	0.475		
45	55575	0.489		
46	56810	0.519		
47	58045	0.544		Bled TO 4600 PSI ~~REMOVE DIAL INDICATORS~~
48		0.793		Bled TO 4600 PSI
49		0.834		Accepted 4900 Bled TO 4700
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 - 27.5 HRS AFTER INSTALLATION
Tested By - D. Ballrauch Approved By - [illegible]

ROCK BOLT CERTIFICATION TEST
DATA SHEET

Mine WIPP Rock HALITE Rock Bolt No. PTOB1391-6
Hole Location PANEL 1-Roo

Hole Length - 11'-7" Dia. - @ Anchor GAUGED /@ midpoint N/A /@ collar
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 Diameter - 1 1/4"
Length - 12" No. Cartridges - 3

Installation Date - 08/13/91 Time - 11:04 AM Torque - N/A f

Test Date - 8/15/91 Time - 12:50 PM Torque - N/A f
Test Jack - RC-60-CH 9.5A Test Pump - RC-60-CH 9.5 SN 23836
Dial Indicator - AMES CD-0040 Calibrated - DUE 10-4-
Pressure Gauge - U.S. GAUGE DSI #001 Calibrated - 08/06/91

PSI X 100 Pump Pressure	Bolt Tension LBS	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	0.209		
28	34,580	0.226		
30	37,050	0.244		
32	39,520	0.260		
33	40,155	0.269		
34	41,390	0.278		
35	43,285	0.288		
36	44,410	0.298		
37	45,695	0.309		
38	46,930	0.310		
39	48,165	0.321		
40	49,400	0.331		
41	50,635	0.340		
42	51,870	0.349		
43	53,105	0.363		
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~~
~~48~~	~~59,280~~			BLOCK TO 4700 PSI
49	60,315	0.470		AFTER 5 MIN. BLOCK TO 4700
50		0.616		REMOVE DIAL INDICATOR *
51		0.886 *		BOLT YIELD @ 3480
52				
53				TERMINATE TEST @ 1:20 PM

Test Results - ACCEPTED Maximum Pull Force 59,280 LBS
Displacement at Maximum Pull Force - APPROX. .886 IN
Nature of Failure/Yield - BOLT YIELD @ 3480 PSI
Other Remarks @ 49.8 HRS AFTER INSTALLATION; HOLD AT
Tested By - D. Dellinger Approved By - [illegible]

ROCK BOLT CERTIFICATION TEST DATA SHEET

Rock Bolt No. PT081391-6

Mine _____ Rock _____ 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			AT 4900	Holding AT 4700 PSI 1 1/2" RAM 59300# 1:35 PM
56			AT 5000	Holding AT 4700 PSI 1 1/2" RAM 60315# 1:40 1 1/4"
57			AT 5100	Holding AT 5000 PSI 1 1/4" 61350# 1:45 PM
			AT 5200	Holding AT 5000 PSI 1 1/4" RAM 61760# 2:00 PM
			AT 5300	Holding AT 5100 PSI 1 1/4" RAM 62988 2:07 PM
			AT. 5400	Holding AT 5200 PSI 1" RAM 64700# 2:13 PM
			AT. 5500	Holding AT 5300 1 3/8" RAM 65455# 2:25 PM
			AT 5600	Holding AT 5400 PSI 2 1/4" RAM 66690# 2:33
			AT 5700	Holding AT 5500 PSI 2 7/16" RAM 67425# 2:40
			AT 5800	Holding AT 5600 PSI 2 1/2" RAM 69160# 2:45
			AT 5900	Holding AT 5700 PSI 2 5/8" RAM 70875
			08/16/91	BLED TO 5700 PSI 12:40 PM 69330#
				MAX PULL FORCE

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

ATT
WP
Pag

UNDERGROUND DRILLING HOLE FIELD DATA SHEET

Date: 8/13/91

Room/Drift: PANEL 1 - ROOM 2

Purpose of Hole: PULL TEST: DESTRUCTIVE (PWR120177
DT081391-6

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

Depth at Completion: 11'-7"

Diameter: 1 3/8" d 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

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

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 - G80 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 - 1¼"
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 4427 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	8/14	
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		
3500	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	0.399	394 DB 8/14/91	
4100	50635	0.404		
4200	51870	0.409		
4300	53105	0.419		
4400	54340	0.425		
4500	55575	0.435		
4600	56810	0.451		HELD - BLED BACK TO 4600 B.Y.
4700	58045	0.474		REMOVED DIAL INDICATOR DB 8/14/91
4800	59280	0.559		ATTEMPTED 4900 BLED TO 4700
4900	X	X		REMOVED DIAL INDICATOR.
5000				HELD FOR 5 MIN. BLED TO 4900
5100				PUMPED 4800 PSI HOLD AT 4800
5200				WORKED BOLT GO TO 4900
5300				TEST TERMINATED @ 12:05 PM

39,520#

Test Results - ACCEPTED Maximum Pull Force - 59280 #
Displacement at Maximum Pull Force - .559 in.
Nature of Failure/Yield - Bolt YIELD 4600 PSI to 4700 PSI
Other Remarks - @ 25.3 KPS AFTER INSTALLATION
Tested By - D. Ballinger Approved By - [illegible]

Test 1

ROCK BOLT CERTIFICATION TEST
DATA SHEET

Page 2 of

Mine _____ Rock _____ Rock Bolt No. PT081391
Hole Location PANEL - R

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

Bolt Grade - _____ Length - _____ Diameter - _____

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

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

Installation Date - _____ Time - _____ Torque - _____

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

Pump Pressure	Bolt Tension	Extensometer Reading	Displacement	Remarks
~~50~~			Ram 1/8	AT 5800 Holding AT 50
~~55~~				
~~56~~				
~~57~~				

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

UNDERGROUND DRILLING HOLE FIELD DATA SHEET

Date: 8/13/91

Room/Drift: ROOM 2 - PANEL 1

Purpose of Hole: PULL TEST: DESTRUCTIVE (PWR 120175)
P+081391-7

Location: N. END 4+/ W Rib ¢ 45.50 FT. FROM N. BULKHEAD

Depth at Completion: 11'-7"

Diameter: 1 3/8" ¢

Approximate Collar Coordinates

Vertical Angle: VERT. (5° TOLERANCE)

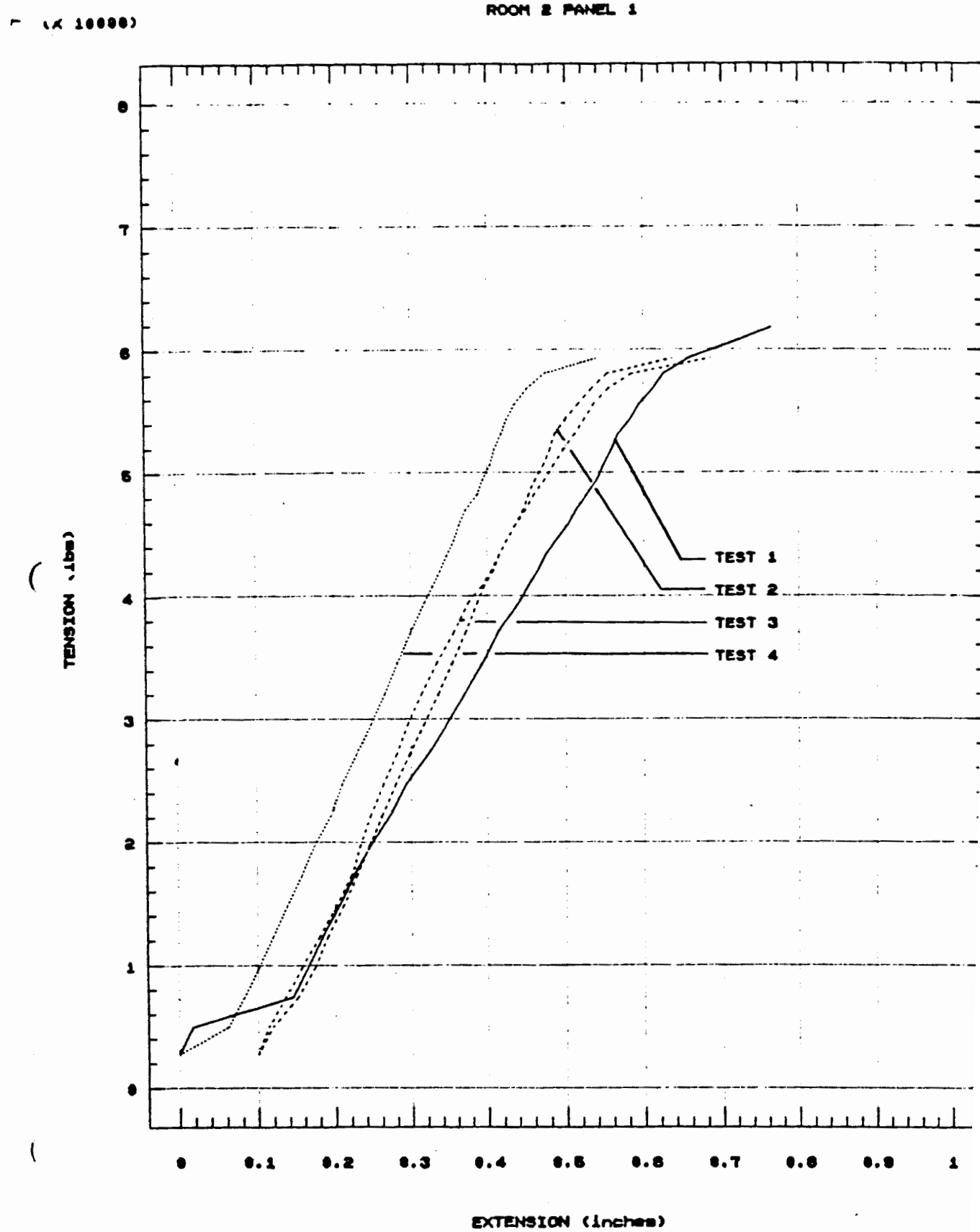
Direction/Azimuth:

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

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

BOLT TENSION VERSUS EXTENSION

ROOM 2 PANEL 1



25 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 - HD0.0204 Diameter - 1 1/4"
Length - 12" No. Cartridges - 3

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

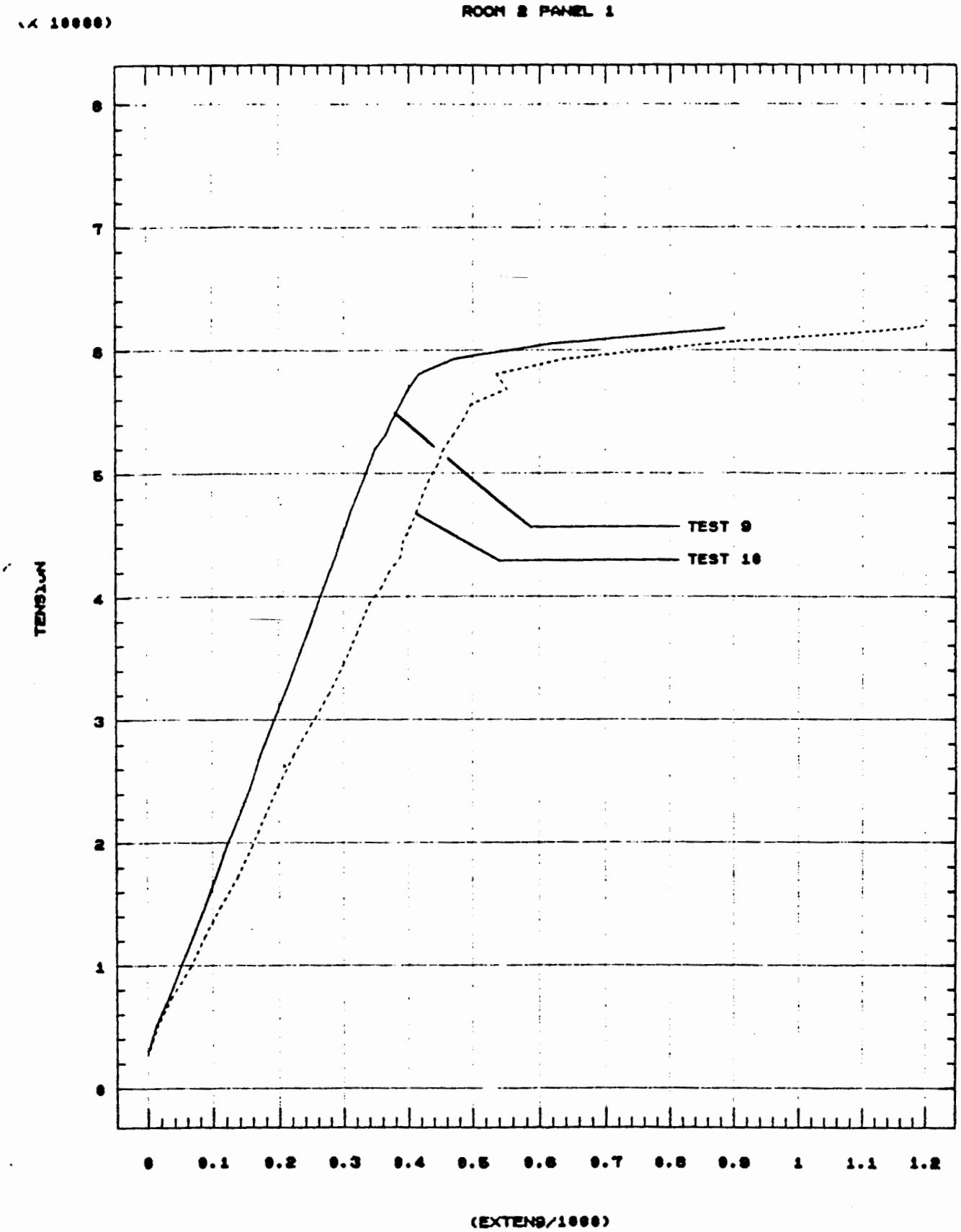
Test Date - 08/16/91 Time - 1:05 PM Torque - N/A ft/lb
Test Jack - RC-60-CH95A. Test Pump - RC-60-CH95 SN 23886
Dial Indicator - AMES CD-0040 Calibrated - DUE 10-4-91
Pressure Gauge - U.S. GAUGE DSI # 001 Calibrated - 08/06/91

PSI X 100 Pump Pressure	LBS Bolt Tension	Extensometer Reading	Displacement	Remarks
2	2470	0.000		
4	4940	0.015		
6	7410	0.035		
8	9880	0.055		
10	12350	0.085		
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	0.359		
34	41,990	0.370		
35	43,225	0.380		
36	44,460	0.391		
37	45,695	0.405		
38	46,930	0.418		
39	48,165	0.431		
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		Pump to 4600 PSI
47	58,045	0.536		REMOVE DIAL INDICATOR
48	XXXXX	0.636		At Impt. 4800 Pump to 4600
49		0.862		At. 4300 Holding 4800 59.8
50		1.171		At 500 Holding 4900 60.3
51		1.333		At 500 Holding 5100 62.9
52		1.714		At 400 Holding 5300 65.4
53			Ram 2 3/4	At 500 Holding 5500 67.92

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 - 83.5 HOURS AFTER INSTALLATION
Tested By - D. Dallmann Approved By - P.K. Brockman

BOLT TENSION VERSUS EXTENSION

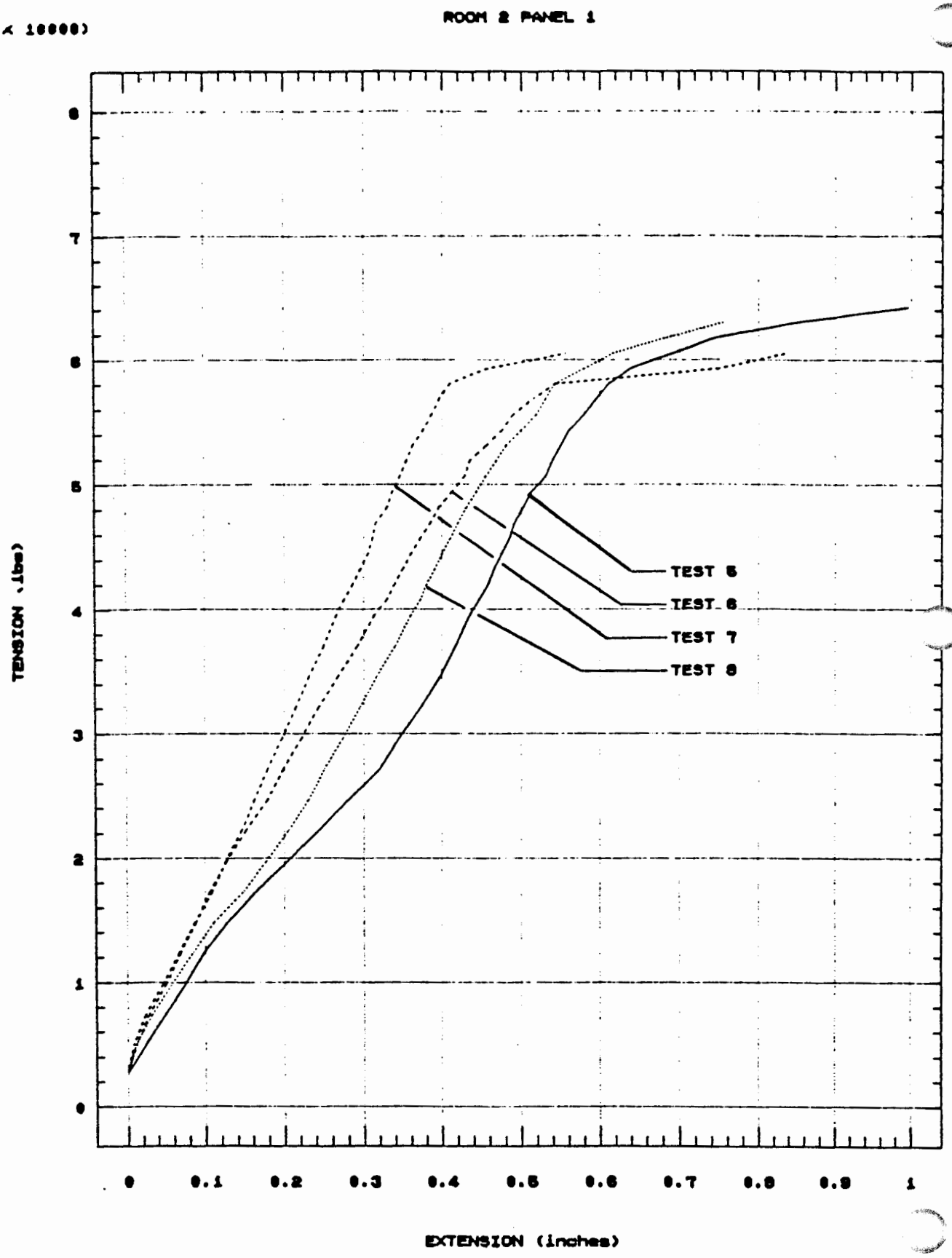
ROOM 2 PANEL 1



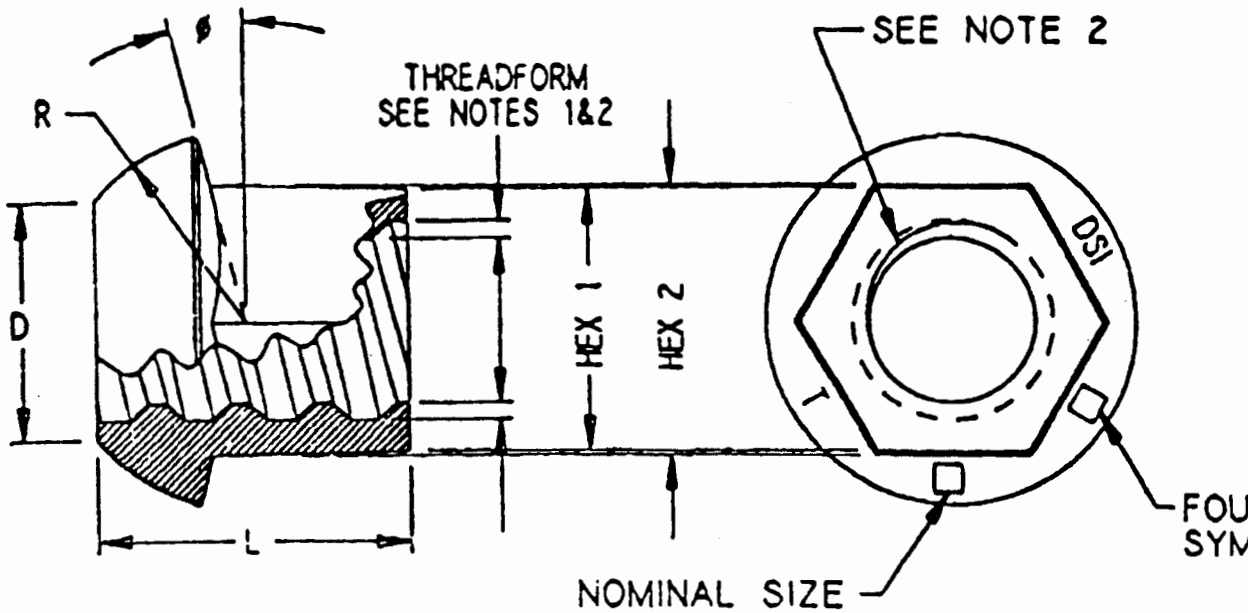
2 AND 3 DAYS CURE

BOLT TENSION VERSUS EXTENSION

ROOM 2 PANEL 1



25 HOURS CURE



NOMINAL SIZE	#6	20mm	#7	[redacted]	#9	#10	PRE
PART NUMBER	B06U28810	B20U28810	B07U28810	B08U28810	B09U28810	B10U28810	B1
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	2.6
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	2.0
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	1.5
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	1.8
Ø DEG.	14	14	14	14	14	14	
THREADFORM: 8341/	743	785	744	745	748	747	
MINOR Ø MIN. (IN./mm)	0.730/18.5	.782/19.9	0.856/21.7	0.974/24.7	1.096/27.8	1.238/31.4	1.
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.
WEIGHT LBS	0.27	0.54	0.5	0.72	0.89	1.20	

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": 6%

TOLERANCES:

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

FOR L: +0.125/-0"

FOR Ø: ±1°

NOTES:

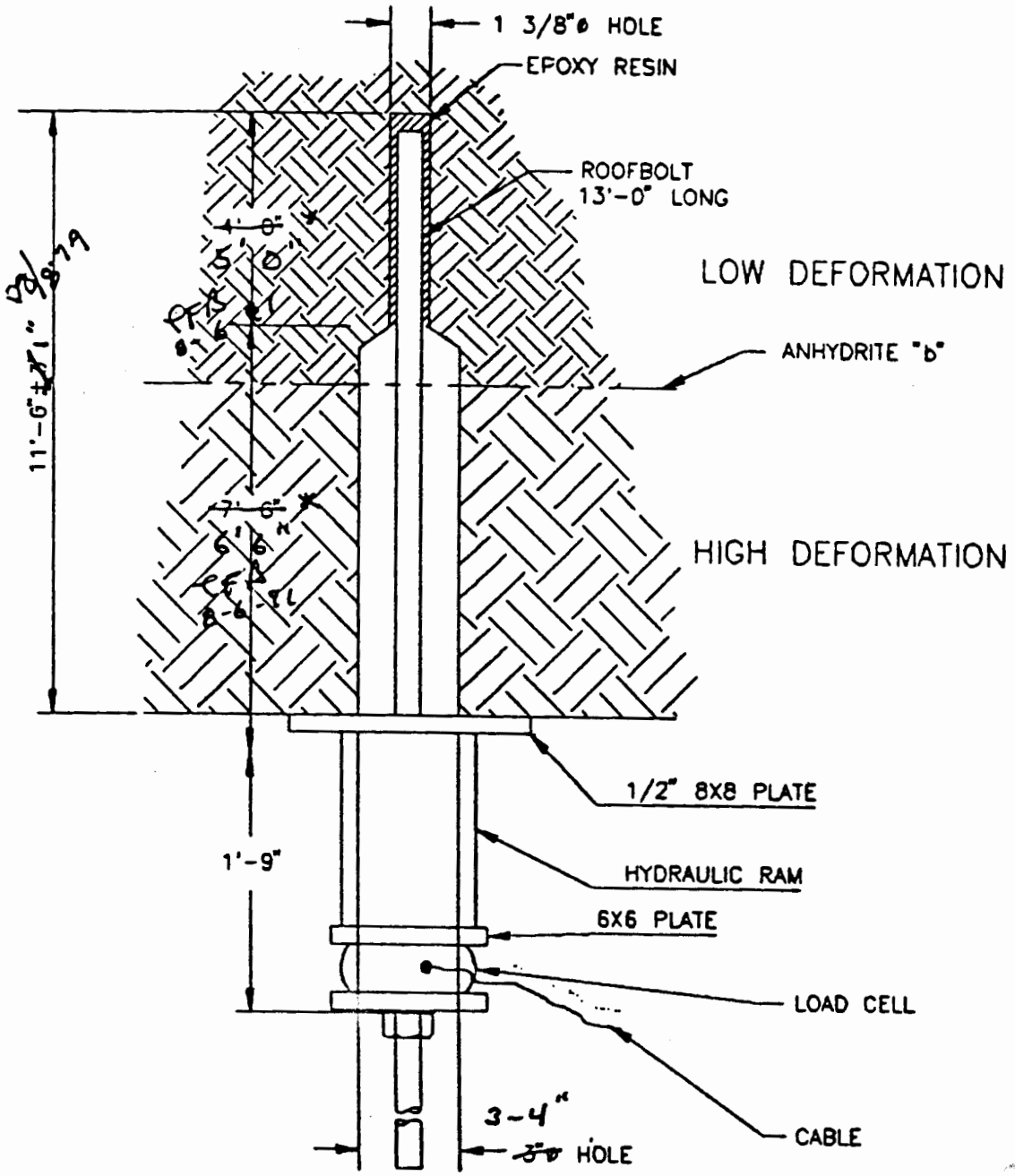
1. THREADFORM AS SPECIFIED WITH EXCEPTION OF
   DIAMETER (MINOR Ø) WHICH IS SPECIFIED IN TA
2. AT BOTH ENDS OF NUT THREADFORM IS MODIFI
   OVER LAST 1/2 TURN AS FOLLOWS: MINOR Ø
   OPENS UP TO MAJOR Ø AT CONSTANT RADIUS.

ATTACHMENT 5

DYWIDAG POST-TENSIONING SYSTEMS	DIMENSIONS: INCH/mm mm FOR REFERENCE ONLY	REV.	DATE	NAME
[redacted]		1	02-20-90	K.L.
		2	08-01-90	K.L.
		3	08-28-90	K.L.
		4	01-29-91	K.L.
	MATERIAL: SEE ABOVE	SCALE 1:1 FOR #7		

DATE 2-12-86 DRAWN J. MALESIC APPROVED:

* CHANGE MADE DUE TO AVAILABLE EQUIPMENT
CFB
8/6/81



ROOFBOLT DETAIL
VERTICAL ALIGNMENT (TOL. 5°)

ATTACHMENT 6

WASTE ISOLATION PILOT PLANT
SUPPLEMENTARY ROOF SUPPORT SYSTEM

APPENDIX B
Destructive Tests

Westinghouse Electric Corporation
Waste Isolation Division
Carlsbad, New Mexico
October 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.

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)
Guaranteed per ASTM	36	54	8	45	60	8	47.4	71.1
Avg. actual per mill cert.	42	65	12	47	72	9	55	83

6/24/91

ATTACHMENT 4.1

Heat # 9-62393

NORTH STAR STEEL MINNESOTA

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

CERTIFIED TEST REPORT

Size 18 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 KSI	YIELD K.S.I.	TENSILE KSI	TENSILE K.S.I.	GAUGE LENGTH (in)	% ELONG	SEND	% R.A.	Specification/Grade
.79	54.9	69.3	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		9606

Grain Size: Cleanliness:

Macro Etch: Specification: Results:

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

* By specification or request or

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 the 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 WH REQUESTED.

ATTACHMENT 4.2

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: _____

_____

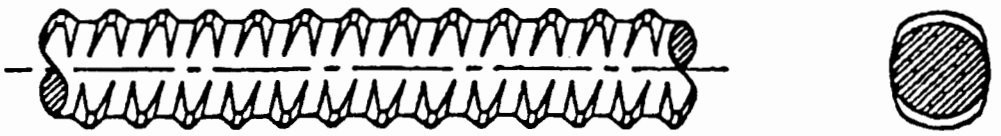
_____

_____

Signature/Department: _____

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

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
[redacted]	TOLERANCES: ± .02 UNLESS NOTED				
DATE: 08-23-90 DRAWN: B. MALECKI APPROVED:	MATERIAL: ASTM A615, GR 60	SCALE NTS			

This drawing, the pertinent [illegible], descriptions, specifications etc. and their contents are the

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 ANCHORA

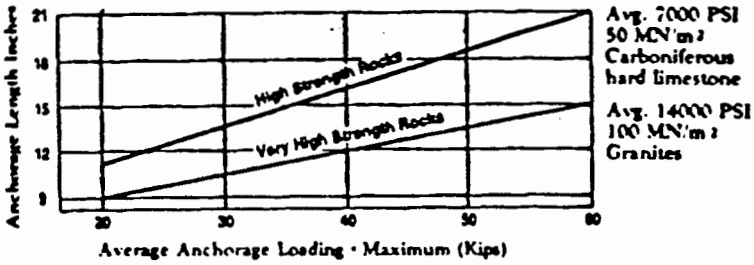
Lokset® cartridges can be ap in underwater anchorage a cations over 24 inches in lengt both concrete and rock. *Co CELTITE® TECHNIK for specific job requirements.*

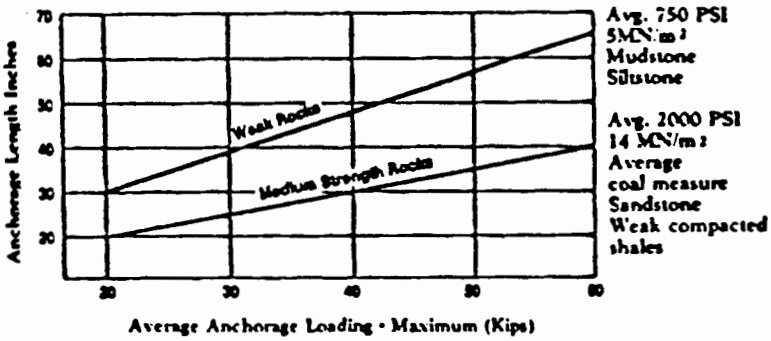
APPLICATION EQUIPME

The hole-drilling equipment tioned in this brochure is gene suitable for spinning in bolts. equipment should be rotary per sive and have provision for inde dent rotation to maintain 100 under load. Under no circumsta should the bolt be simply pu through the resin cartridges as proper mixing can result, prov for possible anchorage failure.

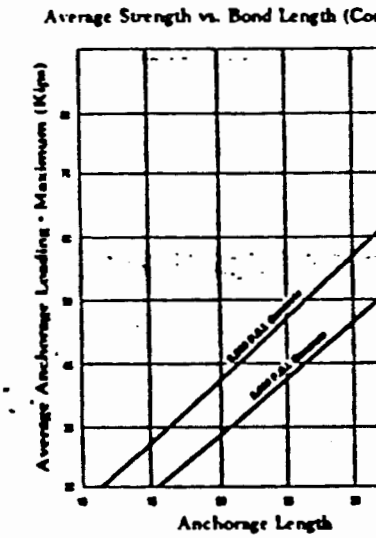
RESIN ANCHORAGE CHARTS

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 PROCEDUR

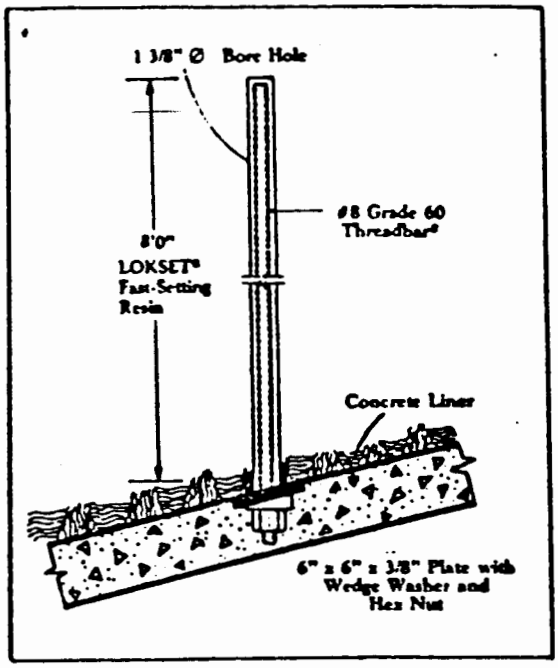
Lokset® cartridges should be stor in a cool, well-ventilated and dry ar away from direct sunlight. High tem perature conditions can reduce she life. Cartridges stored in extrer temperatures should be "normalize at 50-70°F for at least two days pr to use to provide the expected gel time

Pallets should not be stacked. Sto rotation is recommended so that t oldest stock is used first.

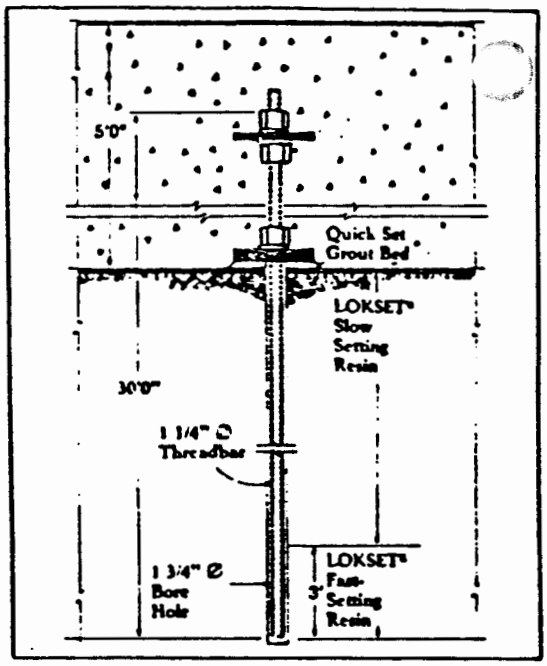
SAFE HANDLING PROCEDURE

REFER TO THE MATERIA SAFETY DATA SHEET. Do not ope or puncture cartridges prior to inser tion. Contents of cartridges may cau mild irritation and contact should b avoided. Use in adequately ventilate area. Avoid prolonged inhalation o vapor. Eye protection should always b used when bolting. If resin contacts th eyes, flush immediately with water fo at least 15 minutes. Seek medica 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.

Dy/G. Sealy & G. Greenfield Celtite

GEORGETOWN, KY : 150 CARLEY CT. 4032
502/863-6800
PRINCETON, WV Rogers St & Capertion Ave 24740 304-425-7
GRAND JUNCTION, CO. 1440 Winters Ave 81501 303-245-4007

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 bo

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

10. Note for stoper Contact Celtite® technical representative.

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

12. Cartridges should be stored in a cool, well-ventilated and dry area av from direct sunlight. High temperature conditions can reduce shelf of cartridges. Stock rotation is recommended so that oldest stoc 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 us 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 responsi 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 ab 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 workma ship NO OTHER WARRANTY, WHETHER EXPRESS IMPLIED OR STATUTORY, INCLUDING ANY WARRANT OF MERCHANTABILITY OR FITNESS F A PARTICULAR PURPOSE SHALL EXIST IN CONNECTION WITH THE SALE OR USE OF ANY CELTITE® PRODUCT, AND ALL SUCH WARRANTIES A HEREBY EXPRESSLY EXCLUDED.

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

U S Patent Nos 3731791, 3915297 Canada No 872075 Other World Patents.

00-00
04-22

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.

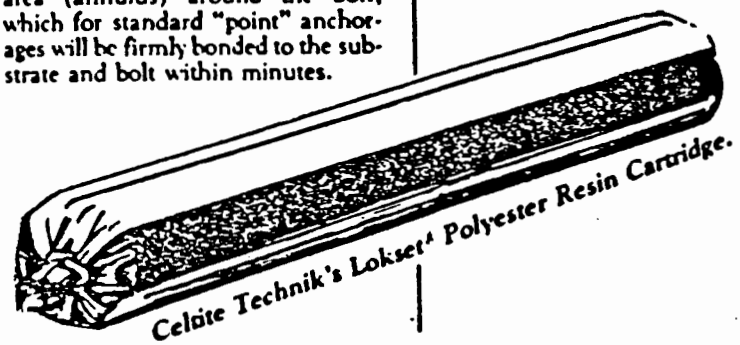
No reaction takes place until the roof or rockbolt is rotated through the cartridge, mixing the components and initiating the curing action. The chemical nature (thixotropy) of the Lockset® cartridge allows the contents to be easily mixed yet minimizes resin displacement after mixing is complete.

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 [illegible] critical jobs such as roof sup[illegible] 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, economic anchoring systems.

ATTACHMENT 3

SECTION 1 - PRODUCT IDENTIFICATION AND USE

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

CANADIAN CONTACT: FOSECO CANADA
GUELPH, ONTAR

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	I C
Polyester resin CAS# N.R.	10-30	N.E.	N.E.	NO	NO	N
Styrene monomer CAS# 100-42-5	5-10	50P	100P	NO	NO	2
Benzoyl Peroxide CAS# 94-36-0	.5-1.5	5M	N.E.	NO	NO	N
Calcium Carbonate CAS# 471-34-1	60-100	N.A.	N.E.	NO	NO	N
Propylene Glycol CAS# 57-55-6	.5-1.5	N.E.	N.E.	NO	NO	N

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 Determin
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 Communicati 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 Sty
PERCENT VOLATILE: 11
SOLUBILITY IN WATER: N.A.
VAPOR PRESSURE(mmHg): 4.5 @ 20C(Styrene)SPECIFIC GRAVITY: 1.75-1.
VAPOR DENSITY: 3.6 as Styrene
EVAPORATION RATE: N.D.
BOILING POINT: 293F
FREEZING POINT: N.D.
pH: Acidic
COEF. OF WATER/OIL DISTRIBUTI
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 in material and collect.
WASTE DISPOSAL METHOD: Dispose of in accordance with Federal, State, 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 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 Reauthorizat 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%

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 Rocco

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

P.F. Broadway 8/7/91

S. C Sethi, Manager
Mine Engineering

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 Bryant

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.

4.0 TEST MATERIALS

4.1 Resin: Celtite Lockset Polyester Resin Cartridge
High Viscosity, Code (H)
Gel Time, Two-to-Four Minutes Code (90) 0204 DG 8.7.91
Cartridge, 3212

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 AIR CONDITIONS THE BAR IS TO BE INSERTED TO FULL DEPTH THE

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 2
5	7.2.7b	CHANGE 40,000# TO READ 3,200 P.S.I
5	7.2.7	CHANGE 500# TO READ APPROXIMATELY 100

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-F

9. Required Approval

	Signature	
PIC All Classes	M. Campbell	8
Originator All Classes	T.E. Brockman	8
Safety As Required	[illegible]	8
Quality Assurance II or IIIA	[illegible]	8
Cognizant Engineer II or IIIA	for T.A. GONZALES D. Bellinger	8
Cognizant Operations Mgr. All Classes	T.E. Brockman	9
Other		
Other		

WP Form 1192; 8/9/90
Page 1 of 1

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.

ROCK BOLT CERTIFICATION TEST
DATA SHEET

Mine WIPP Rock HALITE Rock Bolt No. PTOB00091-3
Hole Location PANEL 1 - ROOM

Hole Length - 11.58 Dia. - @ Anchor GAUGED /@ midpoint N/A /@ collar N
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
Length - 12" No. Cartridges - 3

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

Test Date - 08/09/91 Time - 3:30 pm Torque - N/A ft
Test Jack - RC-60-CH95A Test Pump - RC-60.H95 28886
Dial Indicator - ACME 4822 CD-0040 Calibrated - 10-4-91
Pressure Gauge - KS GAGE DS1#001 Calibrated - 9-6-91

PSI Pump Pressure	LBS Bolt 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 TO 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
~~4800~~	~~59280~~	0.686		BOLT YIELD - FAILURE APP
4800 ATTEMPTED				CONTINUE ~~LOADING~~ PUMPING
				DEF. .686 AT 4600 GAUGE
				.702 AT 4600 ROD
				~~TEST TERMINATED @ 3:45~~

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) APP
Other Remarks - 24 HOURS AFTER INSTALLATION
Tested By - D. [illegible] Approved By - P.E. [illegible]

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. Gallinaire / 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 - 11.58' 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"/32mm
Length - 12" No. Cartridges - 3

Installation Date - 08/08/91 Time - 1:13 pm Torque - N/A ft/
08/8/91

Test Date - 08/09/91 Time - 2:56 pm Torque - N/A ft/
Test Jack - RC-60-CH-25 Test Pump - RC-60-CH-25 23886
Dial Indicator - AMES 4322 LD-0040 Calibrated - 10-4-91
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~~ 8-9-91	2470	
400	0.118	0.100 8-9-91	4940	
600	0.151	0.100 8-9-91	7410	
800	0.174	0.100 8-9-91	9880	
1000	0.191	0.100 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	0.299		27170	
2400	0.317		29640	
2600	0.336		32110	
2800	0.354		34580	
3000	0.372		37050	
39,520# 3200	0.388		39520	HELD FOR 2MIN - BLED TO 3000 psi
3300	0.396		40755	HELD AT 3400
3400	0.408		41990	HOLDING AFTER 2 MIN.
3500	0.415		43225	PS/8/09/91
* 3600	0.425	0.326 *	44460	*ADJUST TO ~~0.328~~ 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~~ 8-9-91		BOLT YIELD - PUMP BLEEDS TO 4700; BACK @ 4600 0.571 AT 4700
				TEST TERMINATED @ 3:05 pm PS/8/9/91

Test Results - ACCEPTED Maximum Pull Force - ~~54810~~ 58045
Displacement at Maximum Pull Force - 0.530 in.
Nature of Failure/Yield - BOLT YIELD (APPARENT) - 08/8-9-91 4700 psig
Other Remarks - 24 HOURS AFTER INSTALLATION
Tested By - A. Baldwin Approved By - P.E.

UNDERGROUND DRILLING HOLE FIELD DATA SHEET

Date: 08/08/91

Room/Drift: PANEL 1 - ROOM 2

Purpose of Hole: PULL TEST : DESTRUCTIVE (PWR 120177)
PT080891-2

Location: NORTH END: APPROX. 4' E OF ROOM 2 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. Balluaith / Mine Engineering

ROCK BOLT CERTIFICATION TEST
DATA SHEET

Mine WIPP Rock HALITE Rock Bolt No. PT080891- 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/

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

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

Test Results - ____ Maximum Pull Force - ____
Displacement at Maximum Pull Force - ____
Nature of Failure/Yield - ____
Other Remarks - ____
Tested By - D. Bellstrom Approved By - R.E. Beach

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'' 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:
TEST COMPLETED 2:30 pm FOR CERTIFICATION.

Signature/Department: D. Ballenaut / Mine Engineering

UNDERGROUND DRILLING HOLE FIELD DATA SHEET

Date: 8/13/01

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. BULKHE

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

ROCK BOLT CERTIFICATION TEST
DATA SHEET

Mine WIPP Rock HALITE Rock Bolt No. PT080891-1 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 - 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 Time - 1:45 pm Torque - N/A ft/lb
Test Jack - RC-60-CH9.5A Test Pump - RC-60-CH9.5A
Dial Indicator - AMES A822 CD-0060 Calibrated - 10-4-91 DUE
Pressure Gauge - A2 GAGE DSI#001 Calibrated - 8-6-91

Pump Psi Pressure	LBS Bolt Tension	Extensometer Reading	Displacement	Remarks
200	2470	0.000		
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		BLED TO APPROX. 2100
2400	29640	0.347		NO IG. PG
2600	32110	0.371		
2800	34580	0.394		
3000	37050	0.414		
3200	39520	0.441		BLED BACK TO 3100 AFTER 2 MIN.
~~3400~~	~~41990~~	0.464		* MISSED READING AT 3300 PSI
~~3600~~				
~~3800~~				
~~4000~~				
~~4200~~				
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 AFTER
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
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

39,520

Test Results - ACCEPTED Maximum Pull Force - 59280#
Displacement at Maximum Pull Force - 0.765 in. 0.842 in.
Nature of Failure/Yield - BOLT YIELD ~~4800 PSI~~ 4800 PSI
Other Remarks - CONTINUED PULL ON 8/12/91 TO 6700 PSI with approx 2.3" elongation
Tested By - D. Wallmann 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. BULK

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. Balhrain 8/14/91

ROCK BOLT CERTIFICATION TEST
DATA SHEET

Mine WIPP Rock HALITE Rock Bolt No. PTOX1391-5
Hole Location PANEL 1-ROOM

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 - H30 .020d Diameter - 1 1/4"
Length - 12" No. Cartridges - 3

Installation Date - 08/13/91 Time - 10:54 am Torque - N/A ft/
Test Date - 08/14/91 Time - 2:38 pm Torque - N/A ft/
Test Jack - RC-60-CH 9.5A Test Pump - RC-60-CH 9.5 SN 23886
Dial Indicator - ALMES CD-0040 Calibrated - DUE 10-4-91
Fressure Gauge - US GAGE DSI #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	0.616		Bind TO 4800 PSI
~~50~~	~~08/14/91~~	0.685		Bind 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 - 0.255
Nature of Failure/Yield - BOLT YIELD @ 4700 PSI
Other Remarks N/A
Tested By - D. Hallmark Approved By - T.E. Brock

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 BULKHEA

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 2:00 pm AND COMPLETED
FOR CERTIFICATION

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

ROCK BOLT CERTIFICATION TEST
DATA SHEET

Mine WIPP Rock HALITE Rock Bolt No. PT091391-4
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 ft/lb
Test Jack - RC-60-CH 9.5A Test Pump - RC-60-9.5 SN 23886 23886
Dial Indicator - AMES CD-0040 Calibrated - DUE 10-8-91
Pressure Gauge - LEG GAGE DSI 8001 Calibrated - 08/08/91 08/06/91

PSI X100 Pump Pressure	LBS Bolt Tension	Extensometer Reading	Displacement	Remarks
2	2470	0.000		
4	4940	0.007		
6	7410	0.023		
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	0.308		
37	45695	0.311		
38	46930	0.313		
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		Bled TO 4600 PSI
47	58045	0.410		Remove DIAL INDICATOR
48		0.456		Bled 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 PSI
Other Remarks - N/A
Tested By - D. Pattison Approved By - [illegible]

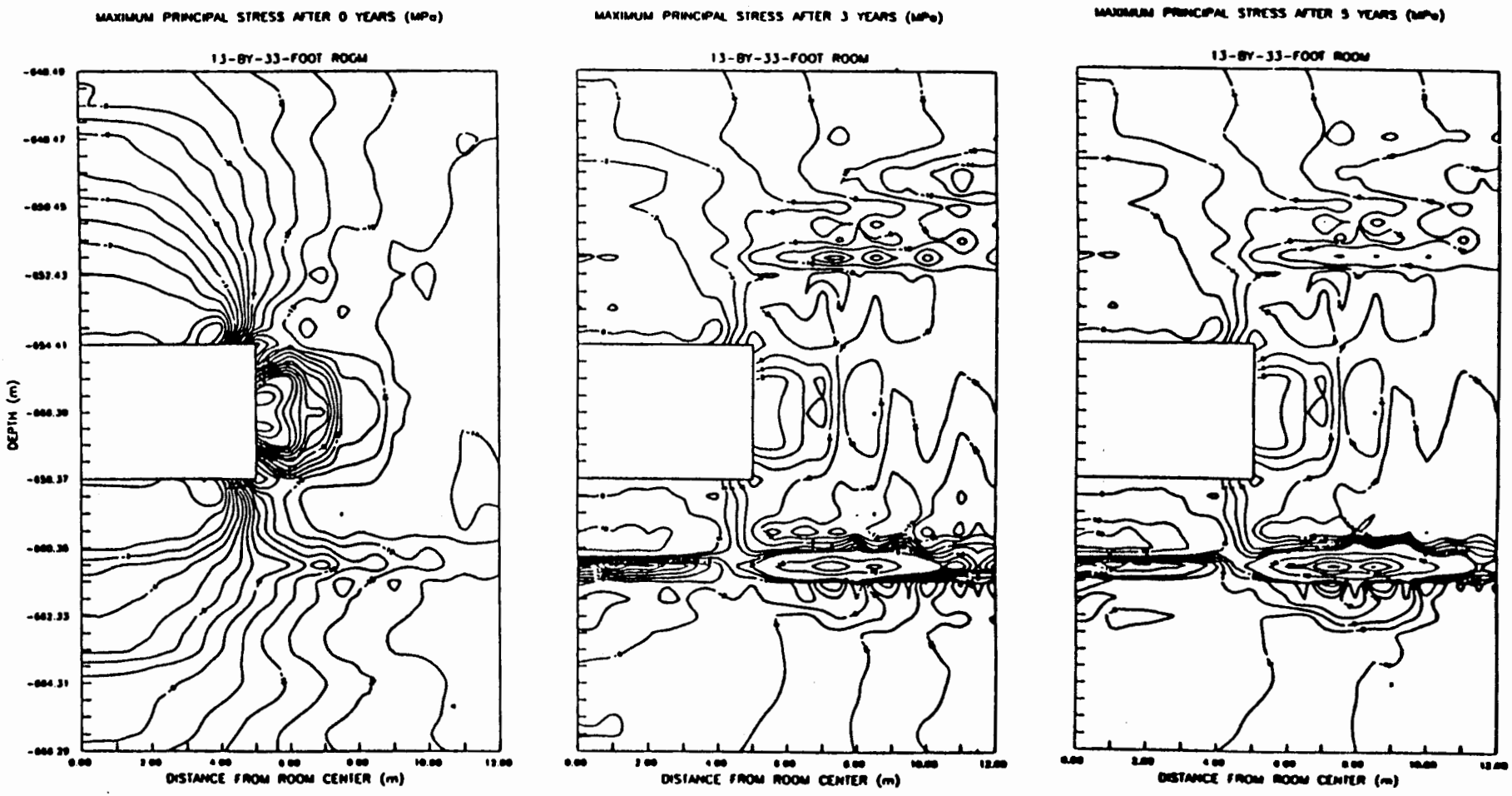


FIGURE 5.10 CONTOURS OF MAXIMUM PRINCIPAL STRESS FOR MODEL

TABLE OF CONTENTS

1.0 Introduction

2.0 Stratigraphy of Repository Horizon

3.0 Properties of Rocks at Repository Horizon

4.0 In Situ Stress Regime

5.0 Structural Response of Excavated Room

5.1 Roof Deformation

5.2 Rock Deformations About Room

5.3 Stresses Around Room

6.0 Performance Requirements for Support System

6.1 Performance

6.2 Geotechnical Design Basis for the Roof Support System for Room 1, Panel 1

6.2.1 Rockload To Be Supported

6.2.2 Vertical Movements

6.2.3 Lateral Movements

TABLE OF CONTENTS

1.0 Introduction

2.0 Stratigraphy of Repository Horizon

3.0 Properties of Rocks at Repository Horizon

4.0 In Situ Stress Regime

5.0 Structural Response of Excavated Room

5.1 Roof Deformation

5.2 Rock Deformations About Room

5.3 Stresses Around Room

6.0 Performance Requirements for Support System

6.1 Performance

6.2 Geotechnical Design Basis for the Roof Support System for Room 1, Panel 1

6.2.1 Rockload To Be Supported

6.2.2 Vertical Movements

6.2.3 Lateral Movements

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. 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.

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 a phenomenological model to be established 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.

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 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.

WASTE ISOLATION PILOT PLANT
SUPPLEMENTARY ROOF SUPPORT SYSTEM

APPENDIX A
Geology and Rock Mechanics

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

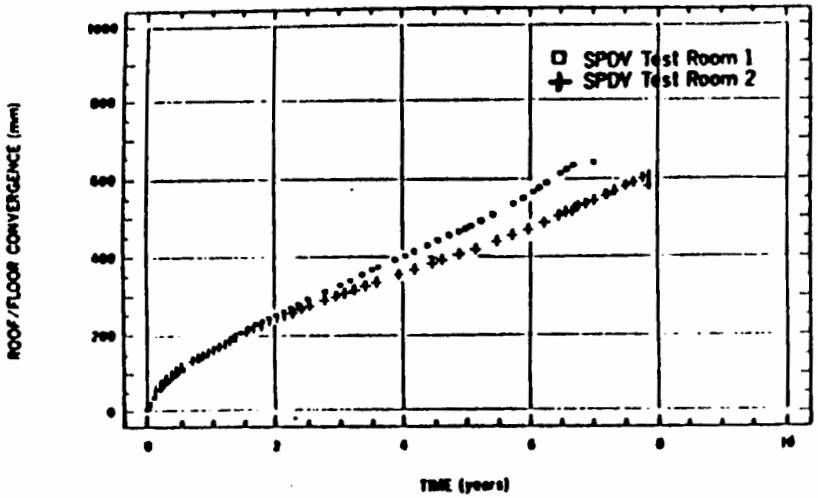
GEOTECHNICAL DESIGN SUMMARY REPORT

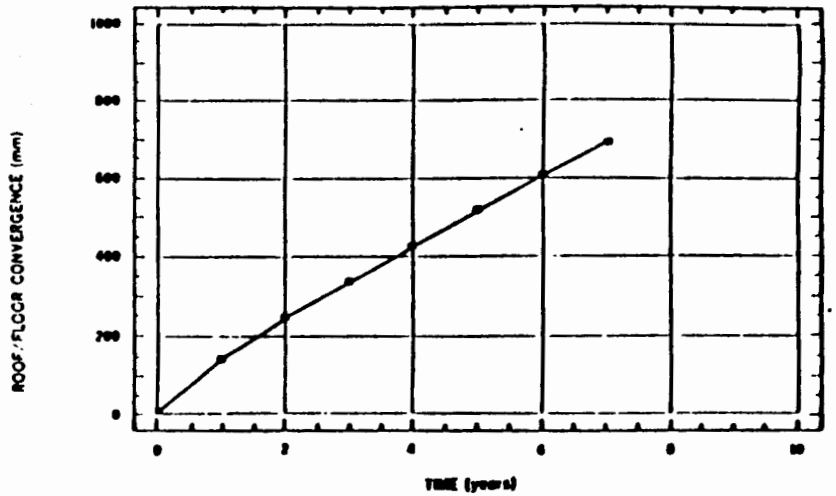
FOR ROOF SUPPORT SYSTEM IN ROOM 1, PANEL 1

FIELD MEASUREMENTS

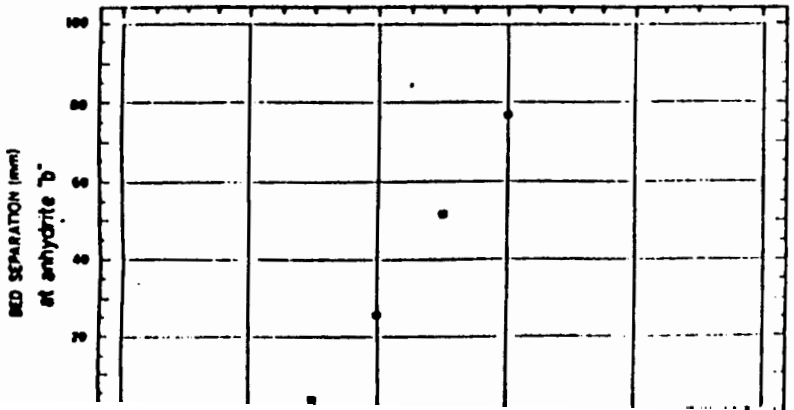
MODEL RESULTS

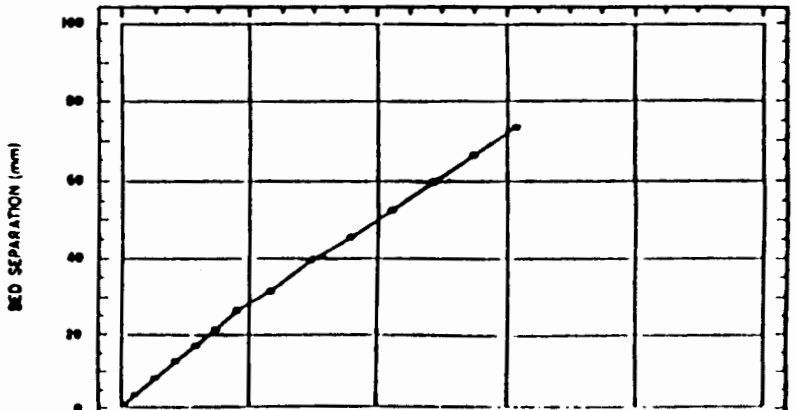
a.

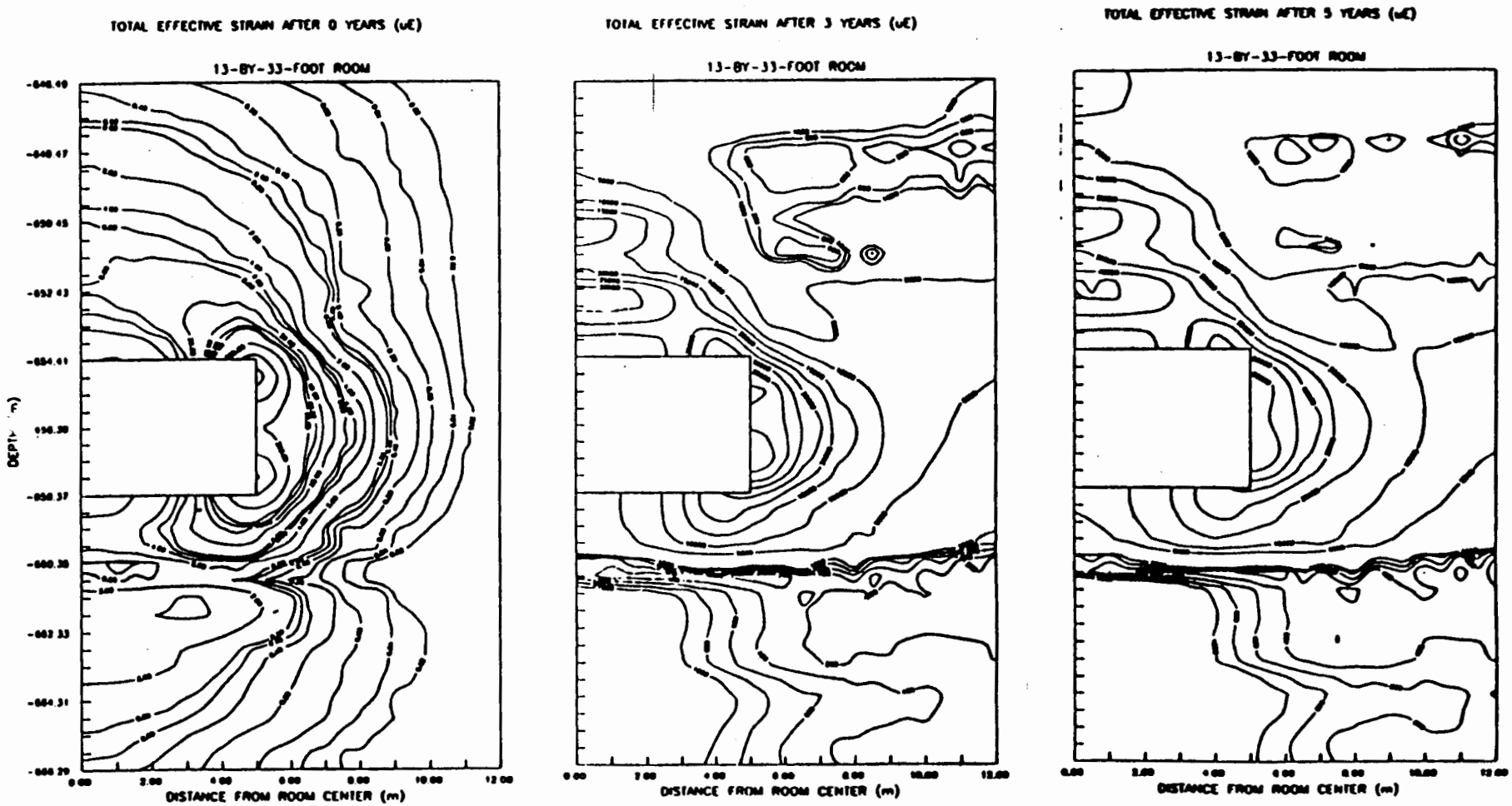




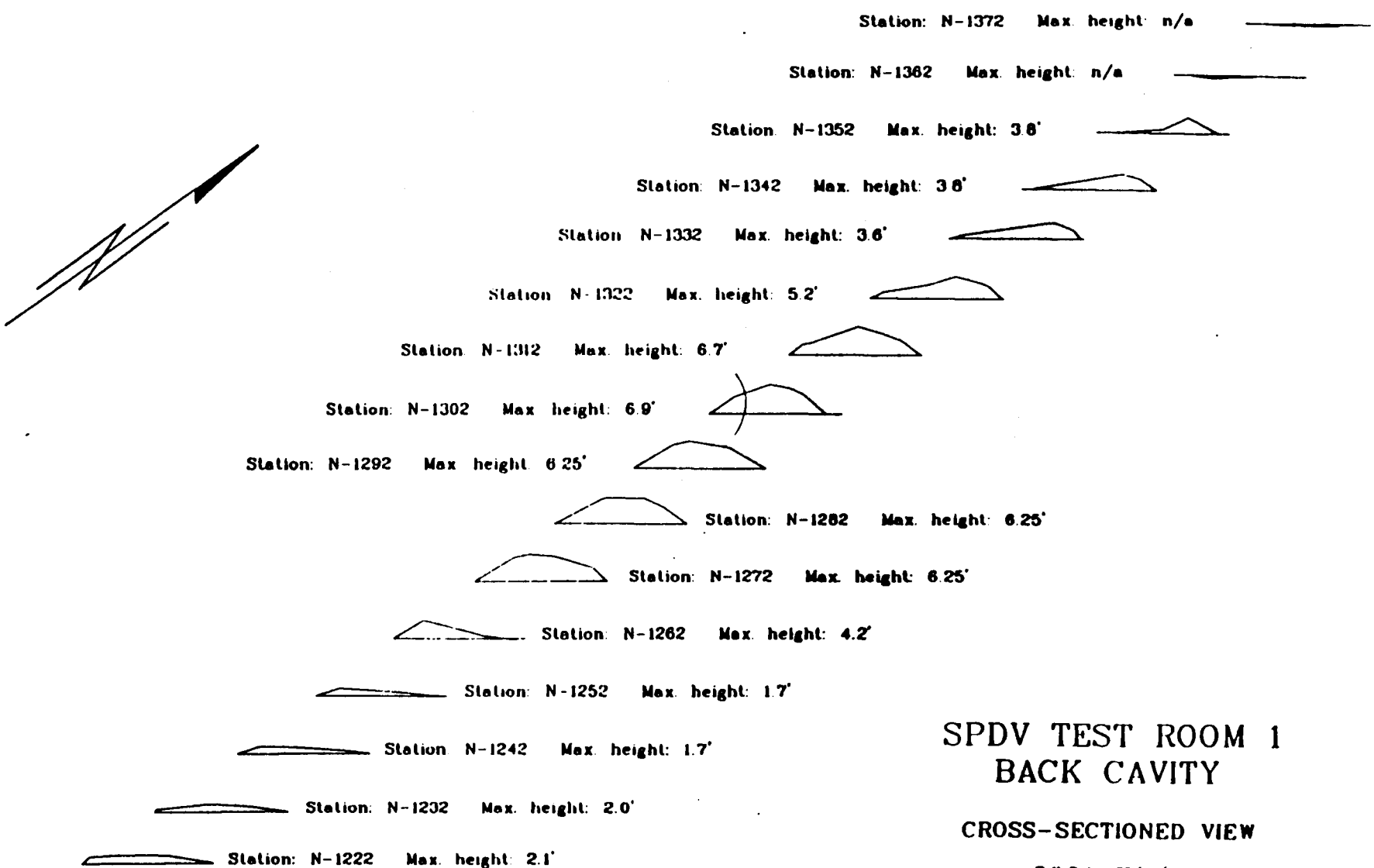
b.







GURE 5.9 CONTOURS OF EFFECTIVE STRAIN FOR MODEL OF 33 FEET WIDE BY 13 FEET HIGH ROOM.



SPDV TEST ROOM 1
BACK CAVITY

CROSS-SECTIONED VIEW

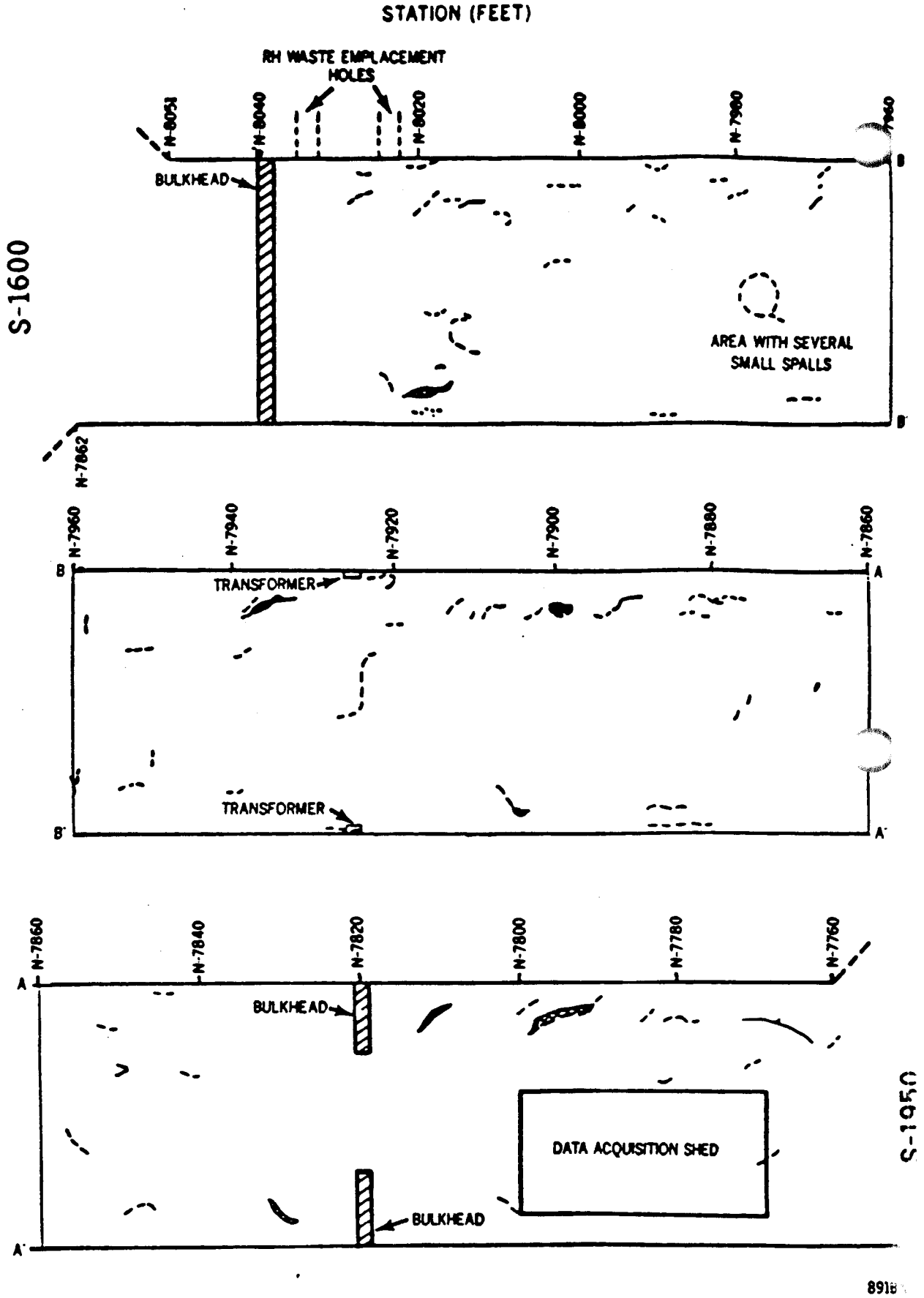
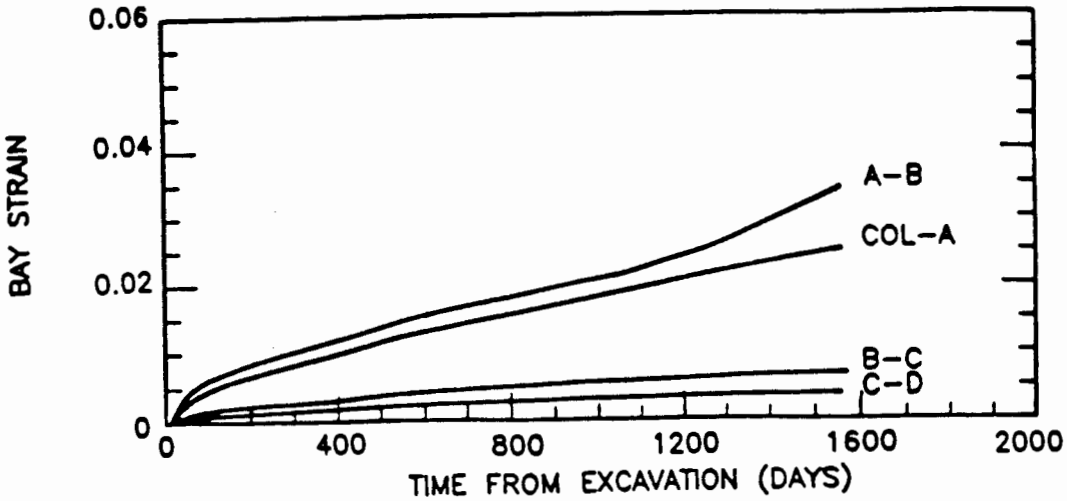
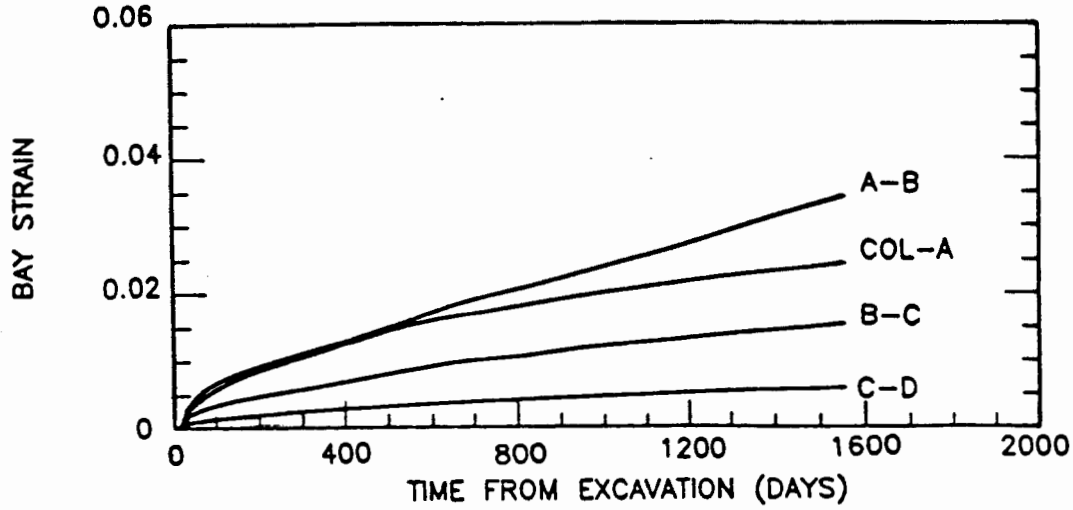


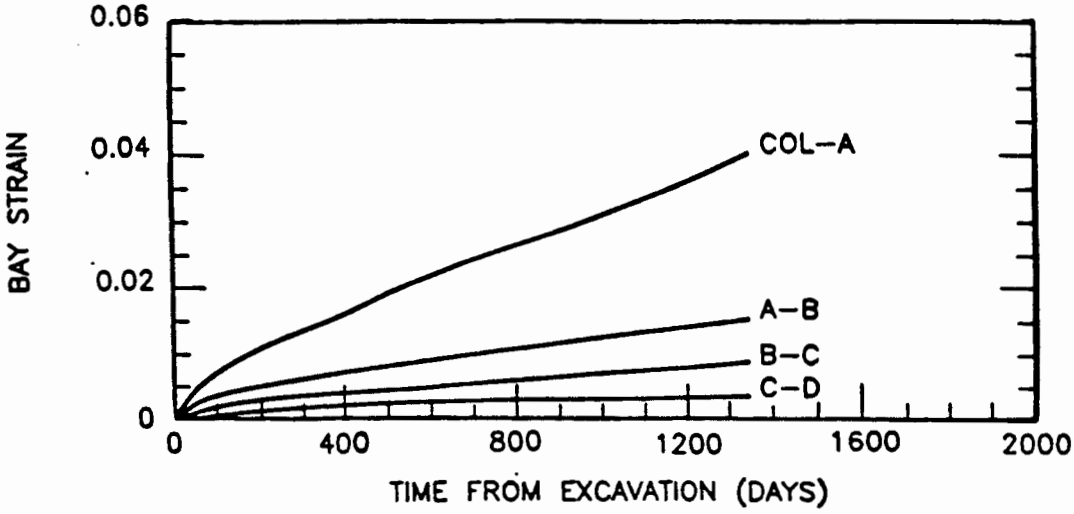
Figure 5.7 Back Fractures, Panel 1, Room 1 mapped on 2/12/91



c. EAST RO
EXTENS
51X G

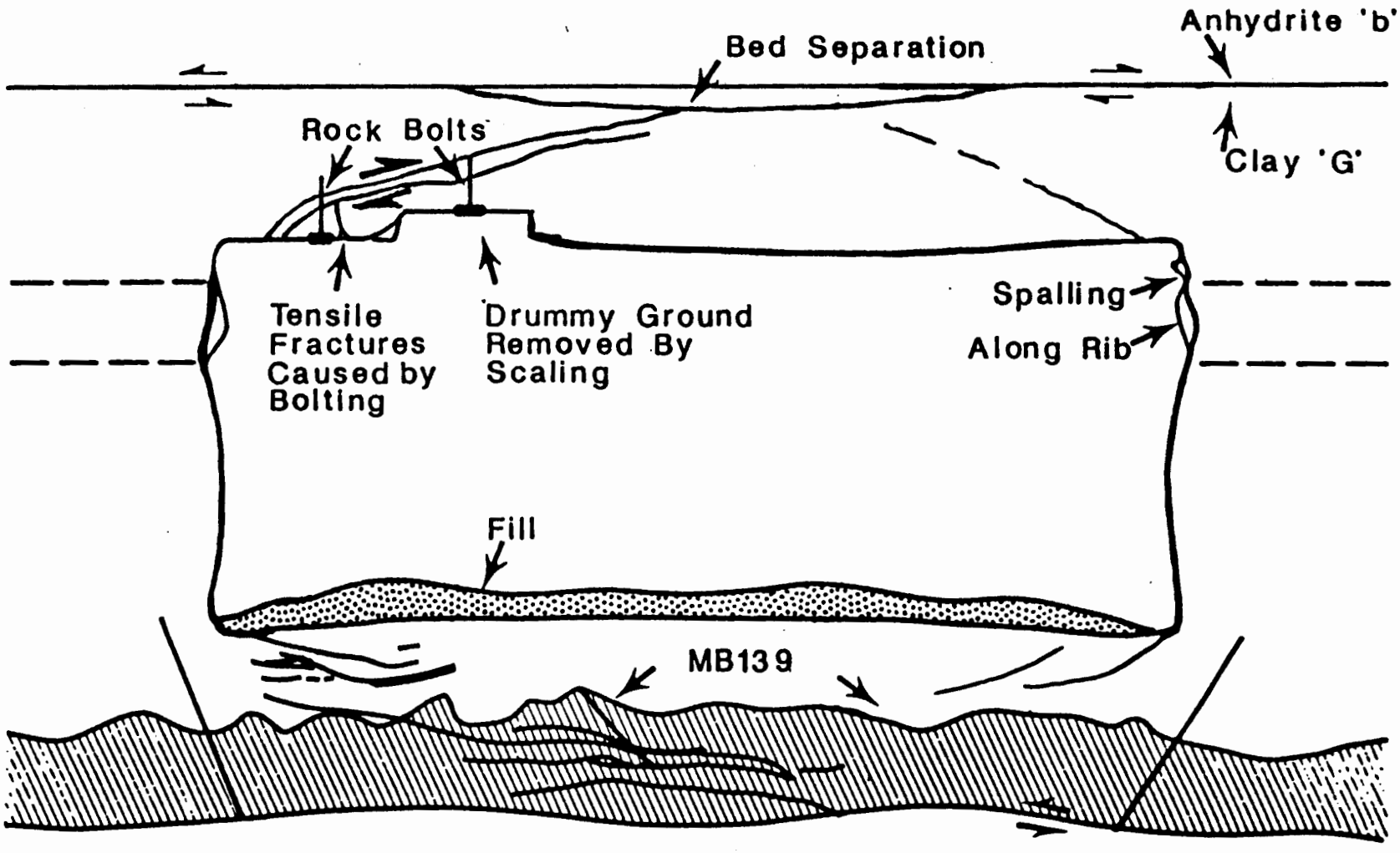


b. MID RO
EXTEN
51X G



a. WEST R
EXTEN
51X G

Figure 5.4 Extensometer Data from SPDV Test Rooms



8918:062Gb

Figure 5.5 Patterns of Fracturing (Idealized excavation effects modified from Borns & Stormont, 1987)

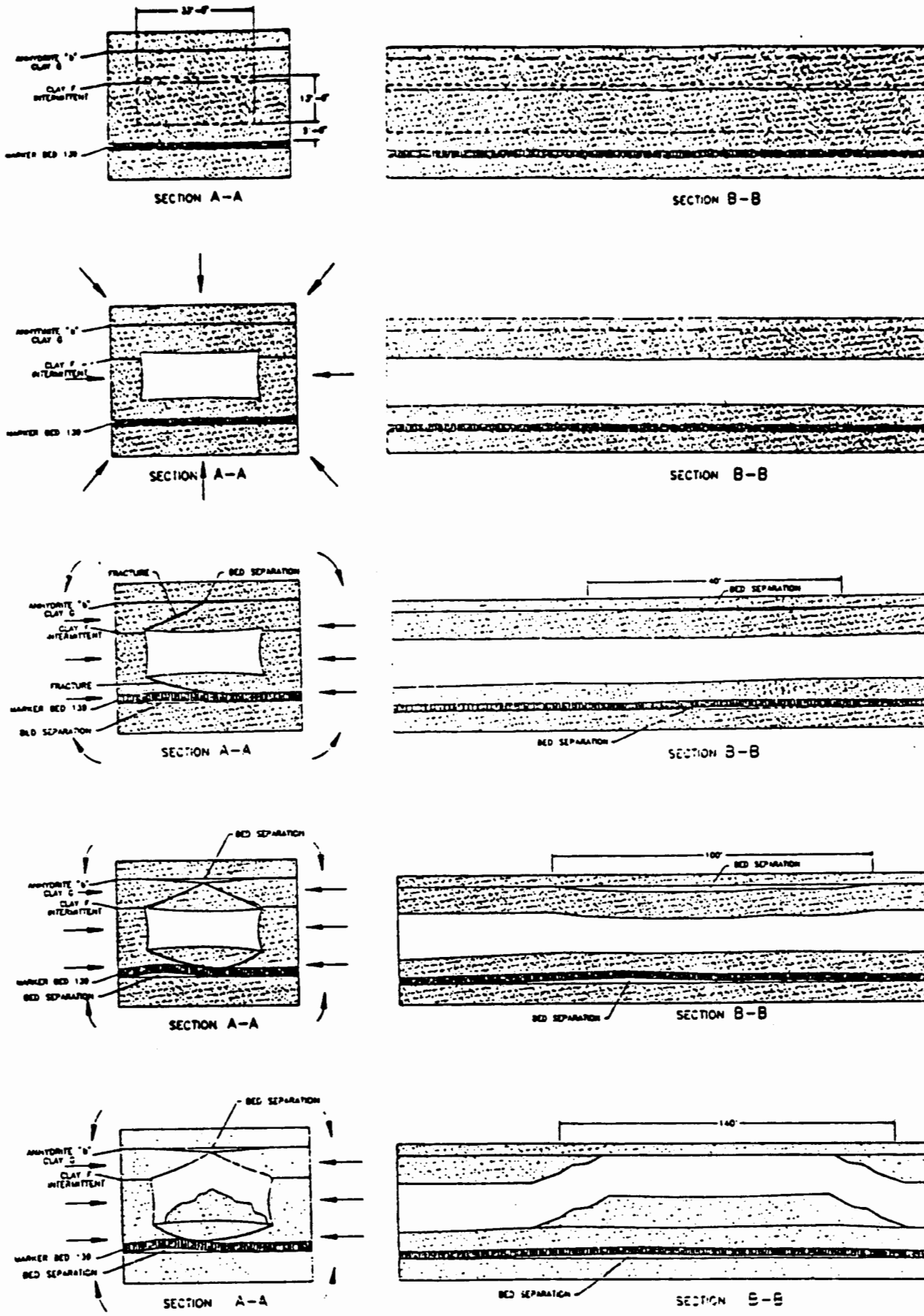
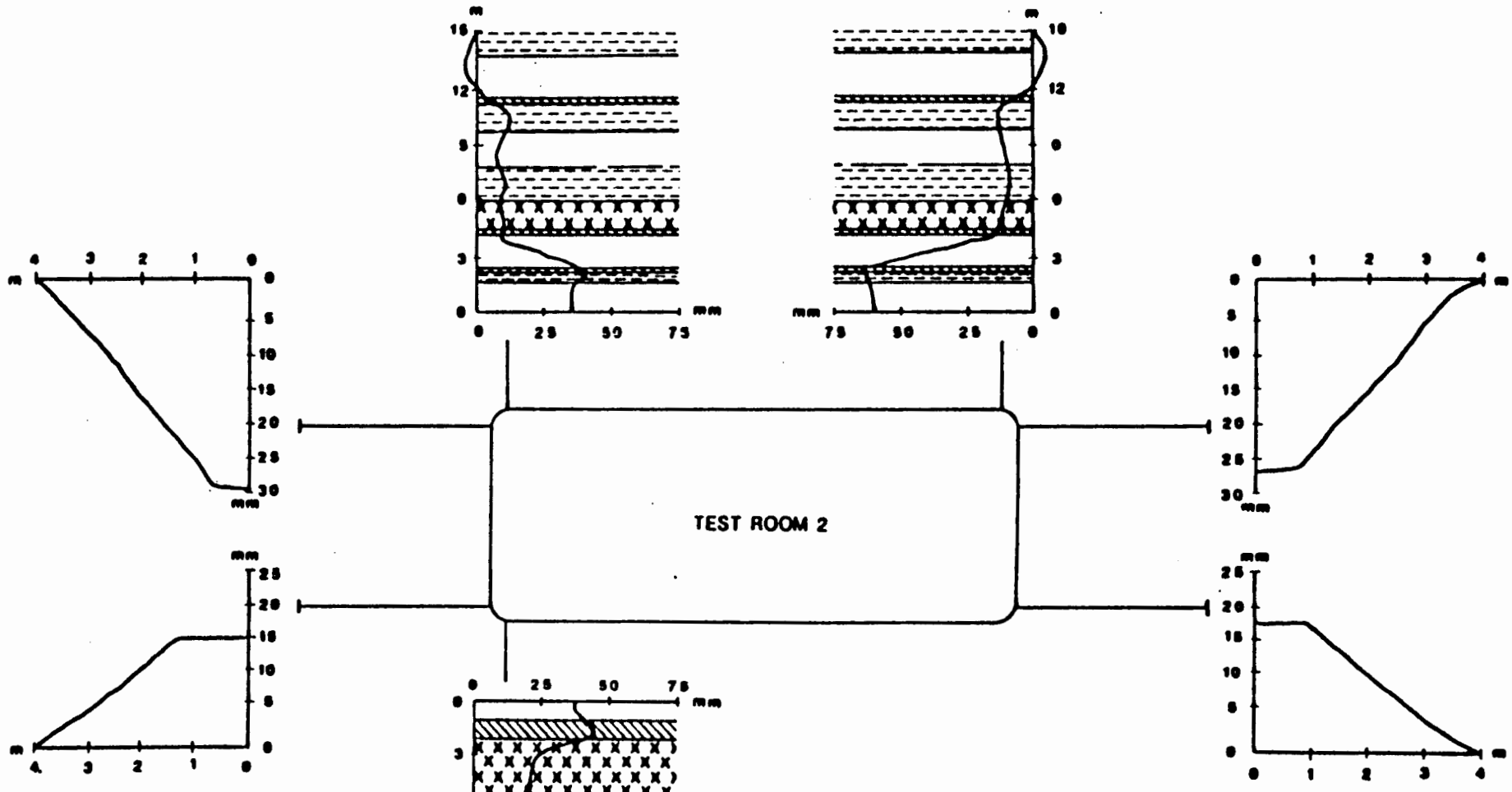


FIGURE 5.2

PHENOMENOLOGICAL MODEL OF ROOF FALL DEVELOPMENT



LEGEND

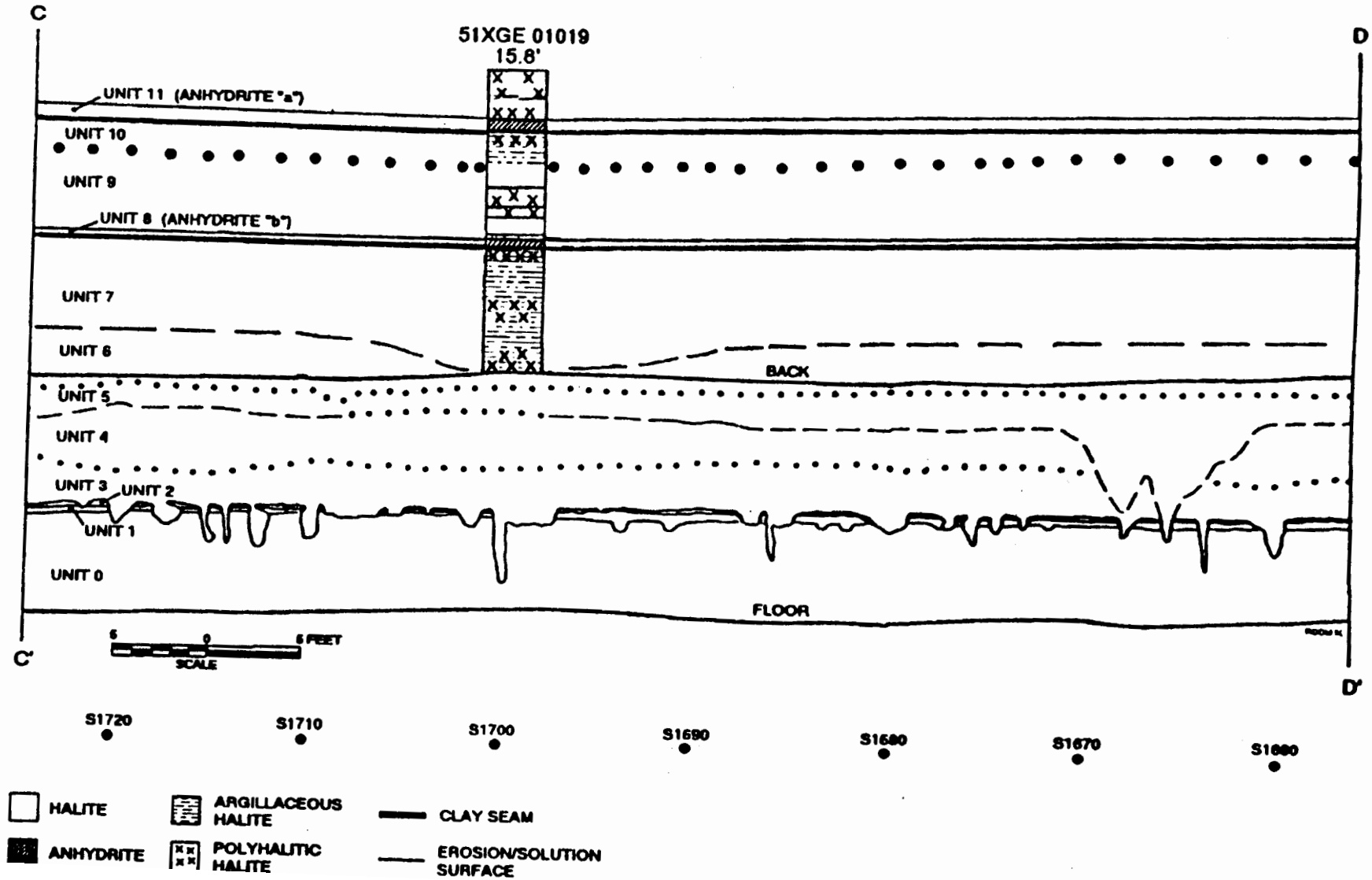
- CLEAR HALITE
- ANHYDRITE
- ARGILLACEOUS HALITE
- POLYHALITIC HALITE

NOTES

1. MOVEMENT PERPENDICULAR TO BORE HOLES MEASURED ON 03/04/07 RELATIVE TO INITIAL READINGS TAKEN ON 04/05/03.
2. VIEW LOOKING NORTH.
3. SCALES FOR HORIZONTAL AND VERTICAL INCLINOMETERS ARE NOT THE SAME.

918:0626

FIGURE 2.3c GEOLOGIC CROSS-SECTION OF PANEL 1, ROOM 1
View West



SPDV Test Room 1 Convergence Date
Combination of Instr. 8001 and 14001

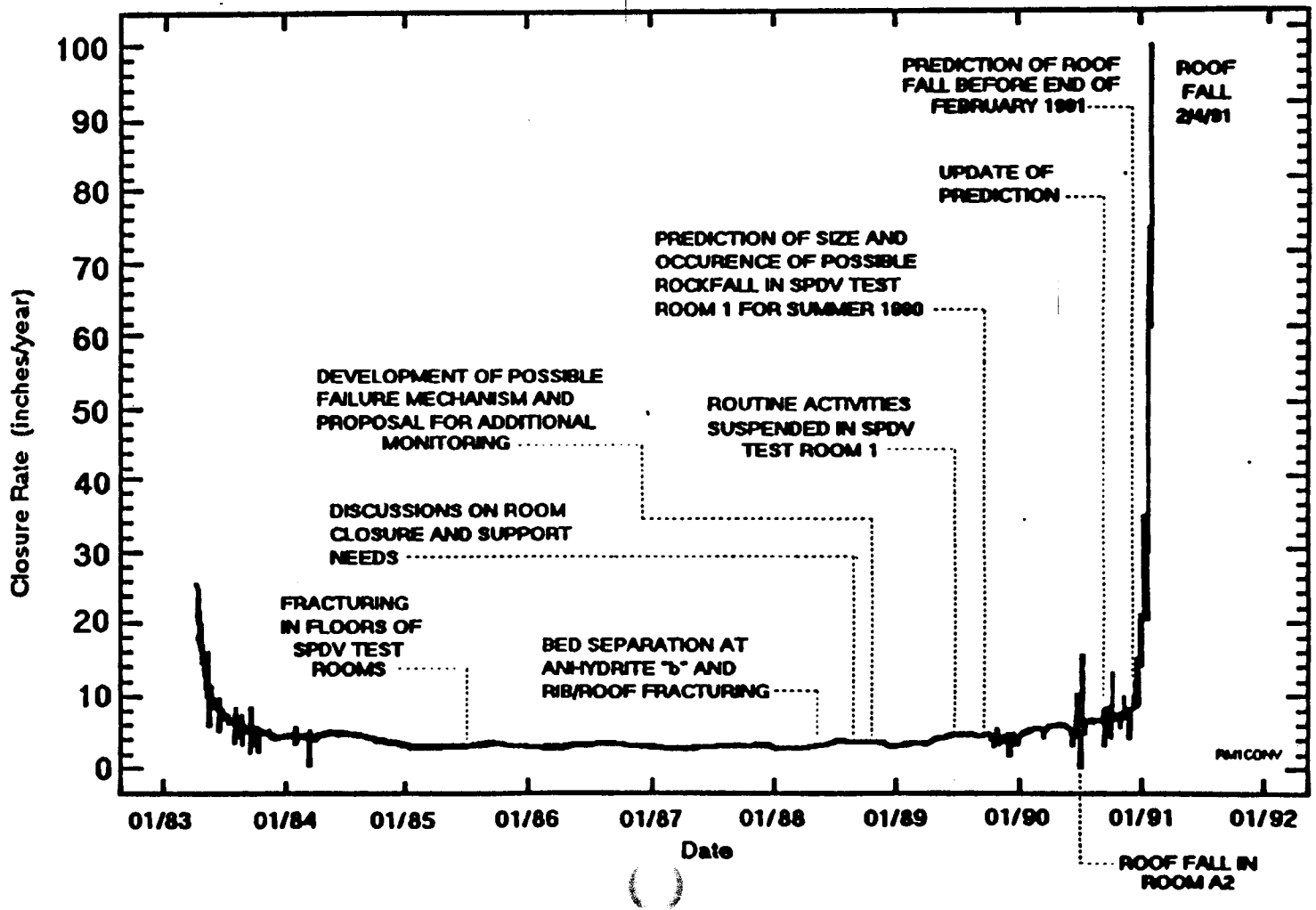


FIGURE 2.3a GEOLOGIC CROSS-SECTION OF PANEL 1, ROOM 1
View West

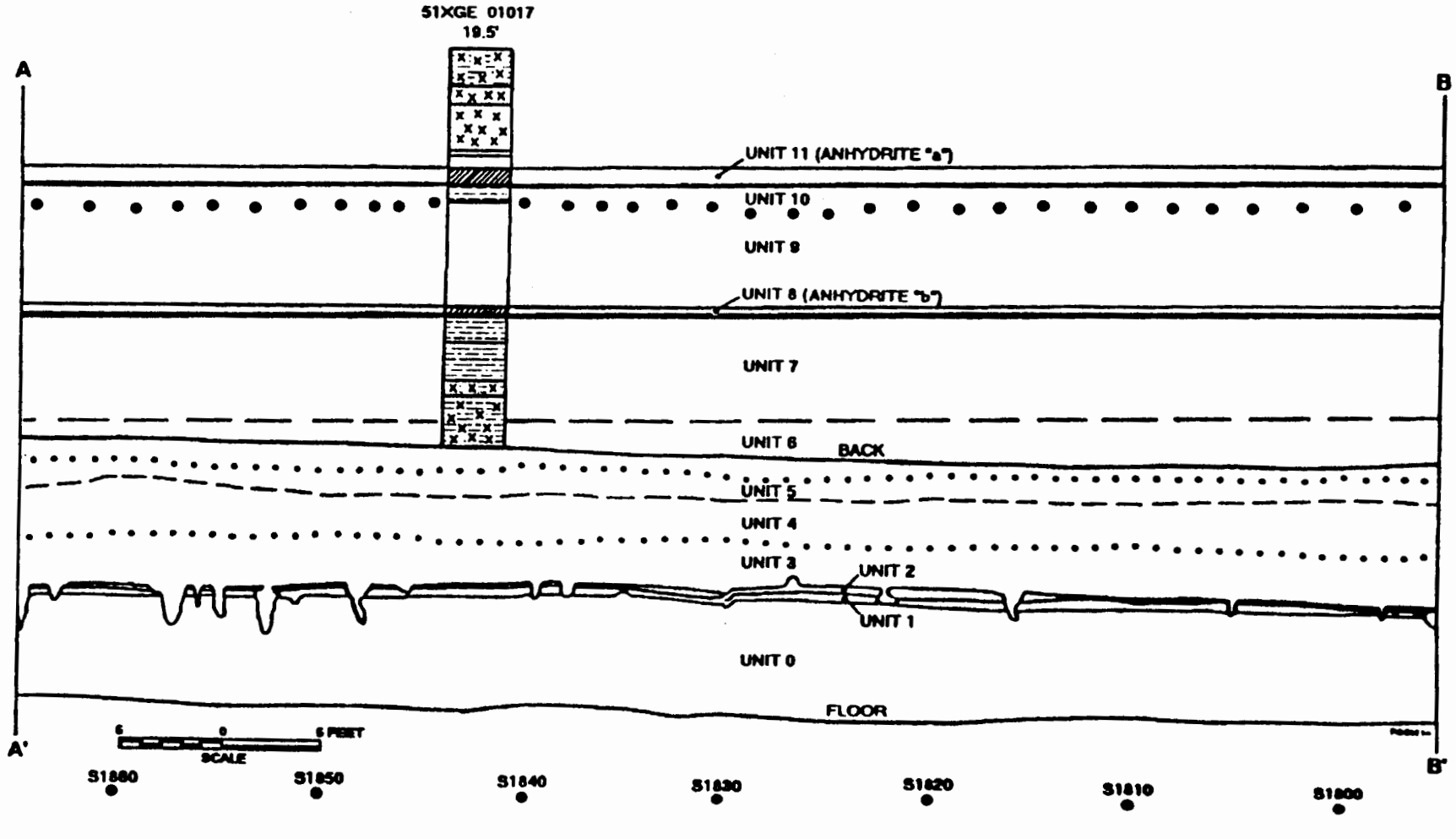
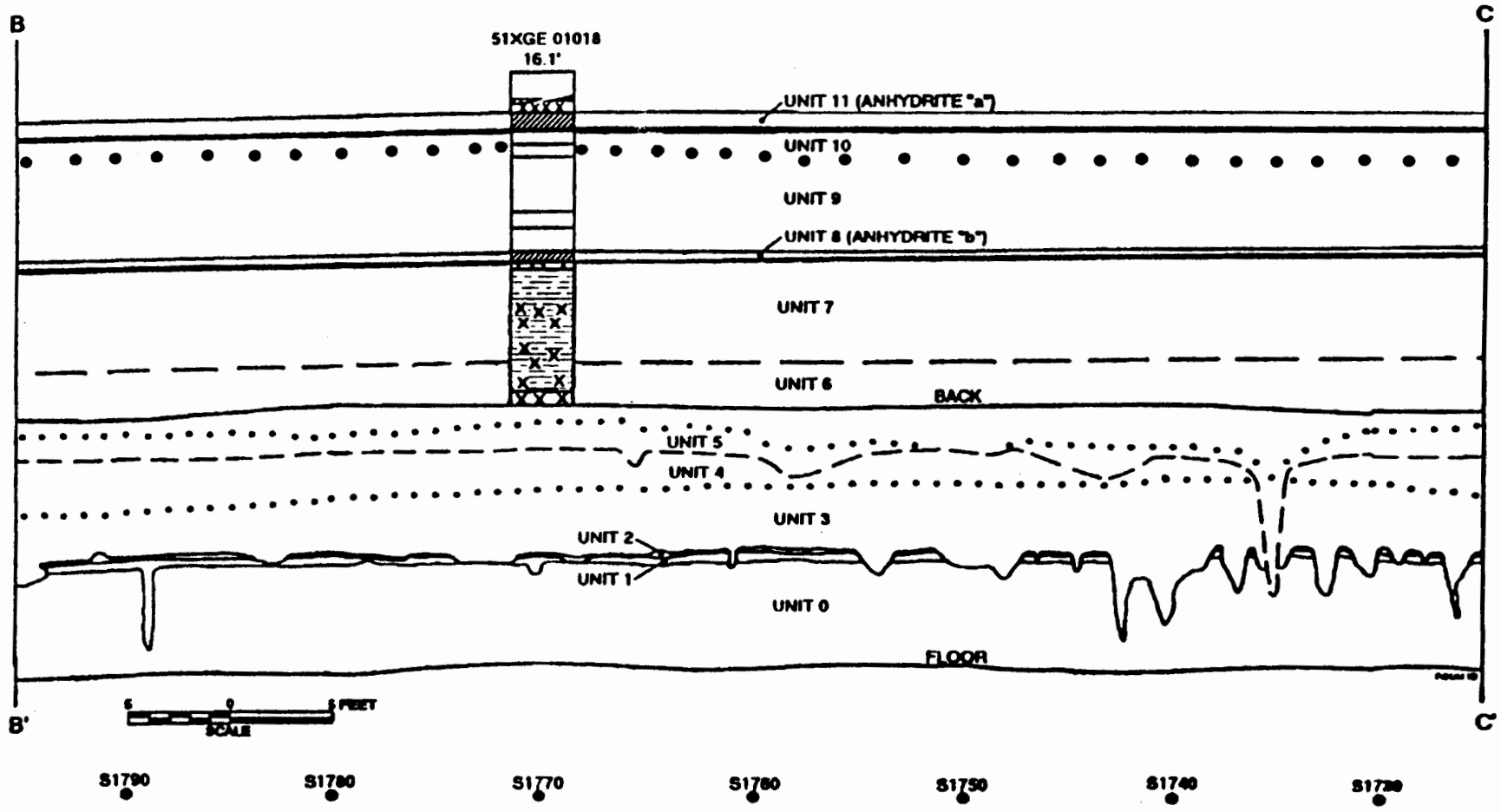
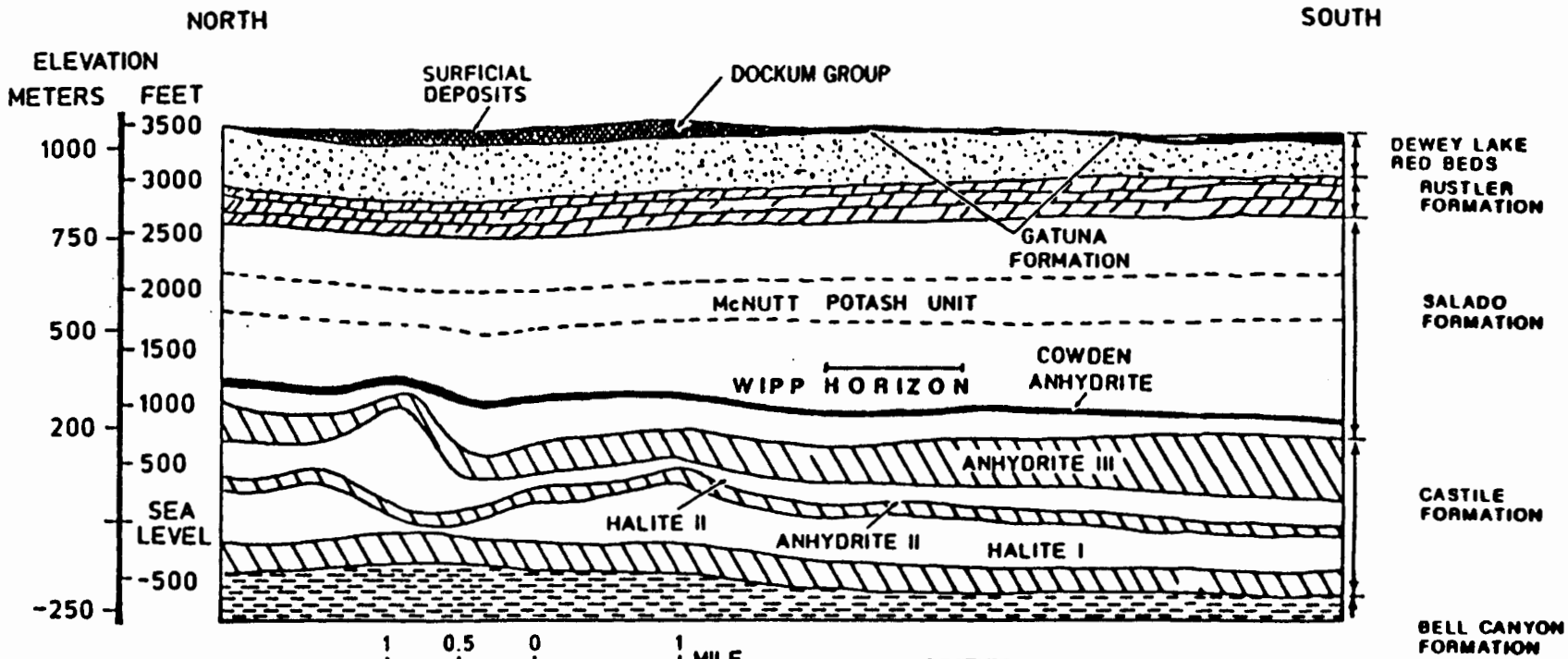


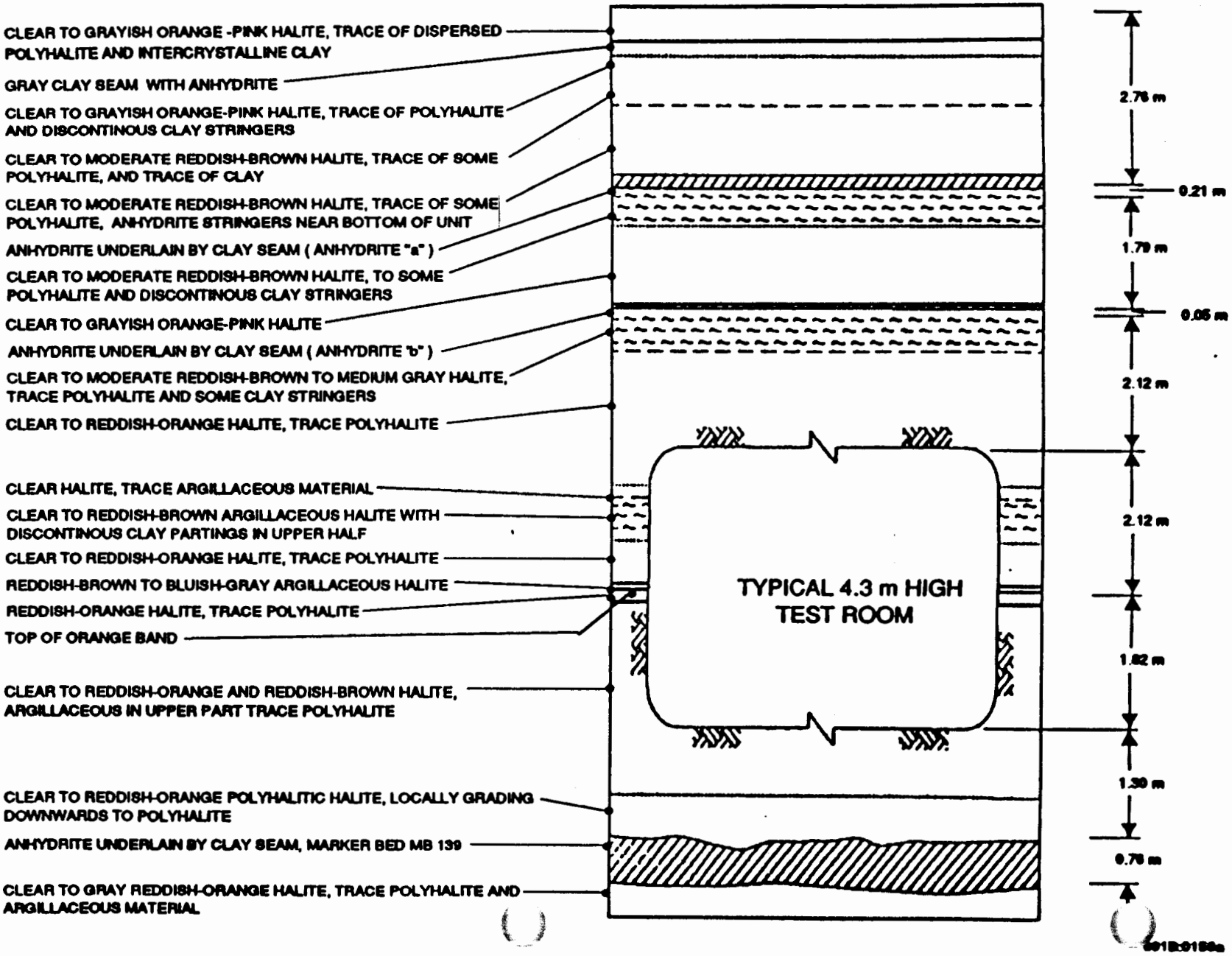
FIGURE 2.3b GEOLOGIC CROSS-SECTION OF PANEL 1, ROOM 1
View West



HALITE
ANHYDRITE
ARGILLACEOUS HALITE
POLYHALITIC HALITE
CLAY SEAM
EROSION/SOLUTION SURFACE



MODIFIED AFTER U.S. GEOLOGICAL SURVEY CROSS SECTION FOR THE U.S. DEPARTMENT OF ENERGY, NOVEMBER 1979


CLEAR TO GRAYISH ORANGE -PINK HALITE, TRACE OF DISPERSED POLYHALITE AND INTERCRYSTALLINE CLAY
GRAY CLAY SEAM WITH ANHYDRITE
CLEAR TO GRAYISH ORANGE-PINK HALITE, TRACE OF POLYHALITE AND DISCONTINOUS CLAY STRINGERS
CLEAR TO MODERATE REDDISH-BROWN HALITE, TRACE OF SOME POLYHALITE, AND TRACE OF CLAY
CLEAR TO MODERATE REDDISH-BROWN HALITE, TRACE OF SOME POLYHALITE, ANHYDRITE STRINGERS NEAR BOTTOM OF UNIT
ANHYDRITE UNDERLAIN BY CLAY SEAM (ANHYDRITE "a")
CLEAR TO MODERATE REDDISH-BROWN HALITE, TO SOME POLYHALITE AND DISCONTINOUS CLAY STRINGERS
CLEAR TO GRAYISH ORANGE-PINK HALITE
ANHYDRITE UNDERLAIN BY CLAY SEAM (ANHYDRITE "b")
CLEAR TO MODERATE REDDISH-BROWN TO MEDIUM GRAY HALITE, TRACE POLYHALITE AND SOME CLAY STRINGERS
CLEAR TO REDDISH-ORANGE HALITE, TRACE POLYHALITE
CLEAR HALITE, TRACE ARGILLACEOUS MATERIAL
CLEAR TO REDDISH-BROWN ARGILLACEOUS HALITE WITH DISCONTINOUS CLAY PARTINGS IN UPPER HALF
CLEAR TO REDDISH-ORANGE HALITE, TRACE POLYHALITE
REDDISH-BROWN TO BLUISH-GRAY ARGILLACEOUS HALITE
REDDISH-ORANGE HALITE, TRACE POLYHALITE
TOP OF ORANGE BAND
CLEAR TO REDDISH-ORANGE AND REDDISH-BROWN HALITE, ARGILLACEOUS IN UPPER PART TRACE POLYHALITE
CLEAR TO REDDISH-ORANGE POLYHALITIC HALITE, LOCALLY GRADING DOWNWARDS TO POLYHALITE
ANHYDRITE UNDERLAIN BY CLAY SEAM, MARKER BED MB 139
CLEAR TO GRAY REDDISH-ORANGE HALITE, TRACE POLYHALITE AND ARGILLACEOUS MATERIAL
TYPICAL 4.3 m HIGH TEST ROOM
2.76 m
0.21 m
1.79 m
0.05 m
2.12 m
2.12 m
1.82 m
1.30 m
0.76 m

REFERENCES

Baar C. A., 1977, "Applied Salt-Rock Mechanics 1, The In Situ Behavior of Salt Rocks", Elsevier Scientific Publishing Company.

Cook, R. F. and W. M. Roggenthen, 1991, "Fracturing Around Excavations in Salt at the WIPP", U.S. Symposium on Rock Mechanics, Oklahoma City.

Dreyer W., 1972, The Science of Rock Mechanics, Part 1, The Strength Properties of Rocks, Trans Tech Publications.

Greenwald, H. P. and H. C. Howarth, 1938, "Compression Tests of Roof-salt Slabs by Potash Salt Pillars", Report of Investigations, U. S. Bureau of Mines RI 3386.

Hansen F.D., "1979 Quasi-Static Compression and Creep Behavior of Bedded Salt from Southeastern New Mexico," SAND79-7045, prepared by Sandia National Laboratories.

Hansen F.D. and K.D. Mellegard, 1979, "Creep Behavior of Bedded Salt from Southeastern Ne Mexico at Elevated Temperature," SAND79-7030, prepared by Sandia National Laboratories.

Herrmann W., W.R. Wawersik, and H.S. Lawson, 1980, "Analysis of Steady State Creep of Southeastern New Mexico Bedded Salt," SAND80-0558, prepared by Sandia National Laborato

Herrmann W., W.R. Wawersik, and H.S. Lawson, 1980, "Creep Curves and Fitting Parameters f Southeastern New Mexico Bedded Salt," SAND80-0087, prepared by Sandia National Laborato

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

Holt R. M., 1991, Personal Communication.

Kreig R.D., 1983, "Reference Stratigraphy and Rock Properties for the Waste Isolation Pilot Plant (WIPP) Project," SAND83-1908, prepared by Sandia National Laboratories.

Miller, H. D. S., 1991 Personal Communication.

Molecke M. A., 1990, "Test Plan: WIPP Bin-Scale CH TRU Waste Tests", prepared by Sandia National Laboratories for the U.S. Department of Energy.

U.S. Department of Energy, 1986, The Design Validation Final Report prepared by Bechtel, DOE/WIPP 86-010.

U.S. Department of Energy, 1989 "The Geotechnical Field Data and Analysis Report, July 1987 to June 1988" prepared by Westinghouse Electric Corporation, Waste Isolation Division, DOE/WIPP 89-0090.

U.S. Department of Energy, 1990, "The Geotechnical Field Data and Analysis Report, July 198 to June 1989", prepared by Westinghouse Electric Corporation, Waste Isolation Division, DOE/WIPP 90-006.

U.S. Department of Energy, 1991a, "The Geotechnical Field Data and Analysis Report July 1989 to June 1990", prepared by Westinghouse Electric Corporation, Waste Isolation Division, DOE/WIPP 91-012.

U.S. Department of Energy, 1991b "Report of the Geotechnical Panel on the Effective Life of Rooms in Panel 1", prepared by Westinghouse Electric Corporation, Waste Isolation Division, DOE/WIPP 91-023.

U.S. Department of Energy, 1991c, "Geologic Mapping of the Air Intake Shaft at the Waste Isolation Pilot Plant", prepared by IT Corporation, DOE/WIPP 90-051.

Wawersik, W.R. and C.M. Stone, 1986, Experience with Hydraulic Fracturing Tests for Stress Measurements in the WIPP, Proceedings of the 27th U.S. Symposium on Rock Mechanics, Society of Mining Engineers.

The shear strains are shown in Figure 5.9. 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 14MPa (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 5.10 and 5.11. 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 (U.S. DOE 1991b) 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 the lateral stresses in the beam. With time, the shear strains build up in the beam and eventually result in the development of failure along particular surfaces. This failure initially occurs at room mid length. The localized 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 compressive stress. The failures propagate in the longitudinal direction, gravity effects take over, and 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 falls as a unit.

6.0 PERFORMANCE REQUIREMENTS FOR SUPPORT SYSTEM

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.

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. 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 or triangular shape that forms from low angled fractures starting at the the ribs whose propagation is bounded and controlled by the bed separation occurring at the Anhydrite 'b'.

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 (1.5 inches 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 (1 inch 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 asymmetrical with one side lowering more rapidly than mid span or the other side.

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 (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 should be designed to accommodate a lateral shift of 12mm per year (0.5 inches per year) and bed separation of 25mm per year (1 inch per year) at the clay/salt contact below the Anhydrite 'b' layer once the bond at the interface is disrupted.

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 an 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 pictu 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 where different conditions exist) 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. Its various stages 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 1 is based on the control of 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 5.1 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 rooms closest to the barrier pillars and are lower in the middle of panels and at the solid abutments. At present, no correlation has been established between closure rate and variables such as excavation sequence and variations in stratigraphy that might explain these differences.

The changes in roof profile have been correlated with time, based on data from SPDV Test Room 1. The room deformation is 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 spanned 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 5.3 covering the period from 1983 to 1987. Figure 5.4 provides the data for roof extensometers.

Bed separation has been identified at the Anhydrite 'b' in the roof after approximately three years. Vertical 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 lateral 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 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 (U.S. DOE, 1991a). Rock deformations about a room also is governed by fracture development. The typical fracture development observed in the wide excavations is shown in Figure 5.5. 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 in the roof of SPDV Test Room 1. This wedge fell 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 5.6 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 5.7 as of the summer 1991.

MODEL DATA

Roof/floor closure for the model is compared with the field data in Figure 5.8. Roof/floor closure rate is 0.1 m per year (4 inches per year) and does not vary significantly with time. This is probably due to the limitations of the steady state of the creep law used for the calculations. A more sophisticated constitutive law that includes transient creep effects has been developed by SNL and would provide a more accurate simulation of the early time performance of the excavations.

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 5.8b. 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. Further investigations of the performance at this layer are required.

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; Herrmann 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. This brittle behavior of salt is generally not characterized by laboratory studies, and therefore, the constitutive law developed to define the creep behavior of salt at the WIPP does not include fracture development or rock dilation. The evaluation of performance 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 Helms 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 (135 lb/ft^3), the initial stresses at the repository horizon are about 14 MPa (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 used for design purposes at the WIPP Project (US 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, havi 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,(13 feet) width of 10m (33 feet), and a length of 91m (300 feet). The rooms are separated by 30m (100 feet) 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.

TABLE 5.1

CLOSURE RATES BY TIME SINCE EXCAVATION
VERTICAL CONVERGENCE DATA

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
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

Notes: Closure rates in inches/year.
Year is determined from date of excavation at instrument.
( ) Estimated based on less than one full year of data.

2.0 STRATIGRAPHY OF REPOSITORY HORIZON

The proposed underground storage facility is located 655m (2150 feet) 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 300m (1000 feet) of impermeable evaporitic deposits separate the facility horizon from the overlying sedimentary rocks and 620m (2000 feet) 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 (40 feet) 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 0.5m (20 inches) to 0.8m (32 inches) thick persistent bed of sulfate (anhydrite and polyhalite), identified as Marker Bed 139 lies about 1.5m (5 feet) 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 up to 150mm (6 inches) have been observed in 100mm (4 inch) diameter bore (Holt, 1991). The bottom of the Marker Bed is sub horizontal and is underlain by Clay "E".

Anhydrite beds, called Anhydrites "a" and "b" occur about 4m (13 feet) and 2m (6.5 feet) 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 high confining pressures and 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).

TABLE 3.1

ROCK PROPERTIES OF THE REPOSITORY HORIZON

	DIMENSIONS	SALT	ANHYDRITE	POLYHALITE
THERMAL PROPERTIES				
Thermal Conductivity	W/(m.K)	5.0	4.7	1.4
Specific Heat	J/(kg.K)	860	860	860
Coefficient of Linear Expansion	K^{-1}	45.0E-6	20-E6	24E-6
MECHANICAL PROPERTIES				
Young's Modulus	Pa	3.1E+10	7.5E+10	5.5E+10
Poisson's Ratio	dimensionless	0.25	0.35	0.36
Bulk Modulus	Pa	2.1E+10	8.3E+10	6.6E+10
Shear Modulus	Pa	1.2E+10	2.8E+10	2.0E+10
Density	Mg/m^3	2.3	2.3	2.3

PROPERTIES FOR SECONDARY CREEP FOR SALT

$$\dot{\epsilon} = D\,\bar{\sigma}^{n} e^{-Q/RT}$$

Where: $\dot{\epsilon}$ = effective creep strain rate

$\bar{\sigma}$ = effective stress

T = temperature in degrees Kelvin
D, n are empirical constants

$D = 5.79E\text{-}36\ Pa^{-4.9}/sec$
$n = 4.9$
$Q/R = 6060\ K$

Where: Q = the effective activation energy for salt, (cal/mole)
R = the universal gas constant, (cal/mole.K)

Further explanation of the definitions for effective creep strain rate and effective stress are provided in DOE-WIPP-86-010, the Design Validation Final Report, Appendix C.

Reference parameter values are based on:

Branstetter, L.J., "Second Reference Calculation for the WIPP" SAND83-2461, Sandia National Laborat
Albuquerque, New Mexico.

Krieg, R. D., Reference Stratigraphy and Rock Properties for the Waste Isolation Pilot Plant (WIPP) P
SAND83-1908, Sandia National Laboratories, Albuquerque, New Mexico.

WASTE ISOLATION PILOT PLANT
SUPPLEMENTARY ROOF SUPPORT SYSTEM

APPENDIX C
Support System Design

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

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
October 1991

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 compour factors of safety when designing an structure. The uncertainty and risk with the work should [illegible] determining the design capacity [illegible] engineer applies a separate factor [illegible] the applied loads, the design capacity and the anchor test load. The actua safety for the anchor will have been co resulting in an overly conservative de

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 m increase or decrease with time depend 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 recom to prevent significant reductions in tra due to stressing anchorage losses or m

3.2.6 Bond Length

The bond length can be estimated by the following equation

The bond length normally is not less th For normal applications the bond be tendon and anchor grout is not critica

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

Pull-out tests may be used to dete ultimate in-situ bond stress between th the anchor grout. Pull-out tests usua that the tendon capacity be increas bond length reduced. In order to ta Pull-out test should not be requir anchors are tested as described in Se

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 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 engineer should consider the critica the anchor application, variations properties and the installation proce

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	2.8 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	2.8 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 Losinger (1968)
Chalk – Grades I - III	0.01 N (N = SPT in 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	0.86 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 [illegible]
Well-cemented mudstones		0.69	2.0 to 2.5	New Zealand Irwin [illegible]
Bunter sandstone	0.40		3.0	Britain Littlejohn (1973)
Bunter sandstone (ultimate compressive stress > 2.0 N/mm²)	0.60		3.0	Britain Littlejohn (1973)
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 (0.45C$_u$)	3.0	Britain Littlejohn (1970) C$_u$ = undrained cohesion
Weak shale		0.35		Canada Golder Brawner (1971)
Soft sandstone and shale	0.10 to 0.11	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 ÷ 30 (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

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 (Bergeret (19
	0.50			Switzerland (Wanner
	0.70			Switzerland (Comte
		1.20 to 2.50		Switzerland (Comte
	[illegible]		2 to 2.5 (Temporary) 3 (Permanent)	Italy (Mascardi (197
	0.69	2.76	4	Canada (Golder Brow
	1.4	4.2	3	USA (White (1973)
		15 % to 20 % of grout crushing strength	3	Australia (Langworth
Concrete		1.38 to 2.76	1.5 to 2.5	USA (PCI (1974)

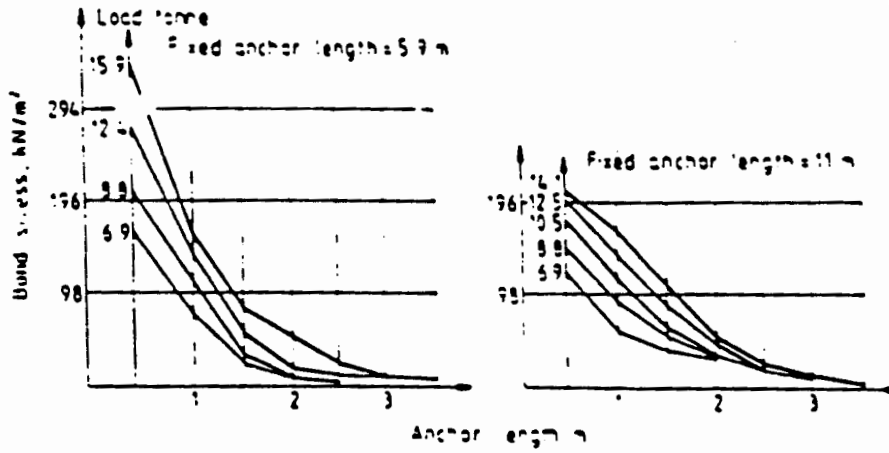


Figure 56. Distribution of bond along fixed anchor length (after Berardi (1967))

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 com pacted or displaced during the casing insta no residual grout pressure was left at the grout soil interface on completion of the A might reduce to a value approximating light of experience, this reduction is now unduly pessimistic since even with tremie the full hydrostatic head of the grout is a anchor interface, which creates a contact than $K_0 \sigma'_v$ in normally consolidated grou

As a consequence, even for the tremie gro it is difficult to envisage a value of A less purposes

TITLE			PAGE
FACTOR OF SAFETY - WEDGE PULLOUT			1
PROJECT	AUTHOR / DATE: C. FRANCKE 9/5/91	CHK'D BY / DATE: JR Tyler 9/4/91	CHK'D BY
SO	CALC NO	FILE NO	GROUP

PROBLEM: DETERMINE THE FACTOR OF SAFETY (FoS) 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
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 Excavation Bechtel

METHOD: THE SHEAR RESISTANCE OF THE WEDGE ABOVE CLAY 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.

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

Check

WESTINGHOUSE WASTE ISOLATION DIVISION

TITLE				PAGE
BOLT WORKING LOAD FOR 10' x 2' SPACING				1
PROJECT	AUTHOR / DATE	CHK'D BY / DATE	CHK'D BY	
	C. FRANCKE 9/5/91	JR Tyson 9/6/91		
SD	CALC NO	FILE NO		

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

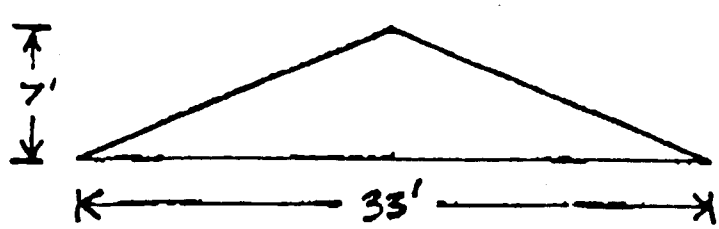
ASSUMPTIONS:

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 EA BOLT WILL BE CALCULATED AND MULTIPLYED BY THE DENSITY OF THE ROCK TO DETERMINE THE LO ON THE BOLT.

CALCULATIONS:

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



CROSS-SECTION VIEW

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

WESTINGHOUSE FORM 40339A

TITLE				PAGE
BOLT WORKING LOAD FOR 10' x 2' SPACING				2 OF
PROJECT	AUTHOR / DATE	CHK'D BY / DATE	CHK'D BY	
	C. FRANCKE 9/8/91	JRTyburk 9/6/91		
SUB	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

TITLE			
BOLT WORKING LOAD FOR 10' x 2' SPACING			
PROJECT	AUTHOR C. FRANCKE 9/5/91	CHK'D BY	DATE
S.O.	CALC NO	FILE NO	GROUP

CALCULATIONS: (CONT'D)

WORKING LOAD

THE WEIGHT OF THE ROCK SUPPORTED B
IS DETERMINED BY THE FOLLOWING EQUATIO

$$W = \rho V$$

WHERE: W = WEIGHT OF ROCK (WORKIN
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 PRO
2'x10' LAYOUT WILL BE ~~20~~ 18.9 kip.

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

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 th 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.

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

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

ASSUMPTIONS:

1) THE TENDON IS EXPECTED TO BE GRADE 60 ST
WHICH MEETS ASTM STANDARD A615 (REF.
2) THE MAXIMUM WORKING LOAD ON THE TENDO
IS 18.9 kip (84.1 KN).
3) THE MINIMUM (GUARANTEED) ULTIMATE TE
STRENGTH OF THE TENDON IS 90,000 psi (R
4) THE TENDON IS 1" DIAMETER. Value

REFERENCES:

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

METHOD:

THE WORKING LOAD ON THE TENDON IS CALCULATED.

CALCULATIONS:

THE MAXIMUM LOAD THAT THE TENDON CAN SU
IS DETERMINED BY MULTIPLYING THE ULTIMA
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

TITLE				PAGE
FACTOR OF SAFETY - BOLT TENDON FAILURE				2
PROJECT	AUTHOR / DATE	CHK'D BY / DATE	CHK'D BY	
	C. FRANKE 9/5/91	JRTyborski 9/9/91		
WO	CALC NO	FILE NO	GROUP	

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 DETERM THE REQUIRED BOLT DIAMETER. IN THIS CALCULAT THE SAFETY FACTOR WAS NOT CALCULATED UNT THE END OINCE BOLT DIAMETER AND LOAD W

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 limit referenced

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

ASTM Designation: A 615 – 90

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

[illegible]

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 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 dowels, 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 ¼ 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 Methods for Steel Products for Domestic Shipm

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

2.2 *U.S. Military Standards:*

MIL-STD-129 Marking for Shipment and Storage

MIL-STD-163 Steel Mill Products Preparation f ment and Storage[3]

2.3 *U.S. Federal Standard:*

Fed. Std. No. 123 Marking for Shipment (Civil A

3. Ordering Information

3.1 Orders for material under this specific include the following information:

3.1.1 Quantity (weight),

3.1.2 Name of material (deformed and plain b 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 follow deformed and plain billet-steel bars for concrete reinforcem 60 ft 0 in. long, deformed, Grade 60, in secured lots. A 615 – __ Certified mill test reports are required.

4. Material and Manufacture

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

5. Chemical Composition

5.1 An analysis of each heat of steel shall be m manufacturer from test samples taken preferably pouring of the heats. The percentages of carbon, m phosphorus, and sulfur, shall be determined. phorus content thus determined shall not exceed

5.2 The chemical composition thus determine reported on request to the purchaser or his repres

---

[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 published as A 615 – 58. Last previous edition A 615 – 89.

[3] Annual Book of ASTM Standards, Vol 01.03.

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

[5] Available from Naval Publications and Forms Center, 5801 Tabor Ave., 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] Diameter, in.	Cross-Sectional Area, in.²	Perimeter, in.	Deformation Requirements[C] Maximum Average Spacing	Minimum Average Height	Maximum Gap (Chord of Arc)
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 part 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 [illegible] 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 [illegible] 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 specimen, 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 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 a tension under load using an autographic diagram method

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	5d	5d	
7, 8		5d	
9, 10		7d	
11		7d	9d
14, 18 [C]		9d	9d

[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, scaling, surface irregularities, or mill scale shall not be cause for rejection, provided the weight, dimensions, cross-sectional area, and tensile properties of a hand-brushed test specimen are not less than 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 either 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 is rolled by three or more designation numbers, one tension test and one bend test shall be made from both the highest and the next 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 scratches marked on the specimen before testing, a retest shall be allowed.

14.2 If the results of an original tension specimen do not meet the specified minimum requirements and are within 2000 psi of the required tensile strength, within 10 psi of the required yield point, or within two percentage points of the required elongation, a retest shall be permitted on two random specimens for each original tension specimen taken 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 those listed in 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 a temperature but not less than 60°F.

14.4 If any test specimen fails because of one of the 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 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 manufacturer's works

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 sh[illegible] properly separated and tagged with the manufactur[illegible] test identification number.

18.2 Each producer shall identify the symbols of marking system.

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

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

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

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

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

19. Packaging

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

19.2 *For Government Procurement Only*—When spec in the contract or order, and for direct procurement b direct shipment to the U.S. government, material sha preserved, packaged, and packed in accordance with requirements of MIL-STD-163. The applicab[illegible] ls be as specified in the contract. Marking for ship[illegible] of material shall be in accordance with Fed. Std. No. 12 civil agencies and MIL-STD-129 for military agencies.

The American Society for Testing and Materials takes no position respecting the validity of any patent rights asserted in connection with any item mentioned in this standard. Users of this standard are expressly advised that determination of the validity of any such patent rights, and the risk of infringement of such rights, are entirely their own responsibility.

This standard is subject to revision at any time by the responsible technical committee and must be reviewed every five years and if not revised, either reapproved or withdrawn. Your comments are invited either for revision of this standard or for additional standards and should be addressed to ASTM Headquarters. Your comments will receive careful consideration at a meeting of the responsible technical committee, which you may attend. If you feel that your comments have not received a fair hearing you should make your views known to the ASTM Committee on Standards, 1916 Race St., Philadelphia, PA 19103.

TITLE			PAGE
FACTOR OF SAFETY - ANCHOR BOND			1 of
PROJECT	AUTHOR C. FRANCKE DATE 9/5/91	CHK'D BY J.R. Tybursk. DATE 9/9/91	CHK'D BY
SO	CALC NO	FILE NO	GROUP

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

ASSUMPTIONS:

1.) MAXIMUM WORKING LOAD ON THE ANCHOR IS 18.9 ki
(84.1 kN).
2.) FIXED ANCHOR LENGTH IS 3' (0.91 m).
3.) AVERAGE HOLE (AND ANCHOR) DIAMETER IS 1 5
(0.035 m).
4.) UNCONFINED COMPRESSIVE STRENGTH (UCS) OF W
SALT IS TAKEN AS 25 MPa. (SEE DISCUSSION).
See below

REFERENCES:

1.) G.S. Littlejohn and D.A. Bruce, "Rock Anchors - State of the
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, Power

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

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

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

TITLE			PAGE
FACTOR OF SAFETY - ANCHOR BOND			2
PROJECT	AUTHOR / DATE	CHK'D BY / DATE	CHK'D BY
	C. FRANCKE 9/8/91	JRTyburski 9/9/91	
SO	CALC. NO	SHEET NO	GROUP

CALCULATIONS:

ACCORDING TO REF 1., THE ULTIMATE ANCHOR LOAD 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 1 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:

$$P_a = \pi(0.035m)(0.91m)(2500\ kN/m^2)$$
$$P_a = 250\ kN = 56\ kip$$

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

TITLE			PAGE
FACTOR OF SAFETY - ANCHOR BOND			3 OF
PROJECT	AUTHOR / DATE: C. BRACKE 9/6/91	CHK'D BY / DATE: J R Tyburski 9/6/91	CHK'D BY
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 APPLIC (REF 1,2,3). SAFETY FACTOR IS NORMALLY INTRODUCED I THE CALCULATION BY REDUCING τ, WORKING BOND STRENGTH, TO 25% TO 50% OF THE ULTIMATE BON STRENGTH (REF. 2). THIS WOULD BE EQUIVALENT T A SAFETY FACTOR OF 2 TO 4. AS IS EXPLAINED O PAGE 2 OF THIS CALC, τ WAS NOT REDUCED SO THA THE TRUE FACTOR OF SAFETY COULD BE DETERMINE

See Comment pg 1 of 3

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

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

THE ABOVE ASSUMPTIONS WILL BE VERIFIED BY FIEL TESTING.

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

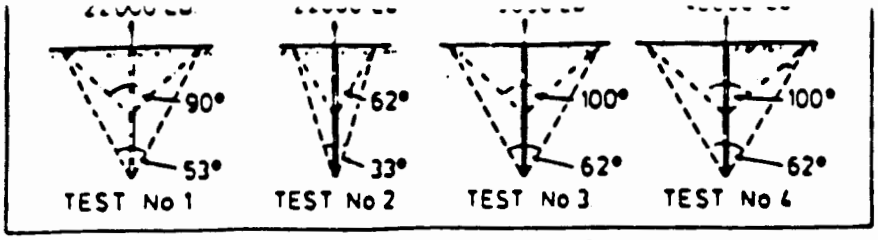


Fig 3 Possible failure modes based on test results at Trinity Clear Creek
(after Saliman and Schaefer 1968)

pies 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 "interhole 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.

on the development of bond or shear region of the rock/grout interface, described by Littlejohn (1972) a in Britain to assume an equivalent distribution of bond stress along the anchor. Thus the anchor load, P is to the fixed anchor design by the e

$$P = \pi d L \tau$$

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

This approach is used in many co e.g. France (Fargeot, 1972), Italy (Cardi, 1973) Canada (Coates, 197 USA (White, 1973)

The rule is based on the fo simple assumptions:

(i) Transfer of the load from th anchor to the rock occurs by a un distributed stress acting over the wh the curved surface of the fixed anch

(ii) The diameter of the borehole i fixed anchor are identical.

(iii) Failure takes place by sliding rock/grout interface (smooth borehc by shearing adjacent to the rock/gr terface in weaker medium (rough hole).

(iv) There are no discontinuities or ent weakness planes along which can be induced, and

(v) There is no local debonding grout/rock interface.

Where shear strength tests are out on representative samples of th mass, the maximum average workin stress at the rock/grout interface not exceed the minimum shear st divided by the relevant safety factor mally not less than 2). This approa plies primarily to soft rocks where t axial compressive strength (UCS) than 7N/mm², and in which holes been drilled using a rotary percussive torque. In the absence of shear st data or field pull-out tests, Lit (1972) states that the ultimate bond is often taken as one-tenth of the u compressive strength of massive (100 per cent core recovery) up to a mum value of 4.2N mm⁻², as that the crushing strength of the grout is equal to or greater than 42N Applying an apparent safety factor of more, which is conservative bea mind the lack of relevant data, the

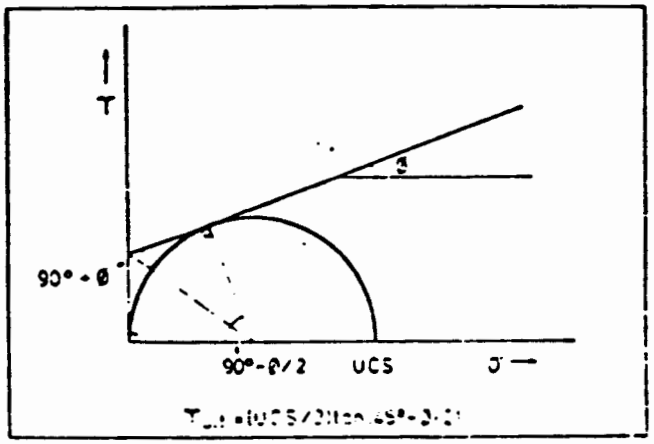


Fig. 4 Relationship between shear stress and uniaxial compressive strength

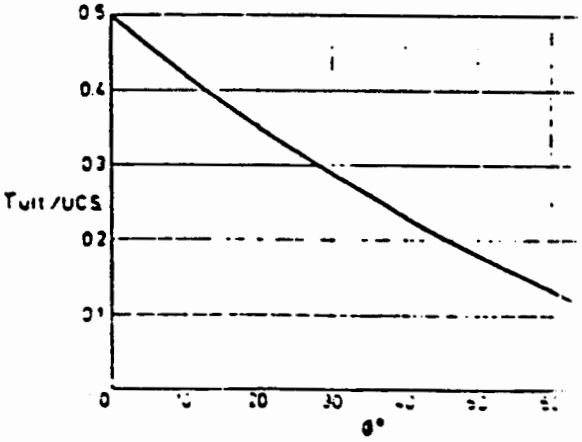


Fig 5 Effect of Ø on τult/UCS ratio

1.5N/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_{ult} 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

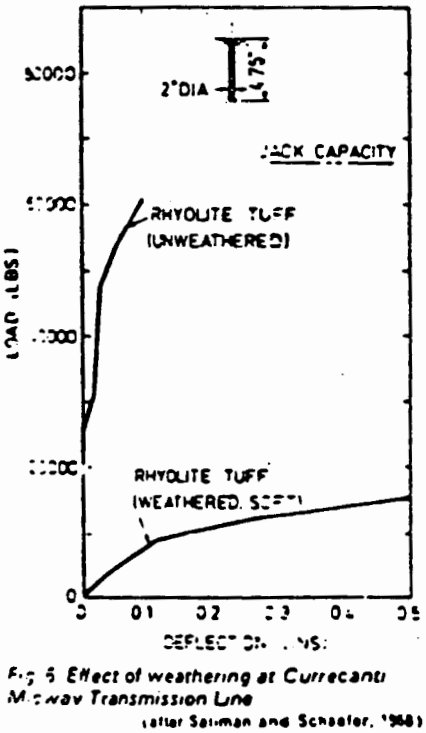


Fig 5 Effect of weathering at Currecanti Midway Transmission Line

(after Salman and Schaefer, 1968)

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	[illegible]
Weathered granite		1.90-2.90		[illegible]
Basalt	1.21-1.38	3.44	2.8-3.2	[illegible]
Granite	1.38-1.96	4.83	3.1-3.5	[illegible]
Serpentine	0.45-0.59	1.55	2.5-3.4	[illegible]
Diabase & basalt		1.72-2.10	1.5-2.5	USA—[illegible]
Metamorphic				
Metamorphic schist	0.70	2.83	4.0	USA—[illegible]
Slate & hard shale		1.83-1.38	1.5-2.5	USA—[illegible]
Calcareous sediments				
Limestone	1.00	2.83	2.4	[illegible]
Chalk—Grades I-III	0.01N (N = SPT blows 0.3m)	0.22-1.07	1.5-2.0 (Temporary) 3.3-4.0 (Permanent)	[illegible]
Tertiary limestone	0.83-0.97	2.76	2.8-3.3	Britain—[illegible]
Chalk limestone	0.86-1.00	2.76	2.8-3.2	Britain—[illegible]
Soft limestone		1.03-1.52	1.5-2.5	USA—[illegible]
Dolomitic limestone		1.38-2.07	1.5-2.5	USA—[illegible]
Arenaceous sediments				
Hard coarse-grained sandstone	2.46		1.76	Canada—[illegible]
Weathered sandstone		0.69-0.86	3.0	New Zealand—[illegible]
Well cemented mudstones		0.69	2.0-2.5	New Zealand—[illegible]
Buntar sandstone	0.40		3.0	Britain—[illegible]
Buntar sandstone (UCS > 2.0N/mm²)	0.60		3.0	Britain—[illegible]
Hard fine sandstone	0.69-0.83	2.24	2.7-3.3	Britain—[illegible]
Sandstone		0.83-1.73	1.5-2.5	USA—PC[illegible]
Argillaceous sediments				
Keuper marl		0.17-0.25 (0.48 C_u)	3.0	Britain—[illegible]
Weak shale		0.35		Canada—[illegible]
Soft sandstone & shale	0.10-0.14	0.37	2.7-3.7	Britain—[illegible]
Soft shale		0.21-0.83	1.5-2.5	USA—PC[illegible]
General				
Competent rock (where UCS > 20N/mm²)	Uniaxial compressive strength—30 (up to a maximum value of 1.4N/mm²)	Uniaxial compressive strength—70 (up to a maximum value of (4.2N/mm²)	3	Britain—[illegible]
Weak rock	0.35-0.70			Australia—[illegible]
Medium rock	0.70-1.05			
Strong rock	1.05-1.40			
Wide variety of igneous and metamorphic rocks	1.05		2	Australia—[illegible]
Wide variety of rocks	0.98			France—[illegible]
	0.50			Switzerland—[illegible]
	0.70			Switzerland—[illegible]
		1.20-2.50		Switzerland—[illegible]
	0.70		2-2.5 (Temporary) 3 (Permanent)	Italy—M[illegible]
	0.69	2.76	4	Canada—[illegible]
	1.4	4.2	3	USA—W[illegible]
		15-20 per cent of grout crushing strength	3	Australia—[illegible]
Concrete		1.38-2.76	1.5-2.5	USA—PC[illegible]

TABLE IV—ROCK/GROUT BOND VALUES WHICH HAVE SEE[illegible] PRACTICE

Rock type	Working bond (N/mm²)	Test bond (N/mm²)	Ultimate bond (N/mm²)	S_m (test)	S_f (ultimate)	Sou[illegible]
Igneous						
Basalt	1.93		6.37		3.3	Britain—Pa[illegible]
Basalt	1.10	3.60				USA—Eber[illegible]
Tuff	0.80					France—Ca[illegible]
Basalt	0.83	0.72				Britain—Ca[illegible]
Granite	1.56	1.72				Britain—Ca[illegible]
Dolerite	1.56	1.72				Britain—Ca[illegible]
Very fissured felsite	1.56	1.72				Britain—Ca[illegible]
Very hard dolerite	1.56	1.72				Britain—Ca[illegible]
Hard granite	1.56	1.72				Britain—Ca[illegible]
Basalt & tuff	1.56	1.72				Britain—Ca[illegible]
Granodiorite	1.09					Britain—Ca[illegible]
Shattered basalt		1.01				USA—Sal[illegible]
Decomposed granite		1.24				USA—Sal[illegible]
Flow breccia		0.93				USA—Sal[illegible]
Mylonitised porphyrite	0.32-0.57					Switzerland—[illegible]
Fractured diorite	0.95					Switzerland—[illegible]
Granite	0.83	0.81				Canada—B[illegible]
Metamorphic						
Schist	0.31					Switzerland—[illegible]
All types	1.20					England—M[illegible]
Weathered fractured quartzite	1.56	1.72		1.1		Britain—Ca[illegible]
Blue schist	1.52	1.67		1.1		Britain—Ca[illegible]
Weak meta sediments	1.10	1.23		1.1		Britain—Ca[illegible]
Slate	0.43					Britain—Ca[illegible]
Fine meta greywacke	1.57	1.73		1.1		Britain—Ca[illegible]
[illegible]	0.35-0.59					[illegible]
Fissured quartzite	0.51					[illegible]
Weathered meta tuff		0.29				[illegible]
Greywacke	0.34					[illegible]
Quartzite	0.93-1.20	1.02-1.32		1.1		[illegible]
Microgneiss	0.95					[illegible]

Rock	Working bond (N/mm²)	Test bond (N/mm²)	Ultimate bond (N/mm²)	τ_m (test)	τ_u (ultimate)	Source
Metamorphic—contd.						
Bersaglio schist	0 06					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 81	1 08		1 3		USA—Bure (1972)
Slate & greywacke	1 08	1 40		1 3		Germany—Ahen (1972)
Various metased-iments			1 97			Germany—Abraham & Pertig (1973)
Micaceous biotite 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 76	2 16				USA—Fold & White (1974)
Mica-schist	0 63-0 74			1 24		USA—Fold & White (1974)
Very-sheared gneiss & mud band	0 07					USA—Fold & White (1974)
Carbonate sediments						
Loamy limestone		0 63				Italy—Berardi (1980)
Fissured limestone & intercalations	1 08	1 19		1 1		Britain—Cementation Co Ltd (1962)
Limestone	0 85					Switzerland—Muller (1966)
Poor limestone	0 32					France—Henneqvin & Cambefort (1966)
Massive limestone	0 39-0 78					France—Henneqvin & Cambefort (1966)
Karstic limestone	0 54					France—Henneqvin & Cambefort (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 36 (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/puddingstone	0 44					France—Solétanche Co Ltd (1968)
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—Drossel (1970)
Sandstone	0 80					USA—Thompson (1970)
Poor sandstone	0 40					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 (1965)
Marl	0 10	0 28		2 8		Italy—Berardi (1967)
Shale	0 30		0 63		2 1	Canada—Hanna & Seaton (1967)
Very weathered shale			0 39			USA—Salman & Schaefer (1968)
Shale	0 13-0 24					USA—Kopasin (1970)
Grey siltstone	0 62					Britain—Universal Anchorage Co Ltd (1972)
Clay marl	0 14-0 24	0 21-0 36		1 5		Germany—Schwarz (1972)
Shale	0 62					Canada—McRoste 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—Beamonte (1961)
Percus. sound geossamer	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—Bujak et al (1967)
Siltstone & mudstone	1 65					Australia—Maddox et al (1967)
Fractured rock (75 per cent shale)						
Poor			0 24			USA—Salman & Schaefer (1968)
Average			0 35			USA—Salman & Schaefer (1968)
Good			0 76			USA—Salman & Schaefer (1968)
Limestone & caly 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

$$\tau_m = 0.007N - 0.12 \text{ (N/mm}^2\text{)}$$

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

Similarly, Littlejohn (1970) illustrates a correlation between N and ultimate bond

$\tau_{ult} = 0.01N$ (N/mm²)

In grades III, II and I of chalk served a range of τ_{ult} of 0.21–1.07 based on test anchors pulled to fa-

Although it would appear from presented in subsequent sections assumptions made in re[illegible]te bond distribution are not [illegible]y, it is noteworthy that few failures countered at the rock grout inter new designs are often based on cessful completion of former [illegible] that is, former working bond [illegible] re-employed or slightly modified ing on the judgement of the desi[illegible]

Table IV contains data abstra[illegible] reports of rock anchor contrac[illegible] ou, the world. In addition to the test, and ultimate bond values, [illegible] sured and designed safety factors vided where available. In certain ca[illegible] fixed anchor diameter has been inte[illegible] facilitate analysis of the data, as p[illegible]

It will be noted that, even for [illegible] type, the magnitude of bond used tice is extremely variable. There ar[illegible] reasons for this, the most impor[illegible] which are:

(*i*) Different designers use differ[illegible] values and safety factors, which related to type of anchor and exten[illegible] anchor testing programme.

(*ii*) "Standard" values for a cert[illegible] type have often been modified [illegible] local peculiarities or irregularities geology.

(*iii*) Factors related to the cons[illegible] techniques e.g. drilling method, procedure, and grout pressure [illegible] ence the results obtained. (The [illegible] these aspects will be discussed in [illegible] Construction.)

On the whole however, [illegible]rou[illegible] that the bond values emp[illegible] are gree consistent with roc[illegible] ar[illegible] petency.

Fixed anchor dimensions

The recommendations made by engineers with respect to length anchor are presented in Table V. [illegible] tain conditions it is recognised [illegible] shorter lengths would suffice, [illegible] the application of a generous f[illegible] safety. However, for a very shor[illegible] the effect of any sudden drop quality along the anchorage zone constructional errors or [illegible] could induce a serious decrease anchor's capacity.

With regard to the choice of an meter several considerations may into account:

(*i*) Type and size of tendon.

(*ii*) The relation of diameter to [illegible] area of fixed anchor and [illegible] anchor capacity, assuming bond.

(*iii*) Ratio of steel area to cross-[illegible] area of borehole for efficient [illegible] tribution and corrosion prote[illegible]

(*iv*) Drilling method and [illegible] and

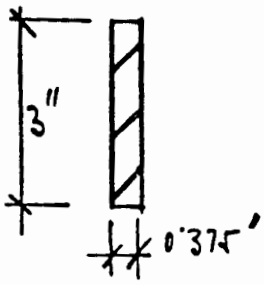
(*v*) Nature of rock in the anchor presence of unconsolidate burden, if any.

The authors find from a surve[illegible] eral hundred commercial anchor[illegible] that no direct relationship may [illegible] ved bearing in mind the [illegible] types, but that most [illegible] the trend indicated in [illegible]

TITLE U/G Room 1, Panel-1; Roof Supp. System			PAGE 9 OF
PROJECT	AUTHOR JMK DATE 08/30/91	CHK'D. BY HS DATE 9/6/91	CHK'D. BY
S.O.	CALC. NO. 6-91-JMK	FILE NO.	GROUP M. & C.E

Splice plate - stresses from bending and shear.

Assume ~~PL~~ 3/8" thick



Moment:

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

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

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

Shear:

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

Combined stress:

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

Bolt at the sliding end.

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

Assume ϕ 5/8"

$A = 3.14 \times 0.31^2 = 0.306\ in^2$

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

Finger tight.

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

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

Data and assumptions for the below calculations:

1. Density of salt material is 135 l
as per Design Basis - U/G Excavati
D-51-W-01, page 6.
2. The total load overburden is bas
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
the room (33'-0).
4. Material of the Rock Bolts is as pe
ASTM A 615 - Grade 60. Yield stres
Fy = 60 000 psi, allowable workin

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

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 11, Panel 1 General Arrangement, Sections and Details

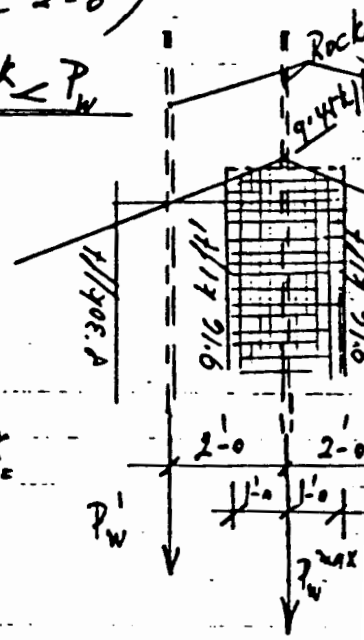
RECORD OF ISSUES

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

TITLE U/G. Room 1 - Roof Support System			PAGE 4 OF
PROJECT	AUTHOR JMK DATE 12/29/91	CHK'D. BY HS DATE 9/6/91	CHK'D. BY
S.O.	CALC. NO. 6-91-JMK	FILE NO.	GROUP M.+C.E.

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

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



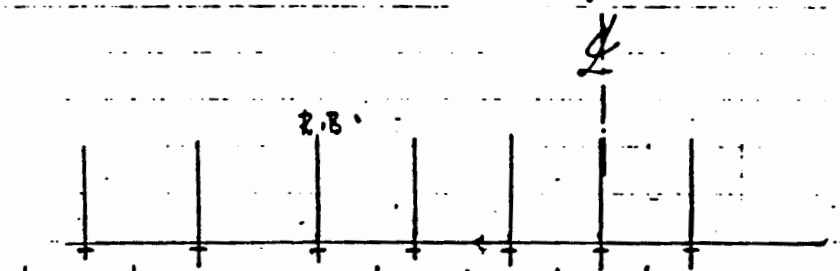
Adjacent Rock Bolt:

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

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 p

=> no overloading occurs.



Spacing of R.Bs. across the room

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

TITLE U/G. Room 1 - Roof Support System			PAGE 3
PROJECT	AUTHOR JHK DATE 8/29/91	CHK'D. BY HS DATE 9/6/91	CHK'D. BY
S.O.	CALC. NO. 6-91-JHK	FILE NO.	GROUP H.&C.E.

stress in tension $F_t = 0.60 F_y =$

$= 36000$ psi. (See PTI-1986).

5. Allowable load on one Rock Bolt is

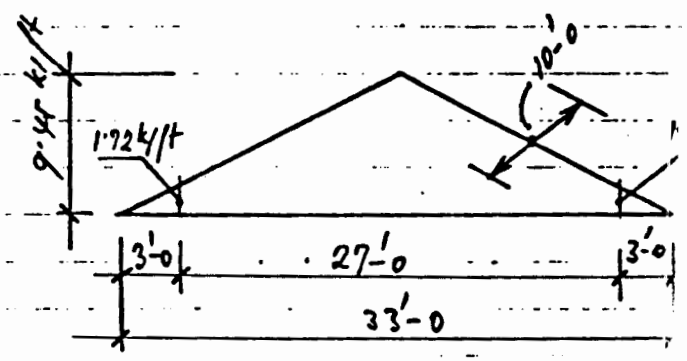
$P_W = A \times F_t = 0.79 \times 36.0 = \underline{28.4^k}$

Triangular load:

Weight of the overburden

$w = (\frac{1}{2} 10.0 \times 33.0 \times 7.0) \times 135 \text{ lb/ft}^3 = \underline{156000 \text{ lb}}$

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



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

TITLE U/G. Room 1 - Roof Supporting System			PAGE 6 OF
PROJECT	AUTHOR JHK DATE 08/30/91	CHK'D. BY HS DATE 9/6/91	CHK'D. BY
S.O.	CALC. NO. 6-91-JHK	FILE NO.	GROUP M. & C.E

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

$M = 3560\ kft \Rightarrow \sigma = \frac{3560 \times 12}{3.21} = 13308\ psi$

Shear in the web:

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

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

$r_{req.} = \frac{1.31}{15 \times 2\pi} = 0.42\ in \Rightarrow r_{used} = 1.00 >$

Use ϕ 2" washer

the rock bolt (see

Bending of the [15 perpendicular to

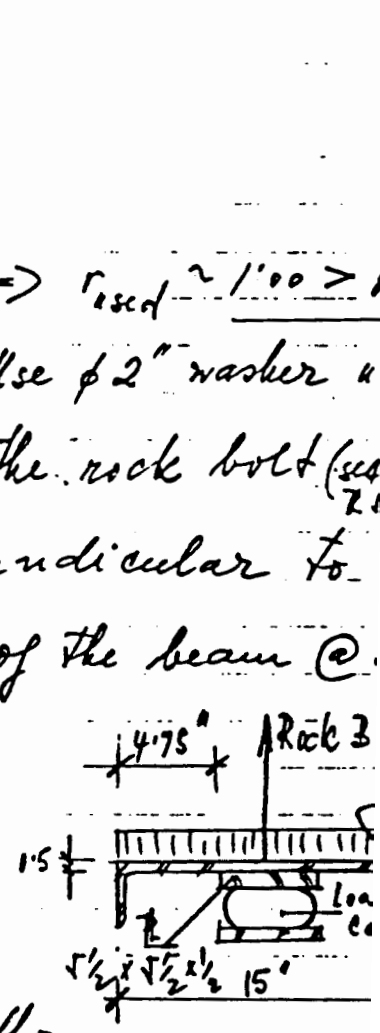
length of the beam @

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

$\doteq 78.9\ psi$

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

$\sigma = \frac{890}{1.042} = 27192\ psi \doteq \sigma_{allow.}$



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

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

TITLE U/G. Room 1 – Roof Support System			PAGE 5 / 9
PROJECT	AUTHOR JMK DATE 8/29/91	CHK'D. BY HS DATE 9/6/91	CHK'D. BY DA
S.O.	CALC. NO. 6-91-JMK	FILE NO.	GROUP M.&C.E.

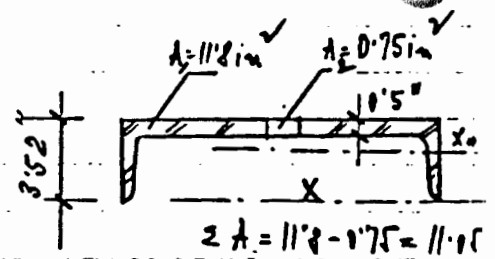
Check of stress in the channel from bending. ($C15 \times 40$; $S = 3.37 in^3$).

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

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

$\sigma = \dfrac{3560 \times 12}{3.37} = 12676$ psi $< \sigma_{allow}$.

Section modulus "S" of the $C15 \times 40$ with a $\phi 1.5''$ hole.



Static moment M_x:

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

$- 0.75 \times 3.27 \ldots - 2.45$

$M_x = 29.88 in^3$

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

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

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

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

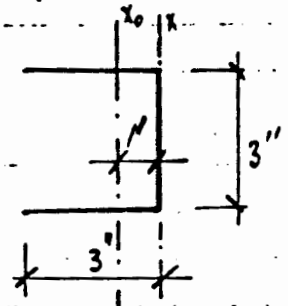
REV. NO.	REV. DATE	AUTHOR	DATE	CHK'D. BY	DATE	CHK'D. BY

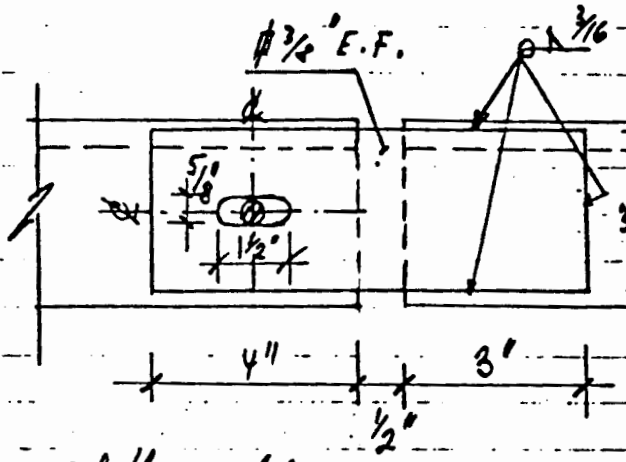
TITLE	U/G Room-1; Panel-1; Roof Supp. System		PAGE 8 OF
PROJECT	AUTHOR JMK 8/30/91	CHK'D. BY HS DATE 9/6/91	CHK'D. BY
S.O.	CALC. NO. 6-91-JMK	FILE NO.	GROUP M. & C.

Design of splice:

- assumed as a flexible pin, allowing ~~some~~ horizontal movement

C.G. of the weld:





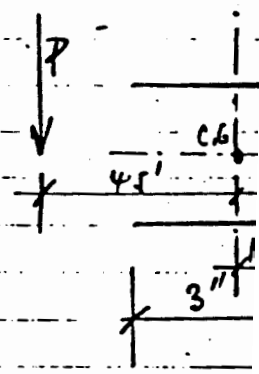
$S_{M_x} = 2(3 \times 1.5) = 9 in$

$L = 3 \times 3 = 9 in$

$x_0 = \frac{9.0}{9.0} = 1''$

Design of the weld:

See table XXIII, page 4-80 AISC.



$k = 1.0$ $a = 1.5$

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

$C = 0.775$

$D = \frac{P}{C C_1 l} = \frac{5.66^k}{0.775 \times 1.0 \times 3.0} = 2.4$

Use 3/16 weld

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

TITLE U/G. Room 1 – Roof Supporting System			PAGE 9
PROJECT	AUTHOR JMK – 08/30/91	CHK'D. BY HS DATE 9/6/91	CHK'D. BY DA
S.O.	CALC. NO. 6-91-JMK	FILE NO.	GROUP H.&C.E.

Shear (in the [18" web) (along the ℞ perimeter)

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

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

Combined stress: (bending + shear)

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

$$\left(\frac{21.19}{23.76}\right)^2 + \left(\frac{2.58}{14.0}\right)^2 = \underline{0.795 + 0.033 = 0.828} < 1$$

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

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
October 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.

TITLE			PAGE
FACTOR OF SAFETY - WEDGE PULLOUT			4
PREPARED	AUTHOR C. FRANCKE DATE 9/8/91	CHK'D BY J R Tyburski DATE 9/8/91	CHK'D BY
B U	CALL NO.	FILE NO	GROUP

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

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

"Geological Characterization Report, Waste Isolation Pilot Plant (WIPP) Site, Southeastern New Mexico" 197 Page

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.

WESTINGHOUSE WASTE ISOLATION DIVISION

TITLE				PAGE
FACTOR OF SAFETY - WEDGE PULLOUT				2 OF
PROJECT	AUTHOR / DATE	CHK'D BY / DATE	CHK'D BY	
	C. FRANCKE 7/26/91	JR T[illegible] 9/4/91		
W.O.	CALC NO	FILE NO	GROUP	

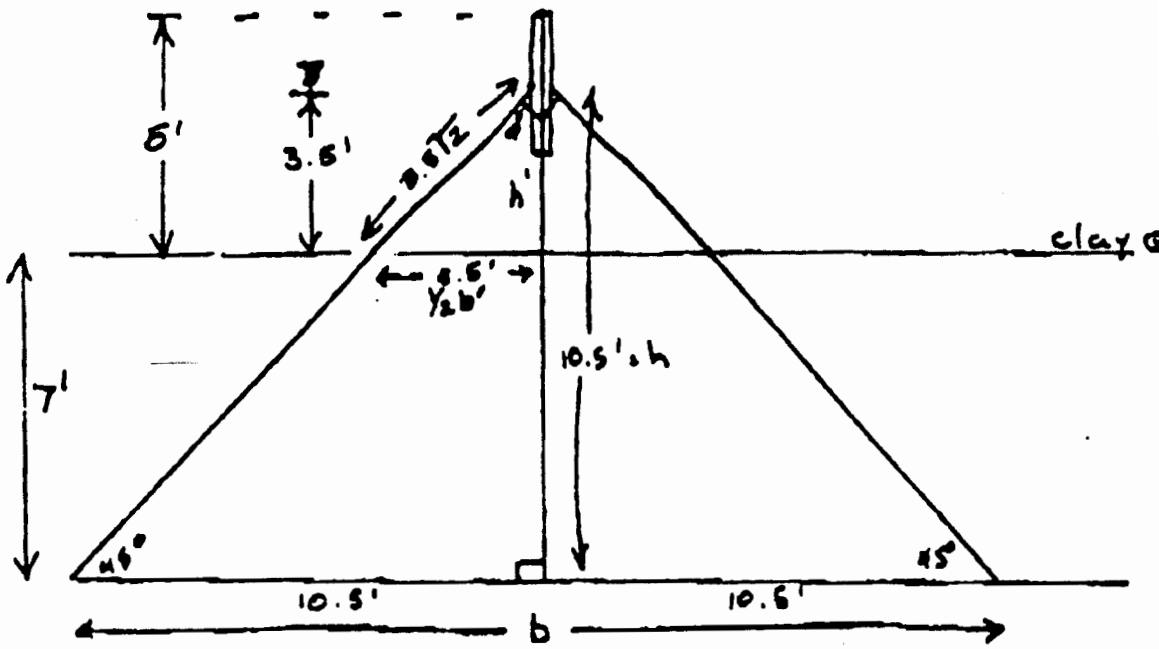


FIG. 1

W = WIDTH = 33'

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

WESTINGHOUSE FORM 49289A

TITLE			PAGE
FACTOR OF SAFETY - WEDGE PULLOUT			9
PROJECT	AUTHOR C. FRANCKE DATE 9/8/91	CHK'D BY J.R. Tybursk DATE 9/9/91	CHK'D BY
A/D	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.

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

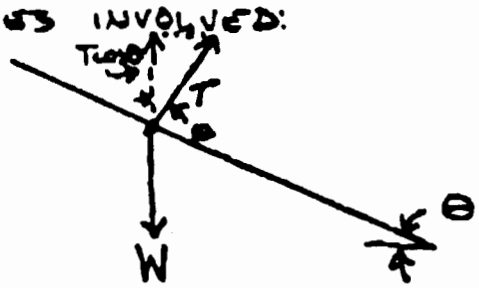
$$W = 491\ kip = 2.18\ MN$$

SHEAR RESISTANCE OF SMALL WEDGE

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

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:



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

WASTE ISOLATION PILOT PLANT
SUPPLEMENTARY ROOF SUPPORT SYSTEM

APPENDIX D
Geomechanical Monitoring Program

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

GEOTECHNICAL MONITORING PROGRAM

FOR ROOF SUPPORT SYSTEM IN ROOM 1, PANEL 1

GEOTECHNICAL MONITORING PROGRAM
FOR ROOF SUPPORT SYSTEM IN ROOM 1, PANEL 1

Table of Contents

Section	Title	Page
1.0	INTRODUCTION	D-1
2.0	MONITORING OF ROOM PERFORMANCE	D-2
	2.1 Room Convergence Measurements	D-2
	2.2 Extensometer Measurements	D-3
	2.3 Survey Measurements	D-3
	2.4 Fracture Mapping of Observation Boreholes	D-3
	2.5 Data Acquisition	D-4
3.0	Monitoring of Support System Performance	D-4
	3.1 Rockbolt Load Cells	D-5
	3.2 Pressure Cells	D-5
	3.3 Cable Elongation	D-6
	3.4 Data Acquisition System	D-6
4.0	Adjustment of Support System	D-7
	4.1 Criteria for Load Adjustment	D-8
	4.2 Analytical Evaluation for Load Adjustments	D-9
	REFERENCES	D-10

GEOTECHNICAL MONITORING PROGRAM
FOR ROOF SUPPORT SYSTEM IN ROOM 1, PANEL 1

Table of Contents
(Continued)

List of Appendices

Appendix	Title
A	Geomechanical Instrumentation System Specifications

GEOTECHNICAL MONITORING PROGRAM
FOR ROOF SUPPORT SYSTEM IN ROOM 1, PANEL 1

Table of Contents
(Continued)

List of Figures

Figure	Title	Page
2-1	Room Performance Instrumentation	D-11
2-2	Wire Convergence Meter Cabling	D-12
2-3	Extensometer Cabling	D-13
2-4	Typical Radial Convergence Point	D-14
2-5	Typical Rod-Extensometer Installation	D-15
2-6	Support System Performance Instrumentation	D-16
2-7	Rockbolt Load Cell Cabling	D-17
2-8	Pressure Cell Cabling	D-18
2-9	Wire Rope Strain Gage Cabling	D-19
2-10	Typical Load Cell Installation	D-20
2-11	Typical Pressure Cell Installation	D-21
2-12	Typical Crackmeter Installation	D-22
4-1	Rockbolt Loading Redistribution	D-23

GEOTECHNICAL MONITORING PROGRAM
FOR ROOF SUPPORT SYSTEM IN ROOM 1, PANEL 1

Table of Contents
(Continued)

List of Tables

Table	Title	Page
2-1	Panel 1 Room 1 Geomechanical Instrumentation Specifications	D-24

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 Appendix A. The design itself, is presented in Chapter 4 Support System, Underground Storage Area, Panel 1, Room 1. 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 would indicate that it will form within the next 7 years in Room 1, Panel 1 unless fracture development in the roof can be controlled. The support system in Room 1 has two purposes. It is designed 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 a 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. 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).

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, 1991a). The Geotechnical Engineering Program Plan supports the Quality Assurance Program implemented at the site (Westinghouse, 1991b) and covers the eighteen criteria that are defined in the Quality Assurance Program for Nuclear Facilities (ANSI/ASME NQA-1-1989).

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 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 compared 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 establihment of the room closure rates and determination 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 monitoring system must meet are given in Appendix A. Figures 2-2 and 2-3 provide 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 wall height. The convergence measurements will establish the rates of room closure for comparison with predicted rates and will be evaluated to determine asymmetric 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 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 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 device capable of measuring length changes. 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, 2m (6.5 feet), 2.3m (7.5 feet), 4m (13 feet), and 4.3m (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, 1.5m (5 feet), 3m (10 feet), 4.6m (15 feet), 7.6m (25 feet) and 15m (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 National Institute of Standards and Testing (NIST).

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 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. Additional boreholes will be installed at the existing locations to monitor the behavior of the roof cross-section. 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, 1986). 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 for fractures immediately following their drilling. This will be carried out using the scratch probe and the borehole camera.

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 once 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.

Remote 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:

- The performance of the structural system that supports the roof.
- 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 will 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 they 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 will be 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 will 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 compensates 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 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.

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

The pressure cells will be placed between the rock surface and support mesh. An important factor to take into consideration when installing a pressure cell is to ensure good contact with the surrounding material and to avoid localized or point loading of the cell. To avoid point loading, each pressure cell will be encapsulated with a concrete based grout. After the grout has set up, the pressure cell will be placed between two steel plates in order to evenly distribute the load on it. The pressure cells will be 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 changed 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 at selected sections within the room that are expected to have the largest deformations and subsequently load the mesh an cables. 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:

- Analog/Digital Loopback - OFF
- 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 anchors will be set to a nominal load of 500 kg (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 will 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 preliminary 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 precisely 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 27.5 MPa (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 percent.

- If the load in one anchor exceeds 27.5 MPa (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 asymmetric room closure, then an asymmetric 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. This assumes that interaction 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 Owen and Hinton (1980) was modified for used in the Salt Repository Program for the disposal of high level radioactive wastes. The FLAC code (ITASCA, 1991) is a proprietary code developed by ITASCA, Inc. to simulate the behavior of geotechnical materials which may undergo plastic flow.

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 asymmetric 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.

REFERENCES

ANSI/ASME, 1989, "Quality Assurance Program for Nuclear Facilities, NQA-1-1989.

ITASCA, Consulting Group, Inc., 1991, "Fast Lagrangian Analysis of Continua (FLAC), Version 3.0", Minneapolis, Minnesota.

Owen, D. R. J., and Hinton, E., 1980, "Finite Elements in Plasticity: Theory and Practice", Pineridge Press Limited, U.K.

U.S. Department of Energy, 1991, "Report of the Geotechnical Panel on the Effective Life of Rooms in Panel 1", DOE/WIPP 91-023

U.S. Department of Energy, 1986, "Interim Field Data Report", DOE-WIPP 86-012.

Westinghouse Electric Company, 1991a, "Geotechnical Engineering Program Plan, WP-07", Waste Isolation Division, Carlsbad, New Mexico.

Westinghouse Electric Company, 1991b, "Quality Assurance Program," WP-13-1, Revision 13, Waste Isolation Division, Carlsbad, New Mexico.

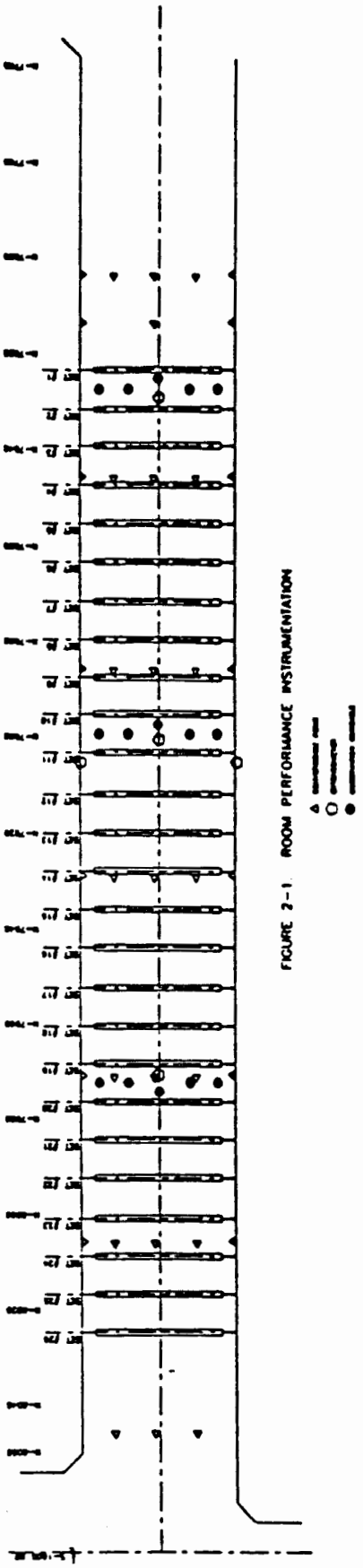


FIGURE 2-1. Room Performance Instrumentation

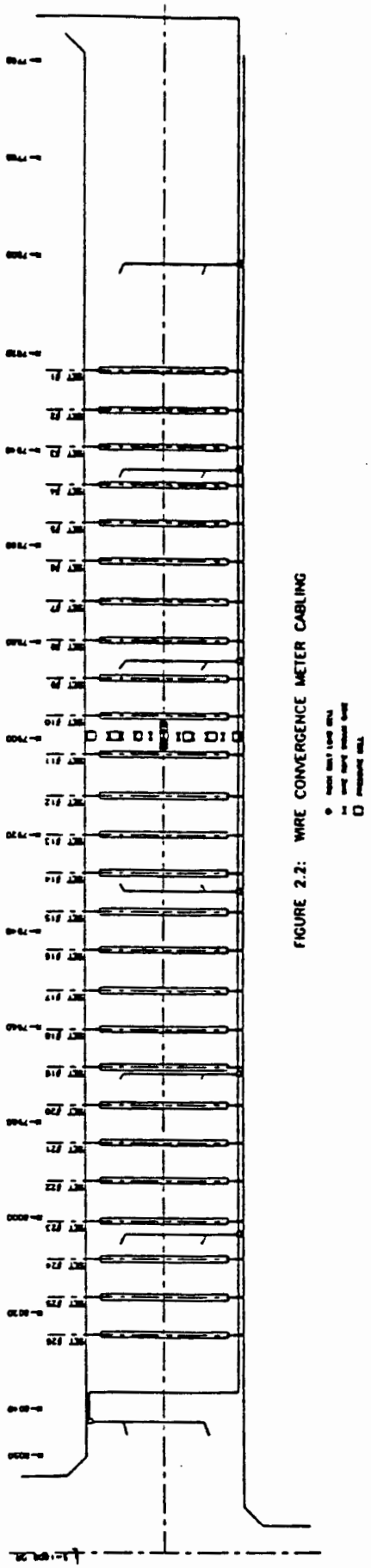


FIGURE 2-2. Wire Convergence Meter Cabling

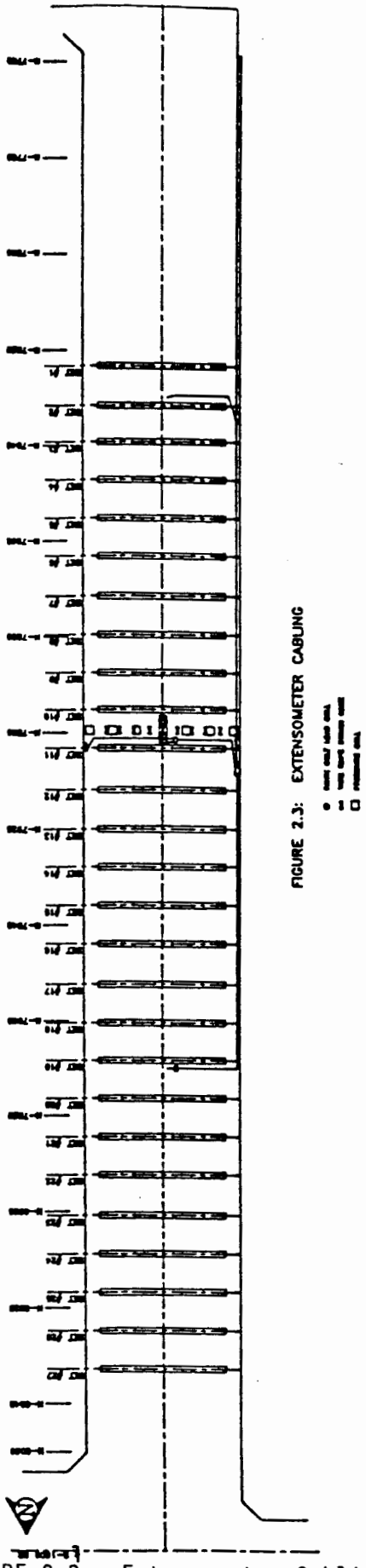
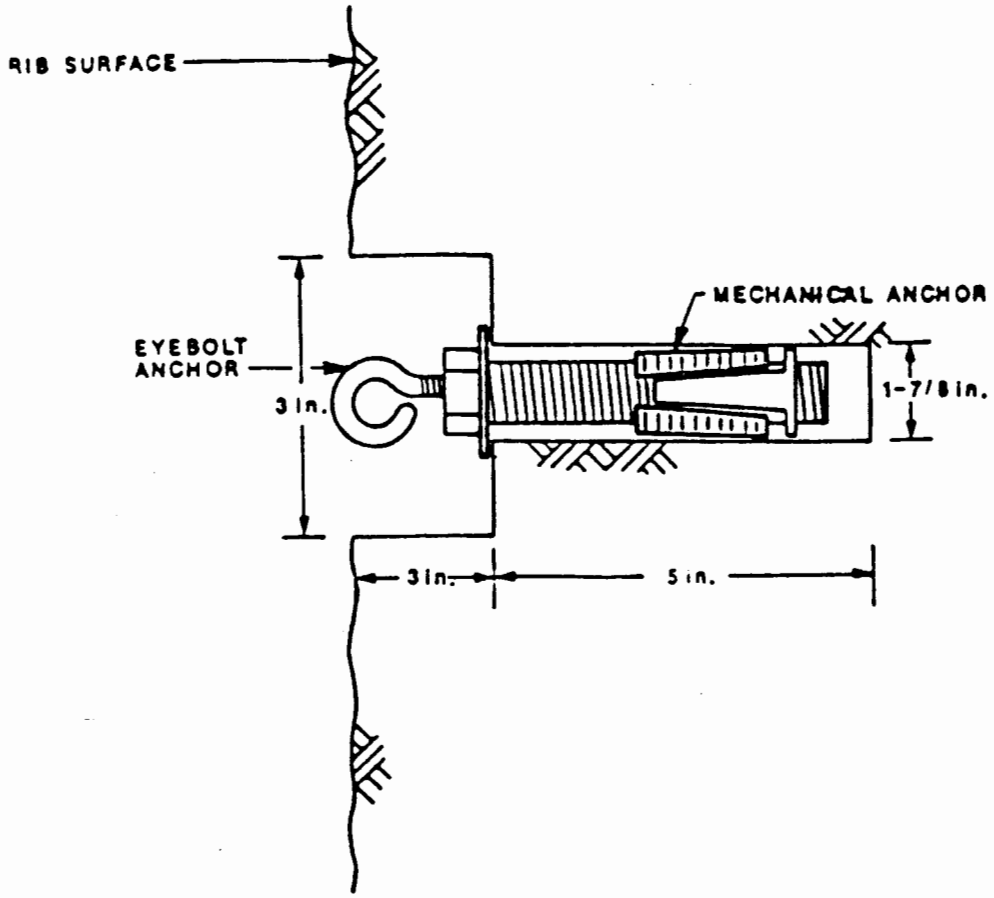


FIGURE 2-3. Extensometer Cabling



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

• *Dimensions approximate*

FIGURE 2-4. Typical Radial Convergence Point

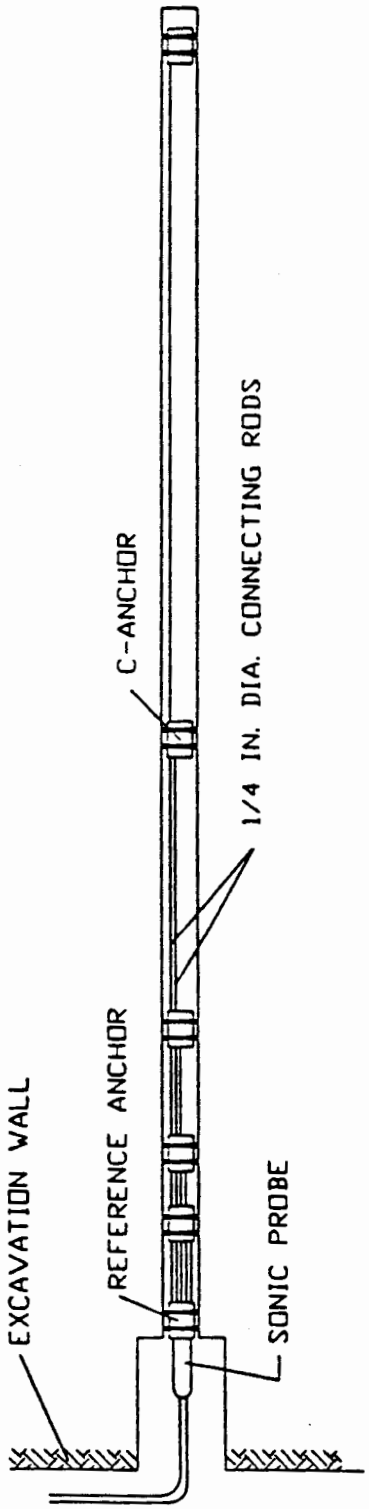


FIGURE 2-5. Typical Rod-Type Extensometer Installation

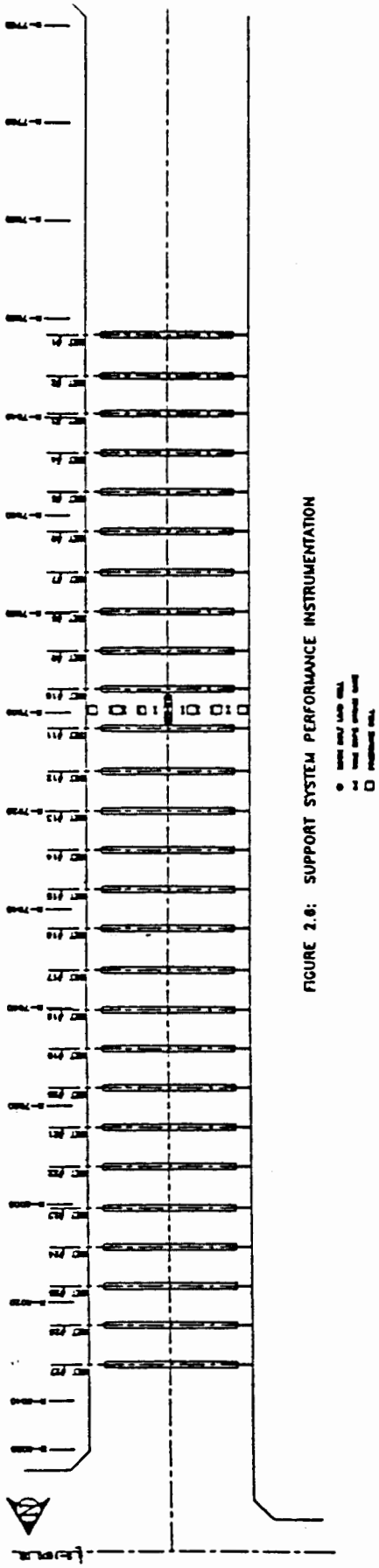


FIGURE 2-6. Support System Performance Instrumentation

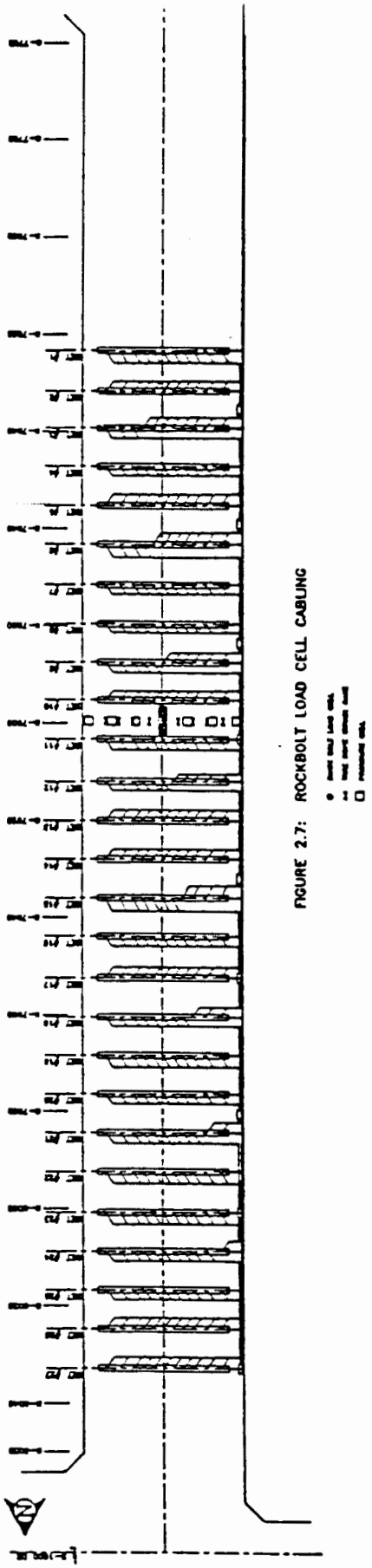


FIGURE 2-7. Rockbolt Load Cell Cabling

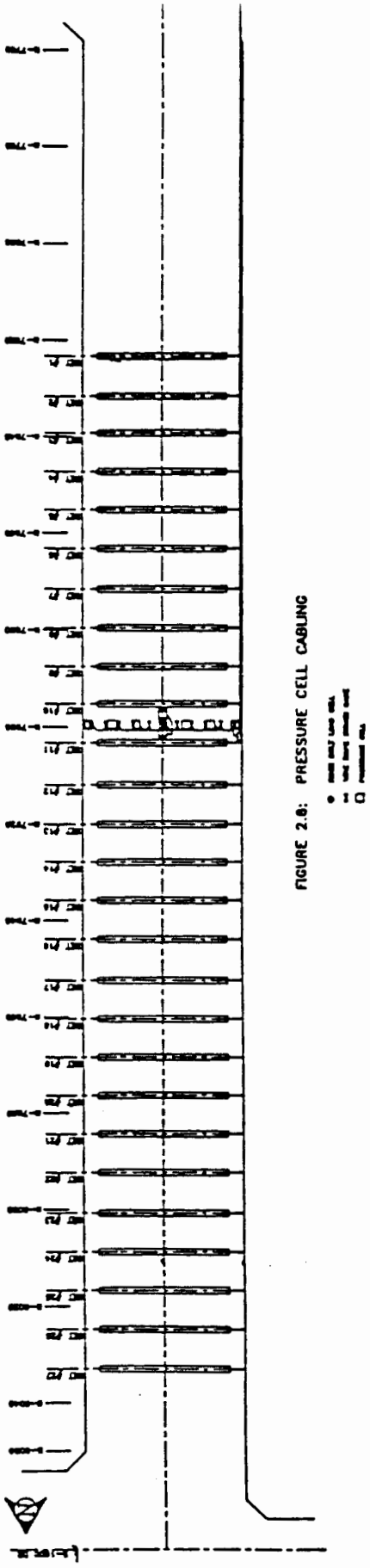


FIGURE 2-8. Pressure Cell Cabling

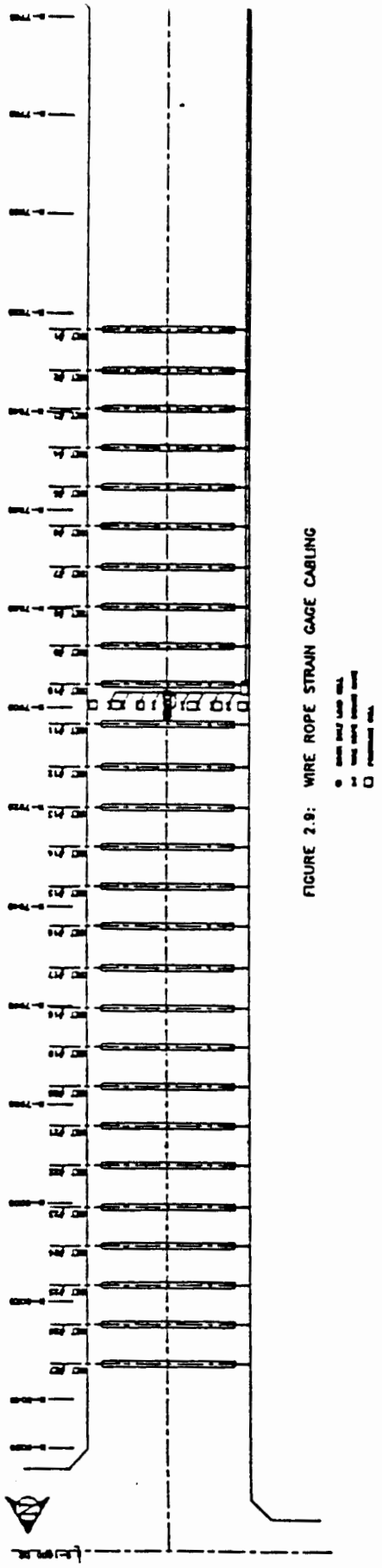


FIGURE 2-9. Wire Rope Strain Gage Cabling

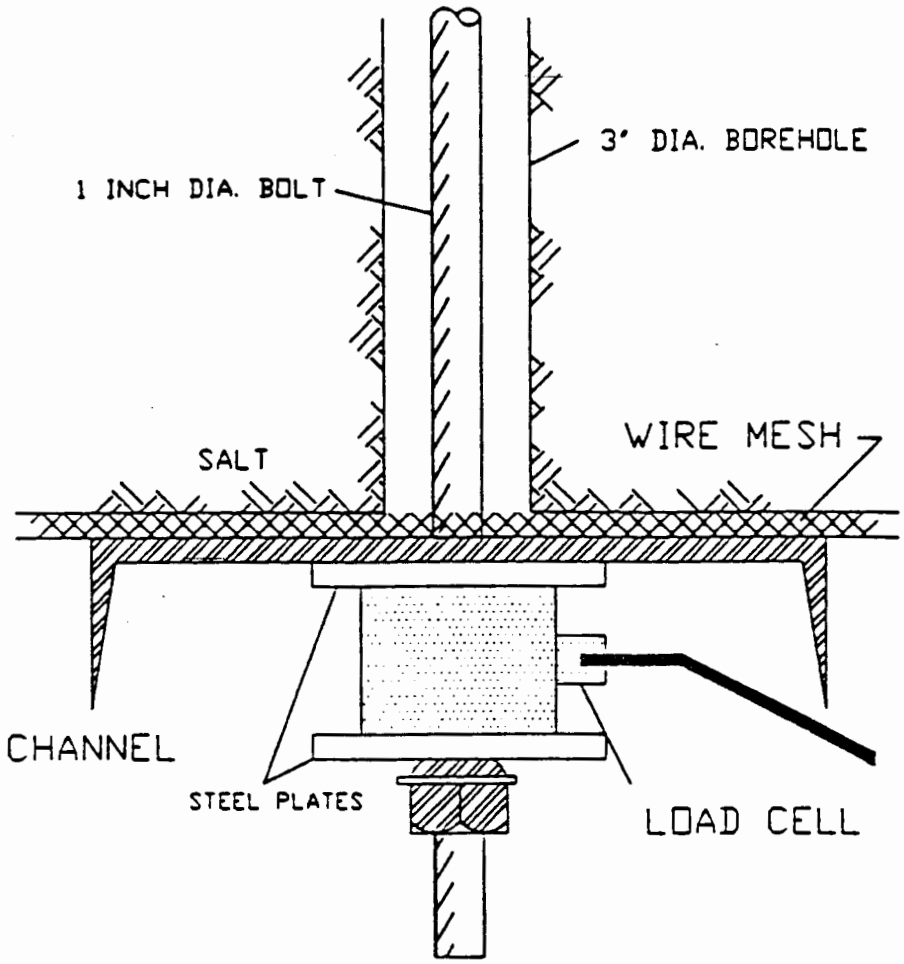


FIGURE 2-10. Typical Load Cell Installation

GEOMECHANICAL INSTRUMENTATION SYSTEM SPECIFICATIONS

MATERIALS/INSTALLATION	SPECIFICATION
	7. Insulation and Jacket For U/G installatins the following materials are approed for use for cable insulation and jacketing as designated below: 1. Jacketing: CPE - Chlorinated Polyethylene CSP - Chororsulphonated Polyethylene - Neoprene 2. Insulation: EPR - Ethylene Propylene Rubber PPE - Polypropylene Ethylene
INSTRUMENTATION CABLES	XLPE - Crosslink Polyefin (Polyethylene) EPDM - Ethylene Propylene Diene Monomer 8. All wires and cables shall be identified at junction boxes and cabinets with wire and cable marker indicating the instrument number of the wired device.
TERMINAL BLOCKS & STRIPS	Specification No.E-P-244 1. Terminal blocks shall be surface-mounted, molded plastic screw-terminals, barrier type, capable of accepting No. 20 through No. 12 AWG wire terminated with solderless ring-tongue compression-type lugs. An 8-inch horizontal clear gap shall be provided between adjacent columns.

MATERIALS/INSTALLATION	SPECIFICATION
	2. Thirty percent unused spare terminals shall be provided for each group of terminal blocks. 3. Terminal blocks shall be grouped by circuit type and segregated according to circuit voltage and destination of field wiring. 4. Not more than wires shall be connected to any one terminal point. 5. Terminal strips for powere connections shall be fitted with covers to minimize shock hazards. 6. Each teminal block marking strip shall be identified. Identification of terminal blocks and terminal points shall be alphanumeric, with each terminal block having unique identifying letters and each terminal point to identify instrument input.
GROUNDING	Specification No.E-P-247 1. Instrument cases, relays, and other switching devices shall be grounded to the cabinet or panel steel structure through theri mounting screws. Additioal grounding shall not be required. 2. For each instrument circuit with a definite ground at the cabinet or panel, a single wire shall be run independently to the applicable ground bus and faatened thereto with a drilled-and-tapped connection, or on a terminal board for wall-mounted cabinets. Each ground point shall be readily accessible for removal, testing, and replacements. 3. Signal cable shielding and grounding requirements shall be included in the wiring diagrams.
FUNCTIONS OF TYPICAL INTERFACE DEVICES	1. Control Unit: When the control unit is activated by the communication controller, either locally or remotely by the GIS Data Logger, it shall, in turn, activate the readout unit and switching module simultaneously to read

MATERIALS/INSTALLATION	SPECIFICATION
	all the sensors and transmit the output of the readout unit to the GIS Data Logger. When the last connected sensor has been read, the unit shall deactivate itself. 2. Switching Module: Sequentially connects the readout unit to the sensors, enabling each one to be read. 3. Remote Reaout/Polling Unit: When the readout unit connected to particular sensor, the readout unit shall energize the sensor, receive its measurement, display the reading, and the transmit the reading to the communication controller through the signal converter. 4. Signal Converter: If the ouput from the readout unit is not in conformance with EIA RS-232-C and ANSI X3.4, a signal converter shall be furnished to generate a serial ASCII signal. The signal converter shall be either an internal subsystem in the control unit or an external unit. 5. Communication Controller: Under the control of the GIS Data Logger, the Communication Controller shall activate the remote readout/polling unit to start the data transmission and deactivated the remote readout/polling unit when data transmission in complete.
FUNCTIONS OF TYPICAL INTERFACES DEVICES	When interrogated by the GIS Data Logger, the Communication Controller shall be able to handle the malfunction or failur of the interface devices. The Communication Controller shall modulate and demodulate the signal for the data link cable/

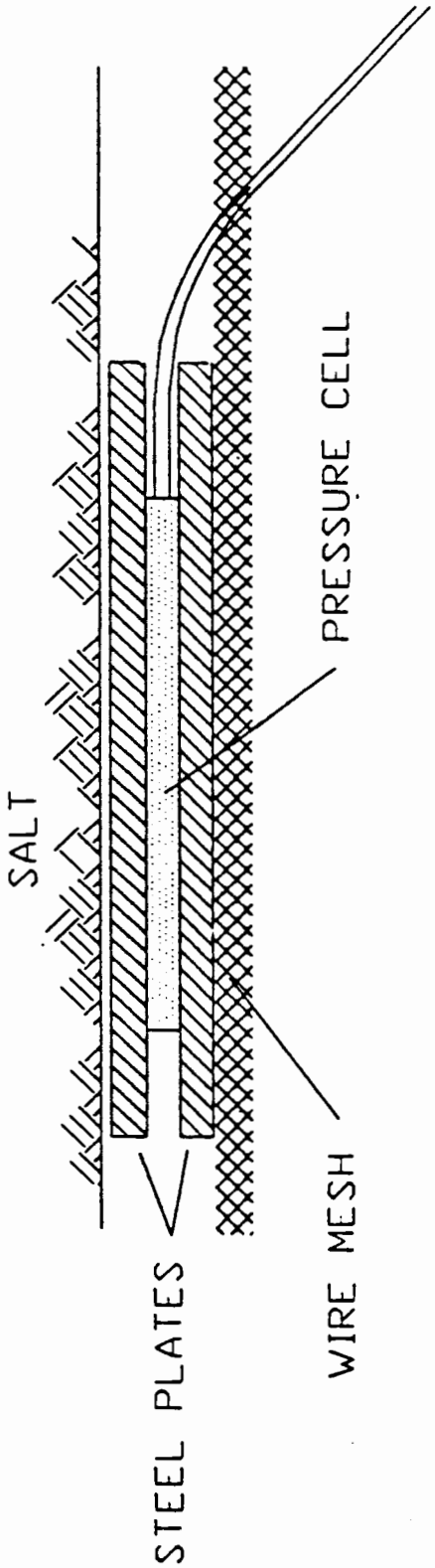


FIGURE 2-11. Typical Pressure Cell Installation

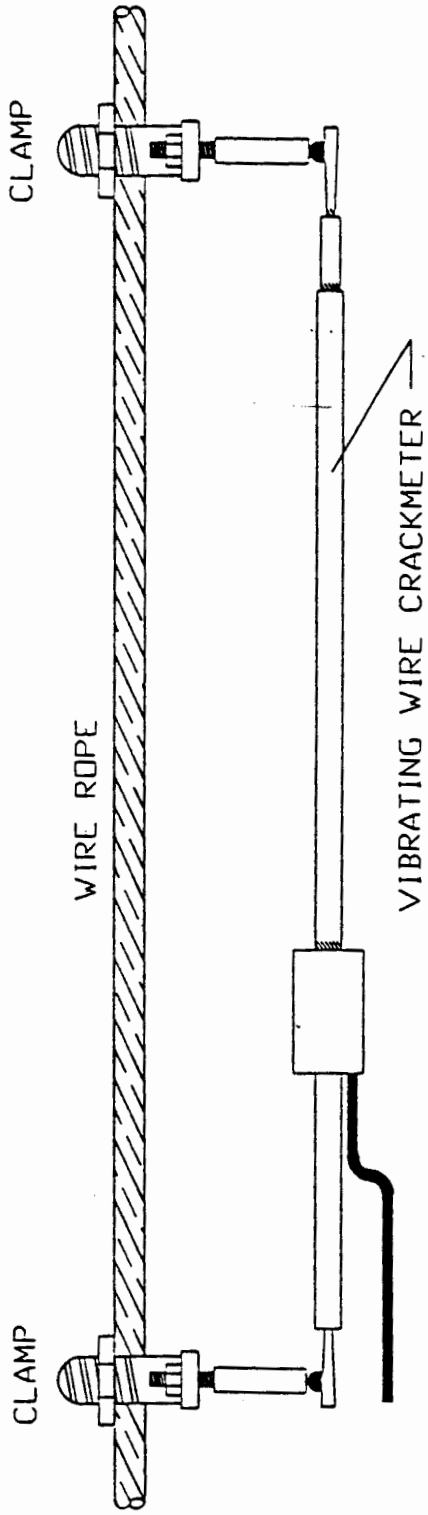


FIGURE 2-12. Typical Crackmeter Installation

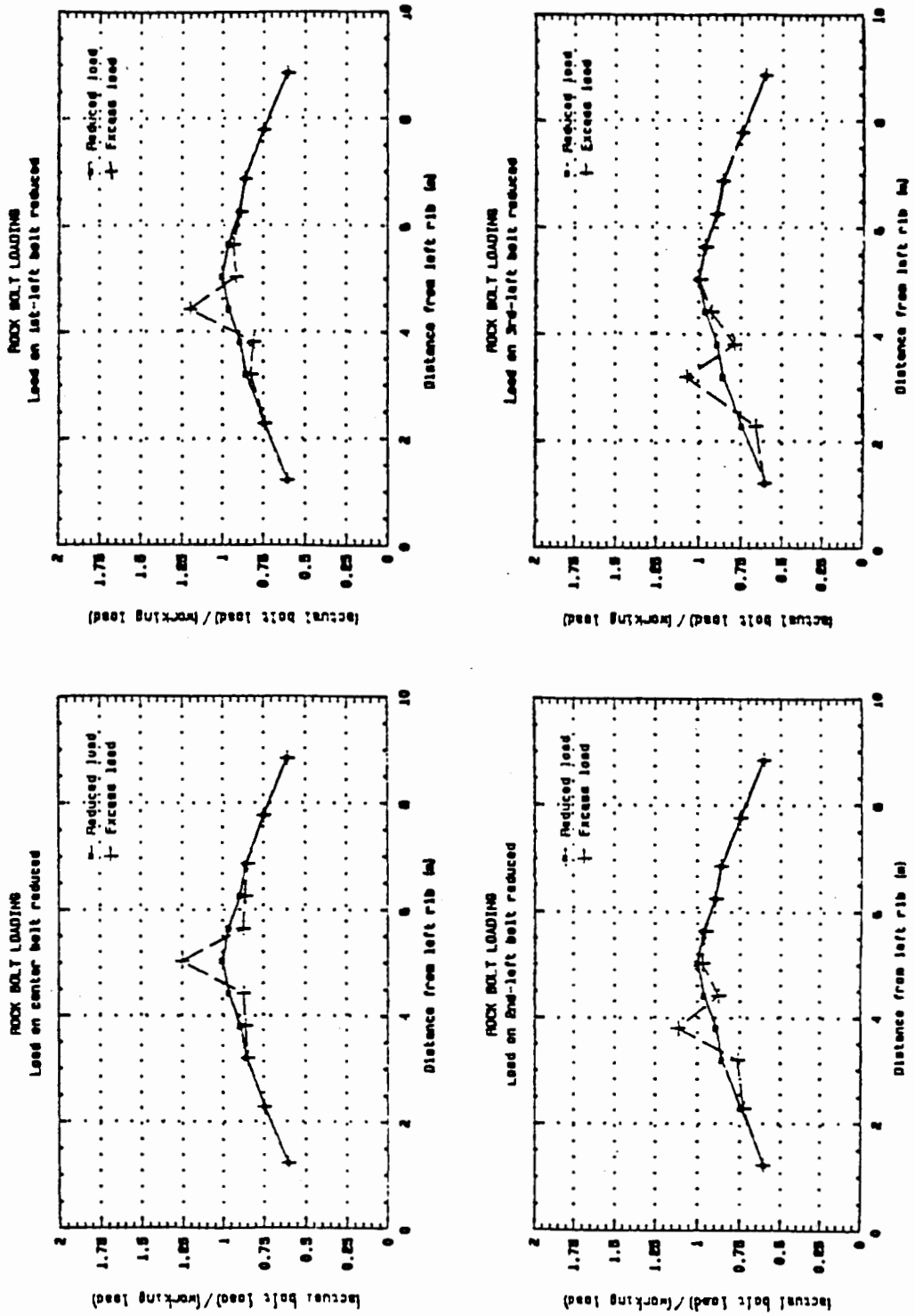


FIGURE 4-1. Rock Bolt Loading Redistribution

Table 2-1

PANEL 1 ROOM 1
GEOMECHANICAL INSTRUMENTATION SPECIFICATIONS

TABLE 2.1

PANEL 1 ROOM 1
GEOMECHANICAL INSTRUMENTATION SPECIFICATIONS

Instrument Type	Measured	Range	Precision	Accuracy	Method
Borehole Extensometers	Cumulative Deformation	0-2 in.	+/- 0.001 in.	+/- 0.001 in.	Sonic Probe
Convergence Points	Cumulative Deformation	2-50 ft.	+/- 0.001 in.	+/-0.005 in.	Tape Extensometer
Convergence Meters Wire	Cumulative Deformation	0-3 ft.	+/- 0.004 in.	+/- 0.032 in.	Rotary Potentiometer
Rockbolt Load Cells	Load	0-50 tons	+/- 40 lbs	+/- 40 lbs	Strain Gage
Earth Pressure Cells	Lithostatic Pressure	0-10 psi	+/- 0.05 psi	+/- 0.05 psi	Vibrating Wire
Crackmeters	Cumulative Strain	0-4 in.	+/- 0.0008 in.	+/- 0.004 in.	Vibrating Wire

APPENDIX A

GEOMECHANICAL INSTRUMENTATION SYSTEM SPECIFICATIONS

MATERIALS/INSTALLATION	SPECIFICATION
CABINET DESIGN AND PERFORMANCE	Specification No.E-P-309 1. Cabinets shall be wall-mounted Maintenance to wall. Maintenance on wall-mounted cabinets shall be from the front. 2. Cabinet enclosures shall be NEMA Type 4X enclosures. 3. Interface cabinets shall be fitted with flame-resistant gaskets and tamperproof screws. 4. All controls,Instruments, and devices shall be shown on the contract drawings. 5. Cabinets shall have their identification permanently marked on the cabinet or panel structure immediately adjacent to their location. Identification shall be clearly visible.
INSTRUMENTATION CABLES	Specification No.E-Q-281 1. Cable shall be suitable for use in dry and wet location, capable of continous operation temperature of 90 degrees Celsius. It shall be UL-listed as Type TC cable in accordance with NFPA 70, Article 340. 2. Cable shall be rated for 300V or 600V insulation, single- or multiple-twisted pairs. 3. Each conductor shall be tinned, soft or annealed copper, Class B seven-strand concentric in accordance with ASTM B 33 and ASTM B 8, Size No. 16 AWG for single pairs and No. 20 AWG for multiple pairs.
INSTRUMENTATION CABLES	4. Cable shields for single-twisted pairs shall be made of aluminum tape wrapped around the conductor and the tinned stranded copper drain wire. 5. Shielding for multiple shielded pairs shall consist of a group shield wrapped around each pair and drain wire and an overall cable shield wrapped around the whole cable assembly and drain wire. 6. Conductor identification shall be by color codes in accordance with NEMA WC 30.

App. B1
Copied incorrectly
correct copy
inserted 06/03/92
keep this as copy of
what was replaced

2.4 SURFACE HYDROLOGY

2.4.1 HYDROLOGIC DESCRIPTION

2.4.1.1 Site and Facilities

The WIPP facility lies within the Los Medanos area in the drainage basin of the Pecos River (Figure 2.4-1). The headwaters of the Pecos River are located northeast of Santa Fe, New Mexico; from there the river flows south through eastern New Mexico and western Texas until it discharges into the Rio Grande. The Pecos River has an overall length of approximately 500 miles, a maximum basin width of 130 miles and a total drainage area of roughly 44,500 square miles(mi^2). Of these, 20,500 $miles^2$ are classified as noncontributing.[1] Climatic characteristics of the basin vary from sub-humid in the mountains to semiarid at the lower elevations.

The Pecos River is about 14 miles west of the WIPP facility at its closest point (river mile 430 above the confluence with the Rio Grande), and has a drainage area of about 19,000 $miles^2$. The slope of the Pecos riverbed in this area is approximately 4.0 feet per mile.[2]

East of the river, near the WIPP facility, which includes the Los Medanos area, no outstanding natural drainage features exist. A few small unnamed drainage channels constitute all the tributaries joining the Pecos from the east within 50 miles north or south of the WIPP facility. From the west, the principal tributaries joining the Pecos north of Carlsbad are Rio Penasco with a drainage area (D.A.) of 1,060 $miles^2$ at river mile 496 (rounded to nearest mile), the Rocky Arroyo (D.A. = 64 mi^2) at river mile 475, and the Dark Canyon (D.A. = 451 mi^2) at river mile 459. Other smaller tributaries flowing from the west are the Little Walt Canyon, Willow Draw, Spencer Draw, and the North Seven, Middle Seven, and South Seven Rivers. Most of these tributaries originate in the Guadalupe Mountains.[2,3] Downstream from Carlsbad, the Black River (D.A. = 436 mi^2) joins the Pecos from the west at a point about 16 miles southwest of the WIPP facility at river mile 436. The Delaware River (D.A. = 689 mi^2) joins the Pecos from the west at river mile 406 and a number of other small creeks and draws join at various points between Carlsbad and Malaga.

In the reach between Carlsbad and Malaga (where the river is closest to the WIPP facility), the bed of the Pecos is made up of sand and gravel. The overbanks adjacent to the main channel consist of sandy soil dominated by mesquite and salt cedars. In general, the right (west) bank has a steeper slope than the left (east bank) and, at a few places, the banks are made up of bare rock. During major floods, river water spreads over a flood plain.

The Pecos River flow above Malaga (river mile 432) is regulated by storage in several dams, including Santa Rosa Lake (Los Esteros Dam-river mile 757), Lake Sumner (Alamogordo Dam - river mile 702), Lake Brantley (river mile 479), Lake Avalon (river mile 467) and several other smaller upstream dams that divert water for irrigation and power production. The locations of dams on the Pecos River and its tributaries are shown in Figure 2.4-1. Salient features of these structures are given in Table 2.4-1. Numerous stream gaging stations are located on the Pecos River and several of its tributaries. These gaging stations are shown in Figure 2.4-1. Information pertaining to several gaging stations, including average, maximum, and minimum recorded discharges, is contained in Table 2.4-2.

The stream gaging station at Malaga, New Mexico on the Pecos River is the nearest station to the WIPP facility. The maximum and minimum recorded flows at this station during the period 1938 to 1985 were 120,000 cubic feet per second (ft^3/s) (August 23, 1966) and 3.7 ft^3/s (October 20, 1976), respectively.[4]

There are no major surface water bodies located within 10 miles of the WIPP facility. Beyond 10 miles, several water bodies lie to the north including Laguna Gatuna, Laguna Tonto, Laguna Plata, and Laguna

2.4.2 PRECIPITATION IMPACTS

2.4.2.1 Local Precipitation Patterns

The WIPP facility is located in the Chihuahuan Desert with an arid to semiarid climate. Precipitation in this area varies between localities and years, but averages roughly 12 inches per year. Mean annual precipitation values at various locations in the region around the WIPP facility are given in Table 2.4-9. Precipitation amounts have also been recorded at the WIPP facility during the periods of May 1976 to February 1980, and June 1985 to February 1987. These data are noted in Table 2.4-10, which indicates that four complete years of precipitation data have been obtained at the WIPP facility. Using 12 inches of precipitation per year as an average, 1977 appeared to be a slightly dry year (9.3 inches), 1979 a normal year (11.8 inches), and 1978 and 1986 very wet years (19.4 and 23.0 inches, respectively).

The probable maximum precipitation (PMP) quantities during "all season" and "winter" for different durations within a 10 miles^2 drainage area in the vicinity of the WIPP facility are listed in Table 2.4-11. The corresponding intensities are shown in Figure 2.4-2. The PMP determinations have been used to design the drainage systems and roof loads for the surface structures at the WIPP facility (discussed in Sections 2.4.2.2 and 2.4.2.3, respectively).

2.4.2.2 Drainage Patterns

In the proximity of the WIPP facility, subsurface geologic formations are covered with smoothly rounded hills of dune sand. Surface drainage patterns in the area are not well defined, and rain that collects in pools between the sand dunes is lost through infiltration into the sand and evapotranspiration. Vegetation generally consists of mesquite, shinnery oak and other plants commonly found in the northern Chihuahuan Desert and southern Great Plains. Land is used primarily for cattle grazing; however, potash mining and petroleum extraction also occur in the area.

The general ground slope in the vicinity of the WIPP facility is about 50 ft/mi. downward from the east toward the west. The average ground slope from north to south is downward about 13 ft/mi. A topographic and surface water divide (Antelope ridge) exists about 10 miles east of the WIPP facility. The activities associated with operation of the WIPP facility are not expected to alter the existing drainage pattern of the area around the WIPP facility.

The local drainage pattern is such that normal surface runoff from contributing areas north, south, and west of the surface structures drains westward into Nash Draw without affecting the WIPP facility structures. Storm water runoff from areas around the surface structures, including that from the east, are diverted away from the structures by a system of peripheral interceptor diversions. As shown in Figure 2.4-3, this drainage system is designed so that storm runoff due to a PMP event on the contributing drainage areas does not flood the plant. In addition, the grade elevations of most roads, tracks, and surface facilities are designed so that storm water will drain away under the most severe conditions. For example, the floors of the Waste Handling Building (WHB) and other surface structures are 0.5 feet above the grade elevation.

2.4.3 PROBABLE MAXIMUM FLOOD (PMF) NEAR THE WIPP FACILITY

2.4.3.1 Flood History

Floods on the Pecos River can be generated by: (1) frontal storms that produce large runoff volumes and high flood peaks; (2) thunderstorms (convective storms) that produce small runoff volumes but very high peaks; and (3) snowmelt in the mountainous portion of the drainage area that produces large runoff volumes, but relatively low peaks.

Between the confluence of the Rio Hondo with the Pecos River (river mile 566) and river mile 484 (location of the former McMillan Reservior, now part of Lake Brantley), the channel capacity of the Pecos River, without any overbank flooding, is about 8,500 ft^3/s. In the reach between river mile 484 and the confluence of Dark Canyon near Carlsbad (river mile 459), the channel capacity is estimated to be 50,000 ft^3/s. From the confluence of Dark Canyon to Red Bluff reservoir (river mile 411), the channel capacity is approximately 20,000 ft^3/s.[2]

Large floods are reported to have occurred on the Pecos River in 1893, 1904, 1905, 1915, 1916, 1919, 1937, 1941, and 1966.[3,4,14] No data are available for the 1893 event. The earliest flood for which discharge information is available occurred on October 2, 1904 following the failure of Avalon Dam. This flood was caused by a storm that lasted five days and covered the upper catchments of the Pecos River, Rio Hondo, Rio Felix, and Rio Penasco. The runoff generated by that storm caused floods throughout the Pecos River basin, and during the flood, the peak flow at Avalon gaging station (river mile 466) probably exceeded 90,000 ft^3/s. The corresponding river stage is not available. The river stage at Red Bluff during this flood (river mile 411) is reported to have reached 28 feet (gage datum = 2,850.05 feet).

Based on the records of the gaging station at Carlsbad (river mile 459), there was another major flood on July 25, 1905, when the peak flow of the Pecos reached 54,900 ft^3/s. The fourth and fifth major floods of the Pecos were on April 17, 1915 and August 7, 1916, when the peak flows at the Carlsbad gaging station reached 80,000 and 85,700 ft^3/s, respectively, with regulation by the McMillan and rebuilt Avalon reservoirs. Another flood occurred in September 1919, when the river stage at the Malaga gaging station, approximately 27 miles downstream from Carlsbad (river mile 432), was recorded at 29.4 feet (gage datum = 2,895.64 feet). The corresponding discharge was 40,400 ft^3/s.

Another major flood on the Pecos River is reported to have occurred on May 30, 1937, when the peak flow at Artesia (river mile 504) and Lake Arthur (river mile 522) was 51,500 ft^3/s. The corresponding stage at Lake Arthur was 21.77 feet (gage datum = 3,327.07 feet). The area in the vicinity of Carlsbad was flooded again on May 22, 1941, when the peak flow at Dam site three (river mile 474) reached 60,000 ft^3/s with regulation by Alamogordo (Lake Sumner), McMillan, and Avalon reservoirs. This flood resulted from intense rainfall over the drainage area of the Pecos River upstream of Red Bluff reservoir. It produced a stage of 35.1 feet at Malaga.

The highest flood of record (through 1986) on the Pecos River occurred on August 23, 1966, when the discharge and stage at Malaga were 120,000 ft^3/s and 42.1 feet, respectively, with regulation by the Alamogordo, McMillan, and Avalon reservoirs. The water surface profile of the Pecos River in Carlsbad during this flood is shown in Figure 2.4-4.

The flood frequency curve of the Pecos River at Malaga is shown in Figure 2.4-5. An additional curve is included in this figure to show the attenuation at Malaga expected to be provided by the Brantley reservoir.

the Pecos River near Malaga are shown in Figure 2.4-7. The water surface elevations of the historic flood of August 23, 1966 are marked on these cross sections, which were used by the USGS to estimate the peak flow of 120,000 ft^3/s that passed down the Pecos on August 23, 1966.

To compute the conveyances of the cross sections shown in Figure 2.4-7, the USGS divided each of these cross sections into four subsections. The values of Manning's n used for each subsection are shown in Table 2.4-13. Based on a comparison of the photographs of the Pecos riverbed with those available in the literature[16] for known values of n, the USGS values of Table 2.4-13 are considered reasonable.

The average water surface slope of the Pecos River in Carlsbad during the flood of August 23, 1966 was 0.00095 (see Figure 2.4-4). The average water surface slope, based on high-water marks near Malaga recorded by the USGS for the same flood, is 0.00123. The energy slopes used by the USGS for its estimates of the flood discharge of August 23, 1966, for two consecutive reaches of 752 feet and 645 feet with beginning and ending cross section shown in Figure 2.4-7, are 0.00113 and 0.00161, respectively. In view of these values, it is assumed that the average energy slope during the PMF on the Pecos River would be about 0.001, i.e., 5.28 ft/mi. For the three cross sections of the Pecos River near Malaga (Figure 2.4-7), weighted average values of Manning's n are computed using the equation:

$$n = \frac{AR^{2/3}}{\sum_{i=1}^{m} \frac{A_i R_i^{2/3}}{n_i}}$$

where

A = total cross sectional area
R = hydraulic radius of cross section, in feet
m = number of subsections in which a cross section is divided
A_i = Area of the ith subsection
n_i = Manning's roughness coefficient applicable to the ith subsection
R_i = hydraulic radius of ith subsection, in feet

Using the average of the weighted roughness coefficients computed previously (Table 2.4-13) and an average energy slope of 0.001, the water surface elevation corresponding to the PMF peak flow of 1,350,000 ft^3/s near the WIPP facility is estimated to be 2,980 feet MSL.

2.4.3.5 Flooding Protection Requirement

As calculated in the previous section, a very conservative estimate of the highest water surface elevation in the Pecos River during a PMF is 2,980 feet MSL. This is about 420 feet below the floors of the surface structures at the WIPP facility, which have an elevation of approximately 3,400 feet MSL. The horizontal and vertical separations between the Pecos River floodplain and the WIPP facility are great enough that floods in the Pecos River cannot affect the surface structures at the WIPP facility. Accordingly, there are no flood protection requirements for the WIPP facility. In addition, as stated in Section 2.4.2.2, the WIPP facility drainage structures will be designed so that ponding during a PMP event does not affect the surface structures.

THIS PAGE LEFT BLANK INTENTIONALLY