2\16

SANDIA REPORT

SAND95–1675 • UC–814
Unlimited Release
Printed November 1995

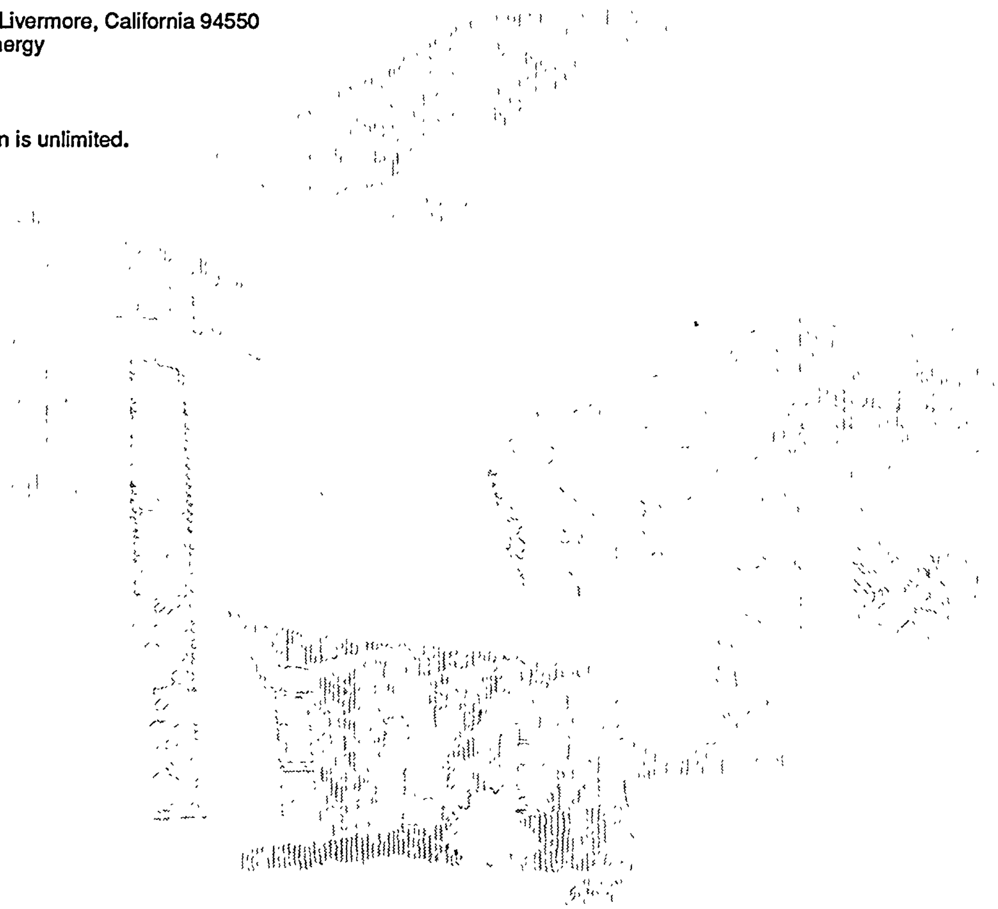
Yucca Mountain Site Characterization Project

Evaluation of Geotechnical Monitoring Data From the ESF North Ramp Starter Tunnel April 1994 to June 1995

YMP Performance Assessment, Applications Department 6313
YMP Las Vegas Operations, Department 6314
Agapito Associates, Inc.

Prepared by
Sandia National Laboratories
Albuquerque, New Mexico 87185 and Livermore, California 94550
for the United States Department of Energy
under Contract DE-AC04-94AL85000

Approved for public release; distribution is unlimited.



"Prepared by Yucca Mountain Site Characterization Project (YMSCP) participants as part of the Civilian Radioactive Waste Management Program (CRWM). The YMSCP is managed by the Yucca Mountain Project Office of the U.S. Department of Energy, DOE Field Office, Nevada (DOE/NV). YMSCP work is sponsored by the Office of Geologic Repositories (OGR) of the DOE Office of Civilian Radioactive Waste Management (OCRWM)."

Issued by Sandia National Laboratories, operated for the United States Department of Energy by Sandia Corporation.

Printed in the United States of America. This report has been reproduced directly from the best available copy.

Available to DOE and DOE contractors from
Office of Scientific and Technical Information
PO Box 62
Oak Ridge, TN 37831

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

Available to the public from
National Technical Information Service
US Department of Commerce
5285 Port Royal Rd
Springfield, VA 22161

NTIS price codes
Printed copy: A08
Microfiche copy: A01

SAND95-1675
Unlimited Release
Printed November 1995

Distribution Category
UC-814

EVALUATION OF GEOTECHNICAL MONITORING DATA FROM THE ESF NORTH RAMP STARTER TUNNEL APRIL 1994 TO JUNE 1995

YMP Performance Assessment
Applications Department 6313
Sandia National Laboratories
Albuquerque, NM 87185

YMP Las Vegas Operations
Department 6314
Sandia National Laboratories
Las Vegas, NV 89109

Agapito Associates, Inc.
Las Vegas, NV 89102

ABSTRACT

Geotechnical monitoring data were developed during excavation of the North Ramp Starter Tunnel (NRST) and Alcove No. 1 to provide the basis for design verification. The NRST was constructed to launch the 7.6-m diameter tunnel boring machine being used to construct the Exploratory Studies Facility main access tunnels, the North Ramp, Main Drift, and South Ramp. Alcove No. 1 was excavated off the NRST to provide access for site characterization testing.

Rock mass quality data and blasting seismic data were collected during construction. Monitoring instrumentation consisted of rockbolt load cells, convergence points, and multi-point extensometers. Instrumentation stations were developed after the installation of final ground support. Rockbolt load and rock deformation data were used to assess stability of the excavation. Convergence data and rockbolt load were used to evaluate the ground support performance. Rock mass quality assessments were used to compare installed ground support to empirical ground support designs.

CONTENTS

Page

Figures

Tables

1.0 Introduction

1.1 Purpose

This report presents the results of instrumentation measurements and observations made during construction of the North Ramp Starter Tunnel (NRST) of the Exploratory Studies Facility (ESF). The information in this report was developed as part of the Design Verification Study, Section 8.3.1.15.1.8 of the Yucca Mountain Site Characterization Plan (DOE 1988). The ESF is being constructed by the U.S. Department of Energy (DOE) to evaluate the feasibility of locating a potential high-level nuclear waste repository on lands within and adjacent to the Nevada Test Site (NTS), Nye County, Nevada. The Design Verification Studies are performed to collect information during construction of the ESF that will be useful for design and construction of the potential repository. Four experiments make up the Design Verification Study: Evaluation of Mining Methods, Monitoring Drift Stability, Monitoring of Ground Support Systems, and The Air Quality and Ventilation Experiment. This report describes Sandia National Laboratories' (SNL) efforts in the first three of these experiments in the NRST.

1.2 Background

The NRST was constructed to launch the 7.6-m tunnel boring machine (TBM) being utilized to excavate the North Ramp of the ESF. The North Ramp is one of two inclined tunnels currently planned to provide access to the potential repository horizon. The North Ramp will be excavated[1] 2,804 m (9,200 ft) to the potential repository horizon. Beyond this point, the

[1]ESF Layout Calculation, BABEAD000-01717-0200-00003, Revision 2.

Topopah Spring Level (TSL) Main Drift will be excavated 3,131 m (10,273 ft) across the proposed repository block by the TBM. The South Ramp will then be excavated 1,921 m (6,302 ft) to connect to the surface. Figure 1-1 shows a map of the locations of the planned ESF access tunnels, and the location of the North Portal.

The NRST was excavated by drill-and-blast techniques in the east face of Exile Hill, a horst-like structure between the Midway Valley and Bow Ridge Faults, in Miocene volcanic tuff rocks of the Paintbrush Group, Tiva Canyon Member, upper lithophysal zone. A geologic cross section through Exile Hill showing the location of the NRST is presented in Figure 1-2.

The NRST excavation consisted of three parts:

- a box cut to form safety benches and to allow safe construction of the tunnel brow,
- a finished size of approximately 10-m (32.8-ft) high by 10-m (32.8-ft) wide tunnel with arched roof excavated to 0+60.2 m, and
- an alcove (No. 1) driven at an angle of 82° to the tunnel to provide access for testing/ characterization.

All parts of the NRST were excavated in generally poor rock mass conditions that resulted from the presence of cavernous lithophysae, minor fault and shear structures, vertical jointing, and weathering effects due to surface proximity. The construction took place in unsaturated, highly fractured rocks and no water flow of construction significance was encountered.

Ground support requirements were relatively heavy due to the poor rock quality. Temporary ground support was provided by split set rockbolts and wire mesh. The permanent support in the tunnel brow consisted of steel lattice girders embedded in fibercrete. Beyond the brow area, final support consisted of 3-m, 4.9-m and 7.3-m (10-ft, 16-ft and 24-ft) long, fully grouted, untensioned rockbolts with a nominal 15.2 cm (6 in) of fibercrete.

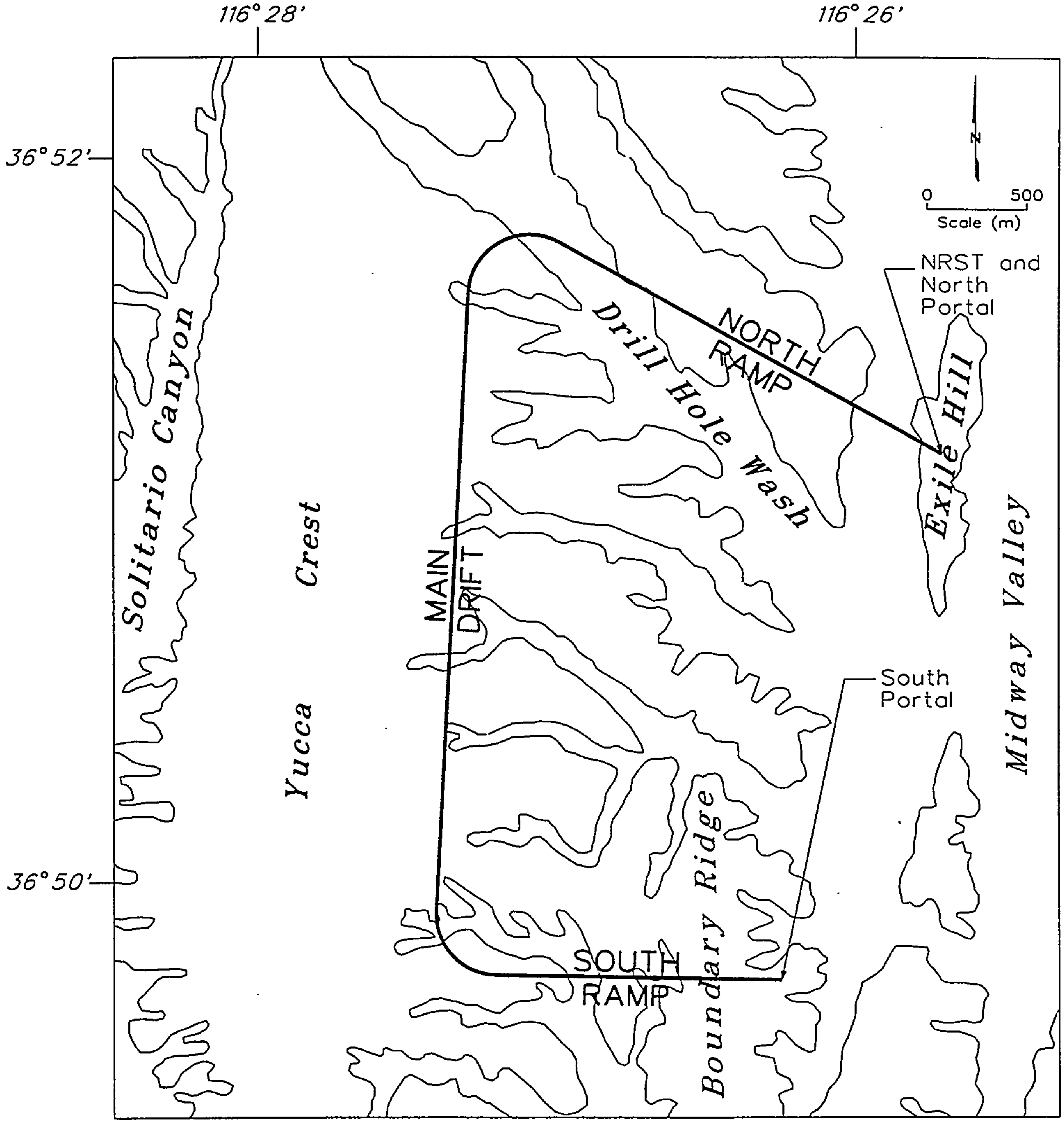


Figure 1-1. Plan map showing the ESF access tunnels.

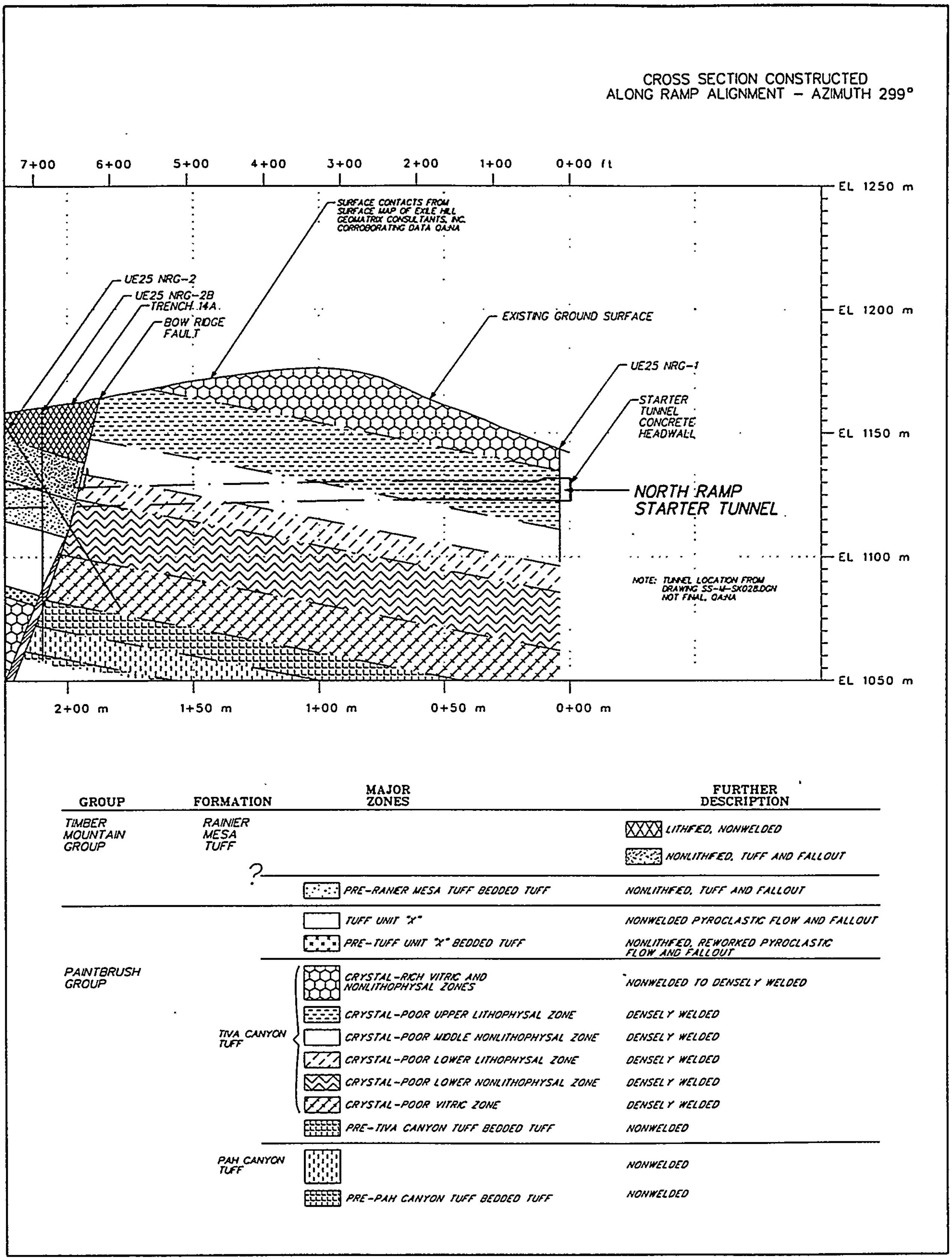


Figure 1-2. Cross section through Exile Hill showing the North Ramp Starter Tunnel.

1.3 Scope

This report presents the results of design verification (DV) studies conducted during construction of the NRST between North Ramp stations 0+00 and 0+60 m. Data were collected in three of the four design verification experiments:

- evaluation of mining methods,
- evaluation of ground support systems, and
- monitoring drift stability.

The overall objectives of these design verification experiments are to monitor and observe the long-term behavior of openings in the range of rock conditions to be encountered in the potential repository host rock, to observe and evaluate the construction of the ESF with respect to implications for repository construction and performance, and to collect information for design of the ventilation systems in the repository underground facility. The ventilation studies were not implemented in the NRST monitoring.

Table 1-1 lists more specific objectives and activities within each of the design verification experiments implemented in the NRST. In Table 1-2, the study activities are correlated with specific investigations that were performed in the NRST and the specific data to be collected.

Two activities, blast seismic monitoring and rock quality evaluation, were conducted in parallel with the excavation process. These data were developed for the upper heading and bench excavations in the tunnel and for the alcove. All other investigations were conducted as the final support was installed, hence, the influence of the excavation process on rock movement was not monitored.

Table 1-1. In Situ Design Verification Study Plan Experiments (8.3.1.15.1.8) Objectives and Activities

SCP Activity No.	Study Plan Experiment	Objectives	Activities
8.3.1.15.1.8.1	Evaluation of mining methods	Record methods and equipment used, describe resulting excavations in terms of conformance to specifications and extent of observable excavation effects, correlate with rock mass quality Q and RMR.	• Rock mass quality evaluation • Monitoring blast vibrations • Evaluate as-built mapping data • Collect construction records on blasting rounds
8.3.1.15.1.8.2	Evaluation of ground support system	Record installed ground support, correlate with rock mass quality, measure load-deformation response of some ground support components, monitor in situ loads on some supports, evaluate performance in terms of deformations and usability of excavations.	• Collect ground support records • Monitor rockbolt loads • Monitor lattice girder convergence • Observe shotcrete cracks
8.3.1.15.1.8.4	Monitor drift stability	Measure cross-drift convergence throughout the ESF and at several intersections, monitor rock movement with borehole extensometers, record rock falls and required maintenance.	• Measure cross-drift convergence • Measure intersection convergence • Monitor rock displacements • Observe rock falls

1.4 Quality Assurance

Information and data presented in this report were developed and documented under a fully qualified Quality Assurance (QA) program. The information was documented in scientific notebooks in accordance with SNL's Quality Assurance Implementing Procedure (QAIP) 20-2. Traceability of the information and data is outlined in Appendix A.

1.5 Report Organization

This report is organized into six sections and appendices. Following this introduction, Section 2.0 describes the NRST construction, the layout, and location of the construction monitoring instruments and the specific activities. Section 3.0 presents summaries of the

Table 1-2. Specific Investigations Conducted in the North Ramp Starter Tunnel

Study Plan Experiment	Study Plan Activity	Specific Investigation	Objective
Evaluate mining methods	Rock mass quality	Scanline mapping	• Define range of rock mass quality using Q and RMR indices
	Blast monitoring	Far-field	• Measure peak particle velocity (PPV), peak frequency in the far-field, develop scaled distance relationship with blasting charge weight
		Near-field	• Measure peak particle velocity close to blast round, correlate PPV with blast damage to rock fabric
	Evaluate as-built mapping data	Obtain USBR detailed mapping data	• Provide rock structural data to correlate with instrumentation and other measurements
	Collect construction records	Obtain constructor records on blasting	• Provide data to correlate with blast monitoring
Evaluate ground support systems	Collect ground support records	Obtain constructor records on support design, installation	• Provide data to correlate with support performance monitoring
		Correlate support with rock mass quality	• Verify adequacy of design approach
	Monitor ground support	Install rockbolt load cells	• Verify performance of rockbolts
		Install lattice girder convergence pins	• Verify lattice girder performance and loading
		Inspect fibercrete cracking	• Verify fibercrete performance
Monitor drift stability	Cross drift convergence	Install convergence pins	• Measure drift closure to verify stability
	Intersection convergence	Install convergence pins and extensometers at tunnel/ alcove intersection	• Verify stability of intersection
	Rock displacements	Install multi-point extensometers	• Verify rock mass stability

construction monitoring data, with the resulting assessments of design/performance verification discussed in Section 4.0. Summary and conclusions are presented in Section 5.0. References are presented in Section 6.0.

Appendices present some specific data from this work and other information that support the results.

2.0 NRST Layout and Configuration of Monitoring Instrumentation

2.1 Introduction

This section of the report describes the construction of the NRST and the associated monitoring and site characterization activities. Excavation of the NRST included three separate elements: the box cut, the starter tunnel, and Alcove No. 1. Monitoring of the construction consisted of geotechnical observations of the exposed rock conditions, measurements of blasting seismic response, and installation of long-term monitoring instruments in the rock mass and on selected ground support components. Detailed structural mapping of the NRST was performed by the U.S. Bureau of Reclamation[1] (USBR) in an associated investigation.

2.2 Layout and Configuration

Construction of the NRST consisted of the three elements of box cut, tunnel, and Alcove No. 1. A plan map showing the layout and configuration of the NRST with respect to the surface topography and surface facilities is presented in Figure 2-1.

2.2.1 Box Cut

The box cut was excavated to remove loose unstable material to allow development of the NRST portal. The tuff rocks of the Tiva Canyon upper lithophysal zone were excavated by mechanical ripping to form a portal face 21.3 m (70 ft) high by 41.1 m (135 ft) wide. The north

[1]Characterization of Structural Features in the Site Area, Study Plan 8.3.1.4.2.2.

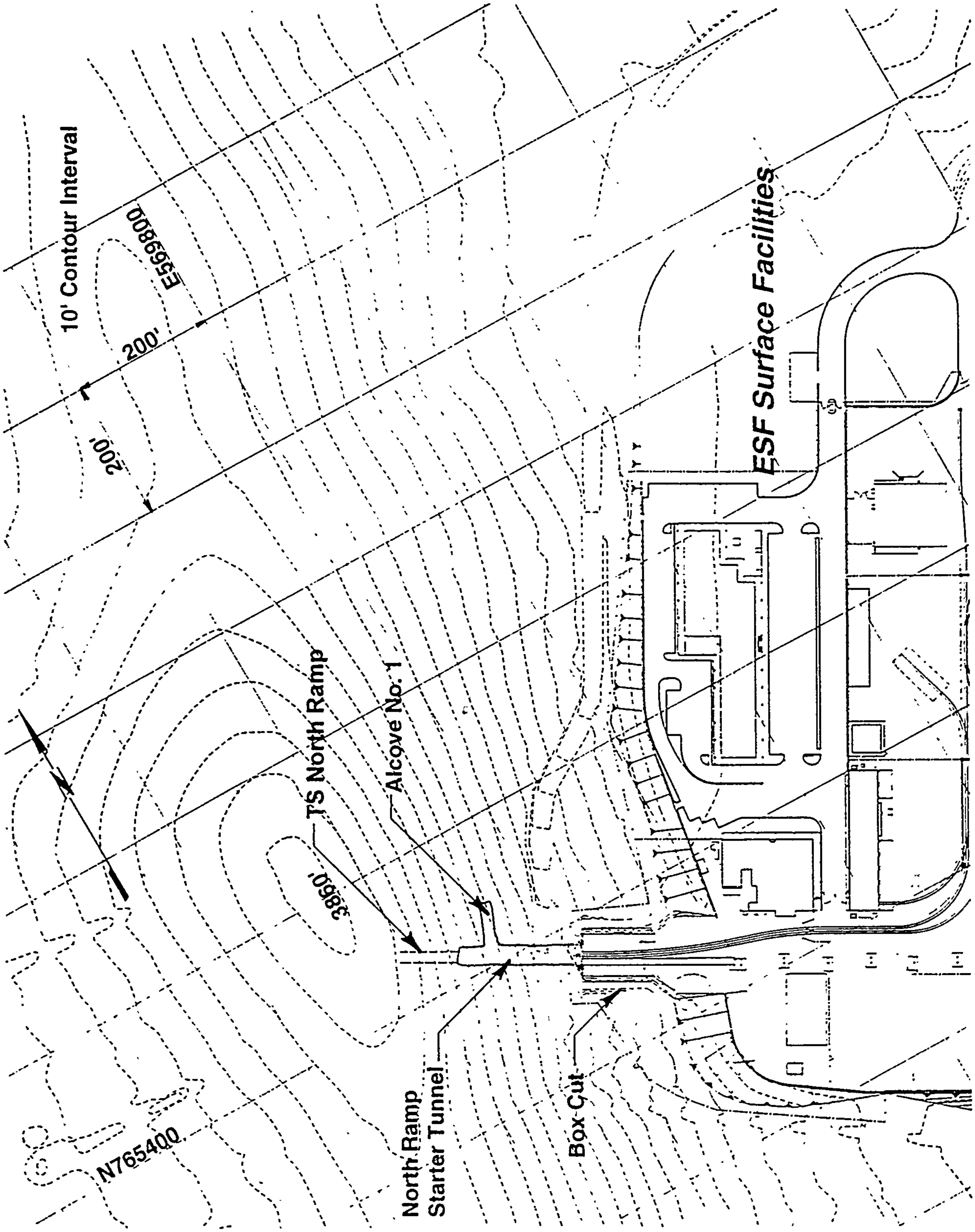


Figure 2-1. Plan drawing of the ESF surface facilities and NRST.

and south walls were sloped by the formation of two benches and covered by 9-gage chain-link mesh secured by split set rockbolts. The portal face was later bolted using 6.1-m (20-ft) long hollow rebar rockbolts which were grouted in place with cementitious grout The portal face was then fibercreted with a minimum thickness of 10.2 cm (4 in) which covered all mesh and bolts.

2.2.2 Tunnel

The tunnel was excavated nominally 9.8 m (32.3 ft) high by 9.9 m (32.5 ft) wide in an arched shape as shown in Figure 2-2. It was constructed in two passes consisting of a top heading and bench. Excavation was by conventional mining using drill-and-blast techniques. Loose rock was scaled and temporary rock support consisting of split set rockbolts with 100-mm by 100-mm (4-in by 4-in) welded wire mesh was installed after each blast round. Final support of fully grouted bolts and fibercrete was installed incrementally after completion of each phase of the excavation.

2.2.2.1 Top Heading. The top heading was accessed from an earth ramp built within the box cut. Figure 2-2 presents a cross section illustrating the nominal tunnel dimensions and shape. The top heading was excavated in two passes, a pilot drive followed by slashing of the north and south ribs. The nominal height was 5.2 m (17 ft) with a 9.9-m (32.5-ft) width. The earth ramp was then removed from the box cut to allow access to the bench.

2.2.2.2 Bench. The bench was excavated by full-face horizontal drilling and blasting. The center bench cut was excavated one round ahead of the north and south cuts; however, after the initial rounds, the slashes were completed and subsequently all three cuts were blasted at the same time. Initial ground support consisting of split sets and welded wire mesh were installed in the ribs.

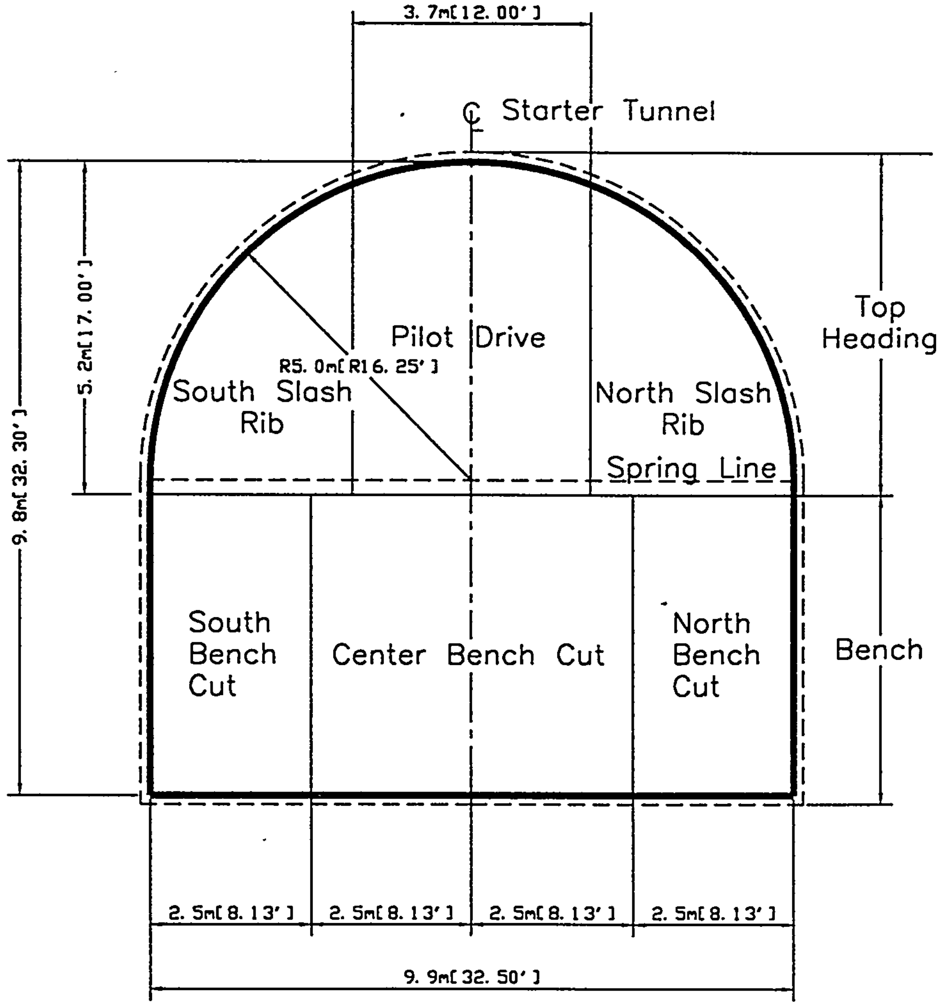


Figure 2-2. Cross section of starter tunnel illustrating top headings and bench configuration (from ESF Package 1A, YMP-025-1-MING-MG135, Rev 3).

2.2.2.3 Ground Support. The final ground support system was installed following removal of the bench to full length of the NRST. This support consisted of untensioned grouted rockbolts between 3 m (10 ft) and 7.3 m (24 ft) long on 1.2 m (4 ft) centers, 152-mm by 152-mm (6-in by 6-in) welded wire mesh, and a nominal thickness of 15.2 cm (6 in) of fibercrete. Steel lattice girders were installed near the portal and were completely embedded in fibercrete. Some additional fully grouted bolts were also installed through the fibercrete.

2.2.3 Alcove No. 1

Alcove No. 1 was excavated in the direction N21°E at station 0+42.7 m (1+40 ft) as shown in plan view in Figure 2-1. The alcove had an arched cross section with nominal height of 5.2 m (17 ft) and a width of 5.8 m (19.0 ft) and was 22.9 m (75 ft) long. The alcove was excavated by conventional drill and blast using full-face blast rounds.

2.3 Construction Monitoring Activities

Construction monitoring activities were conducted in two phases. In the first phase, rock mass quality data, blasting seismic data, and temporary convergence data were collected during construction activities. After completion of the tunnel and alcove, final instrumentation in the form of convergence points, rockbolt load cells, instrumented rockbolts, extensometers, and stress change gages were installed and connected into a central data logging system. This section of the report describes each of these activities and the specific instrumentation installed.

2.3.1 Rock Mass Quality Assessments

Rock mass quality assessments were made incrementally during the excavation of the top heading, bench, and Alcove No. 1. The rock mass quality indices were developed using the Q (Barton et al. 1974) and the RMR (Bieniawski 1979) systems. Individual parameters were subjectively assessed based on descriptive tables published for each system, with the exception of RQD which was assessed using a correlation between fracture frequency and RQD reported by Hudson and Priest (1979). Calculations and parameters are described in Table 2-1.

Table 2-1. Rock Mass Quality Indices Employed in NRST

NGI – Q		RMR	
$Q = \frac{RQD}{J_N} \bullet \frac{J_R}{J_A} \bullet \frac{J_W}{SRF}$		$RMR = C + I_{RQD} + JS + JC + Jw + AJ$	
where		where	
RQD	$= 100e^{-0.1\lambda} + (0.1\lambda + 1.0)$	C	= rock strength rating
λ	= average fracture frequency (m^{-1})	I_{RQD}	= RQD rating
J_N	= joint set number	JS	= joint spacing parameter
J_R	= joint roughness number	JC	= joint characteristics parameter
J_A	= joint alteration number	Jw	= joint water parameter
J_W	= joint water reduction factor	AJO	= joint orientation adjustment
SRF	= stress reduction factor		

Data were developed for different increments of the tunnel length during excavation, depending upon access for description. The RQD estimate was generated for three lines: one horizontal on the wall, one vertical on the wall, and one horizontal across the tunnel or bench face. Fracture frequency was determined by counting fractures along the scanline, with sheared or rubblized zones accounted for by length-weighted averaging using RQD = 0% for the rubblized zone. Lithophysal cavities were considered by counting them as one fracture if the diameter was less than approximately 76.2 mm (3 in) or as two fractures if the diameter was greater than approximately 76.2 mm (3 in). RQD was averaged for the three lines, the average value used as the upper value of the range, and the minimum of the three lines was used as the low value of the range.

Visual observations of the characteristics of the rock were used to estimate the range of conditions existing in the interval being examined for each of the other parameters in Table 2-1. The upper and lower values were then used to estimate a range of Q for each interval of the upper bench and both Q and RMR for the lower bench and Alcove No. 1. The Q system can be used to assess the ground support requirements by using design charts. Appendix F provides background on the design charts and ground support categories.

2.3.2 Blasting Seismic Monitoring

Blasting seismic monitoring was performed to measure peak particle velocity (PPV) in the far-field (>3 m, 10 ft) for the top heading, bench, and Alcove No. 1 blasting, and in the near-field (<3 m, 10 ft) for the initial Alcove No. 1 blasts in an attempt to correlate blast-induced damage with PPV. Each of these activities is discussed separately in the following sections.

2.3.2.1 Far-Field Monitoring. Far-field monitoring was conducted in two separate phases for each of the top heading and bench mining activities. The monitoring was performed using blasting seismograph equipment manufactured by Vibratech Engineers, Inc. of Pittsburgh, Pennsylvania. Two instrument types were utilized, an Everlert 6000 and Everlert II. These systems had similar configurations and specifications and were equipped with one three-axis geophone and one blast air-pressure monitor. Resolutions of the particle velocity were 0.099 mm/sec (0.0039 in/sec) for the Everlert 6000 and 0.1239 mm/sec (0.00488 in/sec) for the Everlert II over the frequency ranges of 2–200 Hz and 2–250 Hz, respectively. Accuracy was specified as ±3 Db over the frequency range and the dynamic range of the instruments was 0–203.2 mm/sec (0–8 in/sec) and 0–254.0 mm/sec (0–10 in/sec), respectively.

Both instruments were event triggered and capable of storing multiple events. Records of individual events were printed intermittently and correlated with individual blasts by using constructor records. Figure 2-3 shows an example of an individual record from the Everlert II. Each record contains the PPV, peak frequency, peak vector sum, the wave form of each of the three transducers, and calibration checks for each transducer. Blast air pressures were not monitored.

The two blasting seismographs were placed at different distances from the face in protected locations. Geophones were coupled directly to rock surfaces by placing sand bags on top of the instrument cases. Five different geophone locations were used to monitor the top heading blasts, and an additional two locations were used for the bench blasts. The coordinates are listed in Table 2-2 and illustrated in Figure 2-4. Geophone stations 1 and 2 were utilized for the initial blasting to form the top heading. After the top heading had advanced several rounds, the blast monitors were moved to geophone stations 3 and 4 for the majority of the top heading blasts. Station 5 was only used for two rounds, and then the geophones were returned to station 3. The box cut ramp was then removed and stations 6 and 7 were established to monitor the bench blasts.

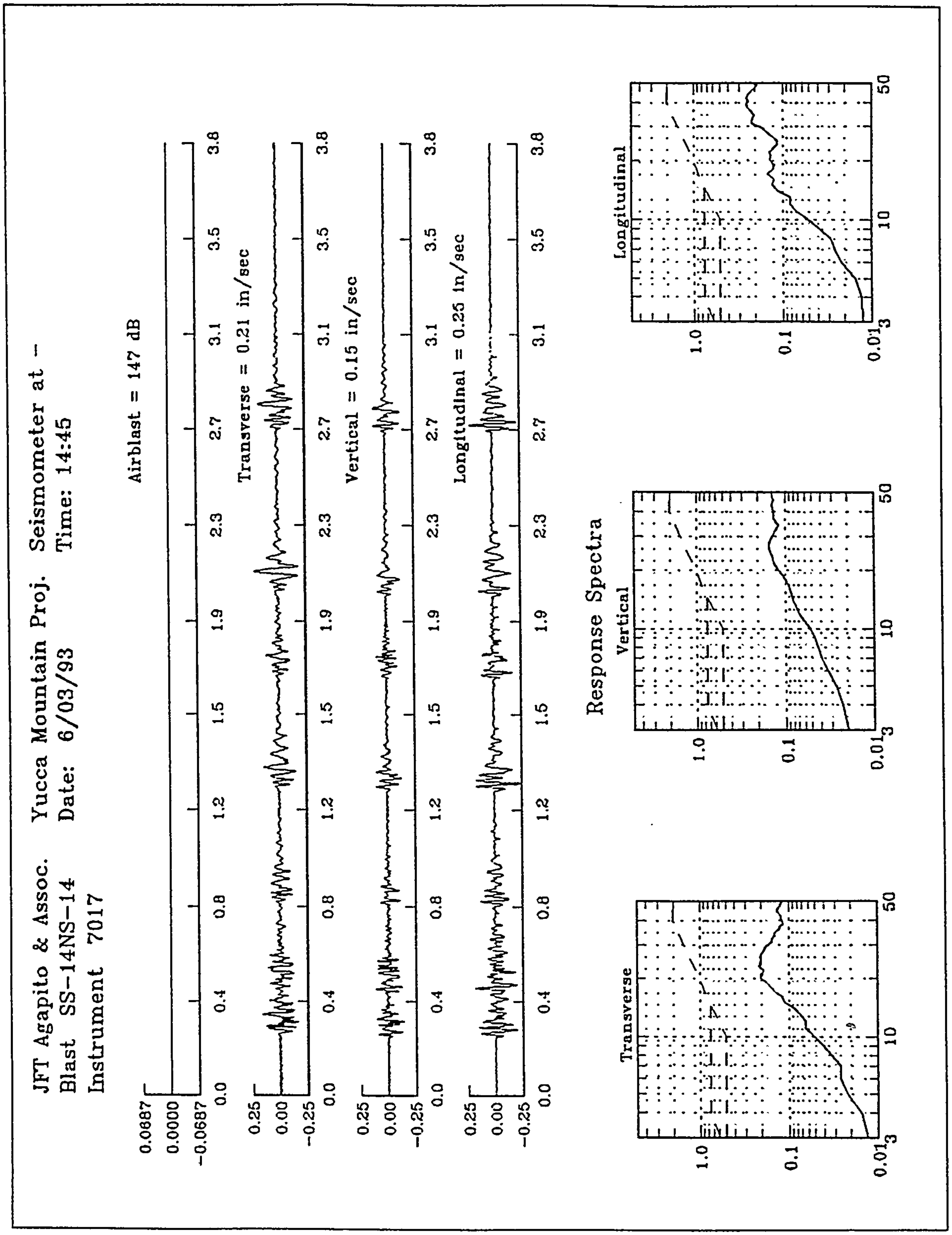


Figure 2-3. Example of Everlert II blasting seismic records.

Table 2-2. Locations of Blasting Seismographs

Geophone Station	North		East	
	(m)	(ft)	(m)	(ft)
1	233,267.9	765,305.2	173,740.7	570,008.4
2	233,253.7	765,258.6	173,799.9	570,202.9
3	233,283.5	765,356.4	173,703.4	569,886.1
4	233,291.1	765,381.3	173,685.8	569,828.5
5	233,286.6	765,366.5	173,678.6	569,804.8
6*	233,290.3	765,378.9	173,686.0	569,828.9
7*	233,291.6	765,383.0	173,675.0	569,793.0

* Preliminary—the locations not utilized for any data in this report.

2.3.2.2 Near-Field Monitoring. Near-field blast monitoring was performed during excavation of Alcove No. 1 to measure PPV at small distances (1 and 2.4 m) from the excavation perimeter and to attempt to observe blasting related damage in the wall of the alcove. Figure 2-5 illustrates the layout of the monitoring holes that were drilled from the NRST parallel to the axis of Alcove No. 1.

Two holes each contained two geophones oriented to measure the compressional and vertical shear-wave vibrations. Vibration in the horizontal shear plane was not measured. The geophones, model GS-20DX-10Hz manufactured by OYO Geospace, were grouted 3 m deep in the monitoring holes located at 1 m and 2.4 m from the Alcove No. 1 perimeter at a height of 2.7 m above the floor. These geophones had a flat response over a frequency range of 15 to <500 Hz and they were coupled to a Hewlett-Packard HP-54112D digital oscilloscope. Recording was done at a rate of 5000 samples per second for the duration of the blast, approximately 12 seconds.

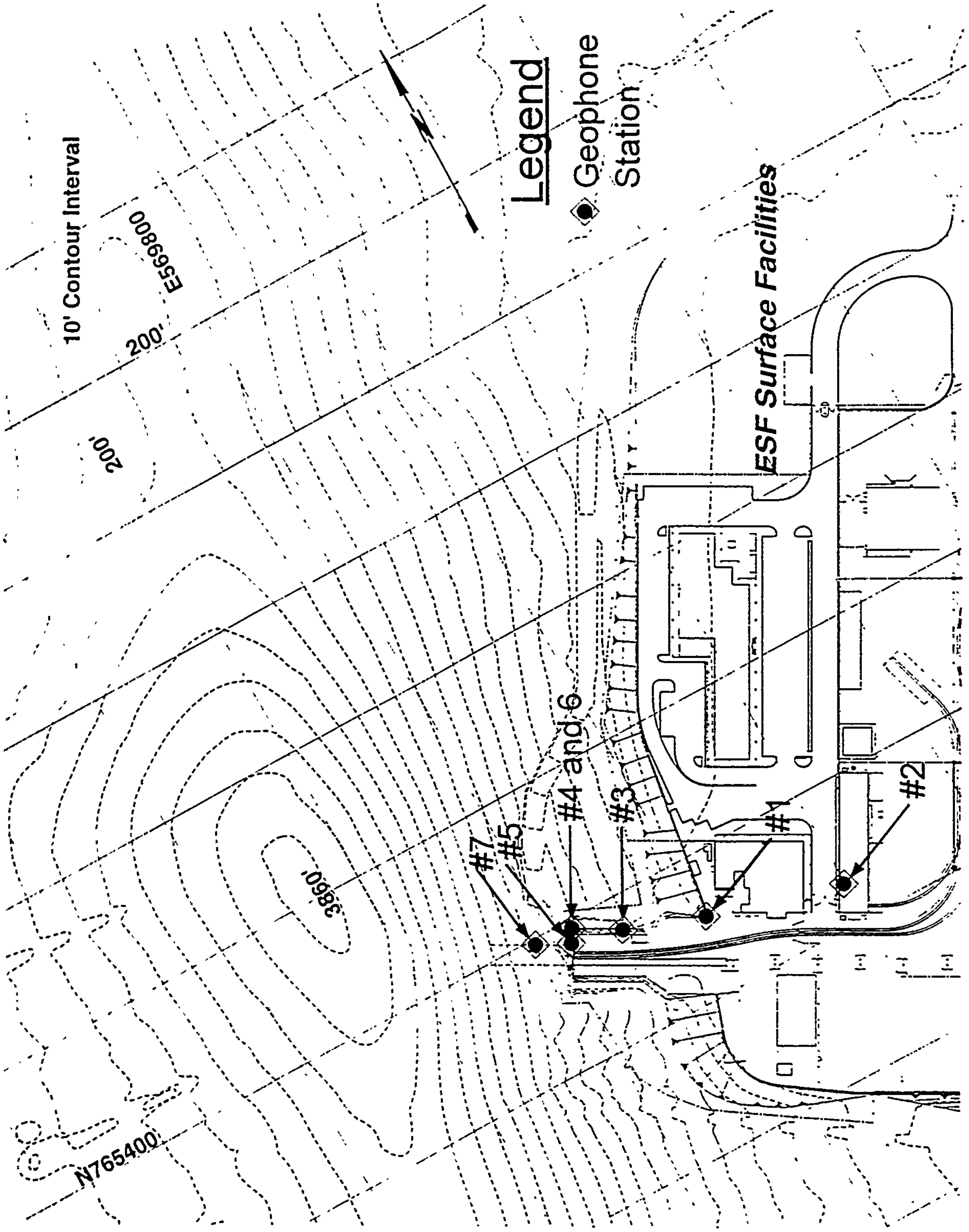
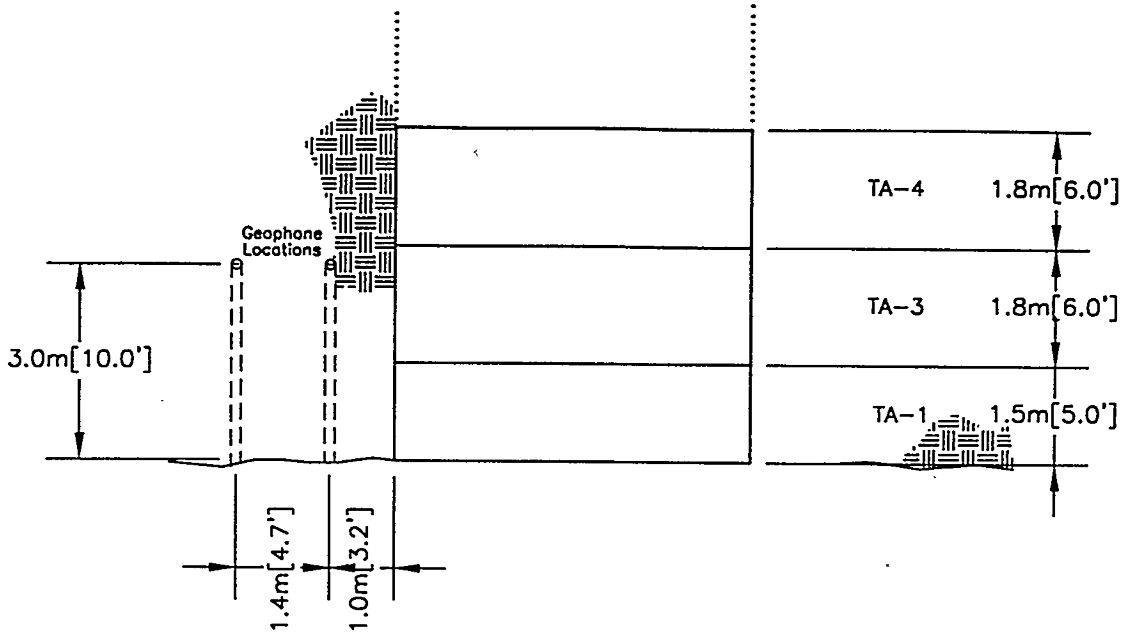
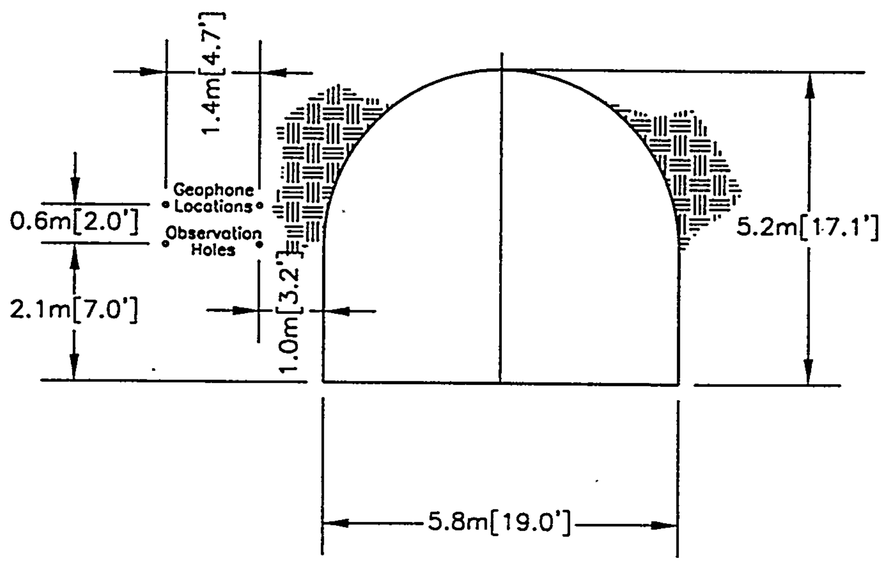


Figure 2-4. Locations of blasting seismographs for NRST blast monitoring.



Plan View



Cross Section

Figure 2-5. Layout of near-field blast monitoring instrument and damage inspection holes.

2.3.3 Ground Support Monitoring

The performance of three types of ground support were monitored:

- lattice girders,
- rockbolts, and
- fibercrete.

The specific monitoring methods and instrumentation are described in the following subsections.

2.3.3.1 Lattice Girders. Steel lattice girders were installed in the first 10 m (32.8 ft) of the NRST and then embedded in fibercrete with a minimum cover of 38 mm (1½ in). The lattice girders were 3-bar model CP70/6/10 manufactured by Commercial Pantex Sika, Inc. of Louisville, Kentucky. The seven lattice girders were installed in eight sections and secured to the rock by rockbolts. The lattice girders were set on wood blocking to provide a footing for the girders prior to their encapsulation in fibercrete.

Convergence point anchors were attached directly to girders No. 4 and No. 7 prior to application of the fibercrete. Figure 2-6 shows a plan drawing of the NRST portal with general locations of the lattice girder instrumentation. Girders No. 4 and No. 7 were located at ramp stations 0+4.6 m (0+15 ft) and 0+9.1 m (0+30 ft), respectively. Figure 2-7 shows a generalized cross section of the opening, the location of convergence points, the chords measured, and the nomenclature used to identify the chords. Appendix B contains a D-size as-built plan drawing of the NRST and Alcove No. 1 with the location of each gage. Appendix C contains a D-size drawing that shows a schematic of the instrument stations in the NRST.

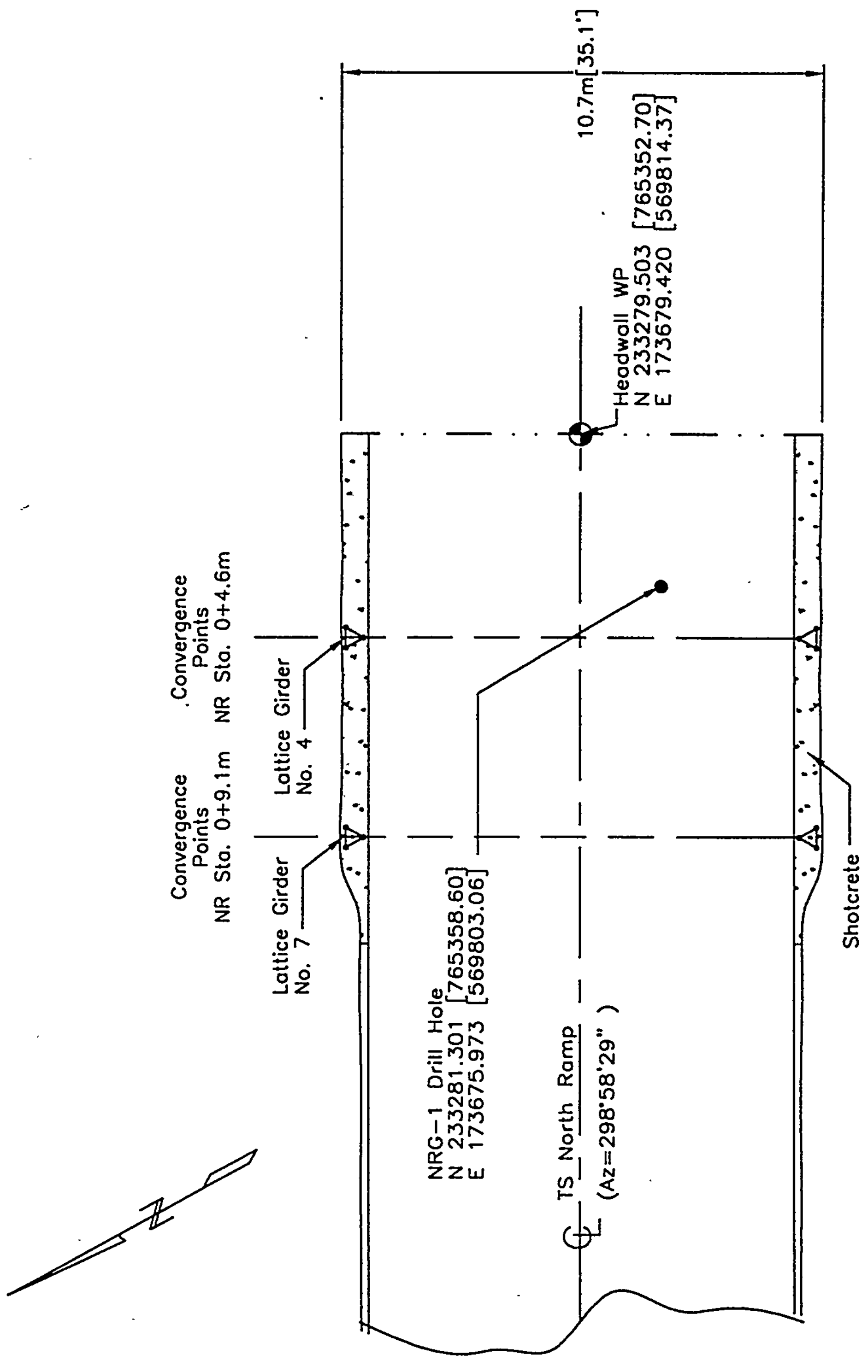


Figure 2-6. Plan drawing of NRST portal area showing location of lattice girders with convergence point stations 0+4.6 m and 0+9.1 m (from ESF Package 1A, YMP-025-1-MING-MG135, Rev 3).

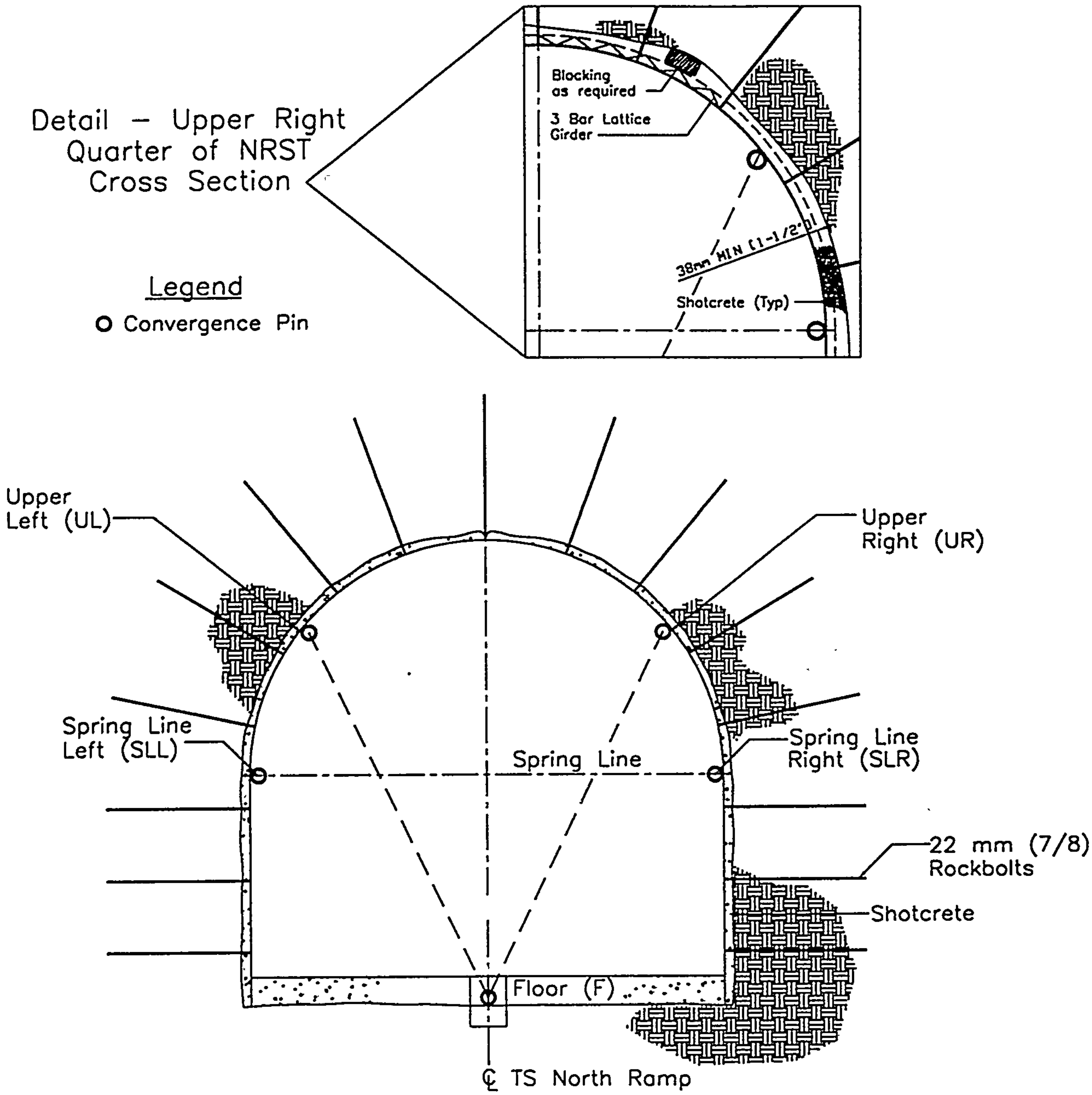


Figure 2-7. Generalized cross section of NRST convergence points installed on lattice girders, looking northwest (from ESF Package 1A, YMP-025-1-MING-MG152 Rev 1 and YMP-025-1-MING-MG153 Rev 1).

2.3.3.2 Rockbolt Load. Two types of instrumentation were installed to monitor rockbolt load:

- rockbolt load cells (RBLCs) and
- instrumented rockbolts (IRBs).

Arrays of the RBLCs were installed at six stations in the NRST and the IRBs at five locations in Alcove No. 1. The locations, number, and types of instruments are listed in Table 2-3.

The face above the portal (highwall station 0+00 m) was rock bolted using 6.1-m (20-ft) long hollow #9[2] rebar bolts with couplings and #7 by 3 m (10-ft) bolts. The bolts were drilled in using sacrificial bits, then grouted in place using cementitious grout. The bolts were untensioned. The locations of the rockbolts and rockbolt load cells on the portal face (highwall) are shown in Figure 2-8.

Rockbolt pattern in the main tunnel of the NRST went through several modifications. The final configuration consisted of fifteen bolts: nine 3–7.3-m (10–24-ft) long bolts above the spring line and six 3-m (10-ft) long bolts in the ribs. All bolts were untensioned grouted, using cementitious grout. The bolts above spring line were #9 (28.6 mm; 1.125 in) bolts and were pull tested to 249.1 kN (56,000 lbs). The rib bolts were #7 (22.2 mm; 8.75 in) diameter and were pull tested to 111.2 kN (25,000 lbs). The typical bolt pattern is shown in Figure 2-9.

The main tunnel bolts and portal face were instrumented with Geokon Model Nos. 4900-45-2 and 4900-40-1.5 vibrating wire rock bolt load cells (RBLCs) with capacities of 200.2 and 177.9 kN (45,000 and 40,000 lbs), respectively. The accuracy was specified at 0.5% of full-scale capacity. Each load cell contains three vibrating wire strain gages, the output

---

[2]#9 rebar refers to rebar with a diameter approximately 9/8 of an inch.

Table 2-3. Location and Type of Instumented Rockbolts

NRST Station* (m)	Alcove No. 1 Station† (m)	Construction Reference‡	Type of Bolt	Type of Instrument	Length†† (m)	Diameter†† (mm)	No. of Instruments	Bolts Pull Tested†† (kN)
0+00 m	N/A	Highwall 1, 9, 10	Grouted	RBLC	3, 6.1	22.2, 28.6	3	NT
0+16.8 m	N/A	R9 C, 2R, 1L	Grouted	RBLC	3, 4.9	22.2, 28.6	3	111.2, 249.1
0+27.4 m	N/A	R16 C, 3R, 2L	Grouted	RBLC	3, 4.9	22.2, 28.6	3	111.2, 249.1
0+33.5 m	N/A	R20 1R, 3R, 2L	Grouted	RBLC	4.9, 7.3	28.6	3	249.1
0+42.7 m	N/A	R26 C, 2R, 3L	Grouted	RBLC	4.9	28.6	3	249.1
0+56.3 m	N/A	R35 C, 2R, 2L	Grouted	RBLC	4.9	28.6	3	249.1
0+42.7 m	0+4.6 m	N/A	Point Anchored	IRB	3	19.0	2	NT**
0+42.7 m	0+11.6 m	N/A	Point Anchored	IRB	3	19.0	1	NT
0+42.7 m	0+14.0 m	N/A	Point Anchored	IRB	3	19.0	1	NT
0+42.7 m	0+17.7 m	N/A	Point Anchored	IRB	3	19.0	1	NT
0+42.7 m	0+23.8 m	N/A	Point Anchored	IRB	3	19.0	1	NT

* Measured from Portal 0+00 m increasing westward.
† Measured from NRST centerline increasing northward.
‡ NRST rockbolts installed in "rings" (R) containing a center (C) bolt, 4 bolts left (L), and 4 bolts right (R) of centerline.
** Not tested — NT.
†† Preliminary

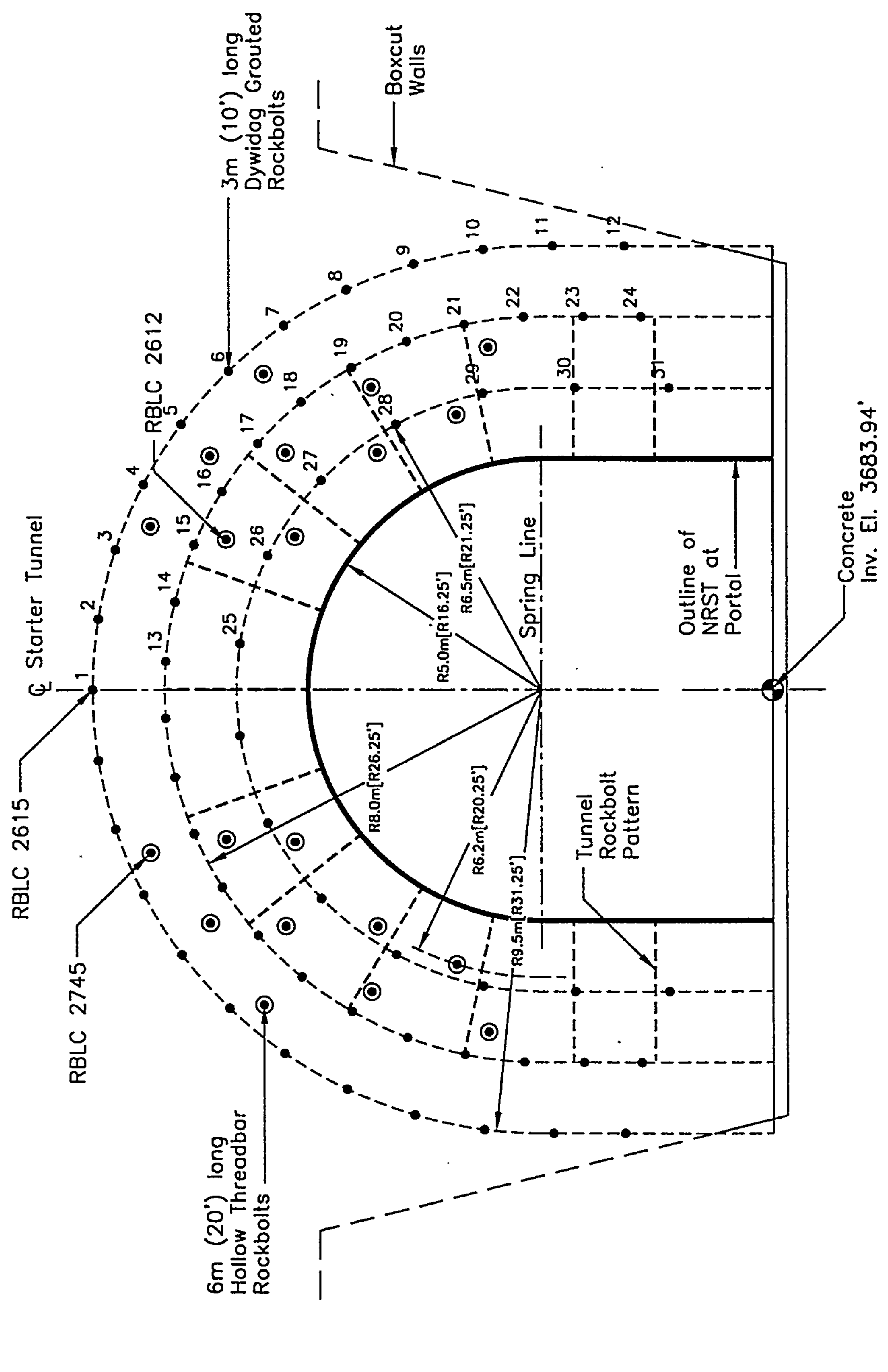


Figure 2-8. Schematic of rockbolt pattern on the portal face (highwall), showing the locations of the RBLCs looking northwest (from ESF Package 1A, YMP-025-1-MING-MG137, Rev 2).

Typical hollow core pumpable cement-grouted 3048 mm (10 ft) nominal rockbolt

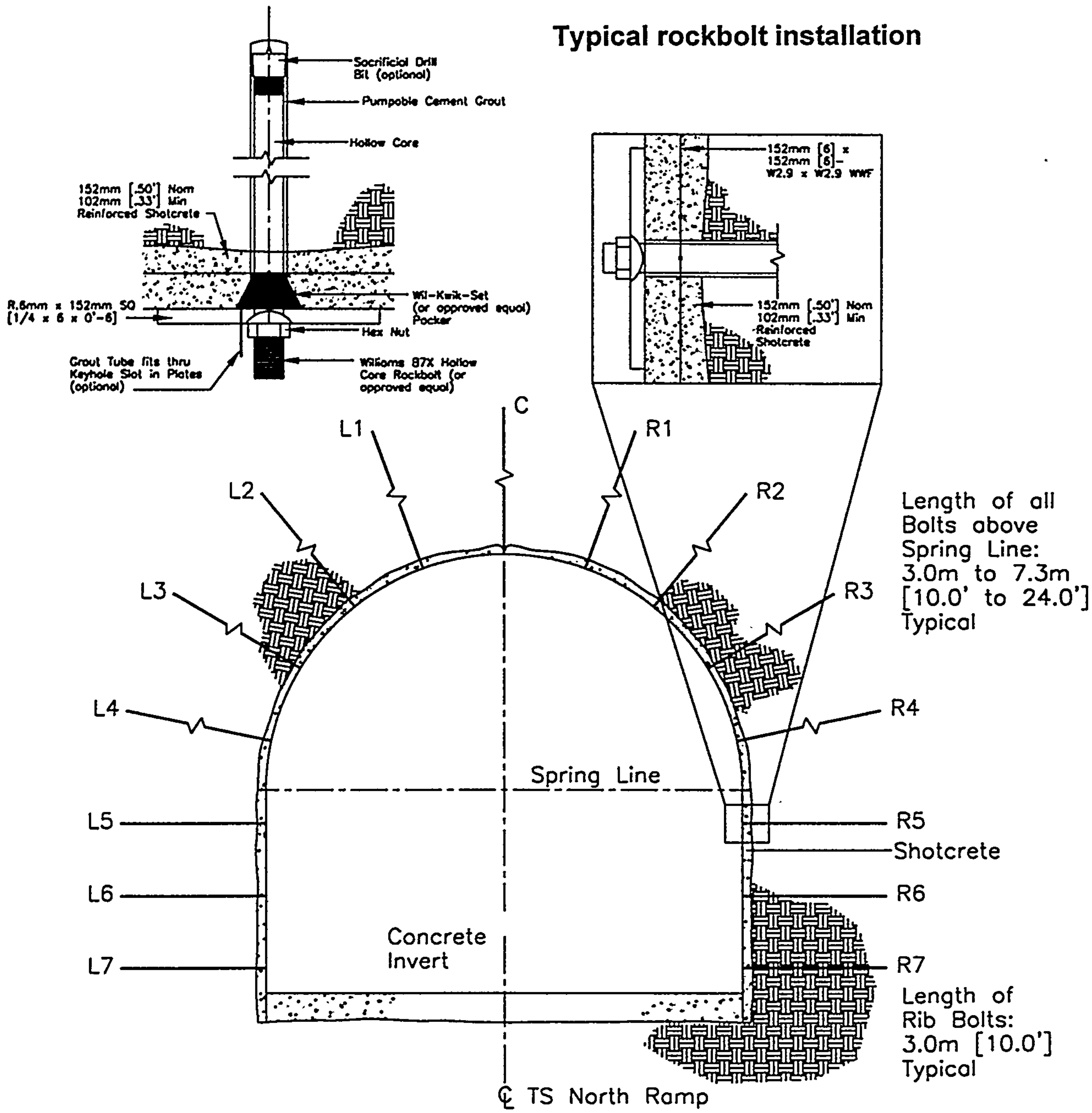


Figure 2-9. General cross section of the NRST showing pattern of grouted rockbolts installed for final support (from ESF Package 1A, YMP-025-1-MING-MG143, Rev 3).

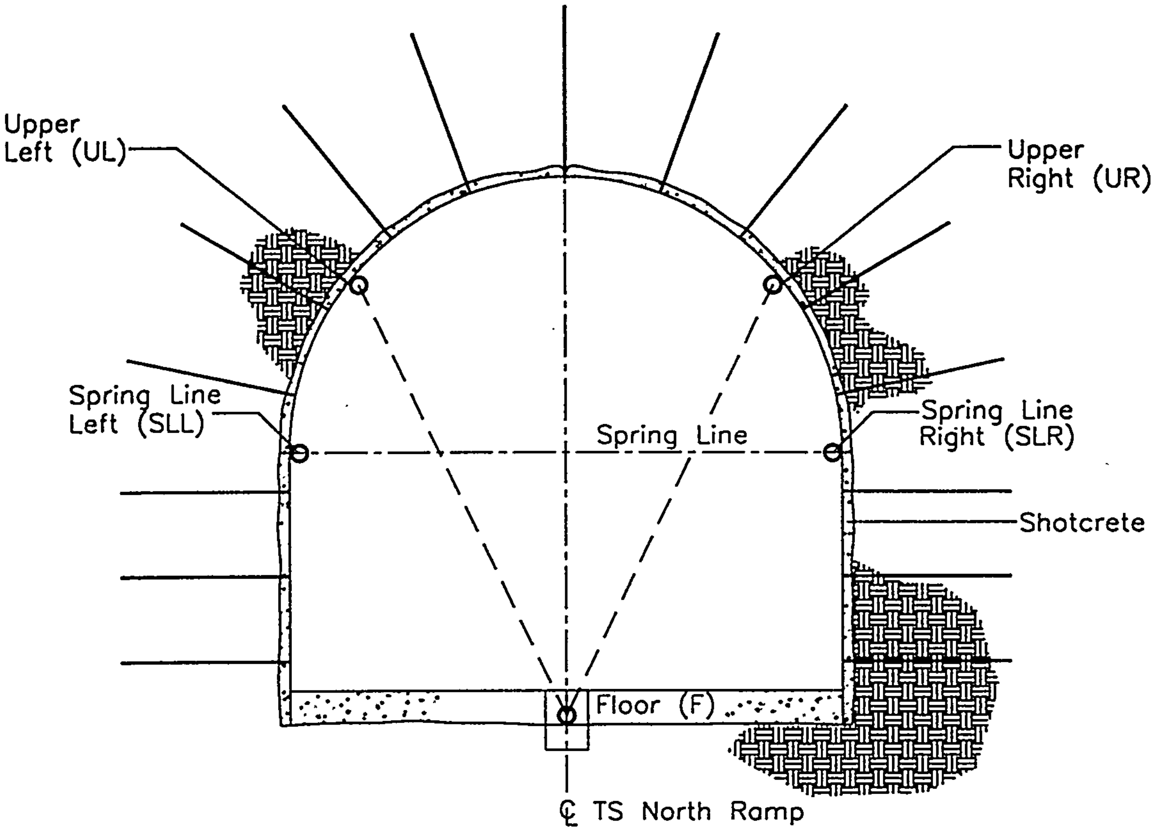
of which is summed and used to calculate the total load. The three transducers allow for nonuniform loading of the load cell at the rough perimeter of the tunnel. The bolts were installed on extensions that allowed attachment through the hollow load cell loaded against a steel bearing plate.

Instrumented rockbolts (IRBs), installed in Alcove No. 1, were Roctest model IRB-H10. The bolts were 19-mm (¾-in) diameter high-strength steel (1060) with 32-mm (1¼-in) expansion shell anchors set at a nominal depth of 3 m (10 ft). The IRB is instrumented with a single vibrating wire transducer placed in a 6.4-mm (¼-in) hole drilled in the center of the bolt head. The transducer is secured in the hole by set screws.

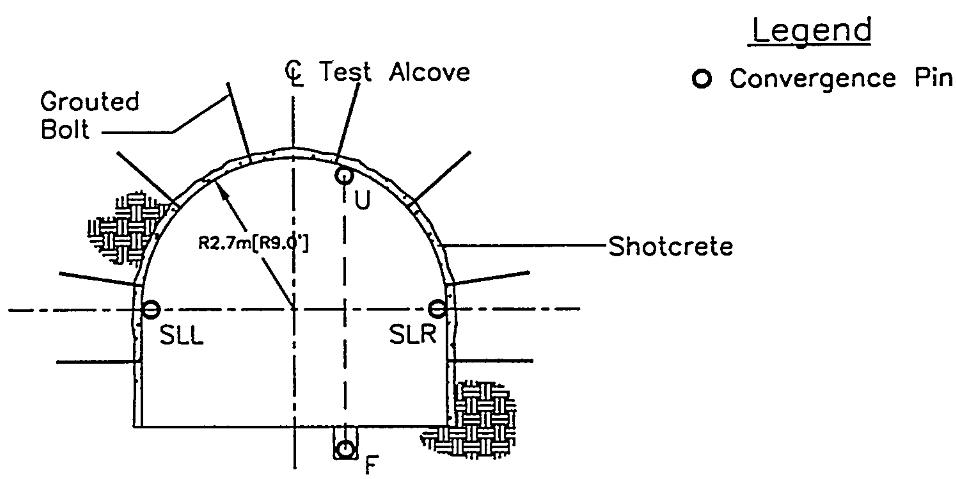
2.3.4 Tunnel Stability Monitoring

Stability monitoring instrumentation in the tunnel and Alcove No. 1 consisted of both convergence measurements and borehole extensometers. Both single- and multi-point borehole extensometers (MPBXs) were installed; convergence was measured using a tape extensometer.

2.3.4.1 Convergence Monitoring. Arrays of convergence points were installed in the tunnel at five stations (0+16.8 m, 0+27.4 m, 0+33.5 m, 0+42.7 m, and 0+56.4 m), corresponding to the locations of the rockbolt load cells. An additional station was installed just past the end of the girders at 0+11 m. The convergence point layout within the tunnel cross section was identical to that of the portal girders. Only vertical and horizontal cords were established in the alcove. Figure 2-10 illustrates the arrangement of convergence points for both the main tunnel and the alcove.



Main Tunnel, looking northwest.



Alcove No. 1, looking north.

Figure 2-10. General convergence pin layout for the Main Tunnel and Alcove No. 1.

The convergence points consisted of eyebolts anchored in a 150-mm (6-in) length of #7 rebar. Installation holes are drilled 38-mm (1½-in) diameter by 0.3-m (1-ft) deep and thereby countersunk 76-mm (3-in) diameter for 76-mm (3 in) to allow the eyebolt to be recessed and protected from damage. The rebars were grouted in place using cementitious grout. Floor points were grouted 0.3–0.6 m (1–2 ft) into the rock through steel pipes set in holes drilled in the concrete floor. Steel covers were installed to protect the floor pins from damage.

Convergence stations installed in Alcove No. 1 consisted of vertical and horizontal chords as shown in Figure 2-10. The vertical chord was offset from the alcove centerline to accommodate the ventilation duct. Alcove No. 1 convergence stations were nominally placed at North Ramp station 0+42.7 m and Alcove No. 1 stations 0+4.6 m, 0+11.3 m, 0+17.7 m, and 0+24.4 m increasing north from the North Ramp centerline.

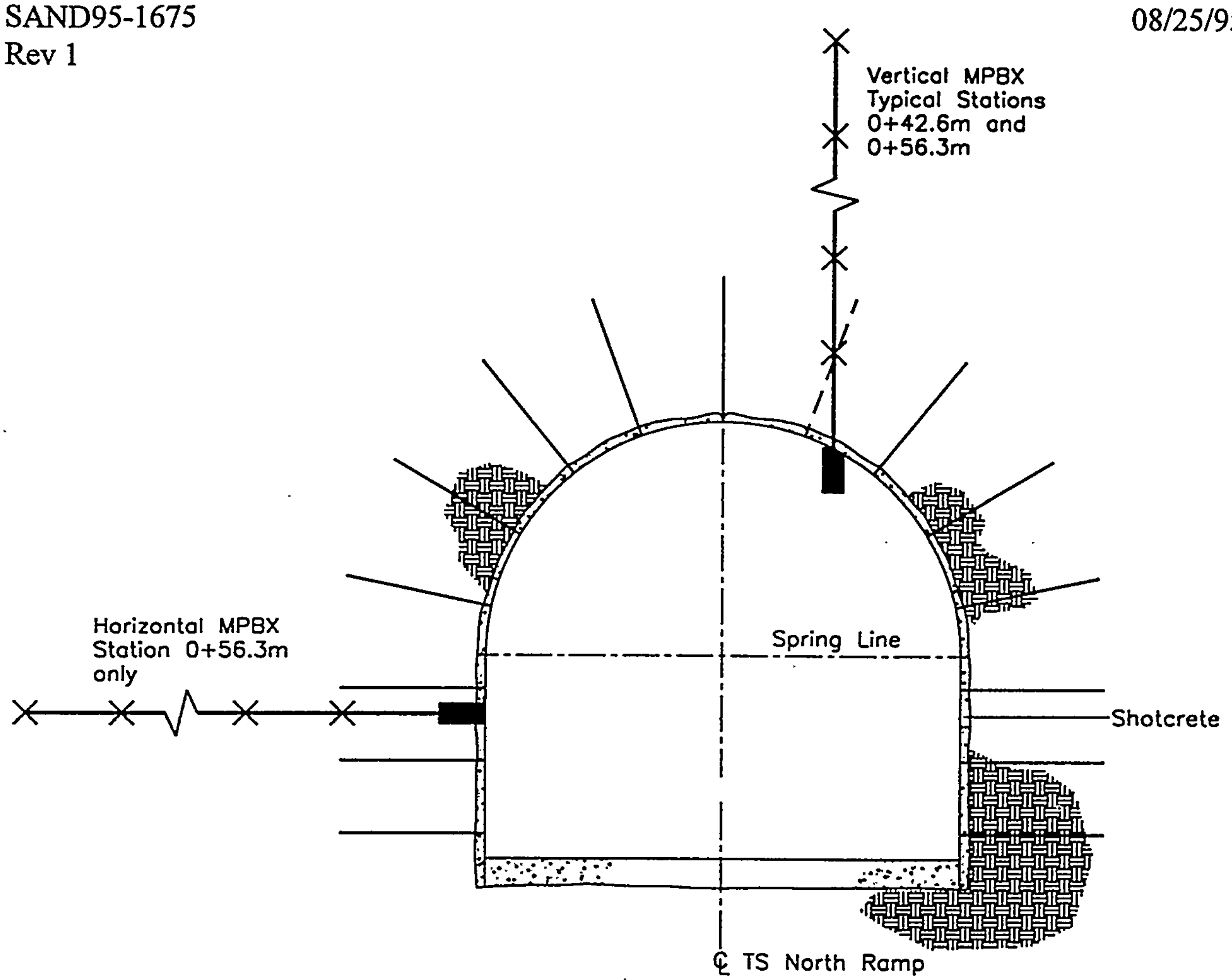
2.3.4.2 Borehole Extensometers. Three MPBXs were installed at stations in the main tunnel, and one MPBX was installed in Alcove No. 1. In addition, two single-point borehole extensometers (SPBX) were installed in Alcove No. 1. Table 2-4 lists the station, orientation, and anchor depths for each of the NRST extensometers. Figure 2-11 shows the typical arrangements of the extensometers within the cross section of both the NRST main tunnel and Alcove No. 1.

All extensometers were Geokon model A5 equipped with model 4450-5 vibrating wire displacement transducers. Borros-type hydraulic anchors were inserted into the boreholes on stainless steel rods. The Borros anchor was used because the large lithophysae that occur in the Tiva Canyon upper lithophysal zone made the borehole diameter very irregular. The anchor had the capability of expanding to 0.3 m (1 ft) to accommodate the borehole irregularity.

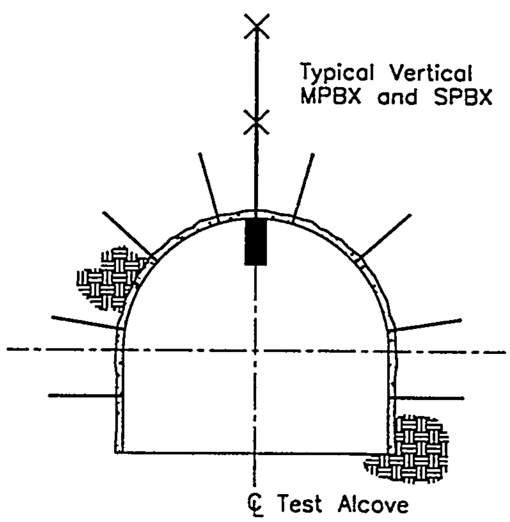
Table 2-4. Location, Orientation, and Anchor Depths for NRST Extensometers

North Ramp Station* (m)	Alcove Type Station† (m)	Type	Orientation	Anchor No.	Nominal Anchor Depth‡ (m)
0+42.7	—	MPBX	Vertical up	0	0
				1	1.74
				2	3.25
				3	4.77
				4	7.81
				5	13.89
0+56.3	—	MPBX	Vertical up	0	0
				1	1.72
				2	3.25
				3	4.77
				4	7.81
				5	13.90
0+56.3	—	MPBX	Horizontal south	0	0
				1	1.71
				2	3.28
				3	4.77
				4	7.85
				5	13.97
0+42.7	0+11.3	MPBX	Vertical up	0	0
				1	2.79
				2	5.84
0+42.7	0+17.7	SPBX	Vertical up	0	0
				1	2.79
0+42.7	0+24.4	SPBX	Vertical up	0	0
				1	2.79

* From Portal 0+00 m.
† From ramp centerline NE along alcove centerline.
‡ Anchor No. 0 is the attachment for instrumented head at the borehole collar.



NRST Main Tunnel, looking northwest.



Alcove No. 1, looking north.

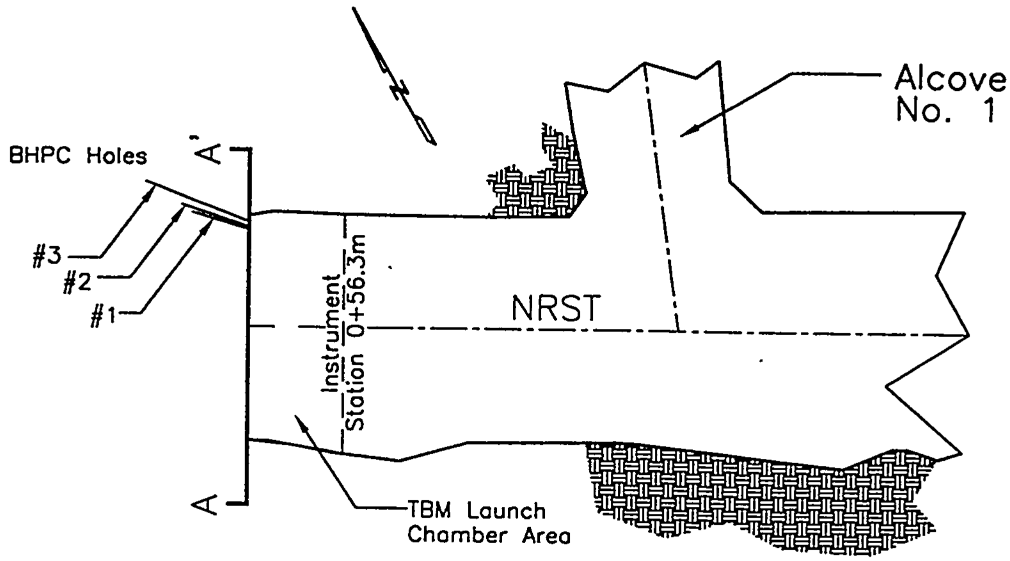
Figure 2-11. Cross sections of the NRST Main Tunnel and Alcove No. 1 showing typical configurations of the extensometers.

2.3.5 Stress Change Monitoring

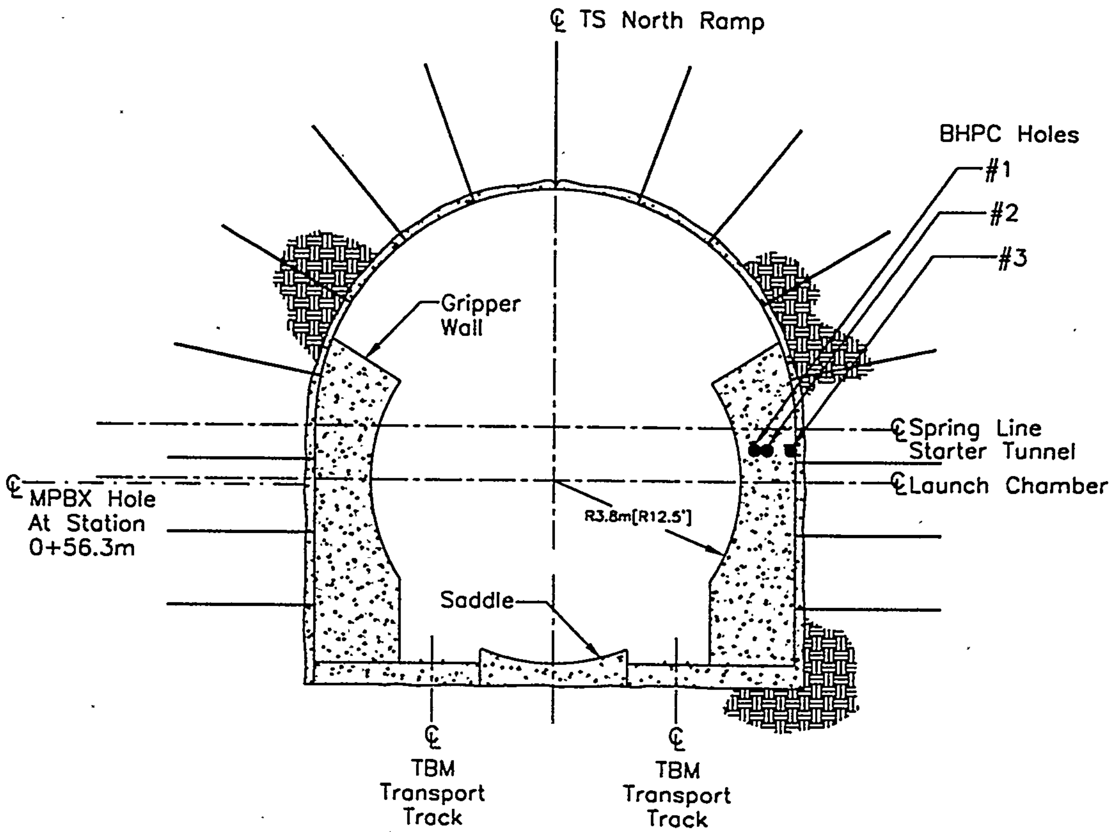
Borehole pressure cells (BHPCs) were installed to measure stress change in rock at the end of the NRST as the TBM excavated past the pressure cells. The BHPCs were installed in three holes drilled radially from the end of the NRST as shown in Figure 2-12, a schematic of the end of the NRST. The BHPCs were installed in three boreholes at different distances from the planned bored wall to be excavated by the TBM. Two BPHCs were placed in each borehole and were oriented to measure changes in the vertical stress and horizontal stress, respectively.

The borehole pressure cells are small flatjacks 51 mm (2 in) wide by 210 mm (8.25 in) long by 6.4-mm (¼-in) thick. These BHPCs, Geokon Model 3200, were precast in grout cylinders. The grout cylinders were then placed at the desired depths in the boreholes, oriented in the boreholes, and then grout was pumped into the boreholes to produce tight contact with the borehole walls. Geokon Model 4500H vibrating wire pressure transducers were used to monitor the flatjack pressure. After grouting the BHPCs, a concrete structure was constructed to aid the TBM in sumping into the face. This structure is shown in Figure 2-12.

The configuration of the individual BHPCs within each hole are listed in Table 2-5. The BHPCs were grouted on June 24, 1994. The grout was allowed to cure until August 3, 1994, then the BHPCs were pressurized to 0.724–0.759 MPa (105–110 psi). The initial pressures are listed in Table 2-5. The pressurized flatjacks bled-off with time and were re-pressurized on October 4, 1994, during the TBM start-up and mining by the BHPCs. Applied pressure ranged from 1.035–1.279 MPa (150–200 psi) and the initial re-pressure levels are listed in Table 2-5.



Plan View



Cross Section A-A', looking northwest.

Figure 2-12. Schematic showing configuration of the TBM launch chamber in the NRST and location of the borehole pressure cells.

Table 2-5. Configuration of Individual BHPCs in Each Hole

Station (m)	Hole No.	BHPC No.	Serial No.	Stress Orientation	Initial Pressure* (MPa)	Repressurization Pressure† (MPa)	Northing Coordinate (m)	Easting Coordinate (m)	Elevation (m)	Radial Distance from Bored Wall (m)
61.41	1	1	22602	vertical	0.324	1.054	233,316.03	173,629.98	1,127.35	0.61
60.25	1	2	22603	horizontal	0.855	1.016	233,315.16	173,630.83	1,127.29	0.31
61.70	2	3	22604	horizontal	0.497	1.039	233,316.41	173,629.86	1,127.37	0.91
60.97	2	4	22605	vertical	0.869	1.099	233,315.85	173,630.38	1,127.33	0.76
62.32	3	5	22606	horizontal	0.559	0.917	233,317.18	173,629.61	1,127.38	1.52
61.61	3	6	22607	vertical	0.759	0.999	233,316.60	173,630.10	1,127.35	1.22

* August 3, 1994
† October 4, 1994

2.4 Data Acquisition and Analysis

2.4.1 Manual Instrument Reading

Instruments were read manually at installation and for the period up to April 30, 1994, when installation of the data acquisition system was complete. Convergence measurements were made using a Terrametrics Model 1600 tape extensometer. Remote reading instruments, with the exception of some temperature sensors, were all based on vibrating wire gages. These instruments were read with a Geokon GK-401 read-out instrument.

2.4.2 Data Acquisition System

A computer-based data acquisition system (DAS) was installed in Alcove No. 1 to automate monitoring of all electronic-based instrumentation. The system layout and connectivity is illustrated by the system drawing in Appendix C.

The DAS was designed and configured by Roctest. It was developed around a CR10 Central Measurement and Control Module manufactured by Campbell Scientific, Inc. The CR10 is a fully programmable data logger/controller built in a small, rugged, and sealed module. The CR10 was located in a junction box in the alcove and communicated with six Roctest RTX-248 analog multiplexers which were remotely located at junction boxes J1 through J6 throughout the NRST and Alcove No. 1.

In the normal operating mode, the CR10 reads each instrument four times a day and stores the data. Data were downloaded to a notebook computer on nominally one-month intervals to protect against overwriting. The DAS output is saved as an ASCII file that is incrementally updated, then backed up on 3.5-in disks and magnetic tape. These ASCII data are the digitized value of the gage output (generally, in the case of vibrating wire gages, this is the frequency squared of the wire). No manipulation of these ASCII data is permitted beyond updating the file with new data.

The data from the DAS are imported into a duplicate spreadsheet file using a macro program. Once verified, the data were separated by gage into individual spreadsheets. Calibration factors are input into calculations in the spreadsheet files and the data are plotted for analysis and presentation.

This page left intentionally blank.

3.0 Construction Monitoring Data

3.1 Introduction

The results of the construction monitoring activities are presented in this section of the report. Rock mass quality evaluations are discussed for each of the top headings, bench and Alcove No. 1. Blasting seismic data are organized according to the top heading, and Alcove No. 1 excavations. The instrumentation data are organized by station/location and can be correlated with the USBR structural mapping data by referring to Appendix D.

Although most of the instrumentation was installed after all excavation had been completed, there were several instruments installed during mining. Figure 3-1 presents a historical schedule of events that indicates the interactions of the various data collection activities. Excavation of the main tunnel was initiated in April 1993 and the top heading was completed by mid-July 1993. During this period, rock mass quality assessment and blast monitoring were the main design verification activities performed. Midway through excavation of the top heading, the 6-m (20-ft) long grouted rockbolts were installed in the portal face (highwall) with three of the bolts instrumented in early June, as described in Section 2.3.3.2. The top heading portions of the portal girders were installed in mid-June and the lower portions were completed at the end of November after excavation of the bench. Excavation of the bench and collection of the rock mass quality data occurred between August and early October 1993. The only instrumentation that was in place during mining excavation were the portal face RBLCs, the upper portion of the lattice girder convergence points, and three additional RBLCs installed at station 0+16.8 m. The remaining instruments in the main tunnel were installed between mid-October and mid-December 1993, during installation of the final support.

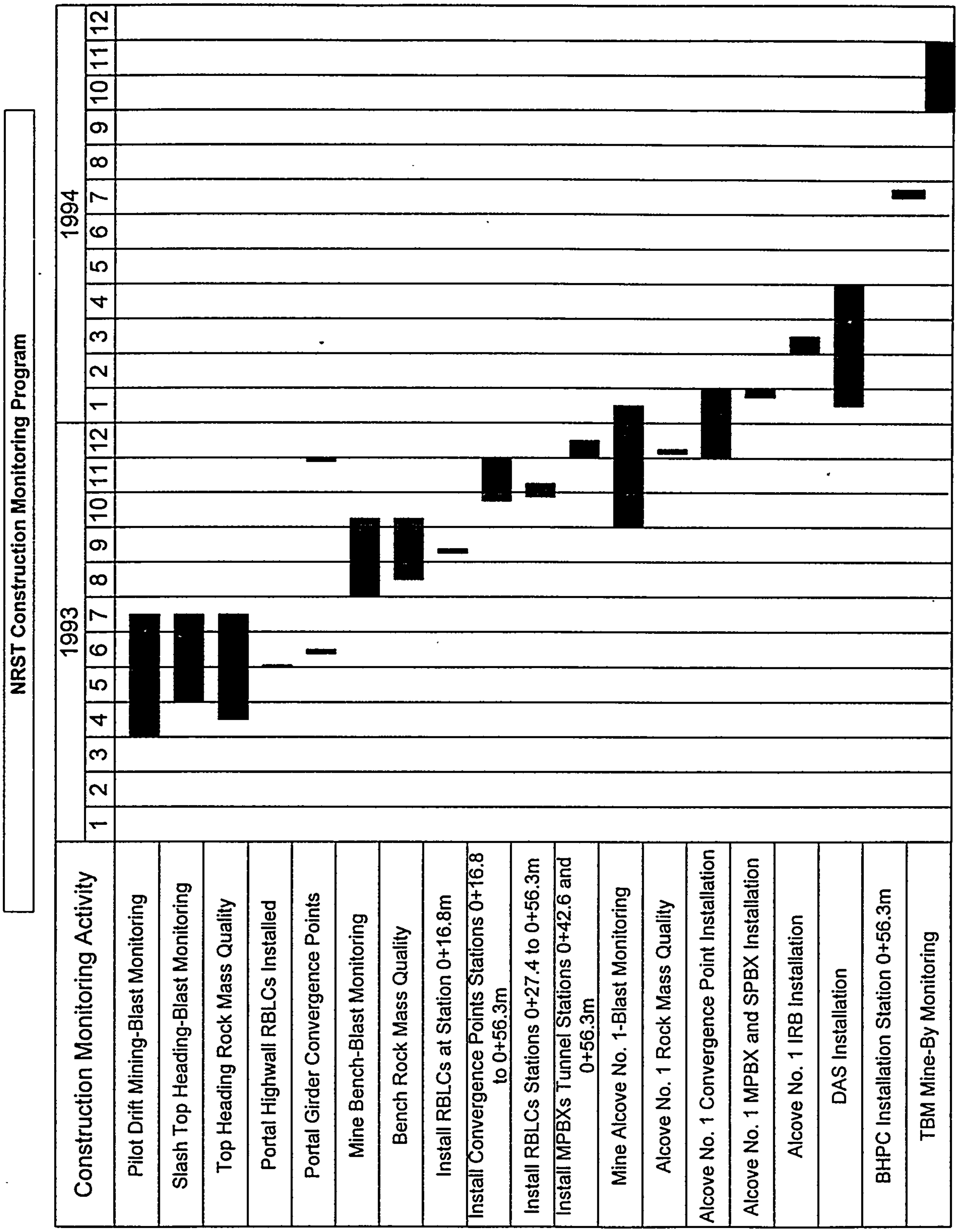


Figure 3-1. Schedule of activities in NRST construction monitoring program.

Excavation of Alcove No. 1 began early October 1993 and was completed by mid-January 1994. Some convergence points were installed during excavation, however, the extensometers and IRBs were installed after completion of mining and during installation of the final support.

Borehole pressure cells were installed and grouted into boreholes near the end of June 1994. The cells were pressurized at the beginning of August 1994. The TBM was sumped-in to begin boring of the North Ramp near the end of September 1994.

These data are derived from a series of scientific notebooks developed during the report period. The traceability of the data into the subsequent analysis file and TDIFs is presented in Appendix A.

3.2 Rock Mass Quality Data

Rock mass quality assessments were conducted using both the Q (Barton et al. 1974) and RMR (Bieniawski 1979) systems. These rock mass quality indices have been utilized as the basis for empirical design of tunnel ground support (Q) and for estimation of rock mass mechanical properties (RMR) in the YMP Drift Design Methodology proposed by Hardy and Bauer (1991). These approaches have also been utilized to project construction conditions along the North Ramp (Brechtel et al. 1995) and as the basis of North Ramp ground support design.

The rock mass quality Q and RMR for assessments performed in the top heading, bench, and Alcove No. 1 are presented in Table 3-1 as a range of the observed conditions. RMR was not evaluated in the top heading. Figures 3-2 and 3-3 graphically compare the range of the assessments as a function of tunnel stationing. The figures include the range of ground support

Table 3-1. Range of Rock Mass Quality for the NRST

	Tunnel Location (ft)	Q_{min}	Q_{max}	RMR_{min}	RMR_{max}
Top Heading	0–10	0.06	0.38	NR	NR
	10–34	0.07	0.24	NR	NR
	34–54	0.15	0.48	NR	NR
	54–72	0.73	1.83	NR	NR
	72–95	0.17	0.86	NR	NR
	97–115	0.68	2.80	NR	NR
	116–129	0.66	1.80	NR	NR
	129–160	0.97	2.28	NR	NR
	160–195	0.31	4.40	NR	NR
Bench	2–40	0.05	0.24	34	61
	40–85	0.63	1.32	34	62
	85–140	0.37	17.40	45	68
	140–172	0.37	17.40	45	68
	172–194	0.16	7.60	37	61
Alcove No. 1	28–88	0.36	2.78	30	63

NR—Not recorded.

categories derived from the empirical ground support design charts developed by Barton et al. (1974) (see Appendix F). Length-weighted average values of Q were calculated by

$$\bar{Q} = \log_{10}^{-1}\left[\frac{\Sigma L_i \times \log_{10}(Q_i)}{\Sigma L_i}\right] \qquad 3\text{-}1$$

where

$\bar{Q}$ = length-weighted geometric mean,

L_i = length of interval i, and

Q_i = rock mass quality Q for interval i.

These values and the range are summarized in Table 3-2.

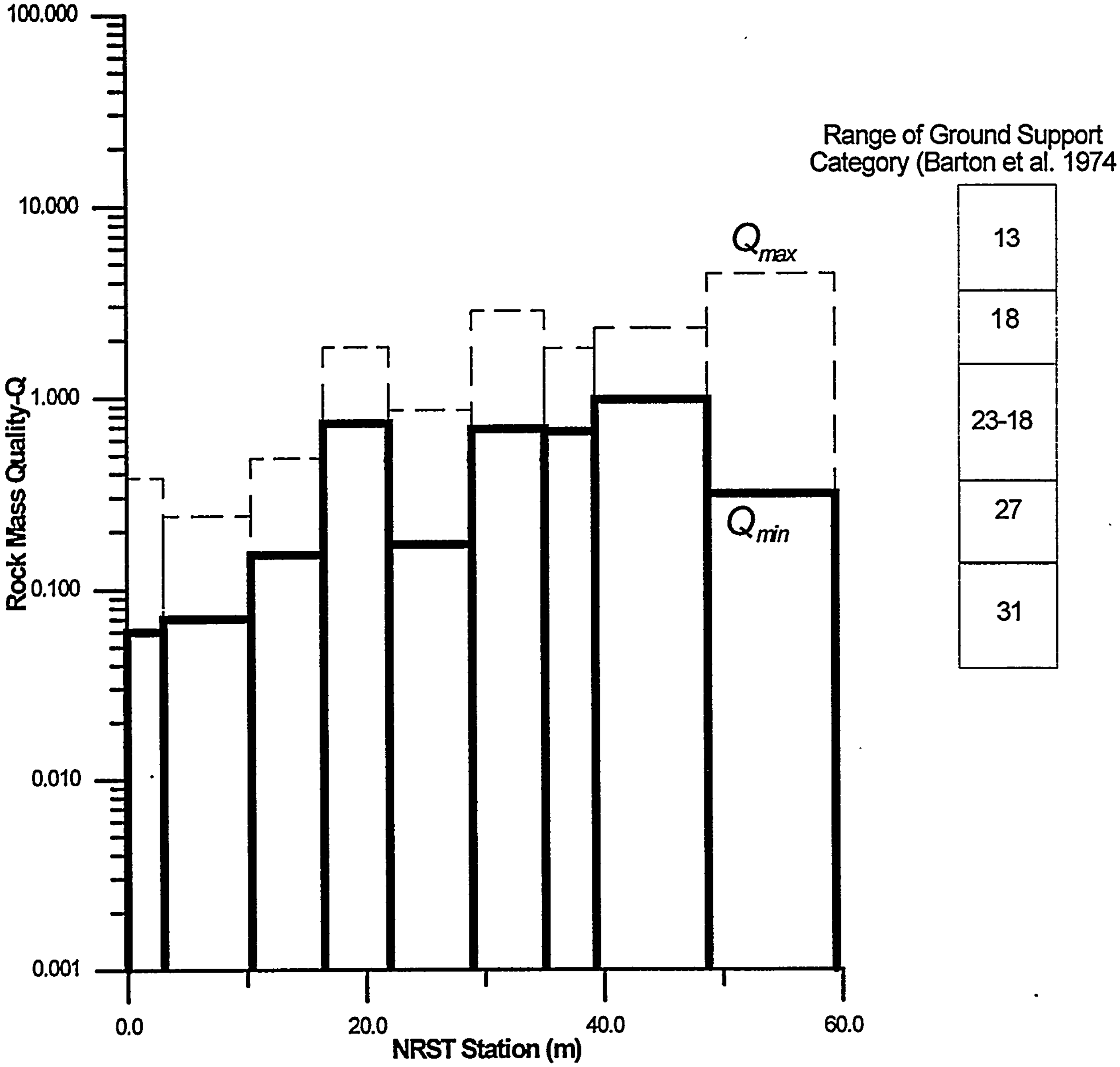


Figure 3-2. Variation of Q_{min} and Q_{max} with NRST station for the Main Tunnel—Top Heading (see Appendix F for definition of ground support categories in Barton et al. 1974).

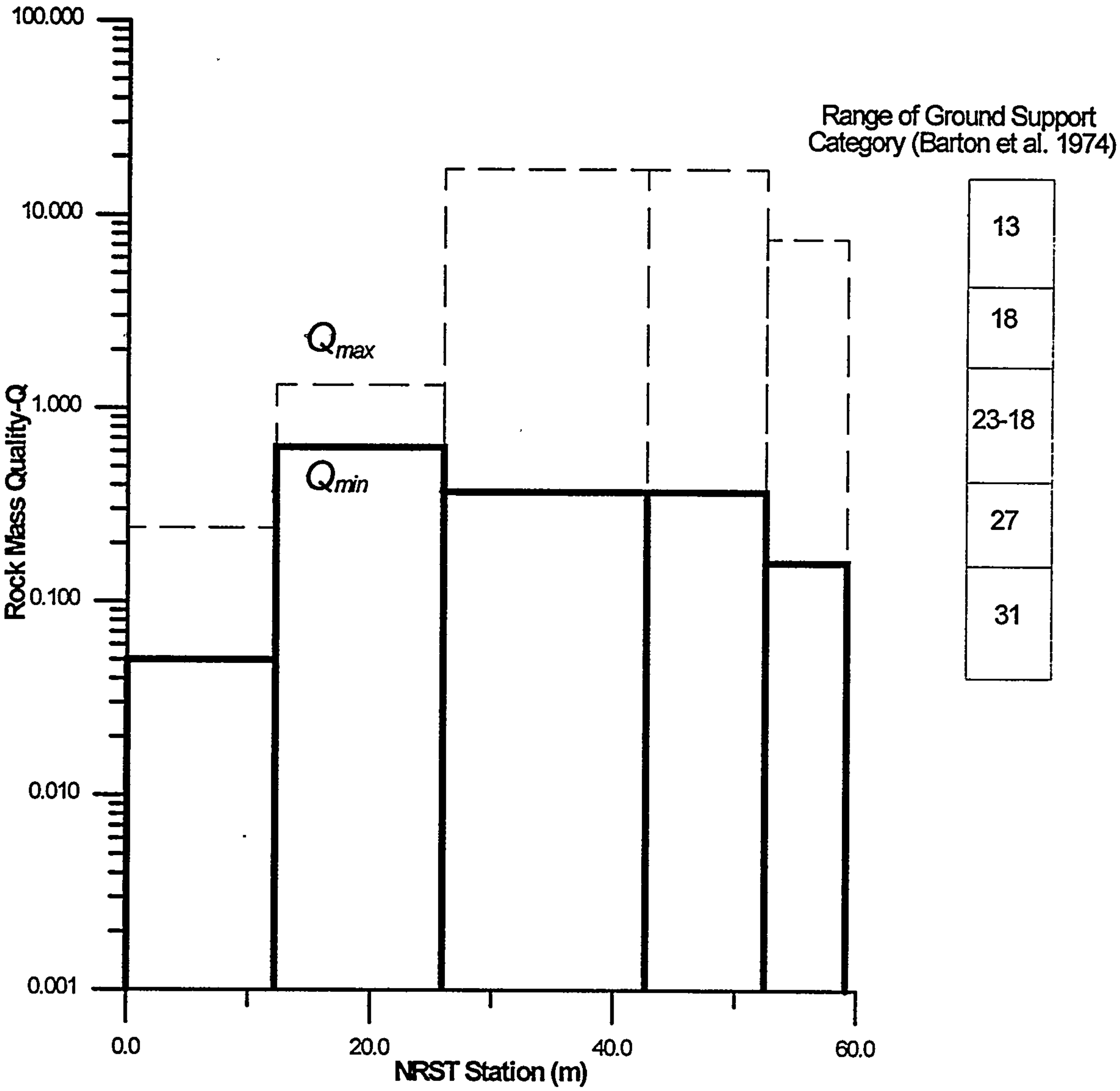


Figure 3-3. Variation of Q_{min} and Q_{max} with NRST station for the Main Tunnel bench (see Appendix F for definition of ground support categories in Barton et al. 1974).

Table 3-2. Length-Weighted Log Mean Values of Q_{min} and Q_{max}—NRST Main Tunnel

NRST Section	Range Q_{min}	$\bar{Q}_{\mathrm{min}}$	Range Q_{max}	$\bar{Q}_{\mathrm{max}}$
Top Heading	0.06–0.97	0.30	0.38–2.80	1.06
Bench	0.05–0.63	0.24	0.24–17.40	1.82
Alcove No. 1	NM*	0.36	NM	2.78

*NM—Alcove included in one assessment.

The tables and figures indicate a general trend for increasing rock mass quality with increased depth of NRST penetration from the portal face. This was due to increased distance from the source of weathering and the gradual cross cutting of the Tiva Canyon upper lithophysal zone which correlated with a gradual reduction in size of the lithophysae that occurred at the bottom of the zone. This change is clearly shown by the two high values of Q (17.40) in the bench excavation near the end of the NRST.

The variation of the individual parameters used to determine Q are shown as frequency of occurrence histograms in Figure 3-4, where all data from the NRST have been grouped together. The Q_{min} data are impacted by the presence of near-vertical shear zones/broken zones that have undergone relatively small movements. These zones were localized but impacted overbreak and ground support requirements locally. The Q_{max} values are representative of the character of the rock between these larger near-vertical structures.

Length-weighted average values of RMR for the bench were 39.4 and 64.4 for the RMR_{min} and RMR_{max}, respectively.

3.3 Blast Monitoring

The seismic records of the blasting events provided several types of information that were useful in assessing performance. The analog records of the data allowed individual events to be correlated with the delays used to detonate groups of holes. This allowed the construction of correlations between scaled distance and PPV, which could be compared with other data to assess comparable performance. It also provided a basis for adjusting the design of the blast to reduce PPV at any given delay. PPVs were derived directly from the seismic records and correlated with details on charge weight and blasting delays taken from blasting records

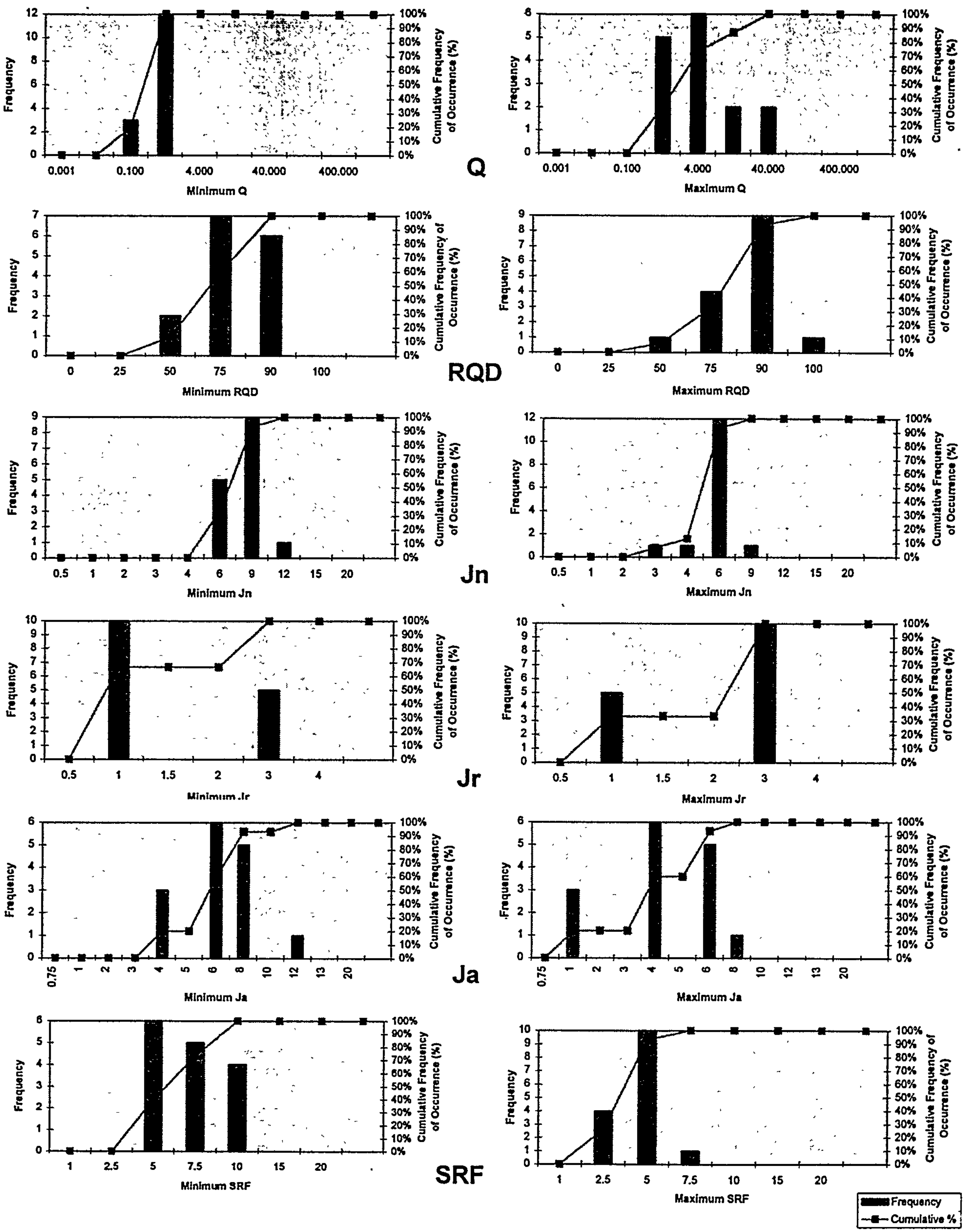


Figure 3-4. Frequency of occurrence of parameters used to determine Q_{min} and Q_{max}—all data.

maintained by the constructor, REECo. These data were derived for the top heading blasting events and the Alcove No. 1 blasting experiments. Data recorded in the bench mining were similar in PPV magnitude, peak frequency and distance, but were not included in the analysis.

3.3.1 Scaled Distance Results for NRST Top Heading Blasting

The scaling of distance is used to develop attenuation relationships between PPV and distance, when both the distance and weight of explosive detonated are varied. The two most common approaches divide the distance by the square root and cube root of explosive weight, respectively.

The attenuation relationship is calculated by Equations 3-2 and 3-3 for the two different scaling approaches (Dowding 1985).

$$PPV = A\,(R / W^{1/2})^{B} \qquad \text{3-2}$$

$$PPV = A\,(R / W^{1/3})^{B} \qquad \text{3-3}$$

where

PPV = peak particle velocity,

R = distance from seismograph to the blast,

W = weight of explosive detonated to produce the PPV, and

A, B = constants.

The two blasting seismic monitors used for recording the top heading blasting were located at five different recording stations as described in Section 2.0. For each blast, the distances between the seismic monitor and the center of the blast were determined using the northing and easting coordinates. Elevations were not considered in calculation of distance.

Each seismic record consisted of an analog record of the blast and the time that corresponded to the occurrence of the PPV. Using this time of occurrence, the corresponding charge weights were extracted from the blasting records.

Curve fits of PPV versus scaled distance (SD), as calculated by Equations 3-2 or 3-3, were performed using a computer program based on the Simplex algorithm described by Caceci (1984). The results of the curve fit are listed in Table 3-3 and plots of the data and curve fits are presented in Figures 3-5 and 3-6. The numerical data are presented in Appendix E.

Table 3-3. Comparison of Curve Fits of PPV versus SD using Equations 3-2 and 3-3

	PPV (in/sec) = A × SD^B		Standard	
Scaling Law	A	B	Deviation	Estimated Error
$SD = R/W^{1/2}$	23	−1.1	0.45	0.047
$SD = R/W^{1/3}$	133	−1.5	0.39	0.040

Figure 3-5 compares the measured PPV to a general relationship reported by Dupont (1980) to be typical for attenuation of blasting energy with conventional explosives. The relationship is

$$PPV = 160\left(\frac{R}{W^{1/2}}\right)^{1.6} \qquad 3\text{-}4$$

The scatter in the NRST blasting data is typical and the results are grouped around the relationship in Equation 3-4. The difference is due to site-specific conditions and would be expected.

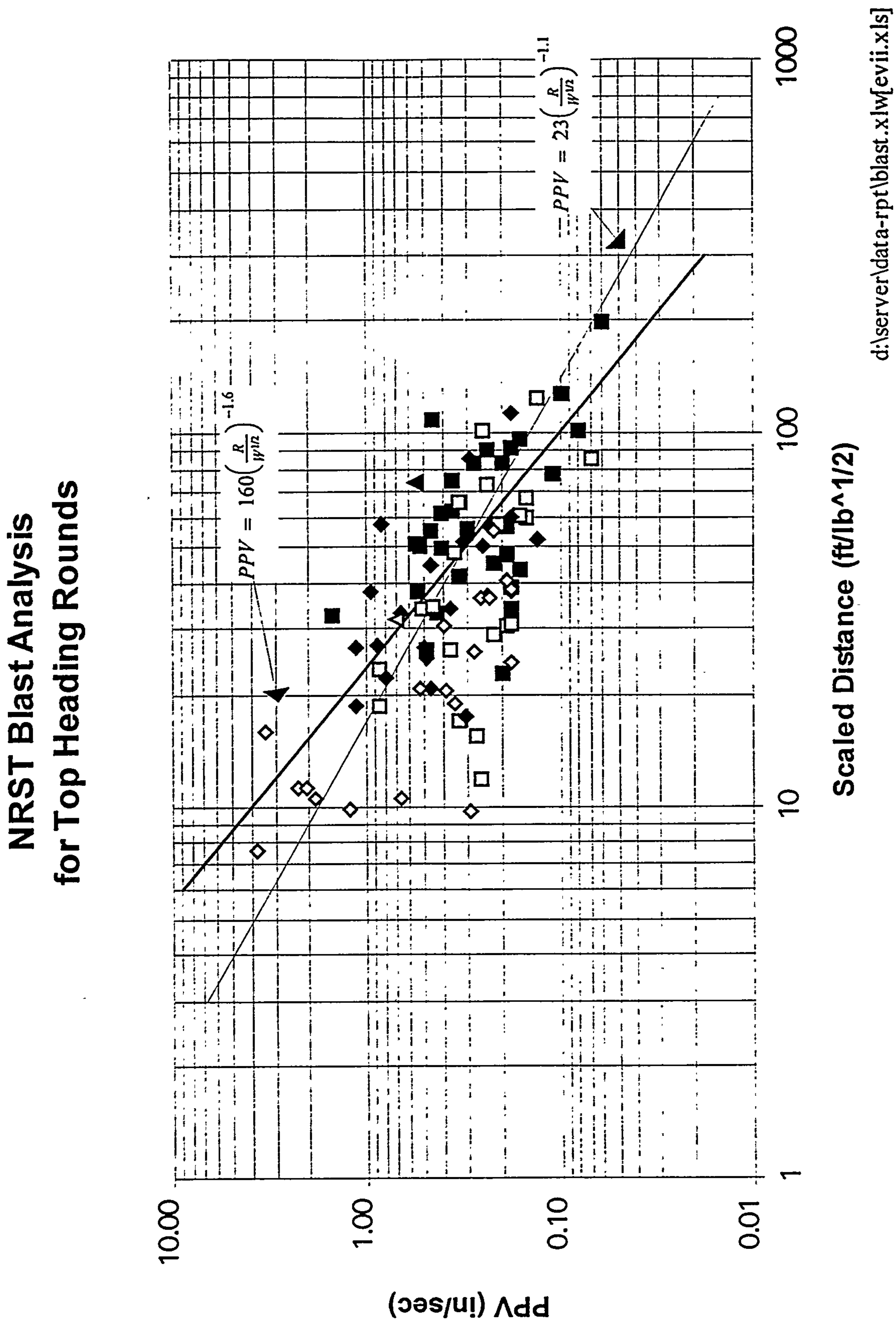


Figure 3-5. PPV versus SD (distance normalized to charge weight to the 1/2 power).

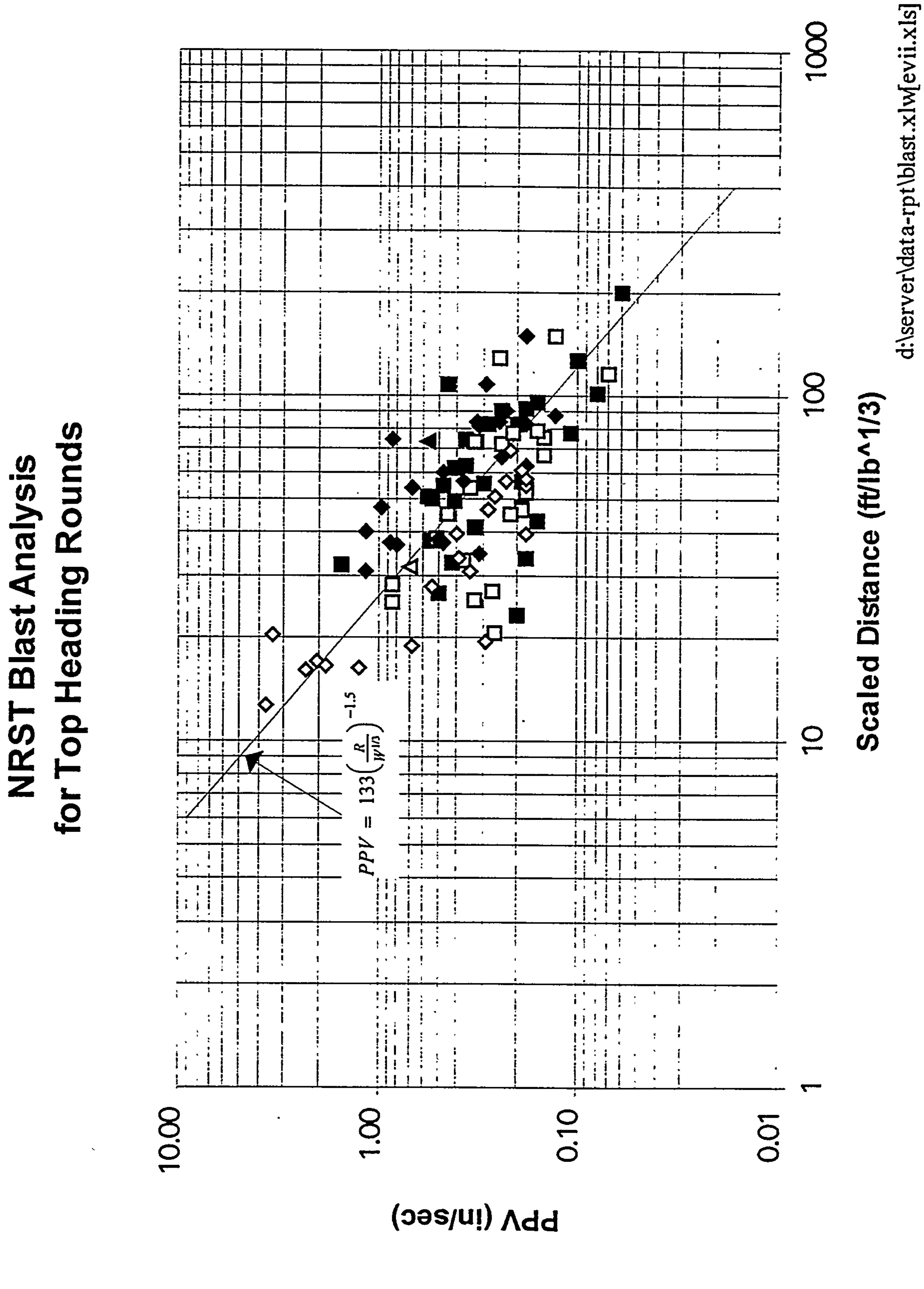


Figure 3-6. PPV versus SD (distance normalized to charge weight to the 1/3 power).

3.3.2 Alcove No. 1 Near-Field Blast Monitoring

Blast damage associated with the excavation of Alcove No. 1 within the NRST was evaluated by monitoring near-field blast vibrations and by making video observations of boreholes in an attempt to determine the extent of damage. A quantitative correlation of PPV with rock damage was then attempted using an approach similar to work done by Holmberg and Persson (1979).

The layout of the monitoring, shown earlier in Figure 2-5, Section 2.3.2.2 was designed to develop data from the first three blast rounds in the Alcove No. 1. Sets of two observation and two instrumentation holes were drilled parallel to the excavation. The observation holes were video logged before blasting and after the fourth round in the Alcove No. 1.

The instrument holes each contained two geophones connected to a four-channel digital oscilloscope. The geophones were oriented to monitor compressional and shear wave vibrations in the horizontal plane. The vertical component was not monitored. During data reduction the vertical component was assumed to be the average of the horizontal components so that a comparison could be made to other triaxial data acquired from the NRST blast monitoring.

The data from three of the first four blasts, TA-1, TA-3 and TA-4, were used for this analysis. Blast TA-2 was not used because it was only a trim round.

To analyze the near-field seismic data, the time of each wavelet peak recorded by the oscilloscope was correlated with a specific delay for each blast. Both single or multiple holes were initiated by a given delay. The asymmetrical design of the blast monitoring layout with respect to the opening resulted in some uncertainty in determining which holes affected the particle velocities measured by the geophones. The burn resulted in a void or disturbed zone in the center of the round, thereby disrupting the wave path to the geophones located on the west side of the round. Because of this, blast holes located east of the blast centerline were not included in the charge weight or distance calculations. The perimeter holes used to determine charge weight were limited to those at the west margin of the round up to a height of 3.3 m (11 ft). The distance from the blast hole to the geophones was determined by using the horizontal distance for delays initiating single holes, however, for multiple holes initiated by the same delay, the horizontal distance to the geometric center of the holes was used.

PPVs were determined from the amplitudes recorded for each wavelet peak associated with a given delay. A vector sum of the particle velocities for each peak was then determined using Equation 3-5:

$$PPV(total) = [PV(comp)^2 + PV(shear)^2 + PV(vert)^2]^{1/2} \qquad (3\text{-}5)$$

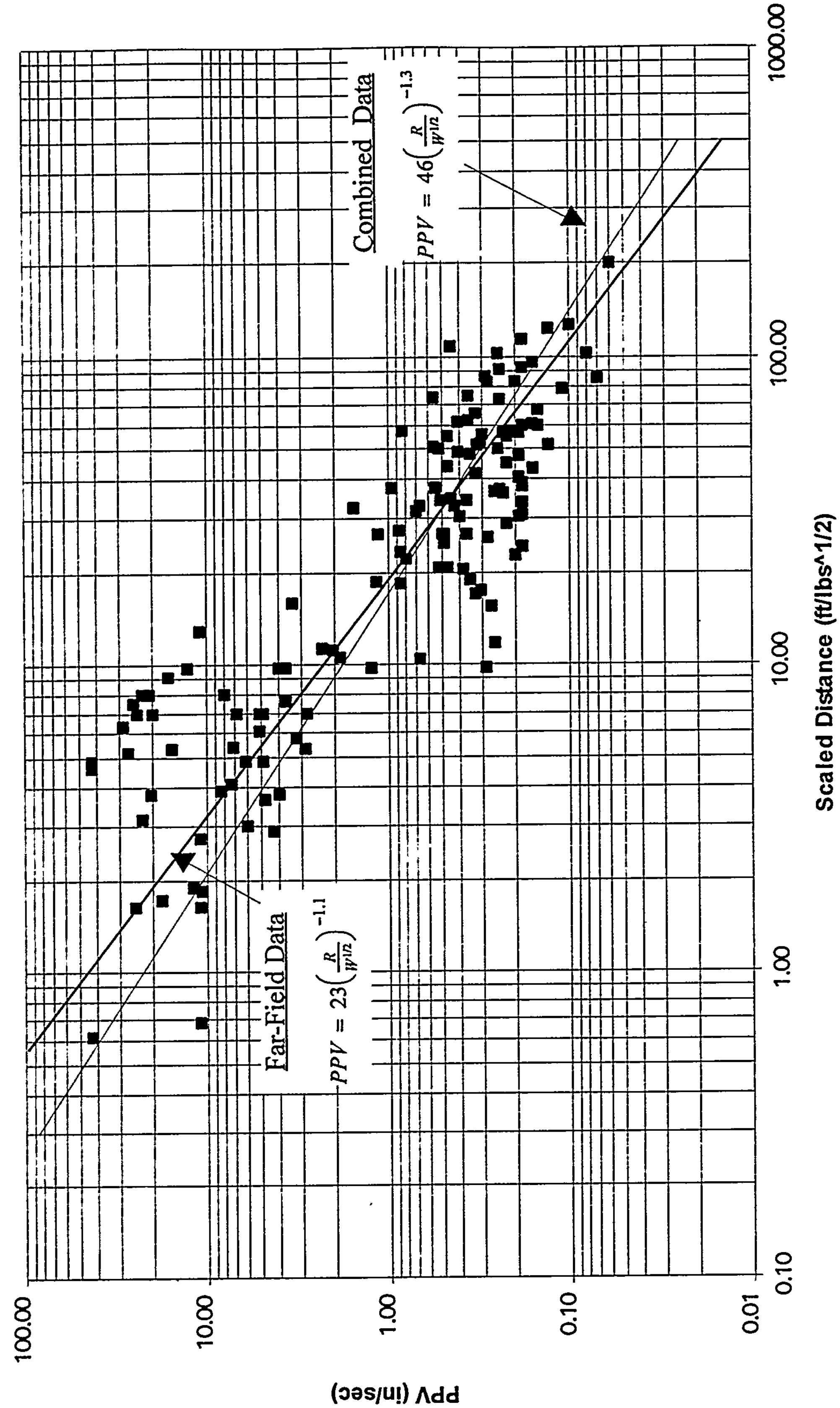
where one of the two measured components is a peak value and the vertical component of velocity was assumed to be the mean of the two horizontal components. This assumption was based upon examination of the far-field blasting records.

The two scaling relationships presented in Equations 3-2 and 3-3 were used, and the near-field data were combined with the far-field data developed for the NRST top headings (Section 3.3.2) to establish a combined correlation between PPV and scaled distance. The resulting linear regressions are listed in Table 3-4 and shown graphically in Figures 3-7 and 3-8. A listing of the near-field data is presented in Appendix E.

Table 3-4. Combined Near-Field/Far-Field Curve Fit

	PPV = A(SD)^B			
	A	**B**	**R^2***	**Standard Error**
SD=R/W$^{1/2}$	46	−1.3	0.76	0.34
SD=R/W$^{1/3}$	89	−1.4	0.82	0.33

*R^2—Correlation coefficient



d:\server\data-rpt\alcvb1.xls[bsim.xls]

Figure 3-7. PPV versus SD for the square foot of charge weight—Top Heading and Alcove No. 1 blast data.

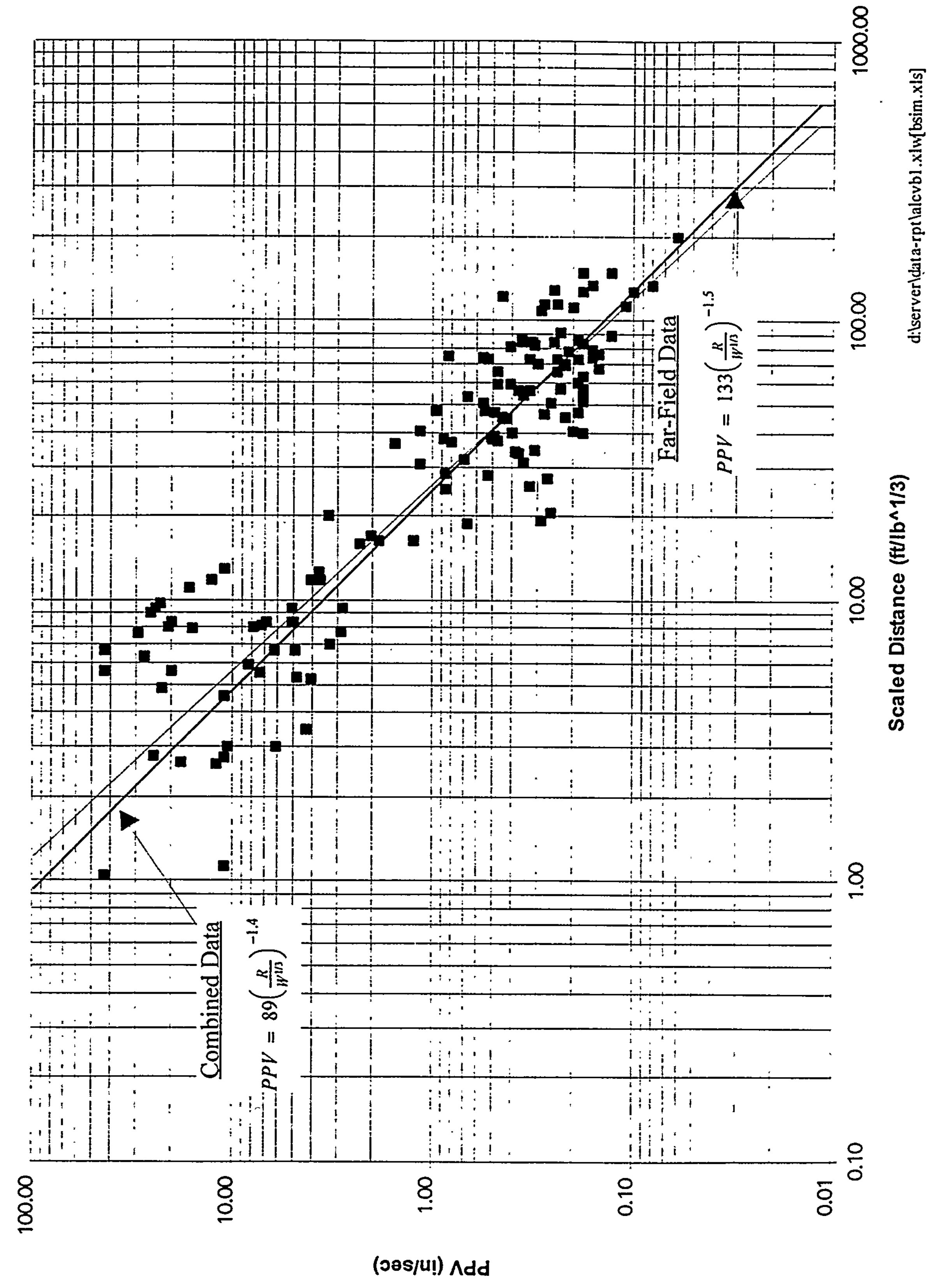


Figure 3-8. PPV versus SD for the cube root of charge weight—Top Heading and Alcove No. 1 blast data.

The curve fits of combined near-field and far-field blast monitoring data are compared to curve fits of the far-field data in the two figures. The curves are quite similar and suggest that extension of the scaling criteria to the near-field is appropriate in the frequency range monitored.

3.3.3 Evaluation of Blast Damage by Borehole Video Logs

The observation boreholes were video borescoped before blasting and after blast round TA-4. The intention of the borescope logging was to determine the difference in the number of fractures before and after blasting as done by Holmberg and Persson (1979). However, grout from nearby rockbolts had penetrated the observation holes between observations. Qualitative observations made during the post-blast video logging indicated that some of the grout had fractured in the holes nearest the Alcove No. 1, 0.91 m (3 ft) from the blast boundary, but not in the holes at 2.4 m (8 ft) from the Alcove No. 1 perimeter. These cracks were attributed to the blasting and were used in place of a fracture counting approach.

Figure 3-9 shows the velocity histories for the holes at 0.91 m (3 ft) and 2.4 m (8 ft) from the Alcove No. 1 wall and compares them to the range of PPV reported to cause rock damage by Holmberg and Persson (1979). A maximum PPV of 1100 mm/sec (43 in/sec) was recorded for the hole at 0.91 m (3 ft) and 650 mm/sec (25 in/sec) for the hole at 2.4 m (8 ft). Holmberg and Persson indicate that blast damage begins in the range of 700–1000 mm/sec (28–39 in/sec). The results of the borehole inspection are consistent with this range.

3.4 Instrumentation Results

The results of monitoring the instrumentation installed in the NRST are presented in this section for the time period between installation and the end of June 1995. The data are presented

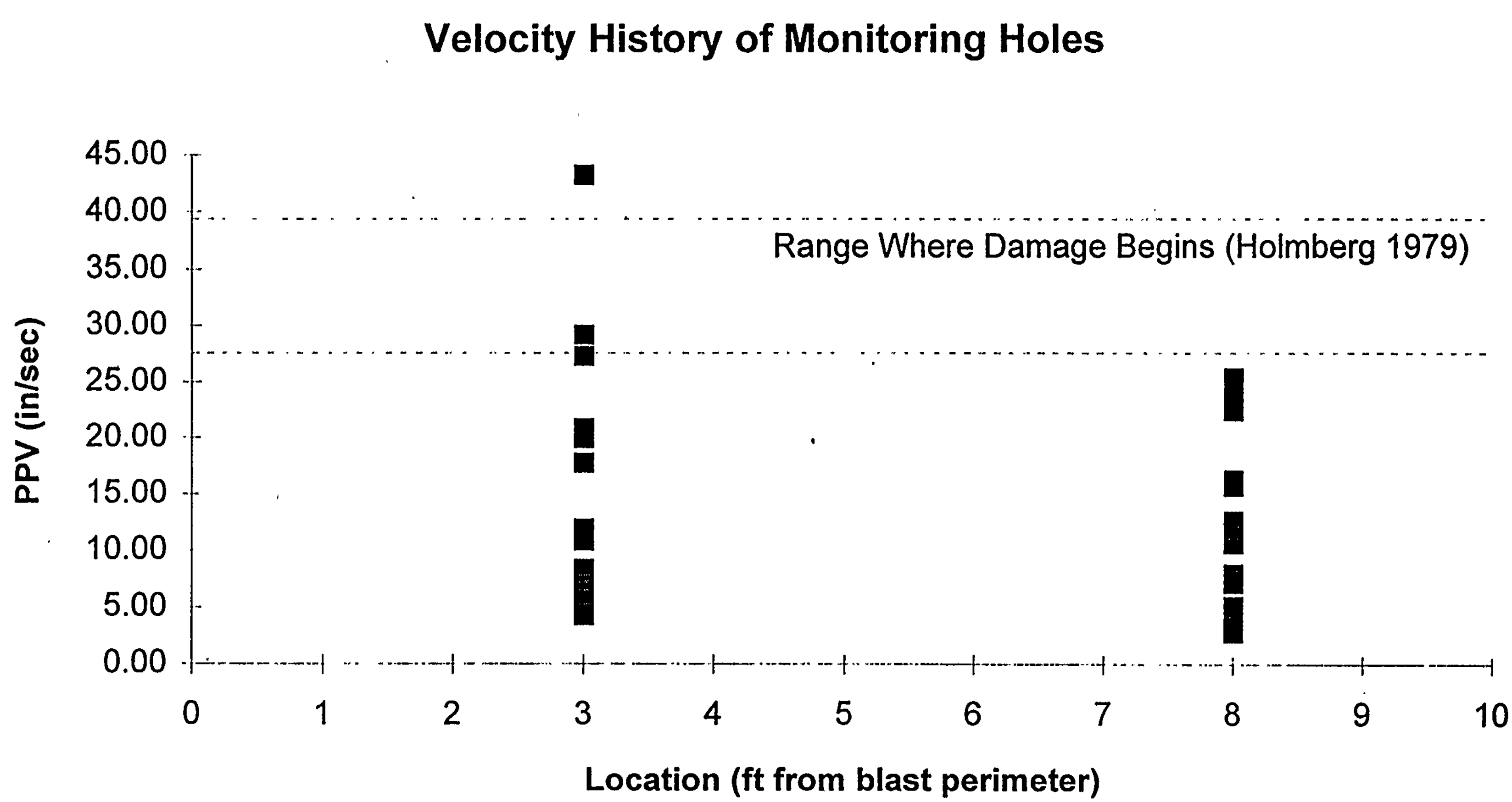


Figure 3-9. PPV at 0.91 and 2.4 m compared to range projected to cause damage by Holmberg and Persson (1979).

by increasing NRST stations (distance) beginning with the portal highwall or face. This section also presents the results of monitoring at station 0+56.3 m as the TBM excavated past the installed BHPC instruments.

3.4.1 NRST Main Tunnel Sections

3.4.1.1 Station 0+00 m—Portal Face (Highwall). Data from the three rockbolt load cells installed at the portal face are presented in Figure 3-10. The rockbolts were installed untensioned and fully grouted along their lengths of 6.1 m (2612 North, 2615 South) and 3 m (2614 Center), respectively. The load cells were attached after grout cure and initial loads were set between 9.79–21.13 kN (2200–4750 lbs). The response of all three cells was characterized by load bleed-off until July 31, 1993, when excavation of the NRST bench was begun. A small increase in load, possibly associated with bench excavation, was observed in 2612 North. This was immediately followed by continued load bleed-off which stabilized around January 1994. Since that time, 2612 North and 2614 South have exhibited long-period oscillations with the load increasing and decreasing within a bracket of roughly 6.67 kN (1500 lbs).

The rockbolts were manufactured from high-strength steel with specified yield loads of 159.9 kN (36,000 lbs) and ultimate load of 239.9 kN (54,000 lbs) for the #7 bolt. Yield load for the #9 bolts was 267 kN (60,000 lbs) and ultimate load was 356 kN (80,000 lbs). The maximum load observed at installation was approximately 21.1 kN (4,750 lbs) or 8% of yield load. Load changes observed at the head of the bolts since installation indicate all stresses at the head of the bolt are currently less than the yield stress.

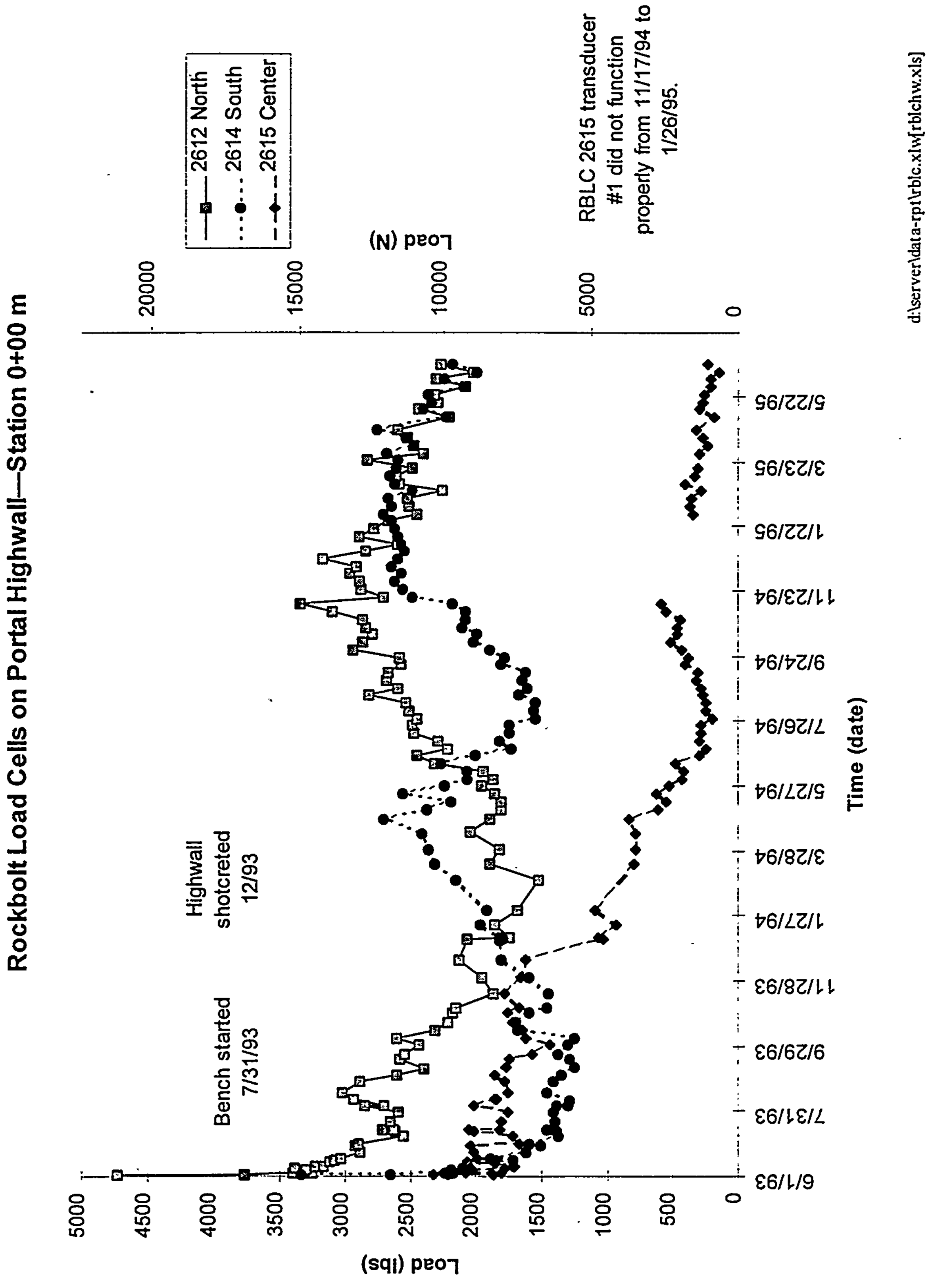


Figure 3-10. Rockbolt load versus time—station 0+00 m, portal face (highwall).

3.4.1.2 Portal Lattice Girder Convergence—Stations 0+4.6 m and 0+9.1 m.

Lattice girder convergence history is presented in Figures 3-11 and 3-12 for girders #4 and #7, respectively. Convergence at both stations exhibited similar characteristics and magnitudes. The change in the horizontal chords (SLL-SLR) indicated generally increasing closure with time from installation to August 1994, then stabilizing beyond this time. The horizontal chords were established at installation before excavation of the bench. Closure stabilized immediately after installation, but then began to increase with excavation of the bench in August and September 1993. Closure of the horizontal chords exhibited another period of stability after removal of the bench was completed, then another sharp increase to the currently indicated plateau.

The floor pins were reestablished after removal of the bench and embedment of the lattice girders in shotcrete. Roof-to-floor closure oscillated with no real trend until roughly December 1994, at which point both stations indicated an increase in the roof-to-floor chord lengths. This period of time corresponds to movement of the TBM into the tunnel and sumping in of the cutter head. These construction activities may have resulted in settlement displacements in the floor that account for the apparent expansion in the vertical direction. The trend of the latest three measurements at lattice girder #4 has been increased roof-to-floor closure at rates between 0.009 mm/day (0.00035 in/day) and 0.0011 mm/day (0.00043 in/day); similar values occur for lattice girder #7.

3.4.1.3 Tunnel Convergence—Station 0+10.7 m. Convergence data for station 0+10.7 m in the main tunnel is presented in Figure 3-13. This station is just beyond the lattice girders in rock supported by shotcrete and rockbolts. The pattern of convergence is similar to both lattice girder stations. The horizontal chord indicates steady closure up to 4.4 mm

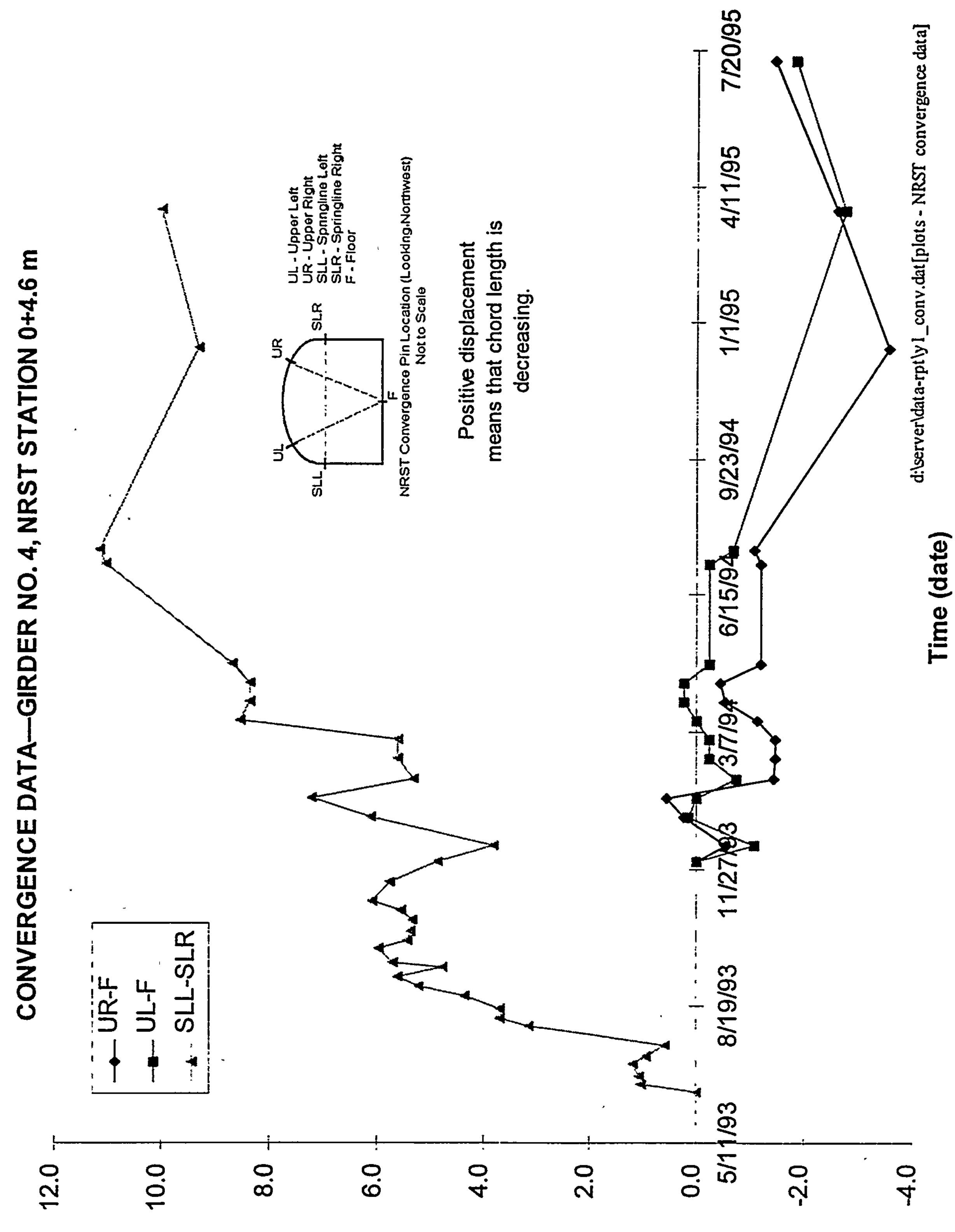


Figure 3-11. Convergence versus time for lattice girder No. 4 at NRST station 0+4.6 m.

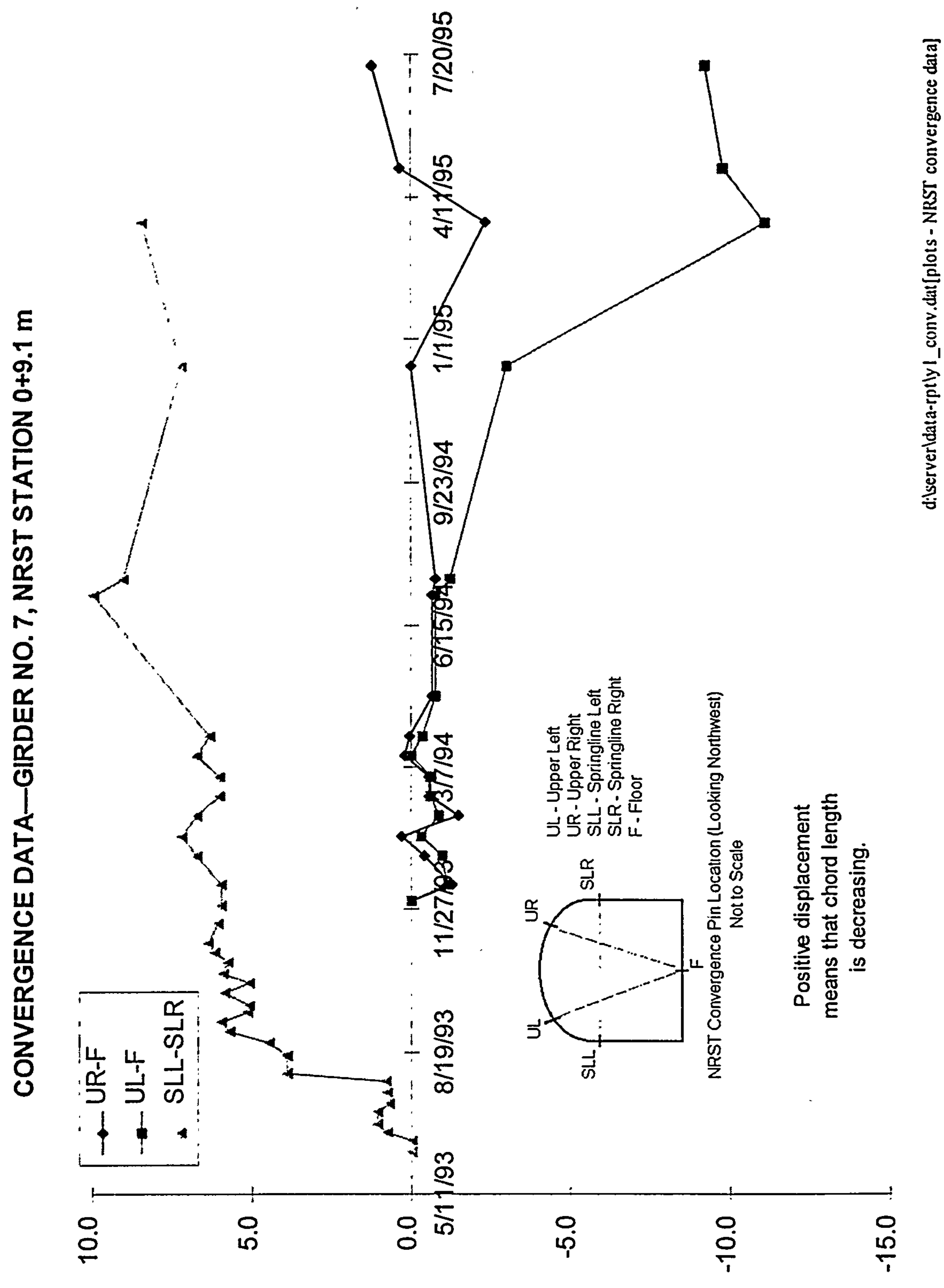


Figure 3-12. Convergence versus time for lattice girder No. 7 at NRST station 0+9.1 m.

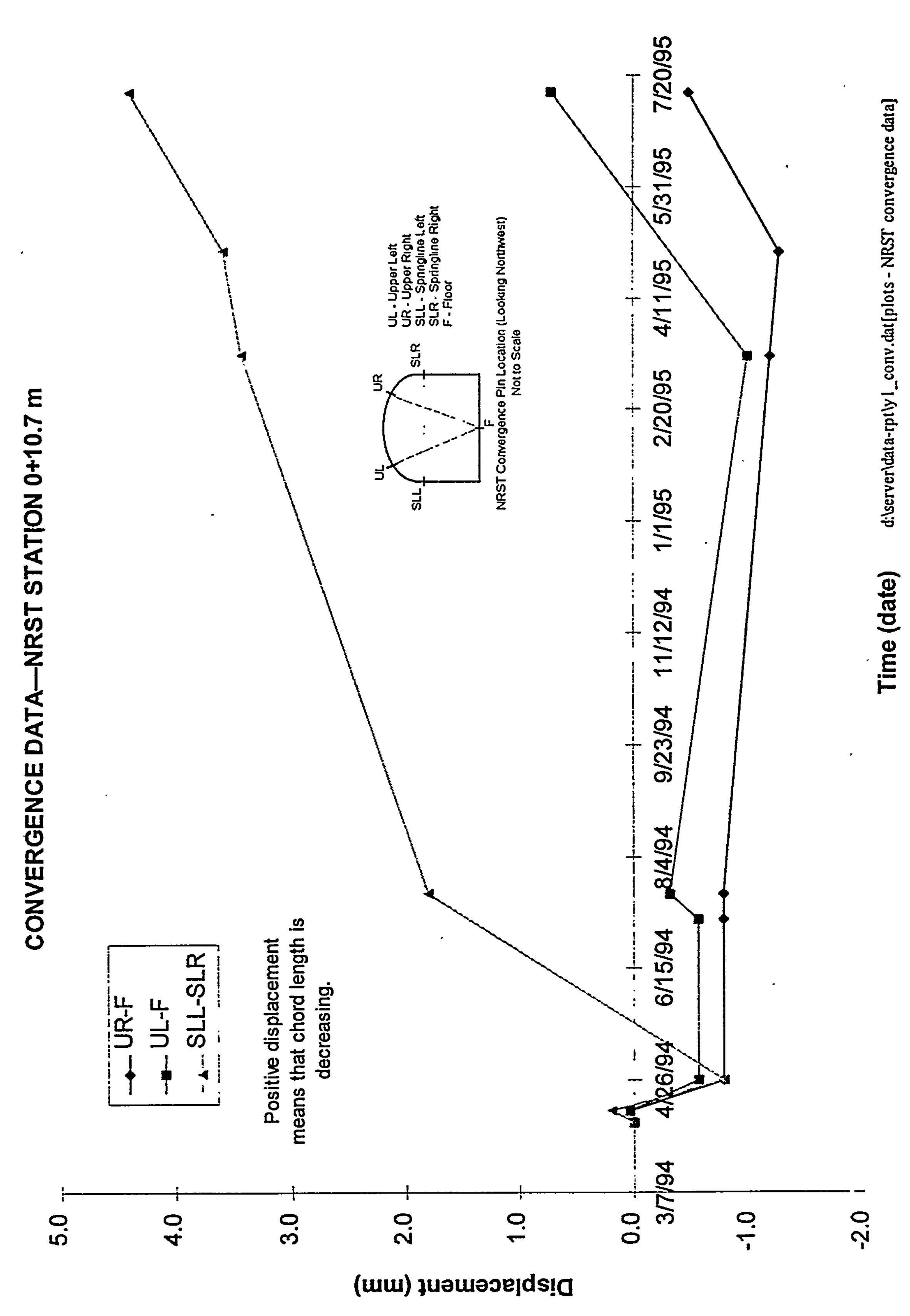


Figure 3-13. Convergence versus time—NRST station 0+10.7 m.

(0.173 in) and a closure rate of 0.0114 mm/day (0.0004 in/day). Roof-to-floor chords indicated very small expansion, 1.3 mm maximum, then a change in direction to tunnel closure in the most recent reading with closure rates of 0.011 (0.0004 in/day) and 0.014 mm/day (0.0006 in/day).

3.4.1.4 Convergence and Rockbolt Load—NRST Station 0+16.8 m.

Convergence data for station 0+16.8 m are presented in Figure 3-14 and indicate behavior very similar to the lattice girders and station 0+10.7 m. All chords were established in November 1993 after installation of the final ground support. The horizontal chord indicates a trend of continued closure with one period where closure rates dropped to zero. The current maximum horizontal closure had reached 7.1 mm (0.279 in) by July 1995 with the closure rate equal to 0.009 mm/day (0.0004 in/day). The roof-to-floor chords showed very small expansions up to the time that the TBM was launched. In late December 1994, the roof-to-floor chords indicated a further expansion (up to a maximum –3.6 mm (–0.14 in) that may be associated with settlement of the floor due to the TBM loading. The roof-to-floor chords have indicated closure since January 1995 with a maximum change of 2.8 mm (0.111 in) and closure rates of 0.020 mm/day (0.0008 in/day) and 0.010 mm/day (0.0004 in/day) for UL-F and UR-F, respectively.

Rockbolt loads for station 0+16.8 m are presented in Figure 3-15. The load history of all three instruments indicate initial load bleed-off followed by very stable load-time behavior. All bolts were untensioned, fully grouted, and pull tested to levels at least 111.2 kN (25,000 lbs) prior to installation of the load cells. The peak bolt load generated by torquing the nut attaching the load cell at the bolt head was 31.6 kN (7100 lbs) or 11.8% of yield strength.

3.4.1.5 Convergence and Rockbolt Load—NRST Station 0+27.4 m.

Convergence data for station 0+27.4 m are presented graphically in Figure 3-16. There is considerable scatter in the data up to July 1994; however, the general character of the curves are

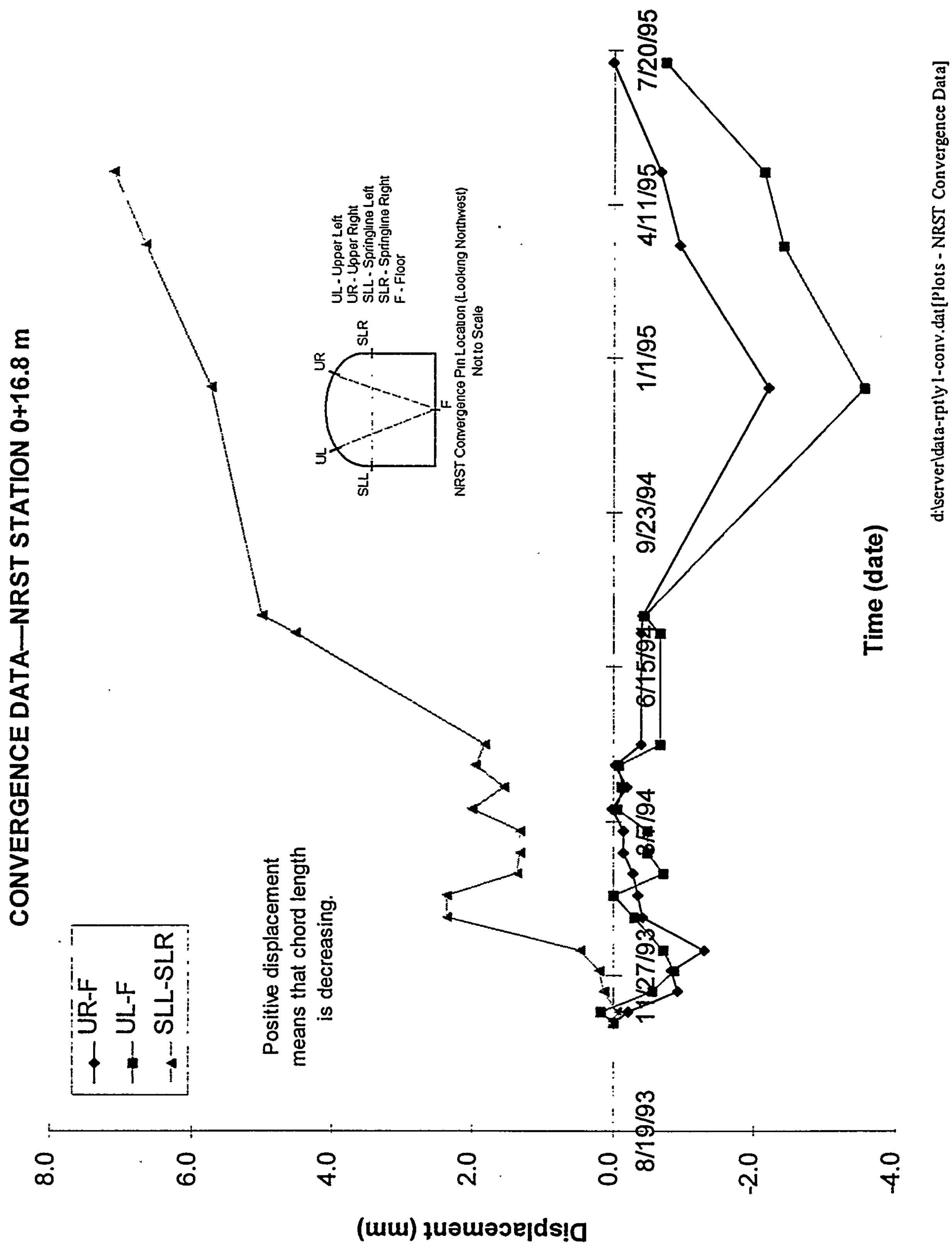
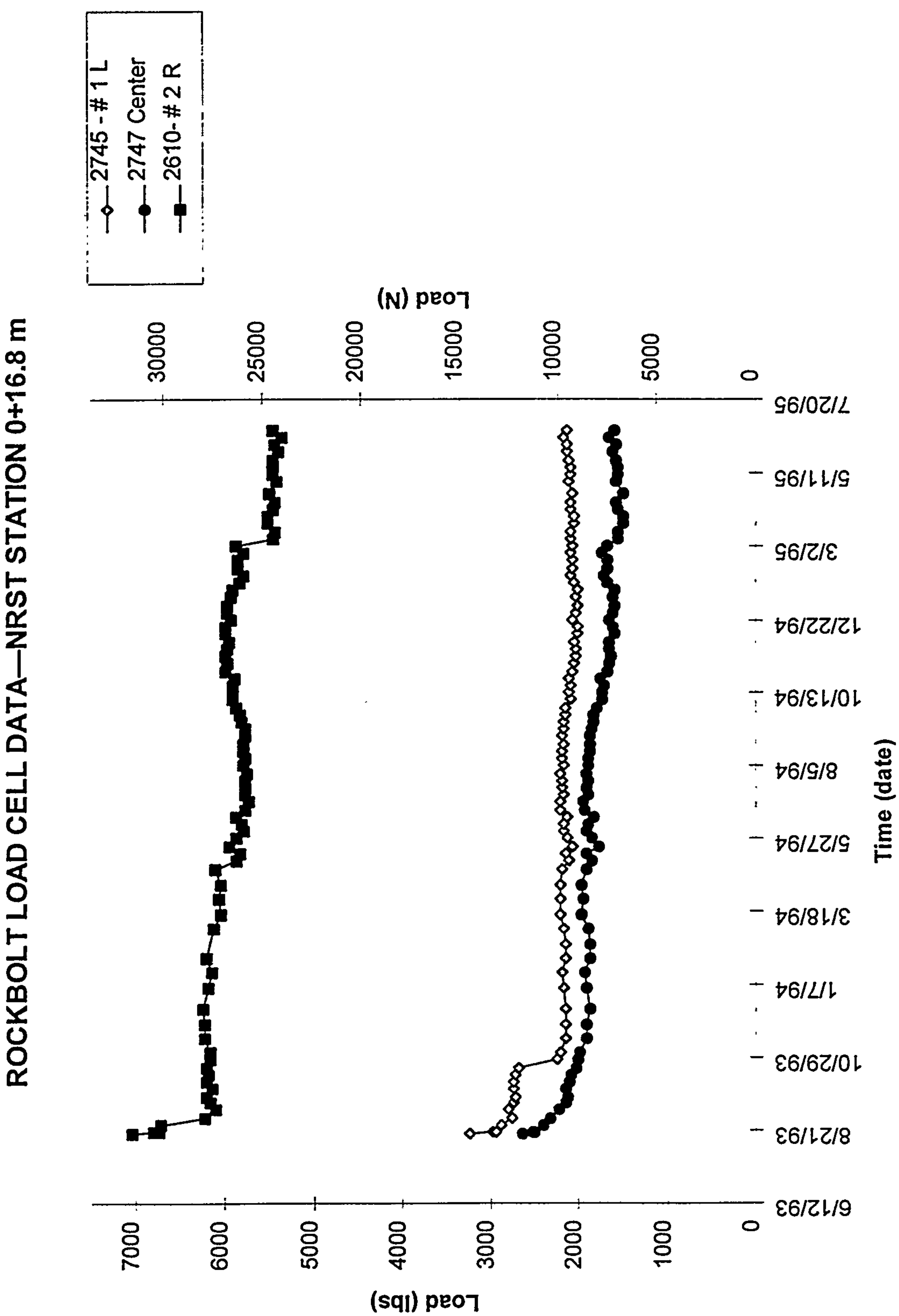


Figure 3-14. Convergence versus time—NRST station 0+16.8 m.



d:\server\data-rpt\rblc.xlw[rblc55.xls]

Figure 3-15. Rockbolt load versus time—NRST station 0+16.8 m.

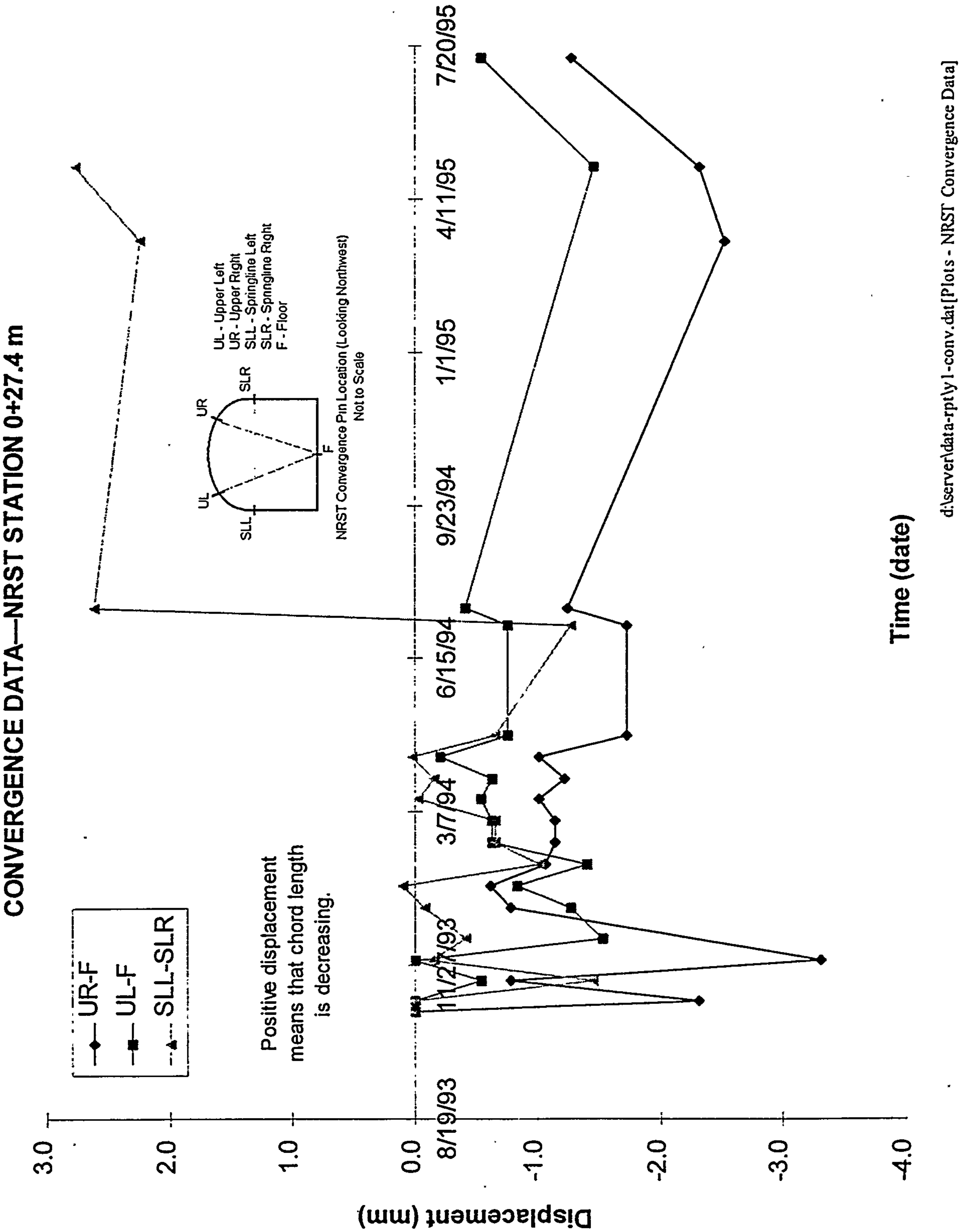


Figure 3-16. Convergence versus time—NRST station 0+27.4 m.

similar to trends at the lattice girders and at stations 0+10.7 m and 0+16.8 m. The horizontal chord indicates a maximum closure of 2.8 m (0.109 in). Peak rates of closure were 0.045 mm/day (0.0018 in/day); however, closure rates have stabilized since mid-June 1994. The roof-to-floor chords show considerable scatter and general expansion up to a peak value of –2.5 m (–0.1 in). This trend changes to closure after January 1995 with rates up to 0.015 mm/day (0.0006 in/day) in July 1995.

Rockbolt load at the head of the grouted bolts is presented in Figure 3-17. Installation loads generated by torquing the nut against the load cell ranged between 27.1 kN (6100 lbs) and 33.6 kN (7550 lbs). After initial load bleed-off, the bolts have been generally stable or have shown slight reductions in load. Peak load at the bolt head is less than 11.2% of the yield load.

3.4.1.6 Convergence and Rockbolt Load—NRST Station 0+33.5 m.

Convergence data for station 0+33.5 m is presented graphically in Figure 3-18. The convergence history is similar to stations 0+4.6 m, 0+9.1 m, 0+10.7 m, 0+16.8 m, and 0+27.4 m with the exception that the roof-to-floor chords indicate continued expansion. Peak closure on the horizontal chord had reached 3.7 mm (0.146 in), but closure rates were near zero at the last reading in mid-March 1995. Roof-to-floor chords have reached a maximum expansive deformation of 3.4 mm (–0.133 in).

Rockbolt load cell data are presented graphically in Figure 3-19. Installation loads varied from 29.3 to 41.4 kN (6580 to 9320 lbs). Load histories have been generally stable with slightly increasing trends in the #3R and #1R locations. Peak installation loads were less than 15.5% of the yield load at the bolt head.

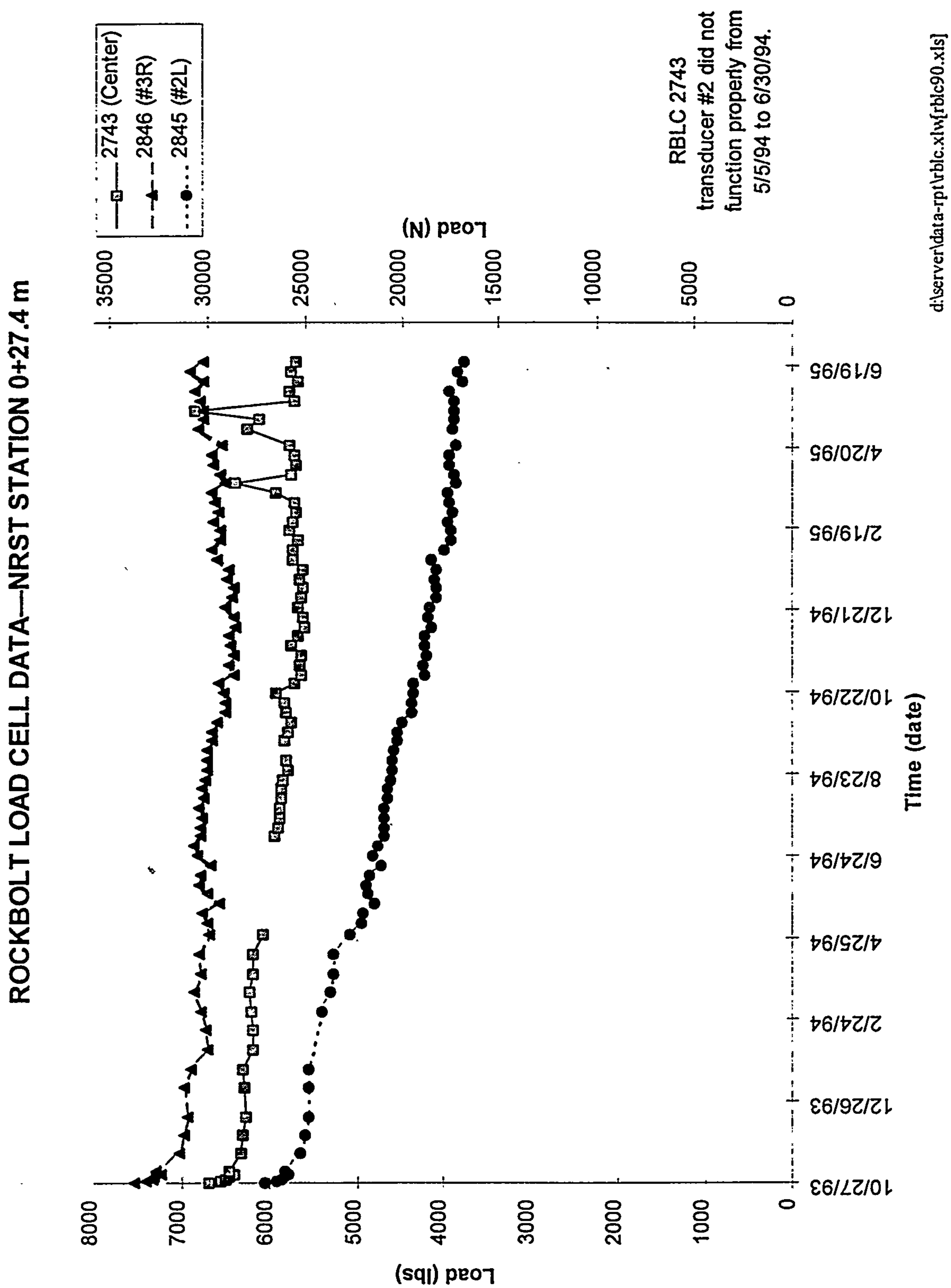


Figure 3-17. Rockbolt load versus time—NRST station 0+27.4 m.

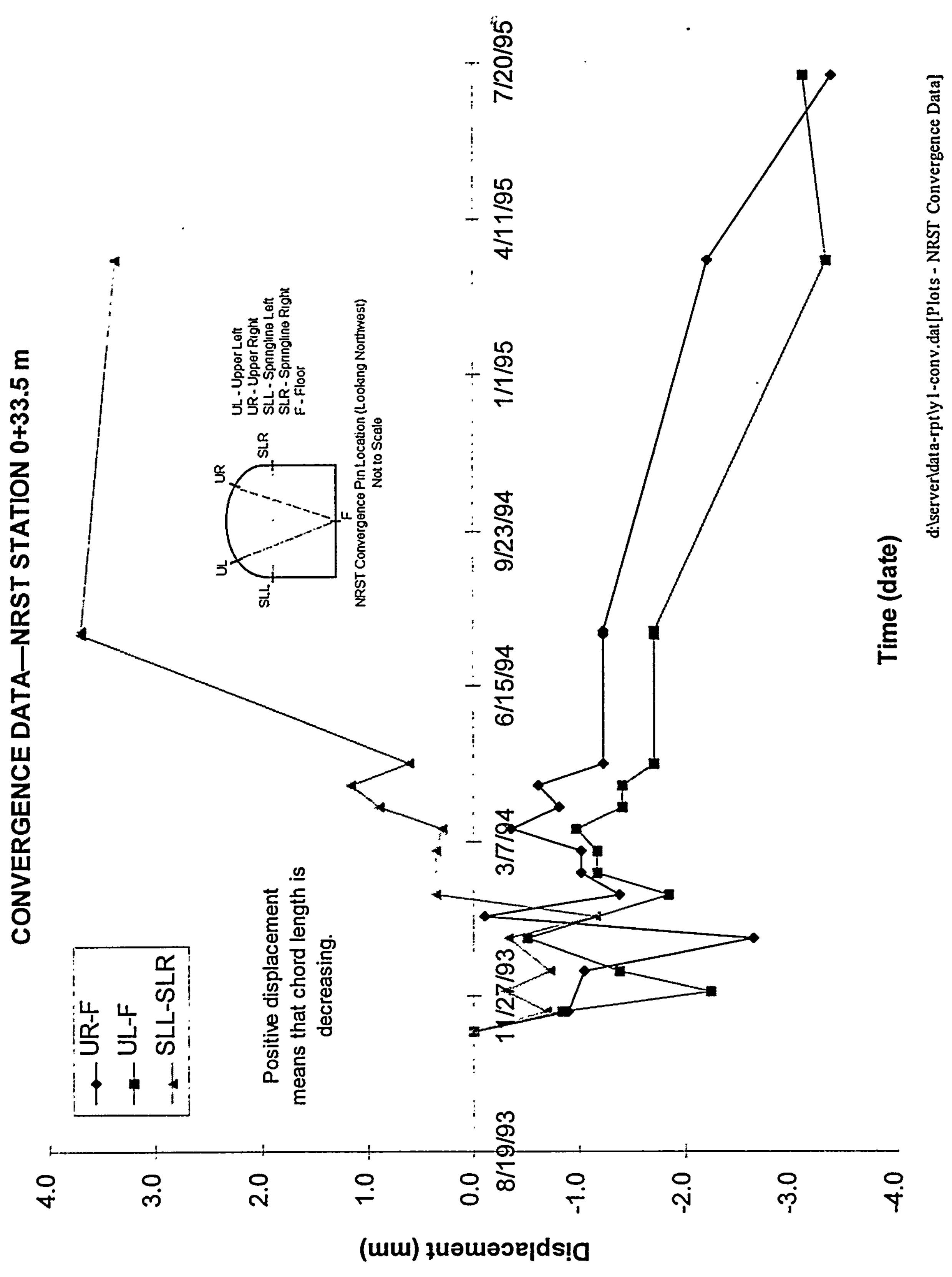


Figure 3-18. Convergence versus time—NRST station 0+33.5 m.

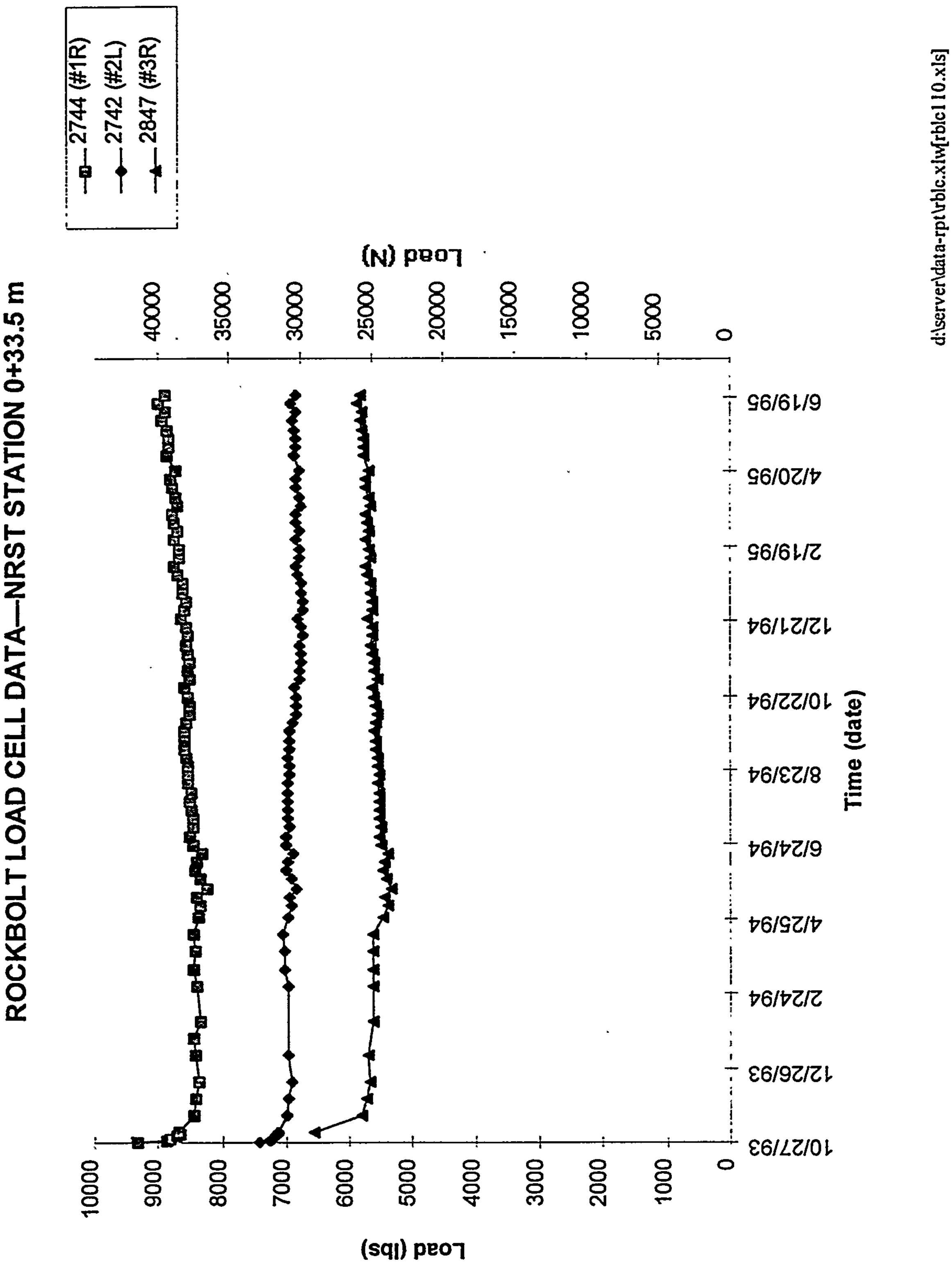


Figure 3-19. Rockbolt load versus time—NRST station 0+33.5 m.

3.4.1.7 Displacement and Rockbolt Load—NRST Station 0+42.7 m.

Convergence data for station 0+42.7 m is presented in Figure 3-20. The arrangement at 0+42.7 m was slightly different because of the intersection with Alcove No. 1. Roof-to-floor chords have the same arrangement, however, two horizontal chords were measured to convergence pins on the west and east sides of the alcove entrance. The pattern of convergence is similar to the previous stations, with the horizontal chords (SLL-A1L, SLL-A1R) showing closure and reaching an apparent plateau with increasing time. Maximum closure had reached 8.4 mm (0.332 in) before reversing direction. Closure rates at March 1995 were a maximum of 0.025 mm/day (0.001 mm/day). Roof-to-floor chords showed general stable behavior until the July 1995 reading with indicated closure rates of 0.024 mm/day (0.0009 in/day) and –0.038 mm/day (–0.0015 in/day) for UL-F and UR-F, respectively.

Data from the five-point MPBX installed vertically at station 0+42.7 m is presented in Figure 3-21. The initial pattern of displacement change was typical, with the deepest anchor showing the greatest change and the shallowest anchor showing the least change. The displacements indicated closure, or movement, of the rock surface into the excavation. All gages abruptly reversed direction around February 15, 1994, and the pattern of change became irregular in all but the anchor at 1.7 m which showed a consistent pattern of closure at a very low rate. The maximum change as of June 1995 was 0.60 mm (0.024 in) recorded at the shallowest anchor. Seven-day interval displacement rates are shown graphically in Figure 3-22, and indicate an oscillatory pattern around zero with relatively low rates of change (0.004 mm/day, 0.0002 in/day) in June 1995.

Rockbolt load as a function of time is shown graphically in Figure 3-23. Following the initial load bleed-off, the bolts exhibit different behaviors; however, the amount of load change is

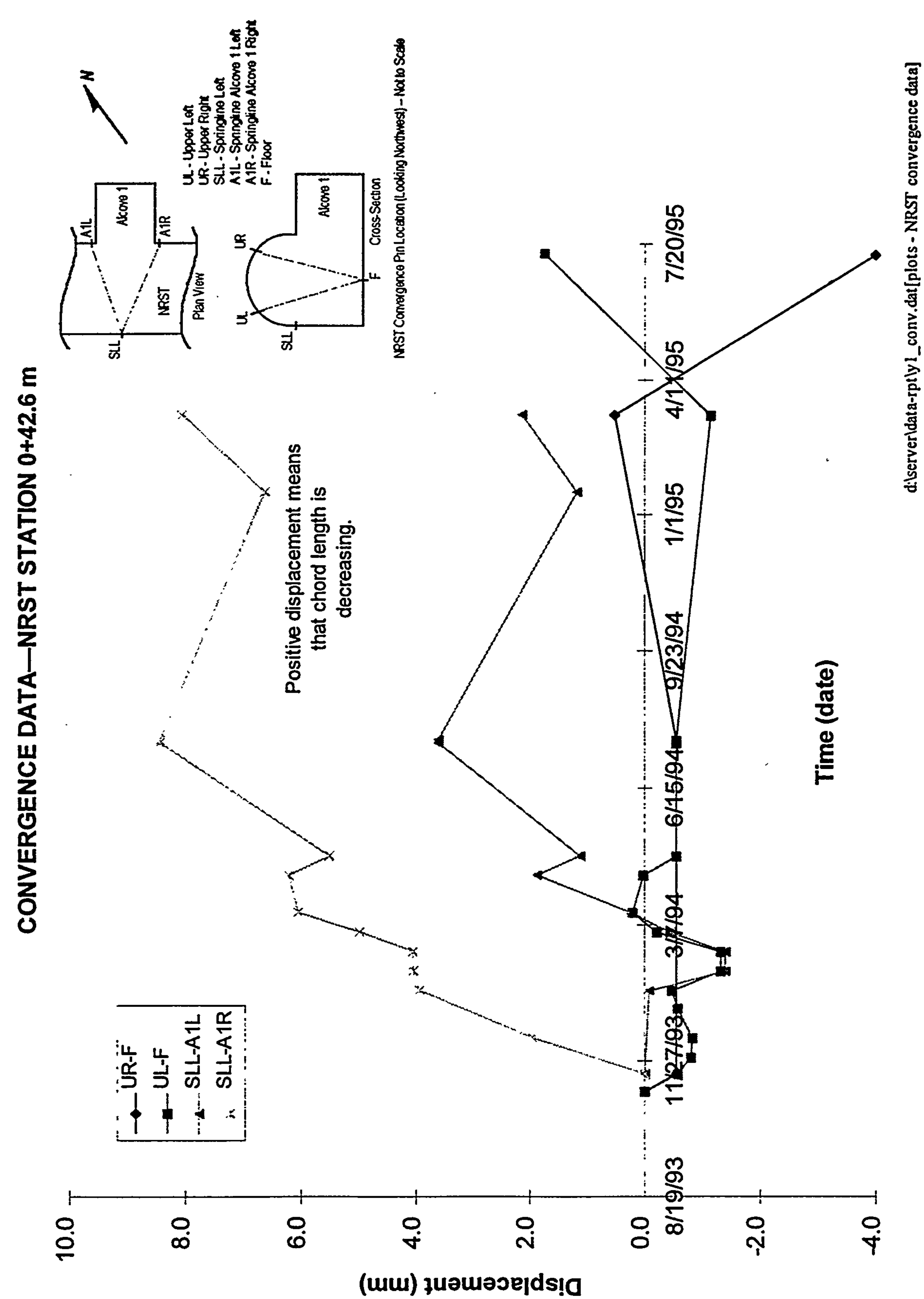


Figure 3-20. Convergence versus time—NRST station 0+42.7 m.

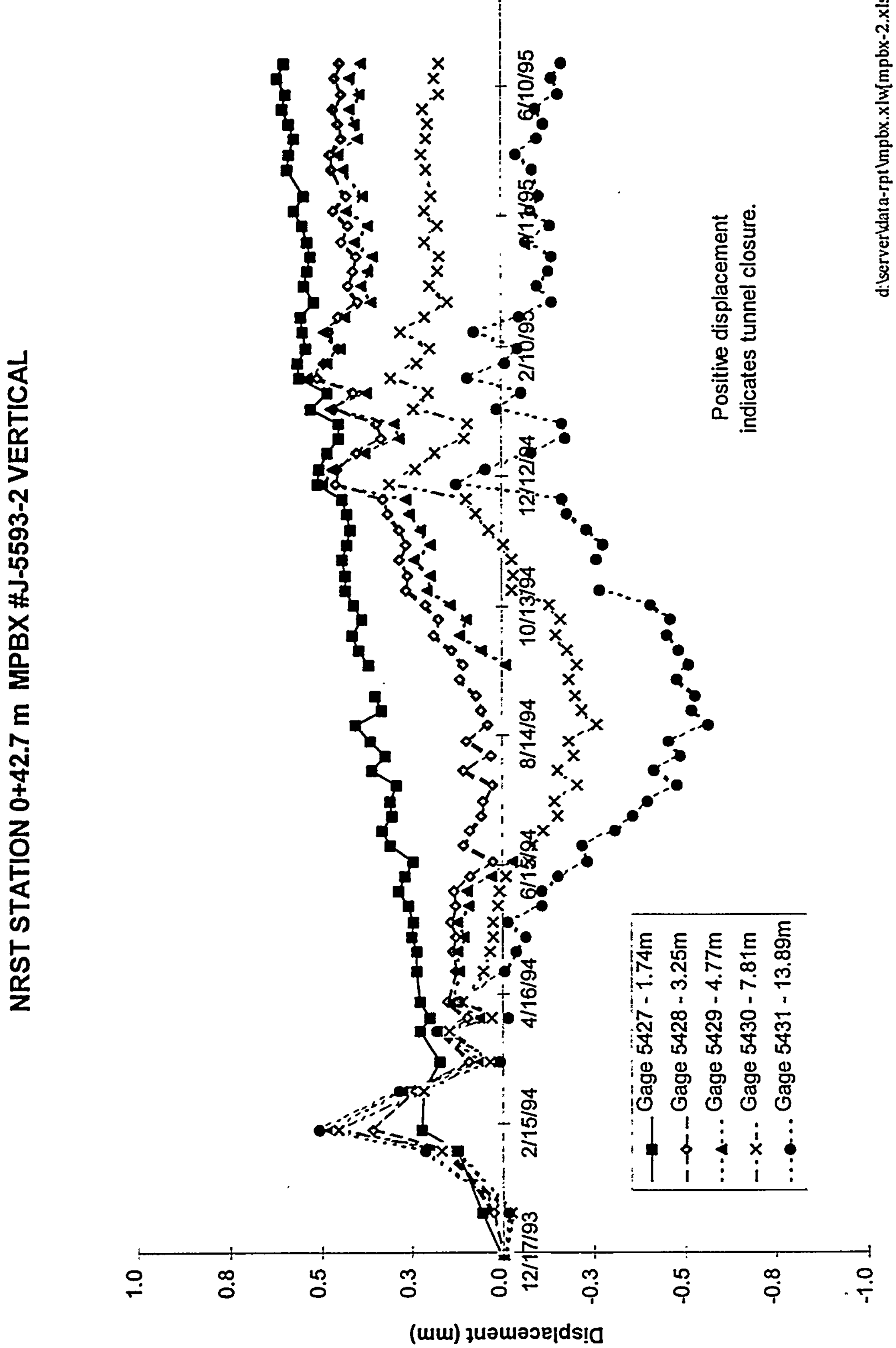


Figure 3-21. Vertical MPBX displacement versus time—NRST station 0+42.7 m.

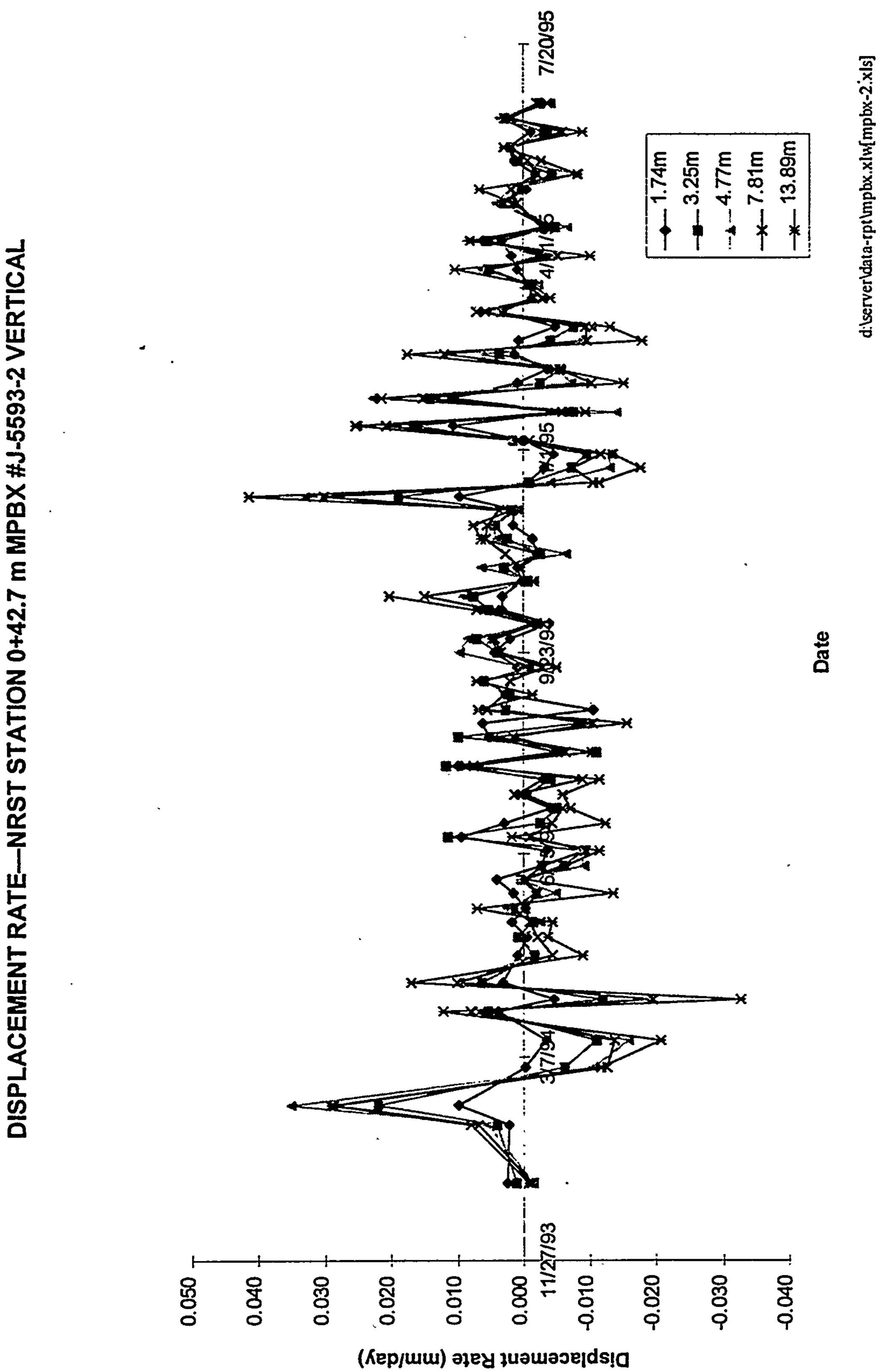


Figure 3-22. Vertical MPBX displacement rates versus time—NRST station 0+42.7 m.

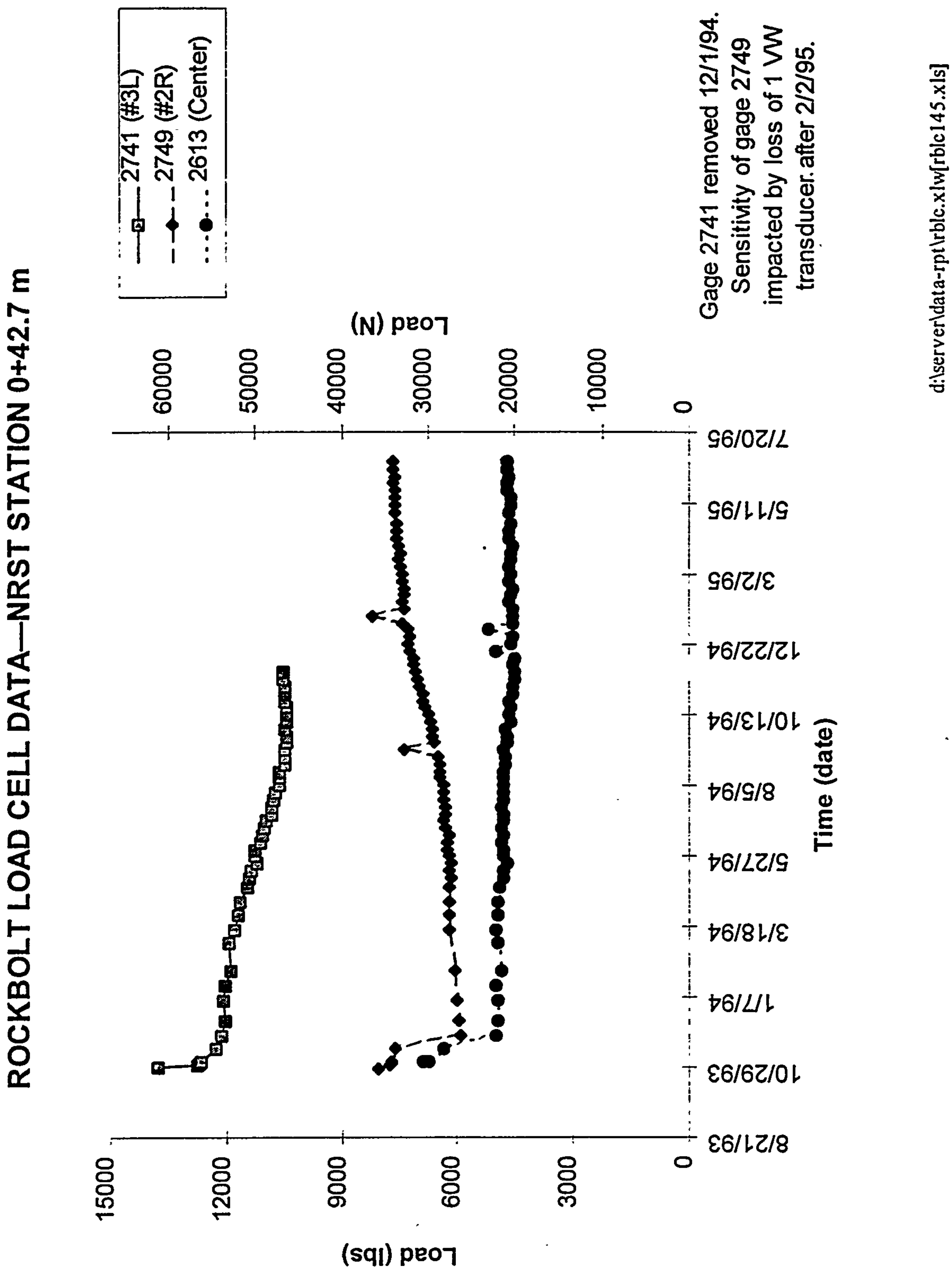


Figure 3-23. Rockbolt load versus time—NRST station 0+42.7 m.

small with regard to the bolt strength. Peak installation load was 23.0% of the bolt yield strength. Rates of change in bolt load were low.

3.4.1.8 Displacement and Rockbolt Load—NRST Station 0+56.3 m.

Convergence data for station 0+56.3 m is presented in Figure 3-24. The two roof-to-floor chords exhibit similar patterns of closure with a peak value of 3.6 mm (0.140 in). The maximum rate of closure was 0.093 mm/day (0.0037 in/day), however, closure rates as calculated from the last two readings were zero for both chords. The horizontal chord showed a maximum closure of 1.4 m (0.056 in). Currently, only the roof-to-floor chords are being measured at this station because the spring line was covered by the concrete bearing structure shown in Section 2.3.5, Figure 2-12, used to launch the TBM.

Rock displacement as measured by the vertical MPBX at station 0+56.3 m is presented in Figure 3-25. This extensometer shows a consistent pattern of displacements between the anchors at each depth, however, the anchor at 3.25 m has the greatest displacement. The displacements at 4.77 and 7.81 m are also transposed. Typically, the deeper anchors will indicate greater displacement than shallower anchors if all deformations are due to rock deforming into the tunnel. The changes exhibited in Figure 3-25 are probably due to minute movements at anchors. This may be due to the irregularity of the hole due to the poor rock quality and the use of the Borros-type hydraulic anchors.

The maximum closure displacement was small up to July 1995 with a value of 2.35 mm (0.093 in). Rates of displacement calculated for seven-day intervals, shown in Figure 3-26, were a maximum of 0.005 mm/day (0.0002 in/day) from the most recent readings in June 1995.

The displacement record for the horizontal MPBX is presented in Figure 3-27. The pattern of change is very similar to that exhibited by the vertical anchor. A special verification

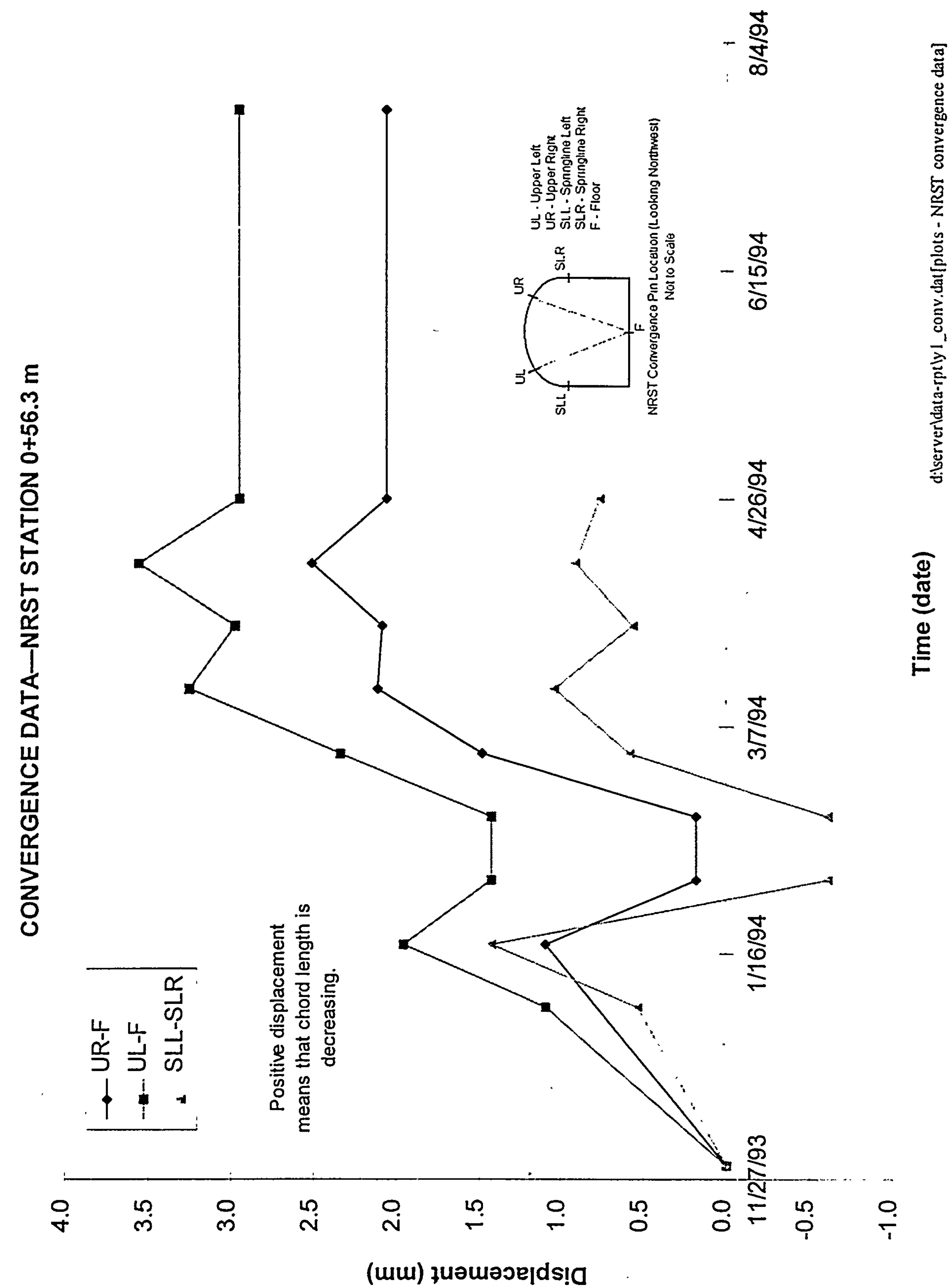


Figure 3-24. Convergence versus time—NRST station 0+56.3 m.

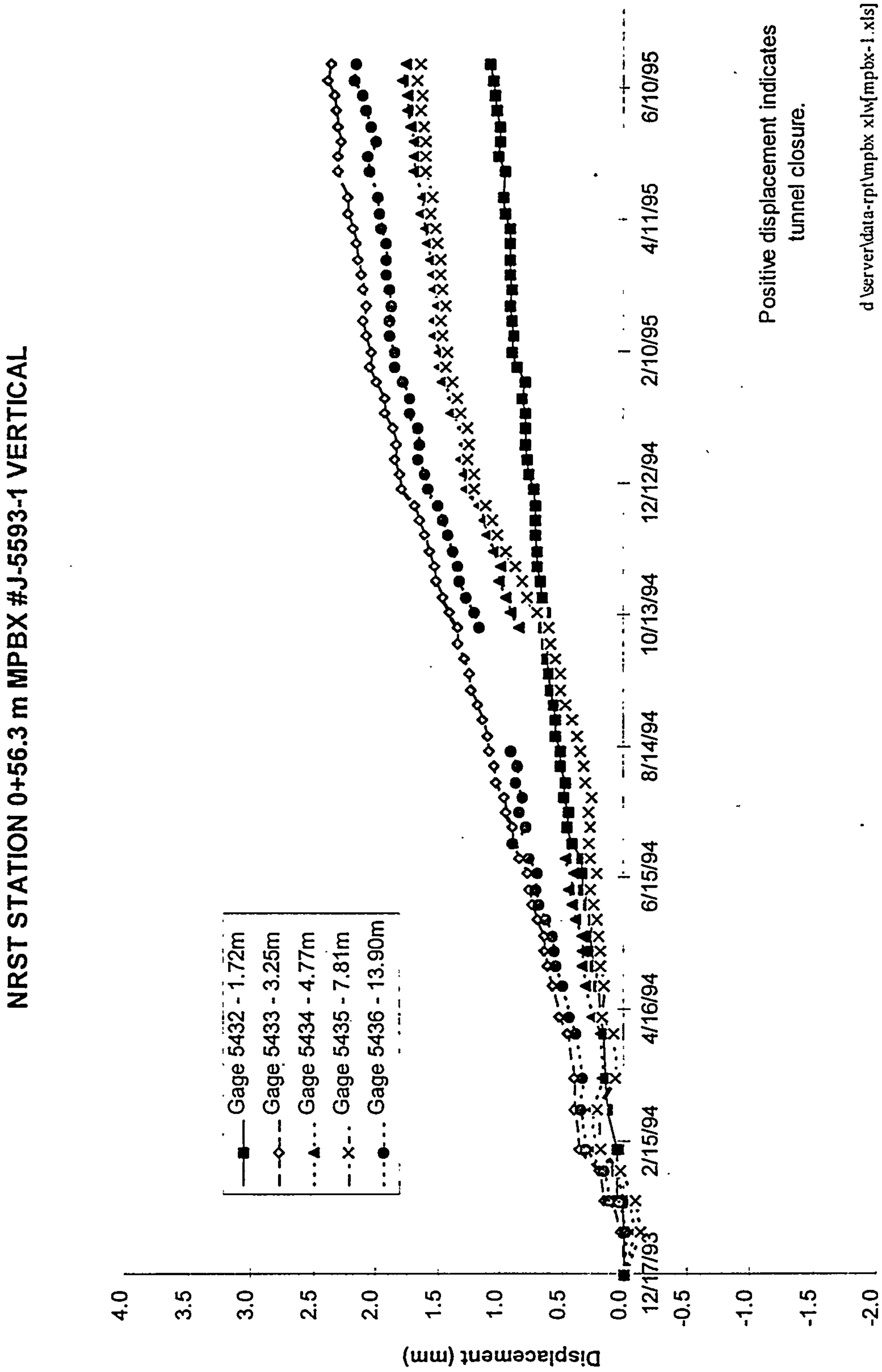


Figure 3-25. Vertical MPBX displacement versus time—NRST station 0+56.3 m.

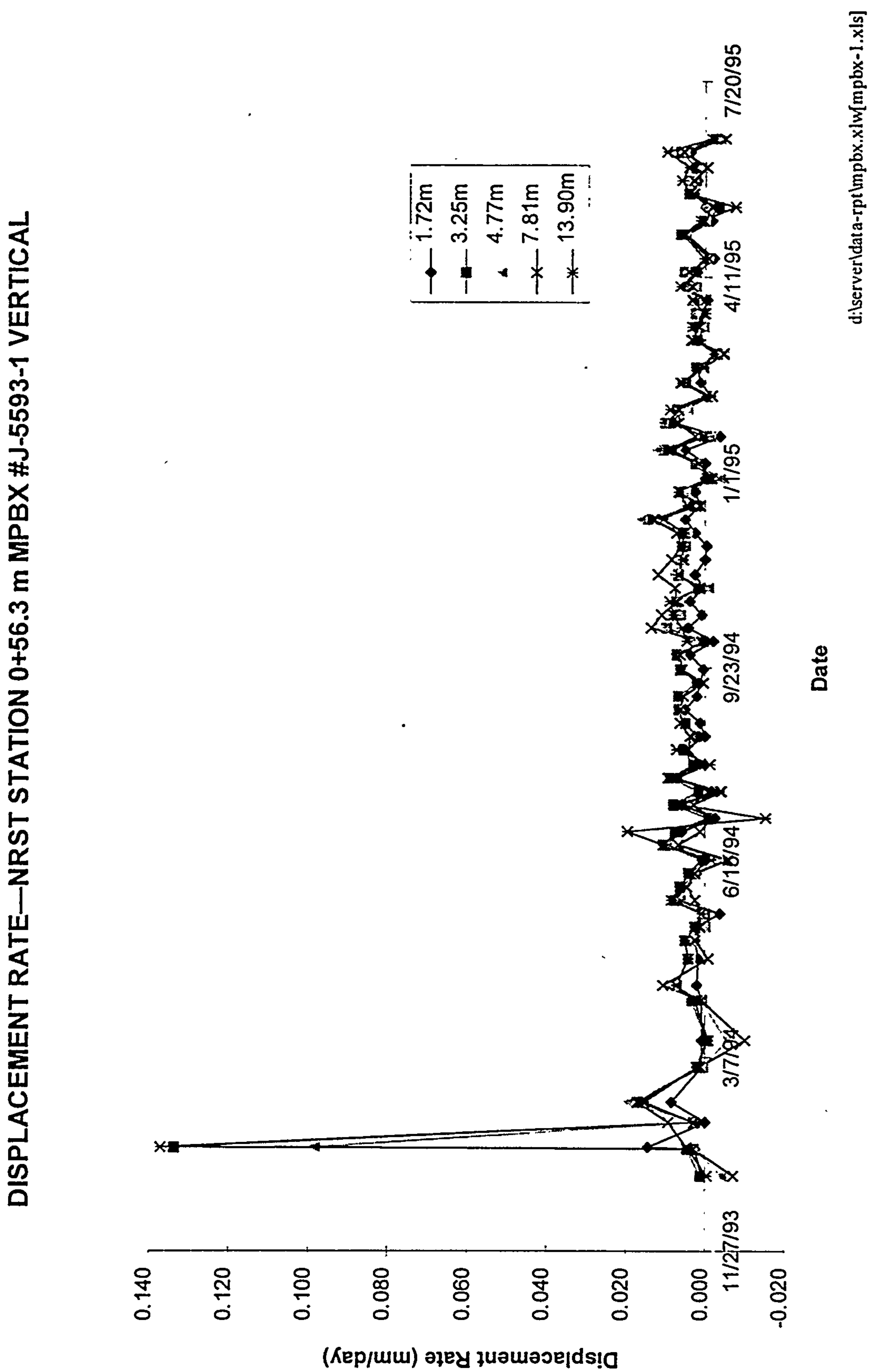


Figure 3-26. Vertical MPBX displacement rates versus time—NRST station 0+56.3 m.

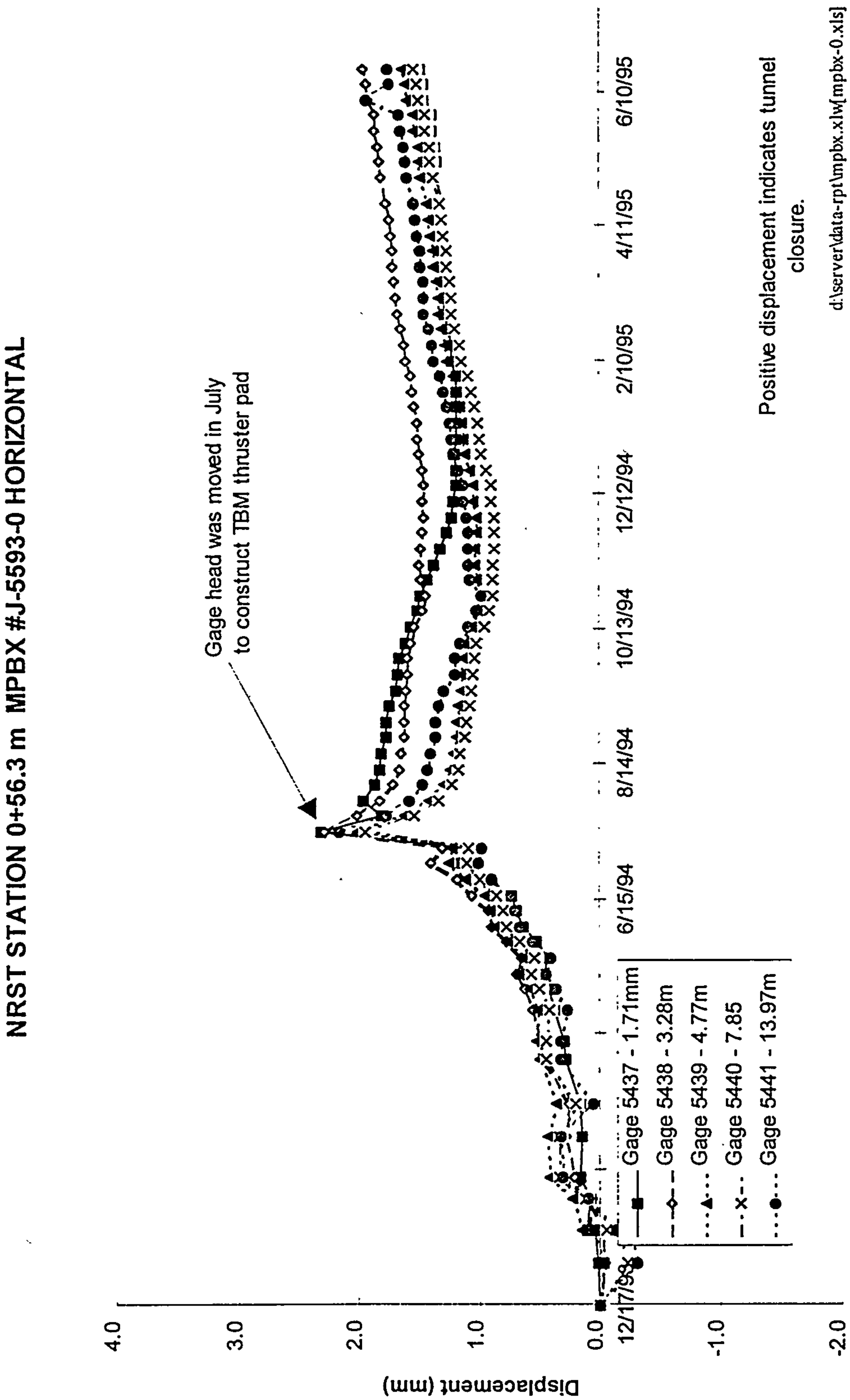


Figure 3-27. Horizontal MPBX displacement versus time—NRST station 0+56.3 m.

exercise was therefore performed on all NRST MPBX instruments to assure that the association between individual transducers and the data acquisition system (DAS) output was correct. The physical verification confirmed the records. Displacements indicated initial closure up to a maximum of 1.41 mm (0.055 in) on June 30, 1994. The peak value was followed by a small decrease in all gages suggesting stabilization on July 7, 1994. At this time, the instrument head was disassembled to install extensions to the rods because the horizontal MPBX was located within the wall where the TBM thruster pad was to be constructed. A concrete form was inserted into the thruster pad to allow access to the MPBX. The record beyond July 7, 1994 reflects the adjustment of the instruments. The displacement then decreases until the end of 1994 and shows a general trend of increasing closure through June 1995. The total displacement is relatively small with a maximum value of 1.95 mm (0.077 in).

Displacements rates were calculated for seven-day intervals and are presented in Figure 3-28. The rates have reduced with time and were a maximum of 0.0035 mm/day (0.0001 in/day) in June 1995.

Rockbolt load cells at station 0+56.3 m have all shown constant load levels with time as shown by Figure 3-29. Peak installation load was less than 16.0% of the yield load.

3.4.2 Alcove No. 1

3.4.2.1 Alcove No. 1 Convergence Stations 0+4.6 m, 0+11.3 m, 0+17.7 m and 0+24.4 m. The results of convergence measurements performed in Alcove No. 1 are presented in Figure 3-30 through 3-33. Convergence at each of the Alcove No. 1 stations display the same general pattern, with initial high rates of closure and with the closure curves tending to reverse by February 1995. The horizontal chord has shown the greatest change at all

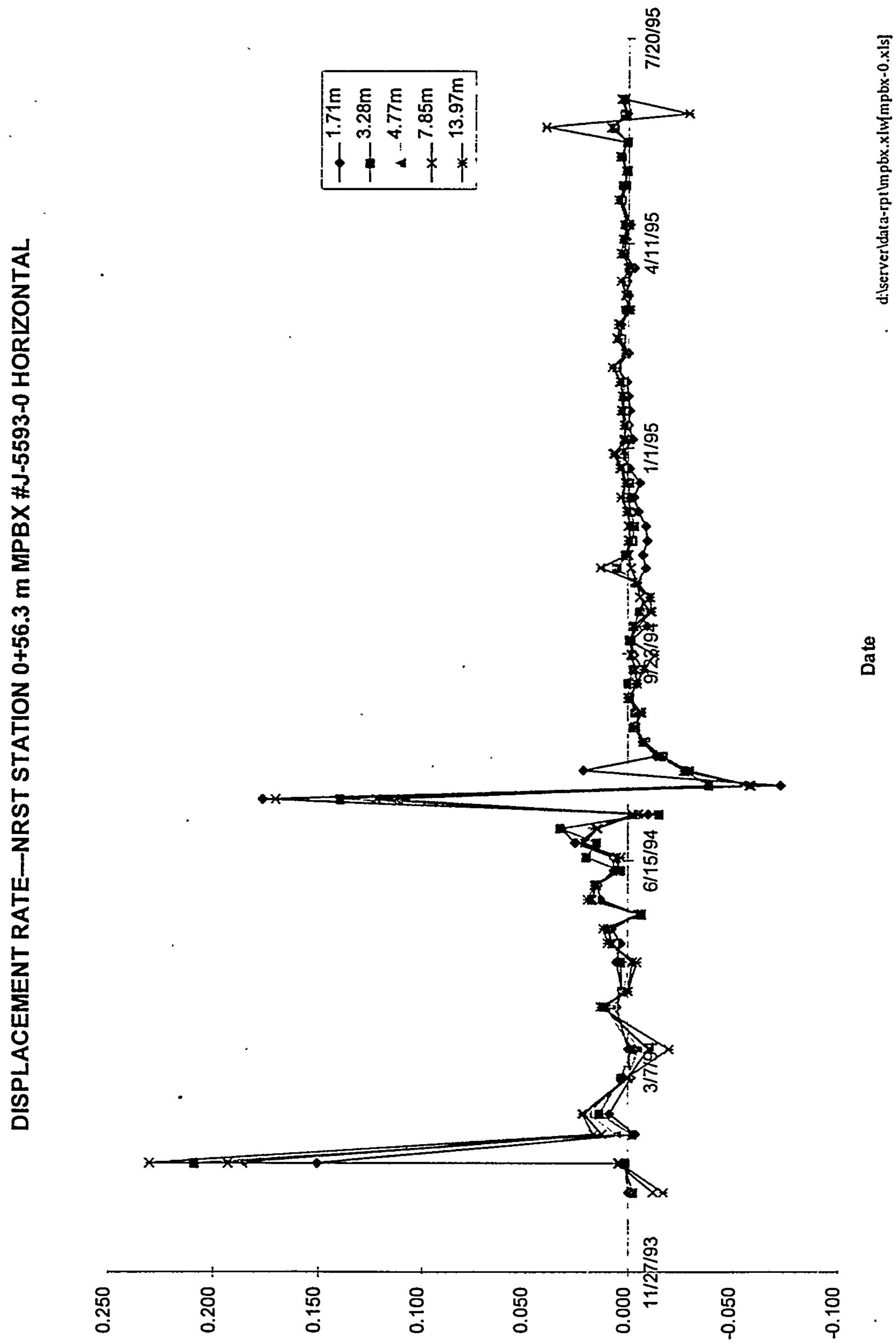


Figure 3-28. Horizontal MPBX displacement rates versus time—NRST station 0+56.3 m.

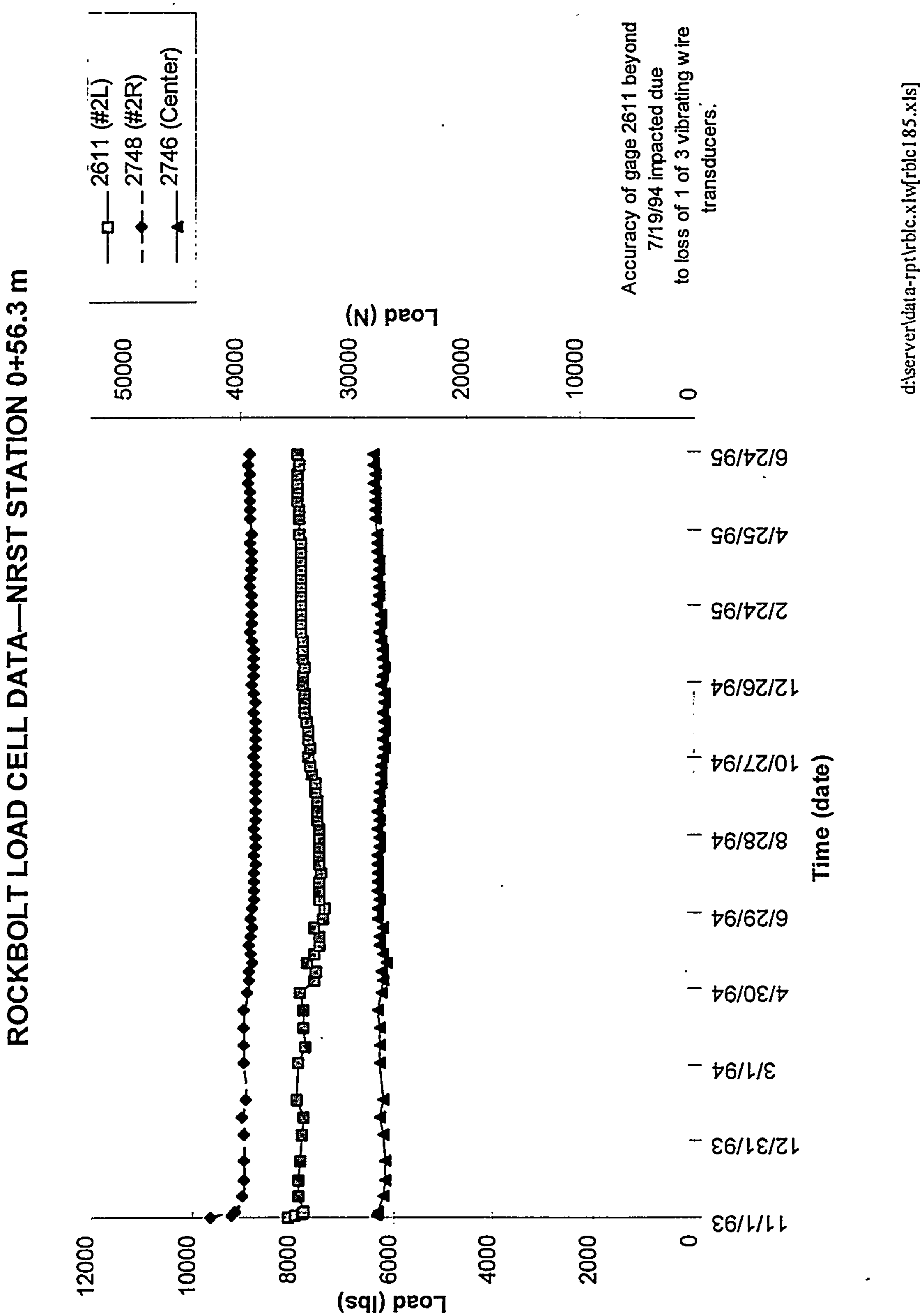


Figure 3-29. Rockbolt load at NRST station 0+56.3 m.

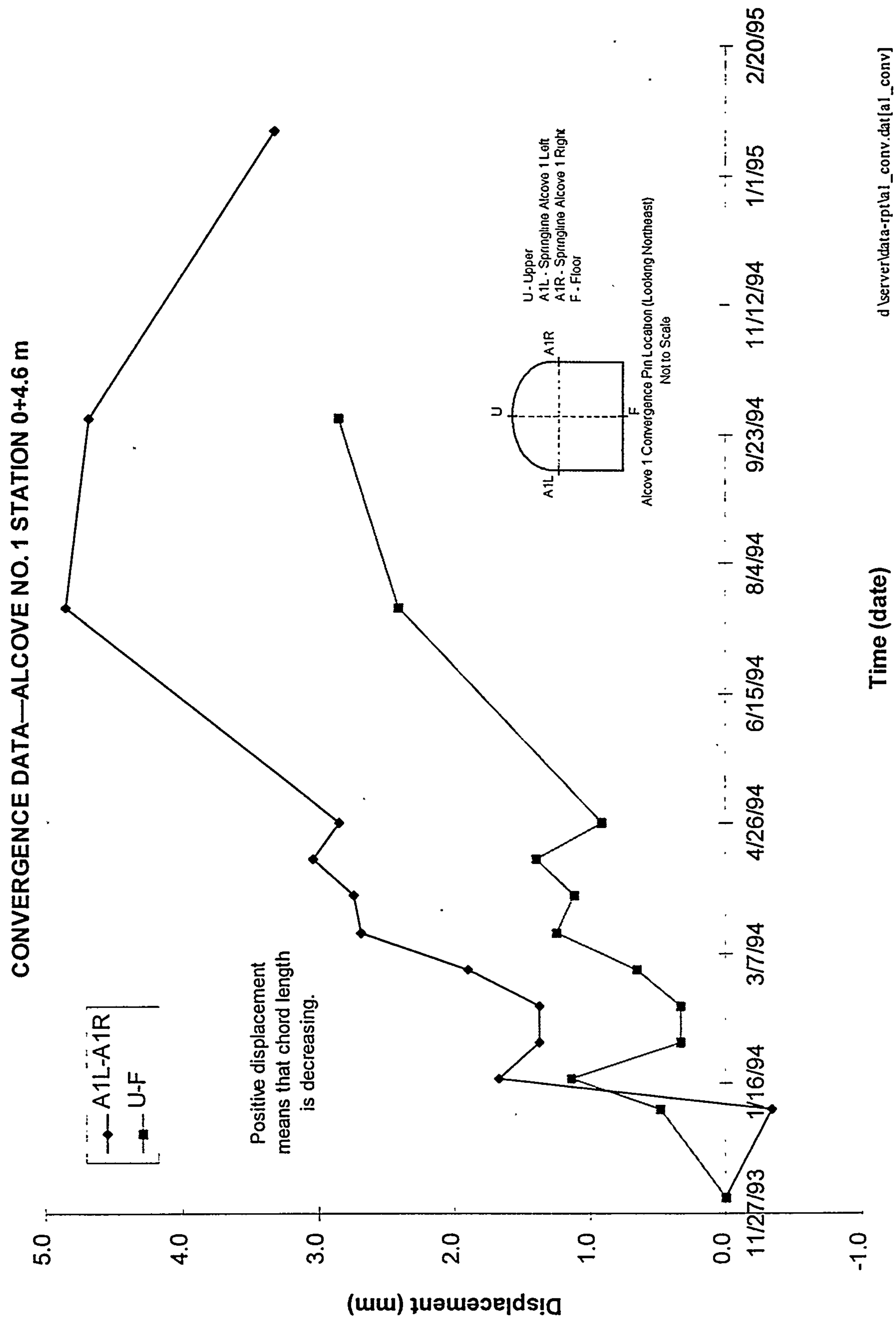


Figure 3-30. Convergence versus time—Alcove No. 1 station 0+4.6m.

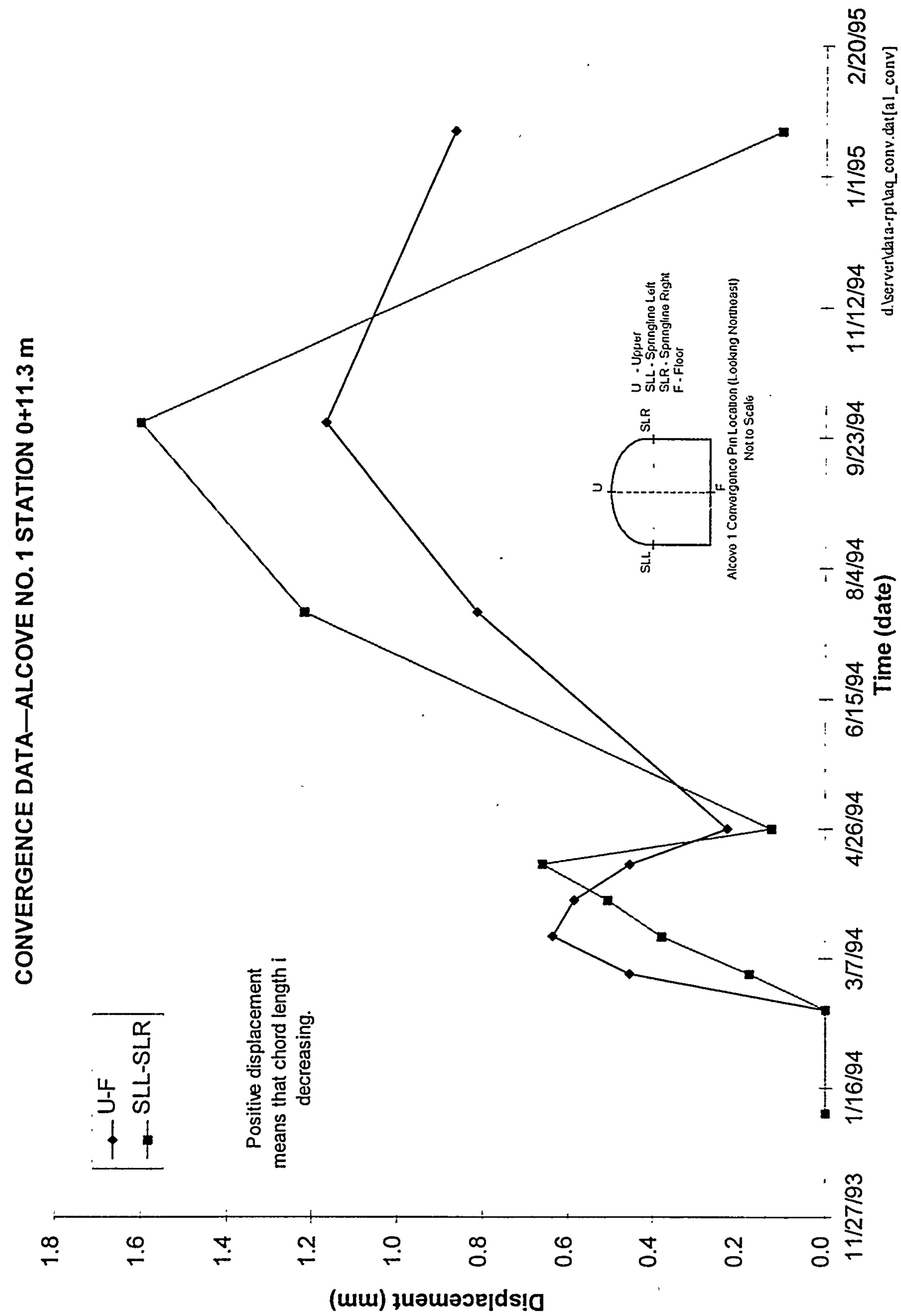


Figure 3-31. Convergence versus time—Alcove No. 1 station 0+11.3 m.

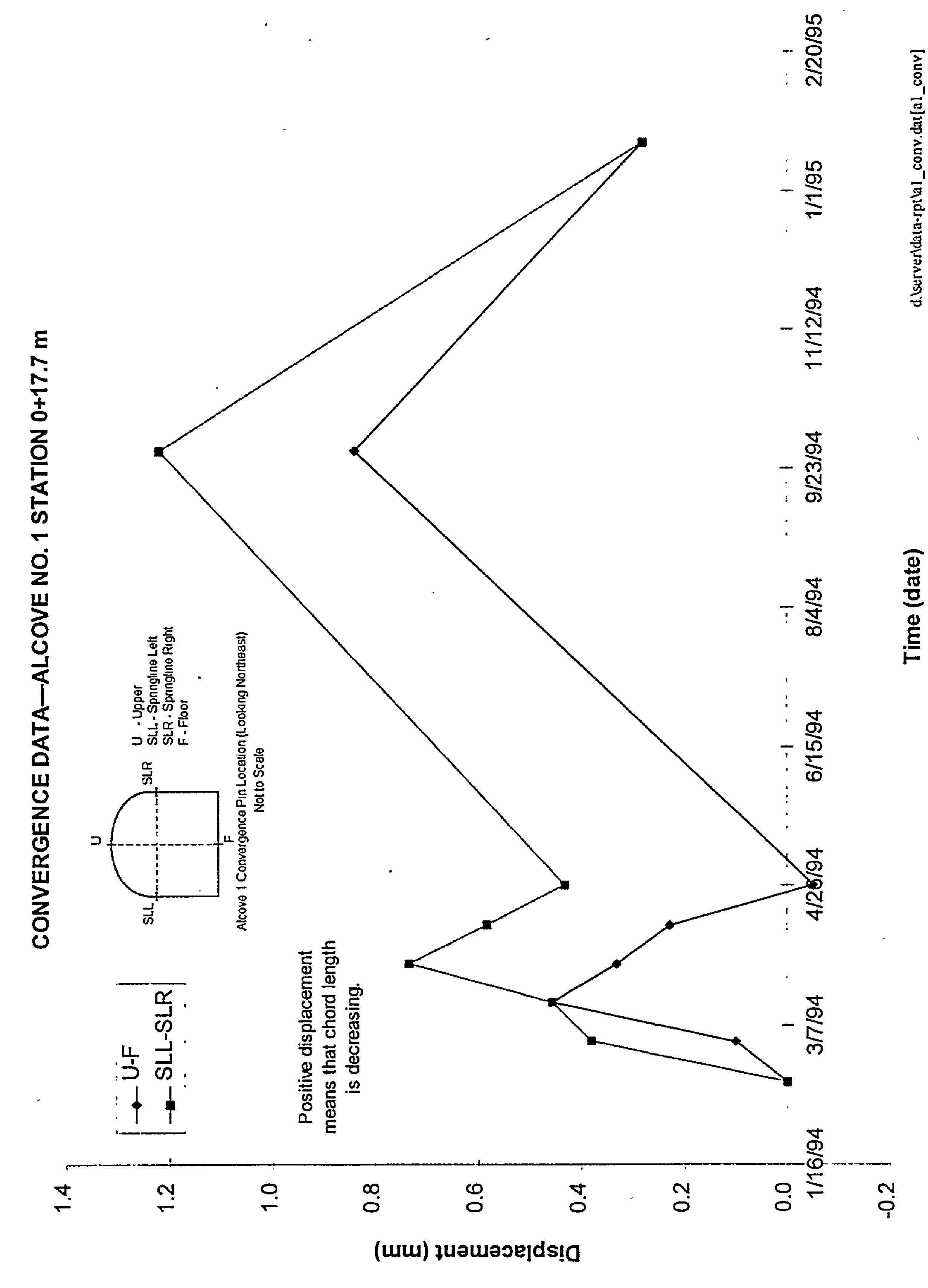


Figure 3-32. Convergence versus time—Alcove No. 1 station 0+17.7 m.

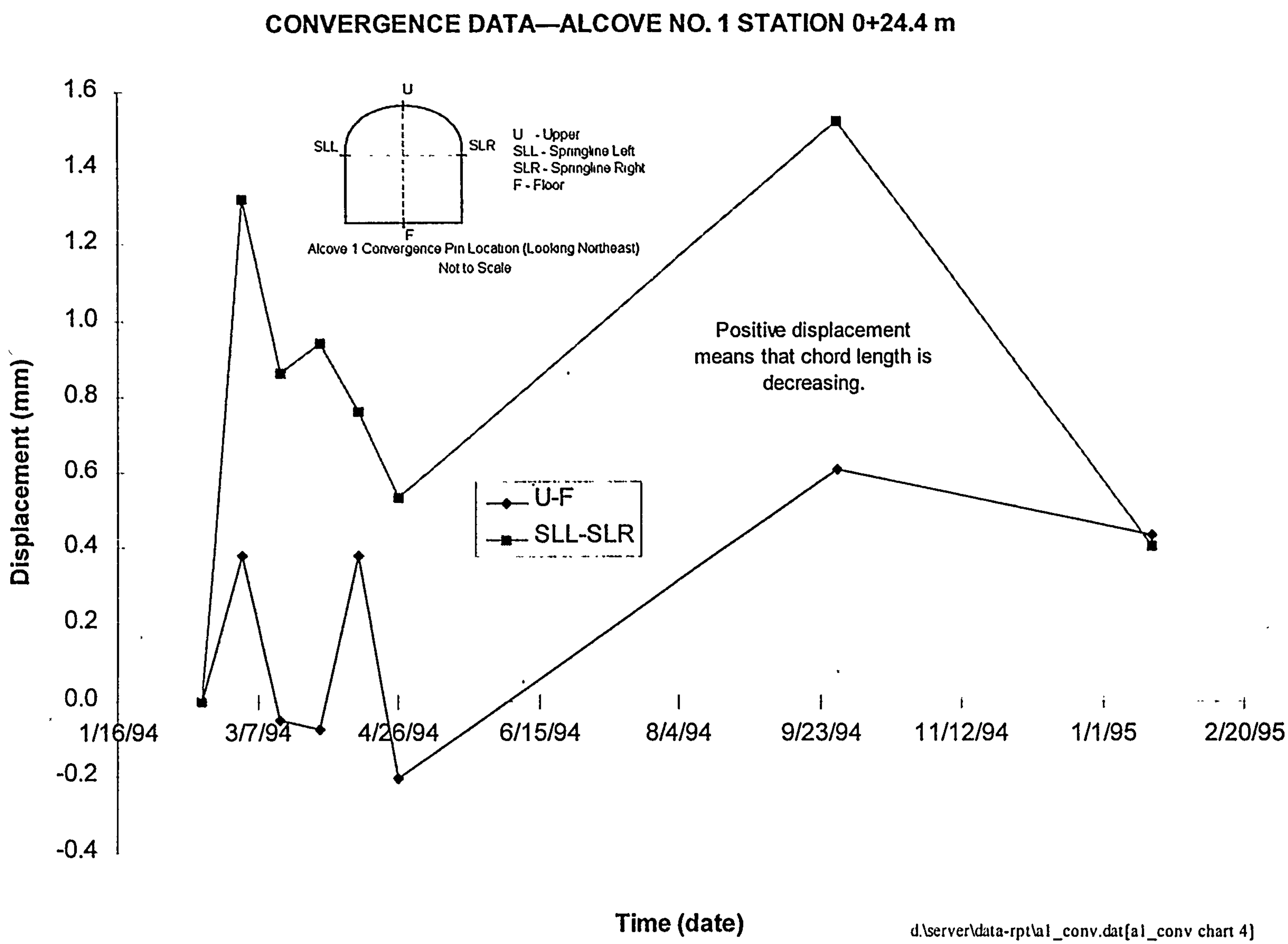


Figure 3-33. Convergence versus time—Alcove No. 1 station 0+24.4 m.

stations with a peak value of 4.85 mm (0.191 in) at Alcove No. 1 station 0+4.6 m. The maximum vertical closure was at the same station and reached 2.85 mm (0.112 in). All stations have shown expansive changes in the most current readings, with the exception of the vertical chord at station 0+4.6 m which had a closure rate of 0.006 mm/day (0.0002 in/day).

3.4.2.2 Alcove No. 1 Extensometer Data—Stations 0+11.3 m, 0+17.7 m and 0+24.4 m. Extensometer data is presented in Figure 3-34 for the vertical MPBX at Alcove No. 1 station 0+11.3 m and the vertical single-point borehole extensometer (SPBX) at Alcove No. 1 station 0+24.4 m. The instruments at both locations indicate closure in the vertical direction, with the maximum value equal to 2.1 mm (0.082 in) occurring at the beginning of September 1994. A slight expansion then occurred, and closure at a reduced rate resumed in December 1994. Closure rates at the end of the monitoring period in June 1995 were –0.004 mm/day (–0.0002 in/day) and –0.002 mm/day (–0.0001 in/day) at Alcove No. 1 stations 0+11.3 and 0+24.4 m, respectively. Displacement rates are shown graphically in Figure 3-35.

The instrument at 0+17.7 m has produced unreliable data due to transducer malfunction.

3.4.2.3 Instrumented Rockbolts—Alcove No. 1 Stations 0+4.6 m, 0+11.5 m, 0+14.0 m, 0+17.7 m and 0+23.8 m. Load data for the instrumented rockbolts in Alcove No. 1 are presented in Figures 3-36 and 3-37. Bolts at Alcove No. 1 station 0+4.6 m include a crown bolt and a bolt located at the east rib. Both these instruments show initial tension bleed-off followed by some increase in load. At the end of the monitoring period, June 1995, the maximum indicated bolt load was 38.4 kN (8,640 lbs). Yield strength of the IRBs is considerably lower than the that were used for permanent support. Due to reduction of area by the hollow center of the 19.1 mm (3/4 in) type 1060 steel bolt required for insertion of the strain gage, yield strength is specified at 68.1 kN (15,300 lbs).

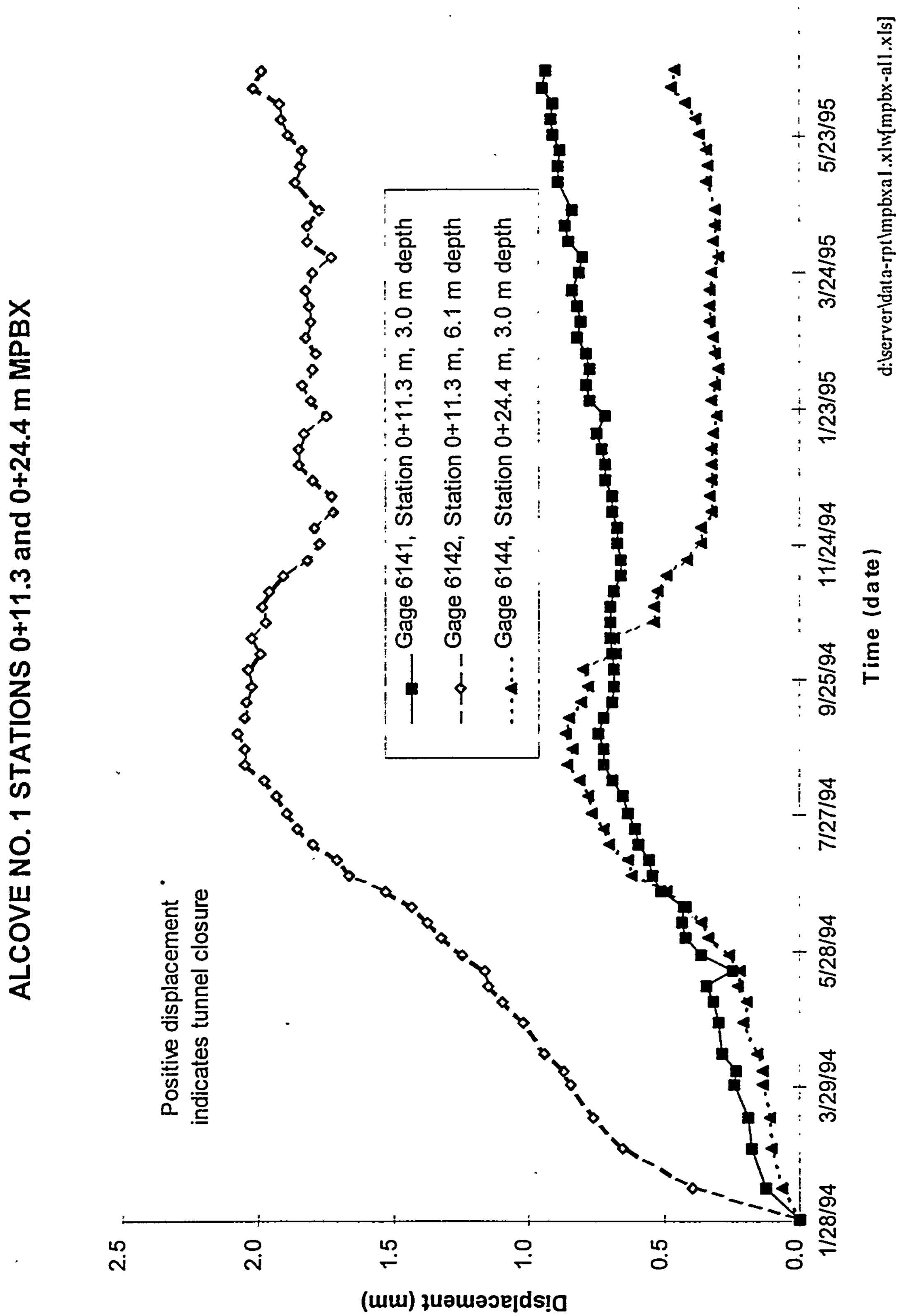


Figure 3-34. Vertical MPBX displacement versus time—Alcove No. 1 stations 0+11.3 and 0+24.4 m.

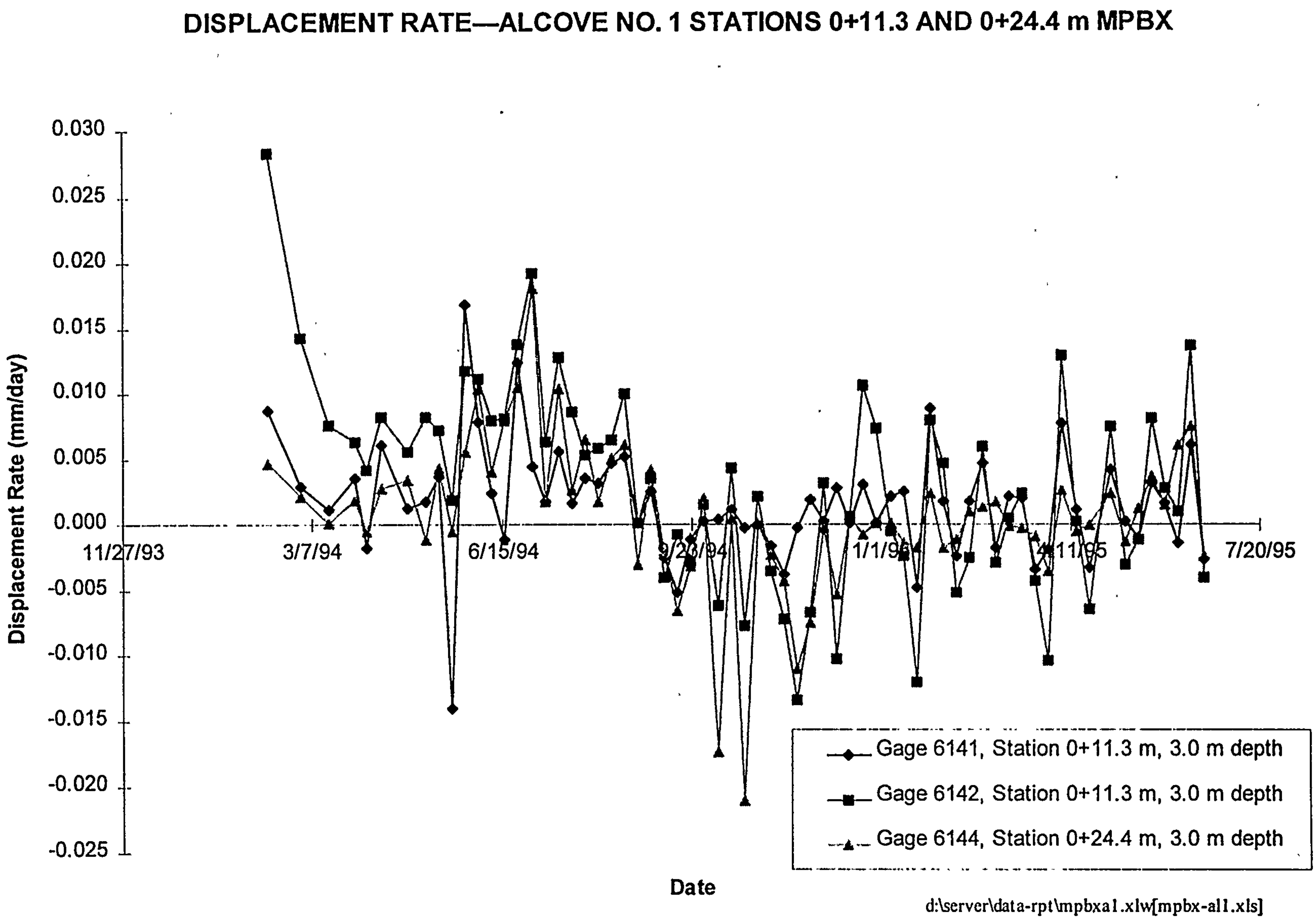


Figure 3-35. Vertical MPBX displacement rate versus time—Alcove No. 1 stations 0+11.3 and 0+24.4 m.

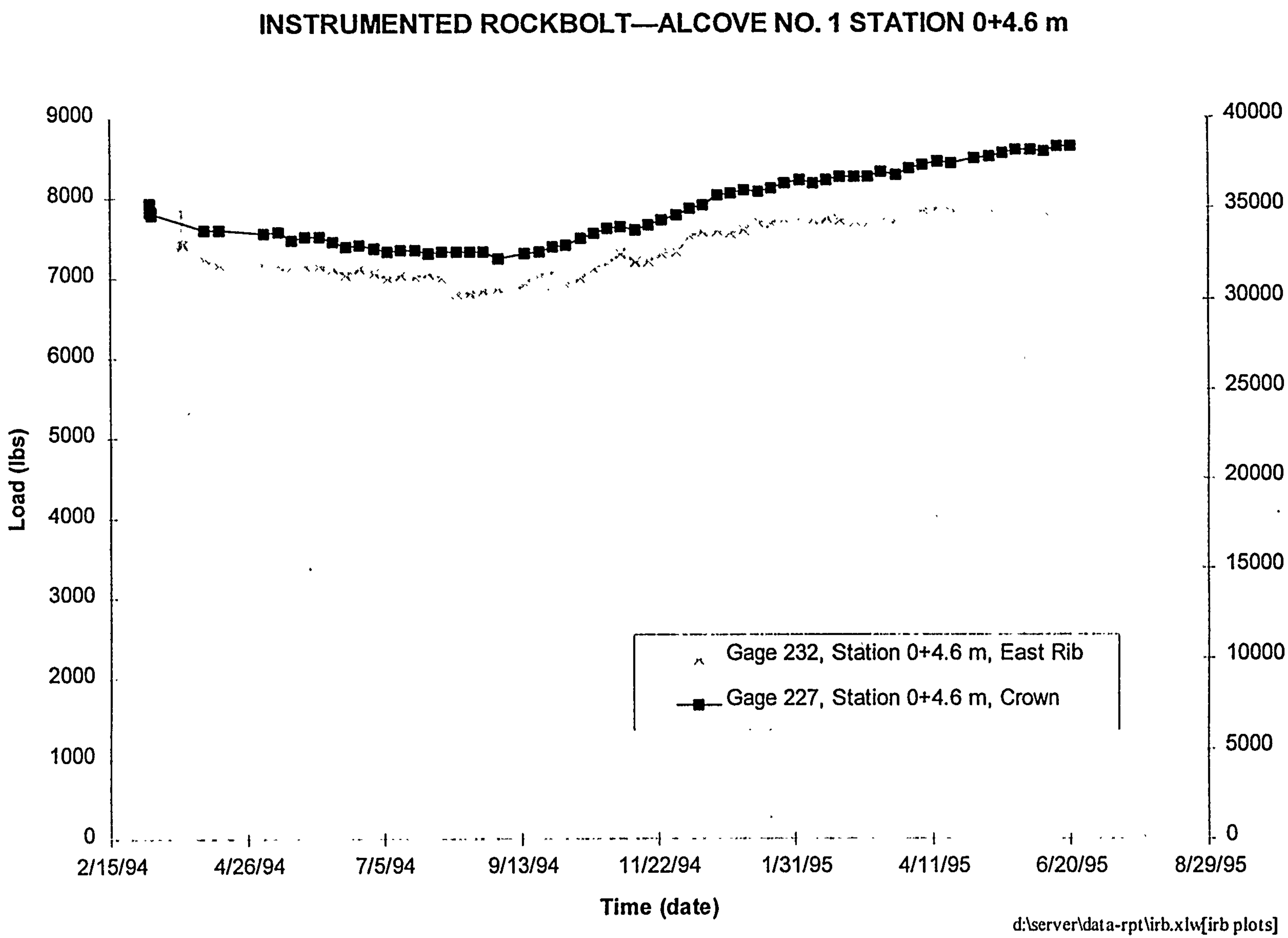


Figure 3-36. Rockbolt load versus time—Alcove No. 1 station 0+4.6 m.

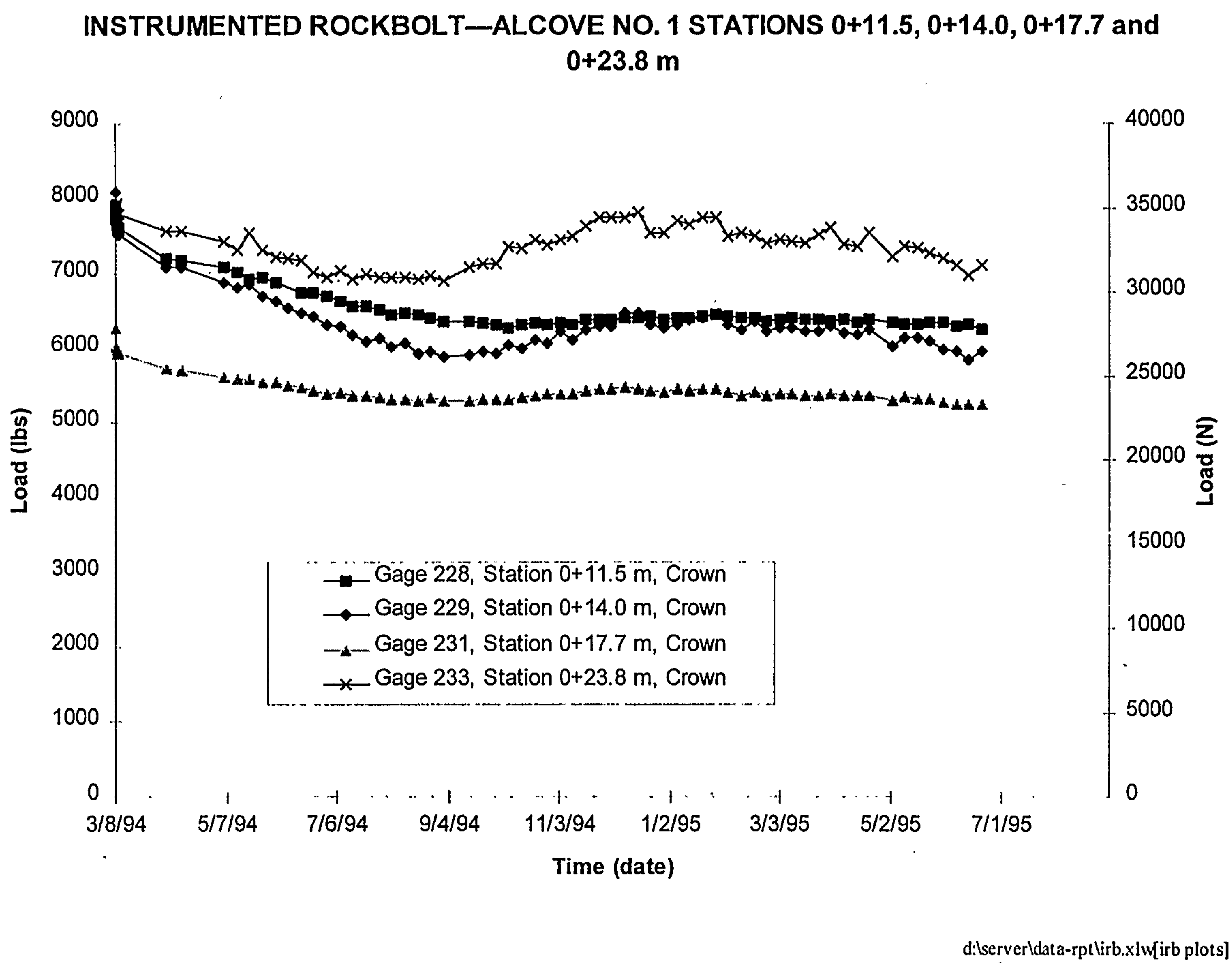


Figure 3-37. Rockbolt load versus time—Alcove No. 1 stations 0+11.5, 0+14.0, 0+17.7 and 0+23.8 m.

Similar levels of load and change in load are indicated for the bolts at Alcove No. 1 stations 0+11.5, 0+14.0, 0+17.7 and 0+23.8 m.

3.5 Fibercrete Condition

The condition of the fibercrete has been evaluated by inspection and mapping of cracks in August 1995. Mapping was performed by visual inspection using a tape referenced to known tunnel stationing. The mapping was limited to the north wall and crown in the Main Tunnel because the conveyor belt and ventilation line obscured the south wall and crown. Visual estimates of aperture width are noted on the maps. The resulting maps are presented in Appendix G.

The observed cracks appear to be primarily related to shrinkage and the installation of rockbolts, hangers and control boxes. They do not appear to be associated with structural conditions within the rock mass. There is no indication of excessive deformation or debonding.

3.6 BHPC Response During TBM Mine-by

BHPCs were installed at NRST station 0+60.2 m to attempt to monitor stress changes induced by the TBM as it excavated past the gages. Location and orientation of the gages was described in Section 2.3.5. The BHPCs were placed in three holes and oriented to measure vertical and horizontal stress change at increasing radial distances (0.3 to 1.5 m) from the tunnel wall near the spring line. Change in stress near the tunnel wall would be induced by the advance of the tunnel past the BHPCs and later by the pressure exerted by the TBM gripper pads.

Placement of the BHPCs was controlled by access limitations at the end of the NRST. Holes had to be drilled from the step between the NRST and machine-mined excavation at 0+60.2 m. The holes had to be nearly parallel to the North Ramp alignment to allow measurement of the radial stress change.

The BHPCs were initially pressurized to levels estimated to be in the range of existing in situ stress, based upon the depth of the tunnel. The initial pressure (see Table 2-5) was used as the reference pressure and subsequent pressures reported as the change in pressure from the initial pressures. Figure 3-38 presents the pressure history of BHPC Nos. 1 and 2 for the period immediately before and after mine-by. Table 3-5 lists the schedule of TBM advance for a distance of 14.2 m from the end of the NRST. BHPC response was heavily impacted by the temperature variations which produced daily pressure changes on the order of 0.3 MPa (50 psi) or roughly 40% of the vertical stress expected at this depth. No stress relief was detected as the TBM sumped in and cut to 0+63 m. Data for BHPC Nos. 1 and 2 for this period are presented in Figure 3-38, and show no detectable offset within the pattern of the daily temperature-induced changes.

Bleed-off of pressure had reduced the internal pressure in the BHPCs to near zero. The BHPCs were, therefore, repressurized on October 4, 1995. This repressurization was followed by rapid pressure reduction which may have masked stress changes as the TBM advanced beyond the gage locations between October 6 and October 17, 1994. Typically, the pressure in a BHPC will decrease for some period of time after repressurization and then stabilize. Both BHPCs Nos. 1 and 2 show this behavior after repressurization. Because mining commenced immediately after the repressurization, the pressure record of the response of the BHPCs to mining was difficult to interpret.

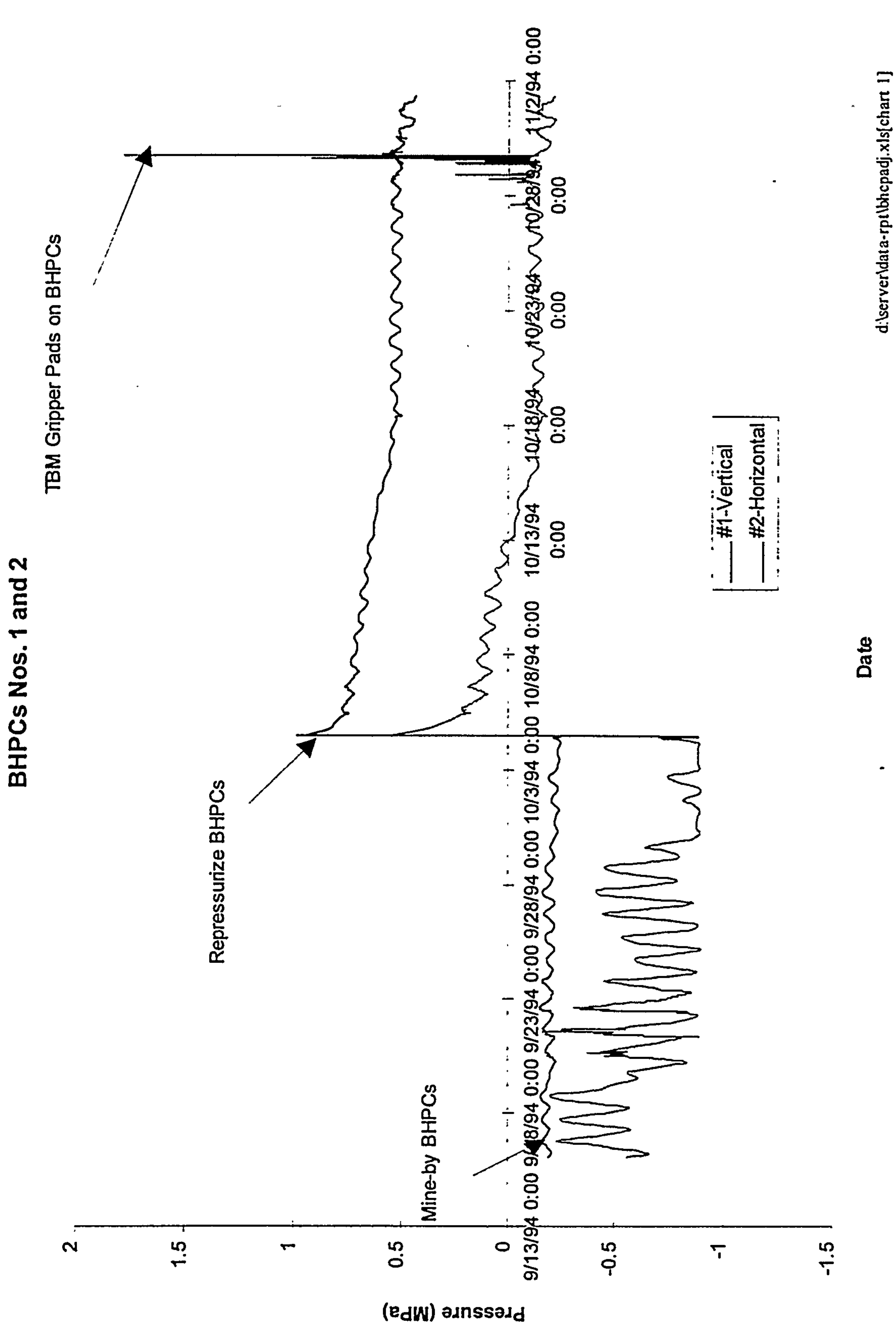


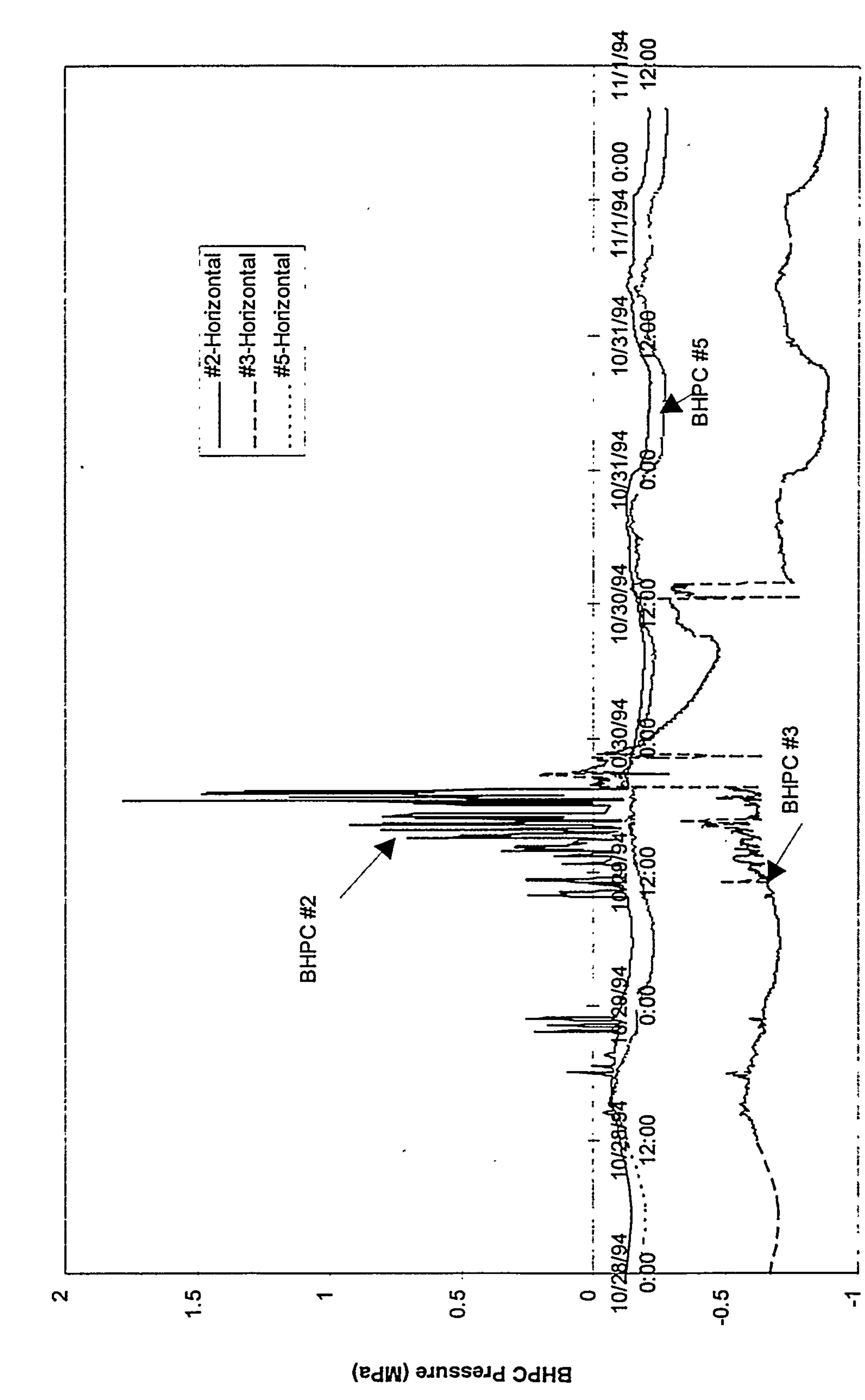
Figure 3-38. Pressure history for BHPC Nos. 1 and 2 for the TBM mine-by period.

Table 3-5. Schedule of TBM Advance Between 0+60.2 and 0+74.4 m

Date	TBM Penetration (m)	Location of Face (m)	Comments
9-19-94	0.00	0+60.2	
9-20-94	1.88	0+62.1	Start excavation, mine-by BHPC Nos. 1, 2, 3 and 4.
9-21-94	0.81	0+62.9	Mine-by BHPC Nos. 5 and 6.
9-22-94	0.14	0+63.0	TBM stopped for adjustment.
10-5-94	1.62	0+64.7	Repressurized BHPCs 10-4-95.
10-6-94	0.60	0+65.3	
10-10-94	0.59	0+65.9	
10-17-94	0.48	0+66.3	
10-27-94	1.80	0+68.1	
10-28-94	1.30	0+69.4	TBM gripper pads directly over gages.
10-29-94	3.75	0+73.1	
10-30-94	0.27	0+73.4	
10-31-94	1.01	0+74.4	

The BHPC pressure stabilized prior to the time at which the TBM gripper pads were directly next to the BHPCs. Each pressure response of BHPC Nos. 1 and 2 to the gripper pad pressure occurred on October 28, 29 and 30, 1994, after which no response was indicated.

The effects of the TBM gripper pads on the BHPCs are shown in Figures 3-39 and 3-40 for the horizontal (radial) and vertical stress changes, respectively. Gripper pad effects became apparent in the horizontal direction as early as October 28, 1994 and were greatest in BHPC No. 2 which was nearest the tunnel perimeter. The greatest change occurred on October 29, 1994 when the gripper pad was immediately over BHPC No. 2. A similar, large change occurred in BHPC No. 3 immediately after response from BHPC No. 2 stopped. BHPC No. 5, located farthest from the tunnel wall, showed very little response.



d:\server\data-rpt\tbmpad.xls[chart1]

Figure 3-39. BHPC response to the TBM gripper pad loads—horizontal direction.

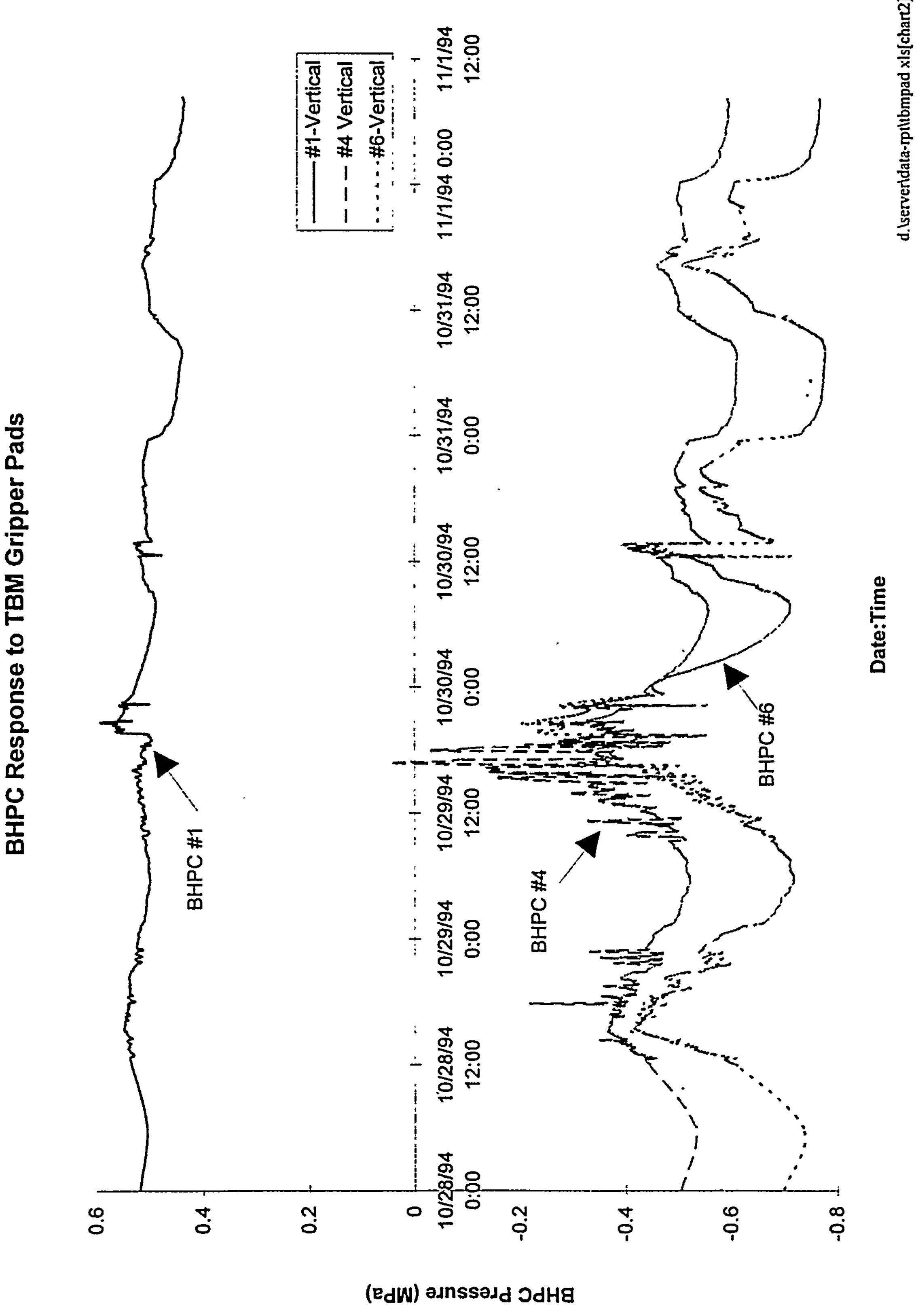


Figure 3-40. BHPC response to the TBM gripper pad loads—vertical direction.

The vertical stress change was less pronounced, as shown in Figure 3-40. BHPC No. 1, closest to the tunnel wall, showed the smallest response with the deepest gages showing the greatest change.

The rock in the area of the BHPC installations was effected by the presence of a large fractured zone that crosscut the NRST face and passed into the north rib just beyond the BHPCs, as shown in Figure 3-41. The existence of this large structure may have impacted the changes of stress that occurred when the TBM mined past the gage, and may have impacted stress distribution during application of the gripper pads.

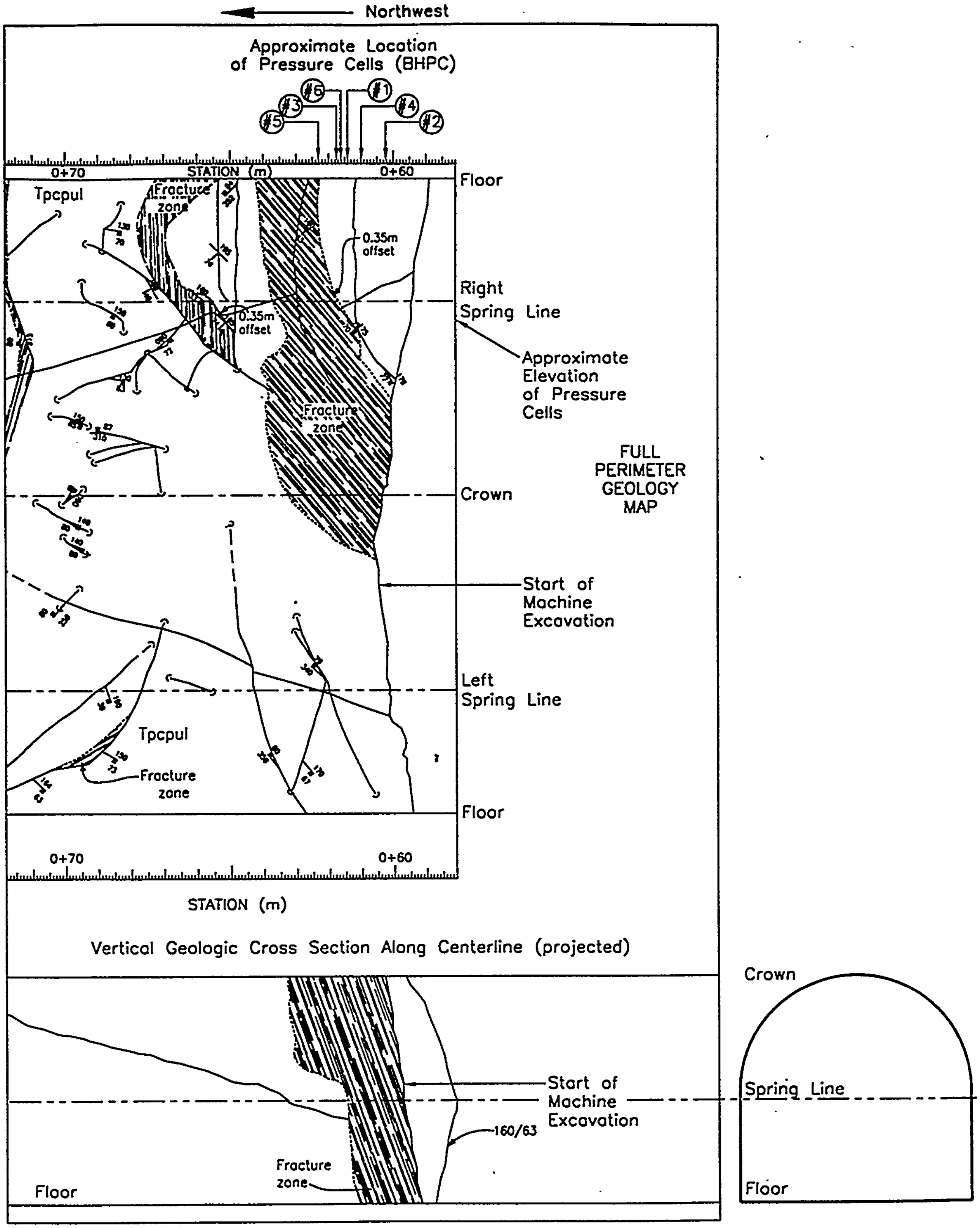


Figure 3-41. Structural geology of the North Ramp in the vicinity of the BHPCs (from detailed line survey from North Ramp of the ESF, stations 0+60 m to 4+00, U.S. Bureau of Reclamation, DTN: GS950508314224.002.

4.0 North Ramp Starter Tunnel—Design/Performance Verification

4.1 Introduction

The monitoring activities performed in the In Situ Design Verification studies are conducted to satisfy requirements in 10 CFR 60, subpart F—Performance Confirmation Program §60.141 Confirmation of Geotechnical and Design Parameters. This section of the Code of Federal Regulations requires a "continuing program of surveillance, measurement, testing, and geologic mapping shall be conducted to ensure that geotechnical and design parameters are confirmed..." It also specifies that "as a minimum, measurements shall be made of rock deformations and displacement changes, changes in rock stress and strain...," and that "these measurements and observations shall be compared with the original design bases and assumptions" to determine if there is a need for modifications of design or construction methods.

Data produced by the design verification activities are identified as repository design data needs (TRW 1995) in the areas of:

- deformation around openings (3.8.1),
- convergence around openings (3.8.2), and
- ground support load and deformation (3.8.3).

The results of the NRST monitoring support the assessment of "Rock Mass Performance Parameters" for the evaluation of emplacement drift design and for development of acceptance criteria for subsurface openings.

The functional requirements of the NRST make stability mandatory because it will provide the only access to the ESF until completion of the Main Drift and South Ramp. As part of the ESF permanent structures, systems, and components, the NRST is required to have a maintainable service life of 100 years (DOE 1995). It therefore controls the progress of all activities associated with ESF construction and testing in the early phases of site characterization. The NRST monitoring provides a basis to assess overall stability, to indicate requirements for maintenance, and to verify success of any maintenance efforts.

Although rock stability is a requirement in several 10 CFR 60 regulations, they do not provide specific ranges that are needed to judge performance. Initial project-imposed goals were defined in the Site Characterization Plan (SCP) (DOE 1988). These tentative goals were summarized by Hardy and Bauer (1991) as a basis for evaluating the YMP Drift Design Methodology and are presented in Table 4-1. Assuming the NRST is a main access excavation, the goals will be used as a basis to evaluate the data observations contained in this report. Other criteria from the general tunneling industry will also be discussed as a basis of comparison.

4.2 Monitoring Mining Methods—Blast Monitoring

During construction of the NRST, the monitoring mining methods (construction monitoring) experiment was confined to monitoring ground motion (PPV), as well as observational assessments of overbreak and damage in the rock surrounding the openings.

Table 4-1. Stability Performance Measures and Goals for Repository Drifts (Hardy and Bauer 1991)

Performance Measure	Tentative Goal from SCP (DOE 1988)	Impact on Design
Limit rock damage	Overbreak < 15 cm average	Selection of construction methods
	Measurable excavation—technique-induced damage limited to within 1 m of the excavation*	
Closure		
Access drifts	Closure rate < 1 mm/yr Total closure in ramps < 7 cm in 100 yr†	Conservative ground support Conservative ground support
Rock Fall		
Main access drifts	No rock falls	Conservative ground support design
Maintenance		
Main access drifts	Inspection and minor maintenance will be performed on a continuing basis	
	Major maintenance frequencies** >100 yr	Selection of ground support materials, conservative design

* SCP recommends blast-induced fracture extent into intact rock < 7.5 cm average.
† Deformation expected to be significantly less.
‡ Not currently in SCP, recommended by Hardy and Bauer (1991).
** SCP recommends > 25 yr, value of 100-yr recommended by Hardy and Bauer (1991).

Both the NRST main tunnel and Alcove No. 1 were excavated using the drill-and-blast mining method. Controlled blasting procedures were implemented to minimize blast-induced damage to the excavation perimeter. Blast damage can result in loosening of the surrounding rock mass which increases ground support requirements and long-term maintenance. Specific Nuclear Regulatory Commission (NRC) regulatory language, 10 CFR 60, subpart E—Technical Criteria §60.133(f), requires "excavation methods that will limit the potential for creating a preferential pathway for groundwater or radioactive waste migration to the accessible environment." The ESF Design Requirements (DOE 1995) estimate the rock mass altered by the excavation will be within 1.5 m of the excavated surface.

Excavation of the NRST produced substantial overbreak on existing structural features in spite of the use of perimeter blasting procedures and in many instances violated the criteria in Table 4-1 of < 15 cm. Perimeter holes did not generally produce idealized half-casts in the tunnel perimeter. A similar result, limited borehole half-casts, had been reported by Zimmerman et al. (1988) during controlled blasting for a mine-by experiment in fractured, welded tuffs in G-Tunnel. Maximum reported overbreak in the G-Tunnel experiment was 0.6 m (2 ft) which occurred in a faulted section.

Monitoring of the blasts indicated that reduced quantities of explosives detonated per delay and the long period between detonations were successful in controlling PPVs. Delays utilized in the blasting could be correlated with individual seismic peaks in the blasting seismic records. Comparison of near-field seismic motion from the Alcove No. 1 blasting to far-field data (see Section 3.3) indicated similar trends in PPV versus scaled distance. The near-field monitoring indicated that PPVs, with dominant frequencies below 300 Hz, dropped below 700 mm/sec (28 in/sec) between 0.91 m (3 ft) and 2.44 m (8 ft) from the Alcove No. 1 perimeter. Visual observations of borehole walls at these distances indicated that motions induced on existing joint structure were sufficient to cause cracking in grout coatings along pre-existing fractures on the inside of the boreholes at 0.91 m (3 ft) but not at 2.44 m (8 ft). Rock damage was reported at PPVs above 700 mm/sec (28 in/sec) by Holmberg and Perrson (1979). The observations suggest that the Table 4-1 criteria of limiting measurable blast damage to within 1 m of the excavation perimeter may have been satisfied.

All blast monitoring conducted in the NRST and Alcove No. 1 used geophones (velocity sensors) with a flat frequency response up to 300 Hz. Recent results reported by Yang et al. (1993) indicate that near-field PPVs are substantially higher at very high frequencies (9000 Hz), and that geophone frequency ranges typically used for blast monitoring are not sufficiently high. They report PPVs of over 6000 mm/sec (236.5 in/sec) at a distance of 1 m and that typical charge weight scaling laws underestimated the PPV substantially at distances of less than 10 m (32.8 ft).

There has been very little data developed on the effects of frequency on intact rock damage. Dowding (1985) emphasized the importance of frequency in controlling damage to structures adjacent to blasting. Assessments of damage to underground structures were generally based on amplification factors that occurred as free-field motion intersected a tunnel. There are currently no data available on the impacts of high frequency, free-field seismic motion on the rock mass very close to the blast. However, future near-field monitoring at YMP should be based on accelerometers and recording systems that can capture the high frequency data.

4.3 Monitoring Ground Support Systems

The ground support installed during the construction of the NRST included rockbolts, steel lattice girders, and fibercrete. Monitoring of rockbolts was accomplished using RBLCs, as well as IRBs. In addition to monitoring within the NRST, three RBLCs were installed on rockbolts on the vertical "box-cut" face above the tunnel portal. Closure pins were attached to the lattice girders in the first 10 m (32.8 ft) to monitor displacement of this component of the ground support system.

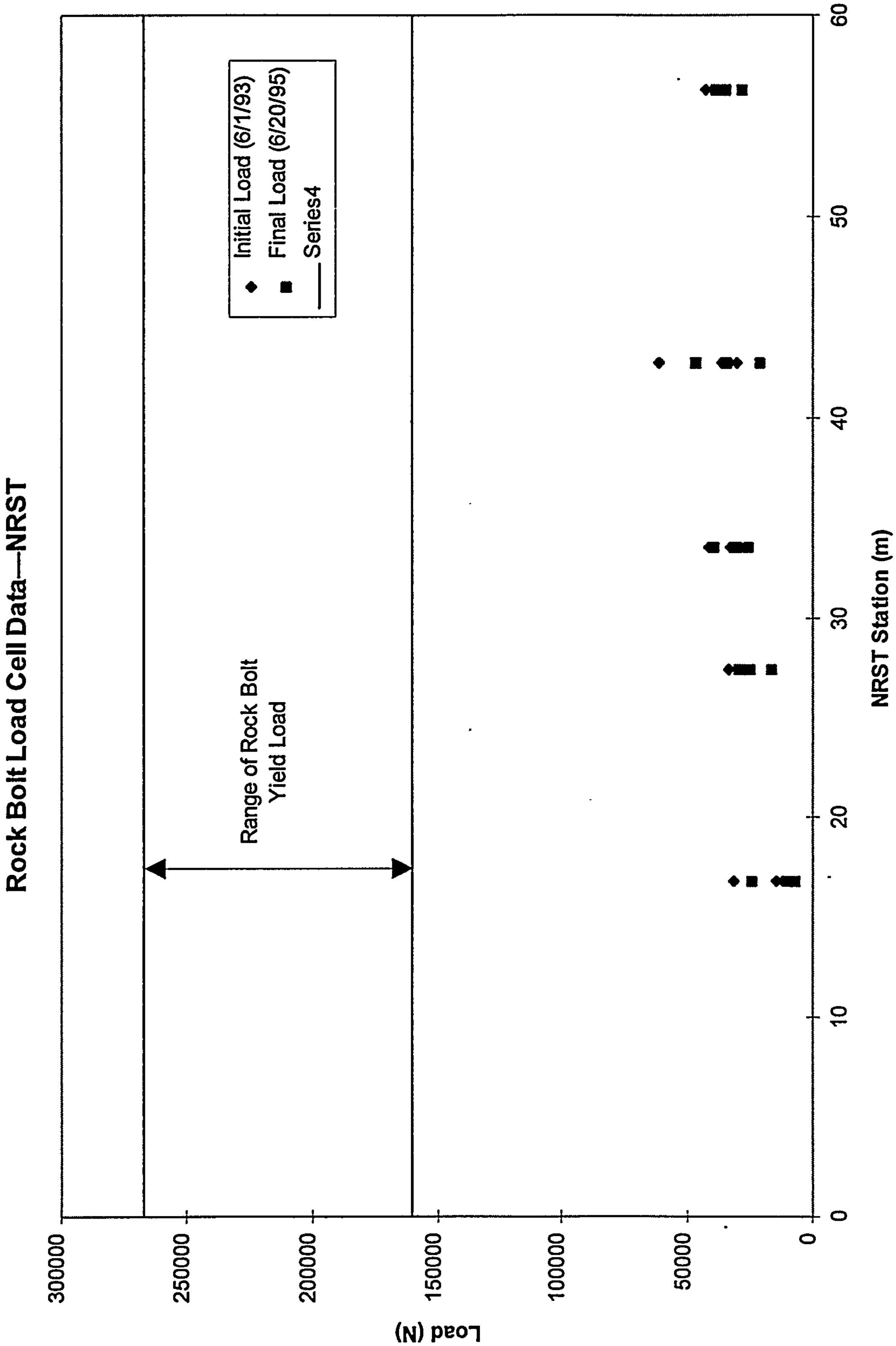
4.3.1 Rockbolt Load

RBLCs were installed in groups of three at stations along the entire NRST, and IRBs were installed on bolts within Alcove No. 1. The permanent ground support in both excavations is based upon fully grouted 3.0 m (10 ft) to 7.3 m (24 ft) long rockbolts. Specifications of these bolts are (Williams 1995):

- nominal diameter from 22.2 mm (7/8 in) to 28.6 mm (1.13 in),
- yield load from 159.9 kN (36,000 lbs) to 269.9 kN (60,000 lbs), and
- ultimate load from 239.9 kN (54,000 lbs) to 355.9 kN (80,000 lbs).

The bolts were installed by drilling the bar into the rock with a sacrificial bit, then grouting the bolt in place with a pumpable thixotropic cement grout. After grout cure, most instrumented bolts in the main tunnel were pull tested to between 111.1 kN and 249.1 kN to confirm the rock-grout-bolt bond.

The RBLCs were attached to the rockbolts with extension couplers and then loaded against a plate set against the tunnel perimeter by torquing up the retaining nut. These installation loads varied between 10.4 kN (2335 lbs) and 42.8 kN (9635 lbs), well below the range of bolt yield strength. All bolts underwent load bleed-off and have settled into generally stable patterns through the monitoring period (installation to June 1995). Figure 4-1 presents a graphical comparison of the installation load and final load for the monitoring period to the bolt specifications.



d:\server\data-rpt\rbload.xls[Rock Bolt Load Chart 2]

Figure 4-1. Comparison of bolt load history to bolt yield strength for different stations in the NRST.

The grouted bolts were instrumented because they are the support system (in conjunction with mesh and fibercrete). Instrumentation of grouted rockbolts is generally not done because the grout column distributes bolt load into the grout/rock column by shear stress. This makes the load cell at the tunnel perimeter insensitive to loading that may occur at some depth into the rock. Bolt failure at some depth into the rock could therefore occur without the load cell seeing load changes in the range of bolt yield or ultimate strength.

This approach was modified in Alcove No. 1 by installing IRBs using conventional anchors without grouting to provide a comparison. Comparison between the IRB performance and RBLC performance indicates similar stable bolt loads with time. Figure 4-2 compares the bolt load at installation and at the end of the monitoring period to bolt yield strength for the Alcove No. 1 IRBs.

4.3.2 Portal Girder Deformation

Deformation of the portal girders embedded in shotcrete is tracked by the convergence data presented in Section 3.4.1.2. These deformations have apparently reached plateaus; however, they have both showed small rates of closure in the horizontal and inclined chords in the most recent measurements. These rates, 0.009 mm/day to 0.011 mm/day, are well below empirical values proposed by Bieniawski (1984) that would suggest pending instability (0.05 mm/day or greater).

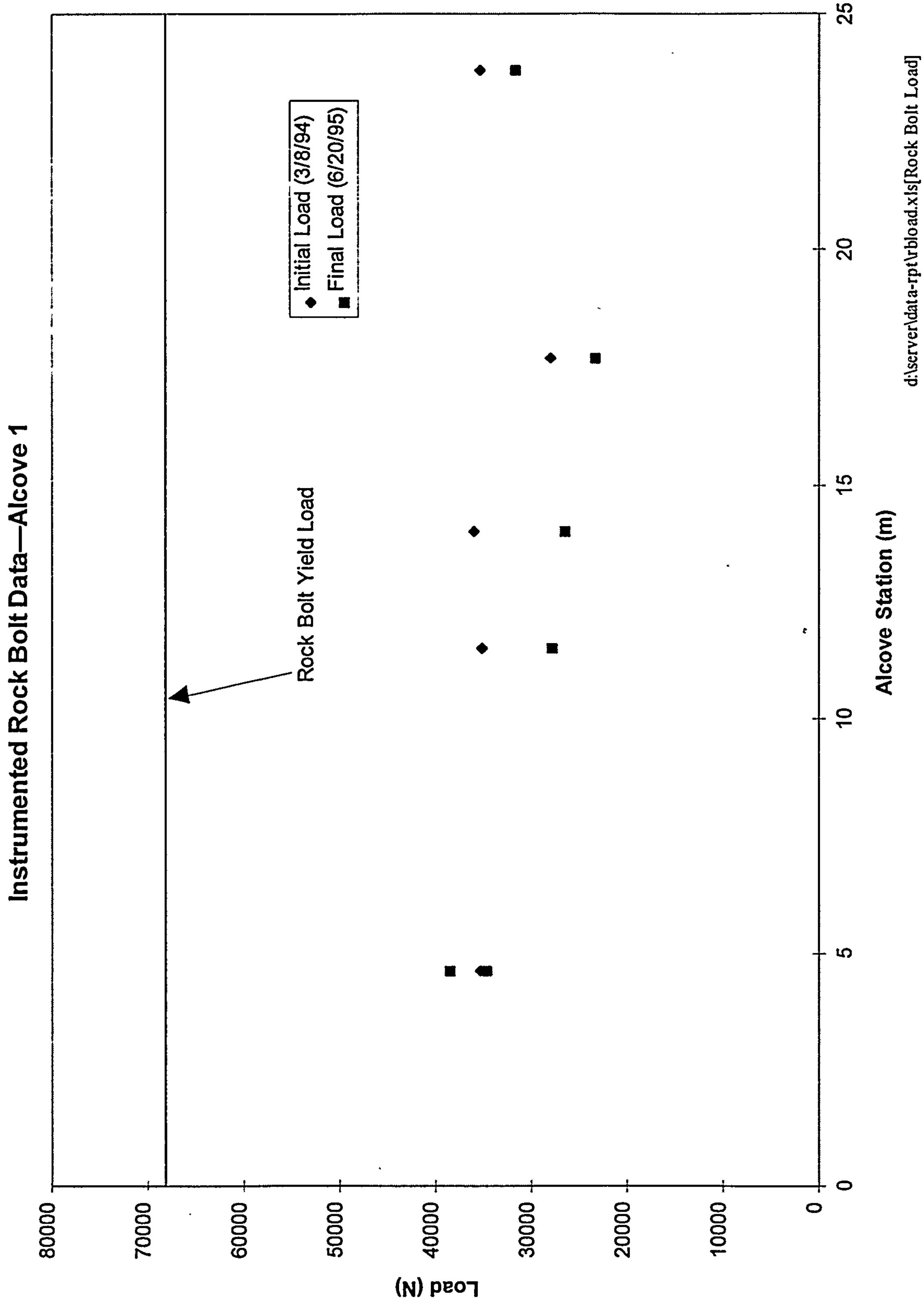


Figure 4-2. Comparison of bolt load history for IRBs in Alcove No. 1.

4.3.3 Visual Mapping of Fibercrete Cracking

Some cracking has occurred in the fibercrete both on the highwall and within the NRST. This cracking was observed shortly after installation and was mapped as part of the In Situ Design Verification Study. The fibercrete in the vicinity of the cracks has been "sounded" and no evidence suggests that the fibercrete has debonded from the surrounding rock. The amount of cracking present is not considered anomalous at this time, however, the cracks will continue to be observed and compared to drift closure and RBLC data. This combined information will be used to determine whether remedial actions are required to arrest additional cracking or deterioration of the fibercrete.

4.3.4 Verification of Ground Support Design

Rock mass quality assessments in the NRST and Alcove No. 1 allowed empirical verification of the ground support installed in the NRST. The results of the assessments in individual sections of the excavations were presented in Section 3.2, Table 3-1, and length-weighted log mean values of Q were listed in Table 3-2. The range of mean values were used as the basis to assess the installed ground support using empirical design data from Barton et al. (1974).

Empirical ground support was correlated with rock mass quality (Q) using a logarithmic scale for over 200 tunneling case histories by Barton et al. (1974). The case history data were used to develop ground support categories that were correlated with both Q and the excavation span divided by a factor called excavation support ratio (ESR). The ESR factor introduces an adjustment for the duty of the excavation. Temporary excavations, for example mine production excavations, have a large ESR (3–5). Underground civilian facilities, major highway tunnels, portals, power stations, etc. have an ESR of 1.0. YMP has adopted an ESR value of 1.0 for most of the ESF openings.[1]

The empirical ground support as described by Barton et al. (1974) is compared to installed ground support in Table 4-2. The table indicates that the installed ground support is consistent with the empirical ground support design approach in all areas of the NRST.

All data collected to monitor ground support performance for the NRST and Alcove No. 1 suggest that the installed ground support is performing within design specifications. Continued monitoring of all ground support instrumentation is required so that any deterioration in ground support performance is recorded and appropriate remedial measures taken.

4.4 Monitoring Drift Stability

Drift stability in the NRST was monitored by making displacement measurements in the rock mass using extensometers and by measuring overall closure using tape extensometers. These measurements have been compared with both empirical criteria and the tentative SCP goals in Table 4-1 to ascertain whether the drifts are performing according to design requirements. Performance requirements for underground structures are generally linked to their functional requirements. The ESF accesses require greater control of potential instabilities to assure that they are not manifested as deleterious rock movement and to allow continuity of function during the site characterization.

---

[1]ESF Ground Support Design Analysis, BABEE0000-01717-0200-00002, Rev. 00D.

Table 4-2. Comparison of Empirical Ground Support versus Installed Ground Support

	Range of Log Mean Q Q_{min}	Range of Log Mean Q Q_{max}	Empirical Ground Support	Installed Ground Support
Lattice Girders	0.067	—	CAT. 35—tensioned, grouted rockbolts on 1-m spacing with 20–75 cm mesh-reinforced shotcrete	Temporary support 3.0-m split sets on 1.2-m spacing with wire mesh, plus final support 3.0-m untensioned, grouted rockbolts on 1.2-m spacing; steel lattice girders on 1.4-m spacing embedded in fibercrete to minimum cover of 38 mm.
	—	1.03	CAT. 27—untensioned, grouted rockbolts on 1-m spacing with 5–7.5 cm mesh-reinforced shotcrete	
NRST Top Heading	0.42	—	CAT. 31—tensioned, grouted rockbolts on 1-m spacing with 5–12.5 cm mesh-reinforced shotcrete	Temporary support 2.0-m split sets with wire mesh, plus final support 3.0-m to 7.3-m untensioned, grouted rockbolts on 1.2-m centers with 152 mm×152 mm, w2.9×2.9 welded wire fabric with 15.2-cm fibercrete, minimum 10.2 cm fibercrete
	—	1.09	CAT. 23—untensioned, grouted rockbolts on 1–1.5-m spacing with 5–10 cm mesh-reinforced shotcrete	
NRST Bench*	0.36	—	CAT. 27—untensioned, grouted bolts on 1-m spacing with 5–7.5 cm mesh-reinforced shotcrete	
	—	2.11	CAT. 18—tensioned, grouted bolts on 1–1.5-m spacing with chain-link mesh	
Alcove No. 1	0.36	—	CAT. 31—tensioned, grouted rockbolts on 1-m spacing with 5–12.5 cm mesh-reinforced shotcrete	3.0-m split sets on 1.2-m spacings, 152 mm× 152 mm, welded wire fabric, minimum 10.2 cm fibercrete
	—	2.780	CAT. 26—untensioned, grouted rockbolts on 1-m centers with 2.5–5 cm shotcrete	

*Q is multiplied by 2.5, as per Barton et al. (1974), to determine wall support category.

Empirical criteria relating measured displacements to underground opening stability were described by Bieniawski (1984):

- rates of displacement of the order of 0.001 mm/day (0.0004 in/day) indicate stable conditions,
- rates of 0.05 mm/day (0.002 in/day) are quite high and dangerous for wide chambers, and
- rates of over 1.0 mm/day (0.039 in/day) are excessive and call for additional support measures.

These criteria compare well with the tentative goal in Table 4-1 of rates less than 1 mm/yr (0.039 in/yr) or an average rate of 0.0027 mm/day (0.0004 in/day).

Displacement rate data from the monitoring records in Section 3.0 have been summarized and presented graphically in Figure 4-3 for comparison against the empirical criteria. The figure presents displacement rates from both convergence and extensometer measurements as a function of tunnel station and indicates that rates at the end of June 1995 are well below the empirical criteria of 0.05 mm/day (0.002 in/day) for wide openings. The NRST width of 10 m (32.8 ft) would be considered at the lower end of the range of wide openings. However, the rates are generally higher than the SCP tentative yearly goal which would produce an average daily closure rate of 0.0027 mm/day (0.0004 in/day). The low displacement rates were confirmed by RBLC records which indicated generally stable trends in bolt load change.

Alcove No. 1 convergence and extensometer data indicated generally expansive convergence deformations at the end of the monitoring period (June 1995). The maximum closure rate was 0.004 mm/day (0.0002 in/day), well below the rate at which concern for stability would be indicated. These results are consistent with expected drift stability for an opening at shallow depth.

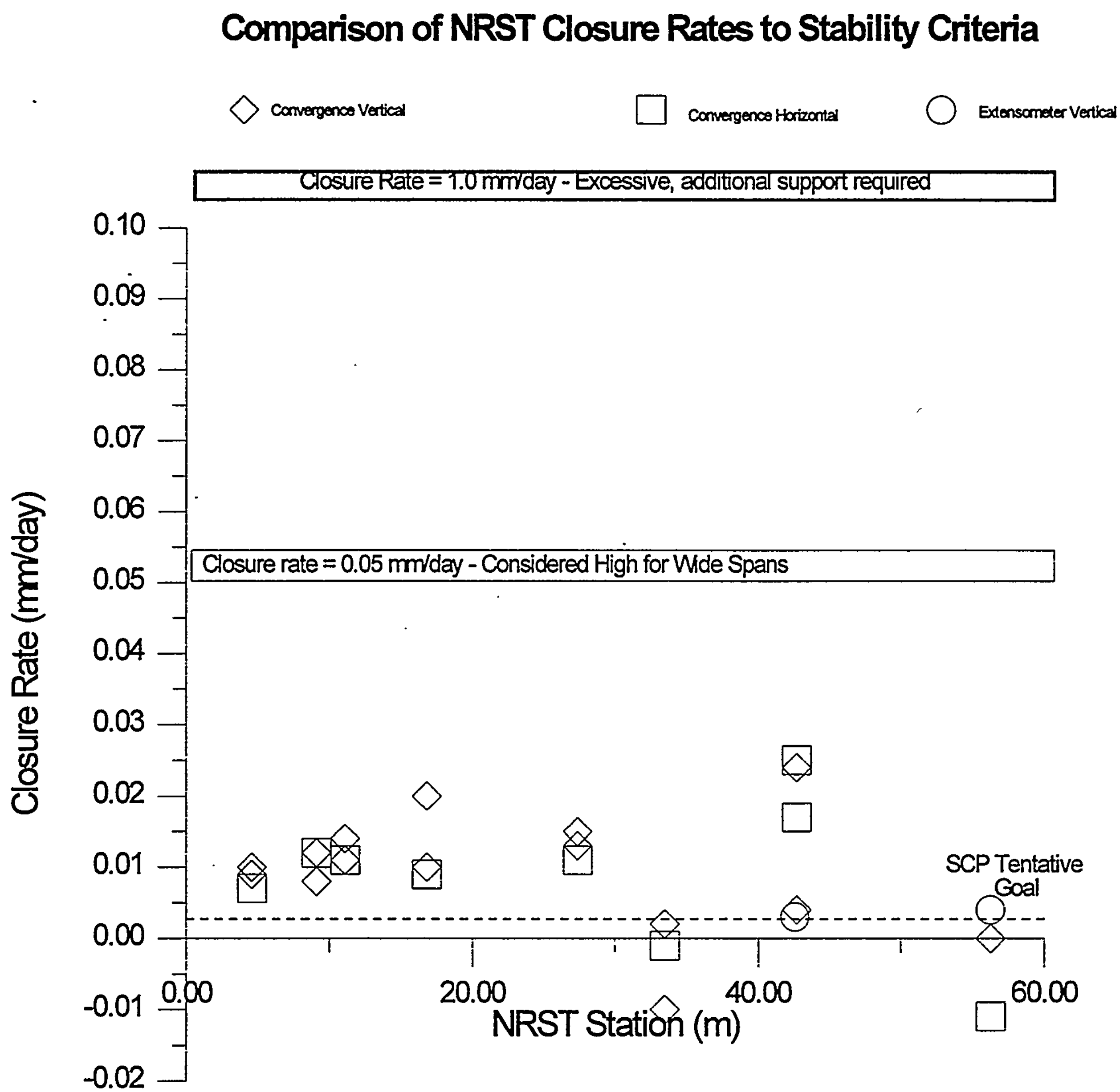


Figure 4-3. Comparison of NRST closure rates to stability criteria.

5.0 Summary and Conclusions

Sandia National Laboratories is conducting ongoing in situ experiments and activities to support design verification of the underground openings for the proposed repository at Yucca Mountain. This report describes these experiments and activities conducted in the North Ramp Starter Tunnel and Alcove No. 1, including evaluation of the mining methods, evaluation of drift stability, and evaluation of ground supports. The measurements in and around the NRST are expected to continue throughout the life of the proposed repository. Therefore, the data presented in this report, April 1993 through June 1995, represent the first of a series of reports covering the design verification activities.

Installed instrumentation included rockbolt load cells, instrumented rockbolts, borehole extensometers, closure pins, and borehole pressure cells. In general, the instrumentation has performed well with the exception of the borehole pressure cells and have provided continuous data since installation. The borehole pressure cells showed excessive pressure bleed-off at low installation pressures (< 2 MPa, 290 psi) and had large daily temperature effects even after efforts to insulate the exposed transducers. BHPC performance may have been impacted by the relatively poor rock quality and high rates of air movement through the rock that may have caused convective temperature changes around the BHPCs.

Blasting seismic monitoring was conducted during excavation and rock mass quality was assessed for various intervals as they were excavated.

Evaluation of the data indicated the following main conclusions:

- The range of rock mass quality data collected in the NRST was large (Q = 0.06 – 17.40) and indicated generally poor rock conditions.
- The installed ground support was consistent with ground support requirements indicated by the rock mass quality (Q) and empirical case history data reported by Barton et al. (1974).
- The performance of the instrumented ground support components, rockbolts, and lattice girders indicated that they are not currently being loaded excessively by ongoing tunnel deformation.
- The magnitude of closure measured in the tunnel after installation of the final ground support has been relatively small. The maximum closure was less than 12 mm (0.47 in) along horizontal chords. Maximum closure measured on inclined roof-to-floor chords was less than 2 mm (0.08 in)
- Closure curves have generally reached plateau values since mid-1994. Since this time, displacement rates indicate both expansion and closure. This behavior is consistent with oscillations around a stable value due to long period thermal response because of the near-surface proximity.
- Closure rates, as measured by convergence points, at the end of the monitoring period, are relatively low by empirical tunneling criteria (< 0.02 mm/day, 0.0008 in/day), but are greater than the SCP tentative goal for average daily convergence of accesses of 0.0027 mm/day (0.0004 in/day). However, the rates reported are the most recent values taken from the end of the monitoring period and may represent cyclic thermal displacements, not continued long-term trends. Closure rates, as measured by vertical MPBXs where data points are collected daily, were less than 0.004 mm/day (0.00016 in/day).

- Blasting monitoring data suggested that peak particle velocities produced by the controlled blasting procedures were consistent with good practice. The rock damage assessment performed suggested that the rock damage may have been consistent with tentative SCP goals.
- The results of monitoring instruments in the NRST indicate overall performance of the NRST that is consistent with the SCP goals for both long-term stability and the functional requirements.

This page left intentionally blank.

6.0 References

Barton, N.R., R. Lien, and J. Lunde (1974). Engineering Classification of Rock Masses for the Design of Tunnel Support, *Rock Mechanics*, 6:189–236, Springer Verlag. (NNA.8700406.0237)

Bieniawski, Z.T. (1979). *Engineering Rock Mass Classification*, Wiley-Interscience Publication, John Wiley & Sons, New York, New York. (NNA.901005.0044)

Bieniawski, Z.T. (1984). *Rock Mechanics Design in Mining and Tunneling*, A.A. Balkema, Boston.

Brechtel, C.E., M. Lin, E.R. Martin, D.S. Kessel (1995). Geotechnical Characterization of the North Ramp of the Exploratory Studies Facility, *SAND95-0488/1*, Yucca Mountain Site Characterization Project, Sandia National Laboratories, Albuquerque, New Mexico. (MOL.19950502.0004)

Caceci, M.S., and W.P. Cacheris (1984). Fitting Curves of Data, *Byte*, May, pp. 340–362.

Department of Energy (DOE) (1988). Site Characterization Plan, Yucca Mountain Site, Nevada Research and Development Area, Nevada, *DOE/RW-0199*, Chapter 8. (HQ0.881201.0002).

Department of Energy (DOE) (1991). Yucca Mountain Site Characterization Project, Study Plan No. 8.3.1.14.2, *Studies to Provide Soil and Rock Properties of Potential Locations of Surface and Subsurface Access Facilities*, U.S. Geological Survey, November. (NNA.911108.0238)

Department of Energy (DOE) (1995). Yucca Mountain Site Characterization Project, Exploration Studies Facility Design Requirements, Vol. 1, Rev. 1/ICN-2, YMP/CM-0019.

Dowding, C.H. (1985). *Blast Vibration Monitoring and Control*, Prentice Hall, Englewood Cliff, New Jersey.

Dupont (1980). *Blasters Handbook*, 16th Edition, Wilmington, Delaware.

Hardy, M.P., and S.J. Bauer, 1991. Drift Design Methodology and Preliminary Application for the Yucca Mountain Site Characterization Project, *SAND89-0837*, Sandia National Laboratories, Albuquerque, New Mexico. (NNA.910808.0105)

Holmberg, R., and P. Persson (1979). Design of Tunnel Perimeter Blasthole Patterns to Prevent Rock Damage, *Tunneling 79*, Institute of Mining and Metallurgy, London, pp. 280–283.

Hudson, J.A., and S.D. Priest (1979). Discontinuities and Rock Mass Geometry, *International Journal of Rock Mechanics, Mining Sciences and Geomechanics Abstracts*, Pergamon Press, 16(6):339–362. (NNA.900403.0399)

TRW Environmental Safety Systems, Inc. (1995). *Repository Design Data Needs*, BC0000000-01717-5705-00012, Rev. 00, Civilian Radioactive Waste Management System, Management and Operations Contractor.

Williams Form Engineering Corp. (1995). Rock Anchor Systems, No. 392, product literature, Williams Form Engineering Corp., Grand Rapids, Michigan.

Yang, R., W.F. Bawden, S. Talebi, and P. Rocque (1993). An Integrated Technique for Vibration Monitoring Adjacent to a Blast Hole, *Canadian Institute of Mining (CIM) Bulletin*, July-Aug, CIM 86(972):45–52.

Zimmerman, R.M., R.A. Bellman, Jr., K.L. Mann, D.P. Zerga, M. Fowler, and R.L. Johnson (1988). G-Tunnel Welded Tuff Mining Experiment Evaluations, *SAND87-1433*, Yucca Mountain Project, Sandia National Laboratories, Albuquerque, New Mexico.

This page left intentionally blank.

Appendix A
Traceability of NRST Data

Data in this report were developed and documented in a series of scientific notebooks. The relationship of the notebook and traceability of the data into this report "Evaluation of Geotechnical Monitoring Data from the ESF North Ramp Starter Tunnel April 1994 to June 1995" are described in Figure A-1. The figure presents a flowchart showing the activities of data collection and linkage between scientific notebooks which are listed by their number (SN #04, for example) and the notebook title. A central analysis file was developed to support preparation of the report and all relevant information from the scientific notebooks was assembled in this file. The data were submitted in the following interim data transmittals:

1. DTN: SNF32120393001.010—RBLC, IRB, and MPBX Data for the NRST/Alcove No. 1 (initial installation through June 20, 1995)
2. DTN: SNF33120393002.001—Convergence Pin Data for the NRST/Alcove No. 1 (initial installation through July 2, 1995)
3. DTN: SNF28021693001.005—Rock Mass Quality Assessments in the North Ramp Starter Tunnel—Top Heading, Bench, and Alcove No. 1
4. DTN: SNF28021693001.006—Map of Fibrecrete Surface Cracks—North Ramp Starter Tunnel
5. DTN: SNF28021693001.007—Blast Monitoring Data—North Ramp Starter Tunnel, Top Heading and Alcove No. 1
6. DTN: SNF28021693001.008—North Ramp Starter Tunnel, Borehole Pressure Cell Data Collected during TBM Mine-By

Traceability of Data for the North Ramp Starter Tunnel Data Report

Sandia National Laboratories YMP WA-0065, Rev 3 and WA-0116, Rev 1

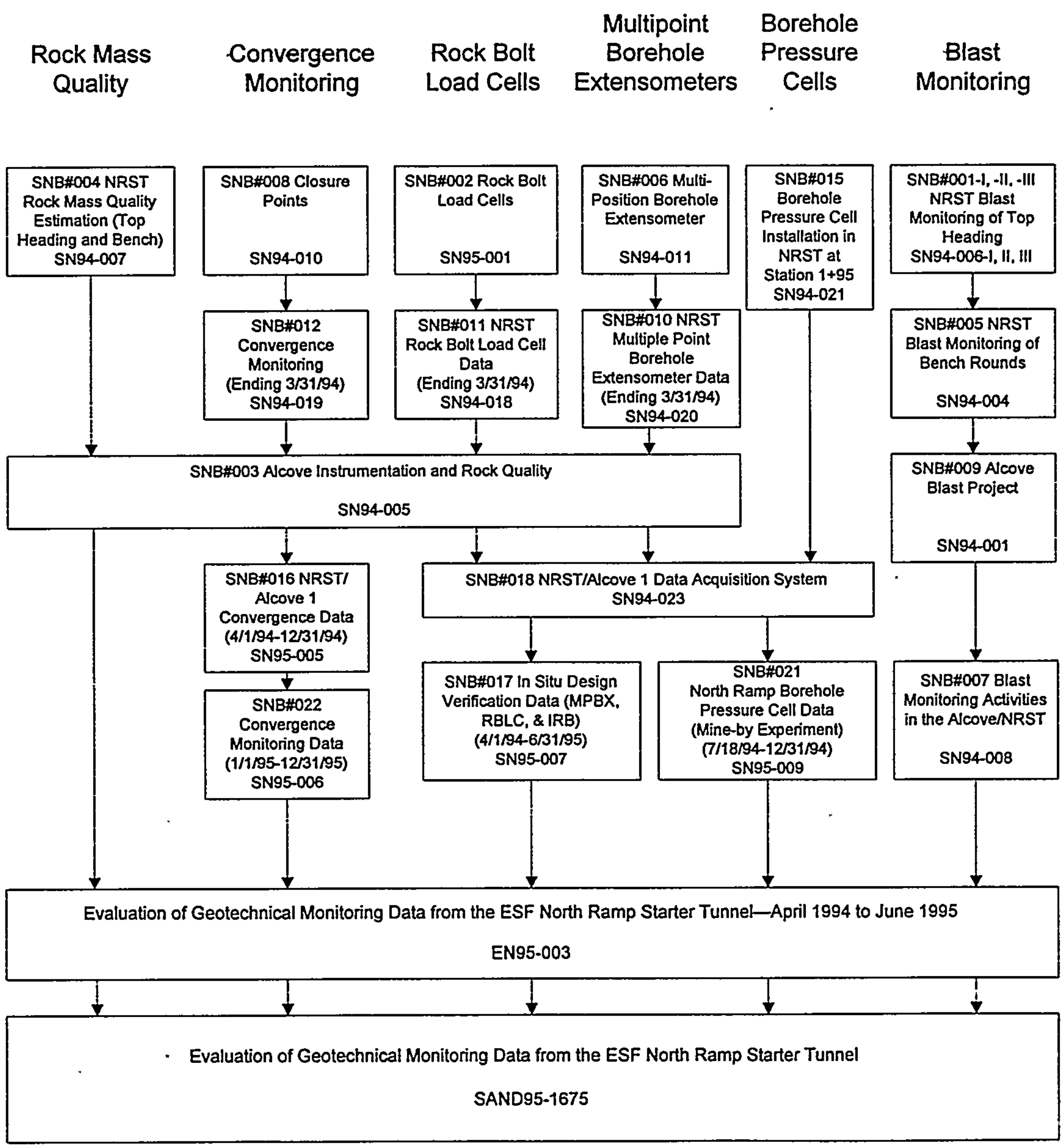
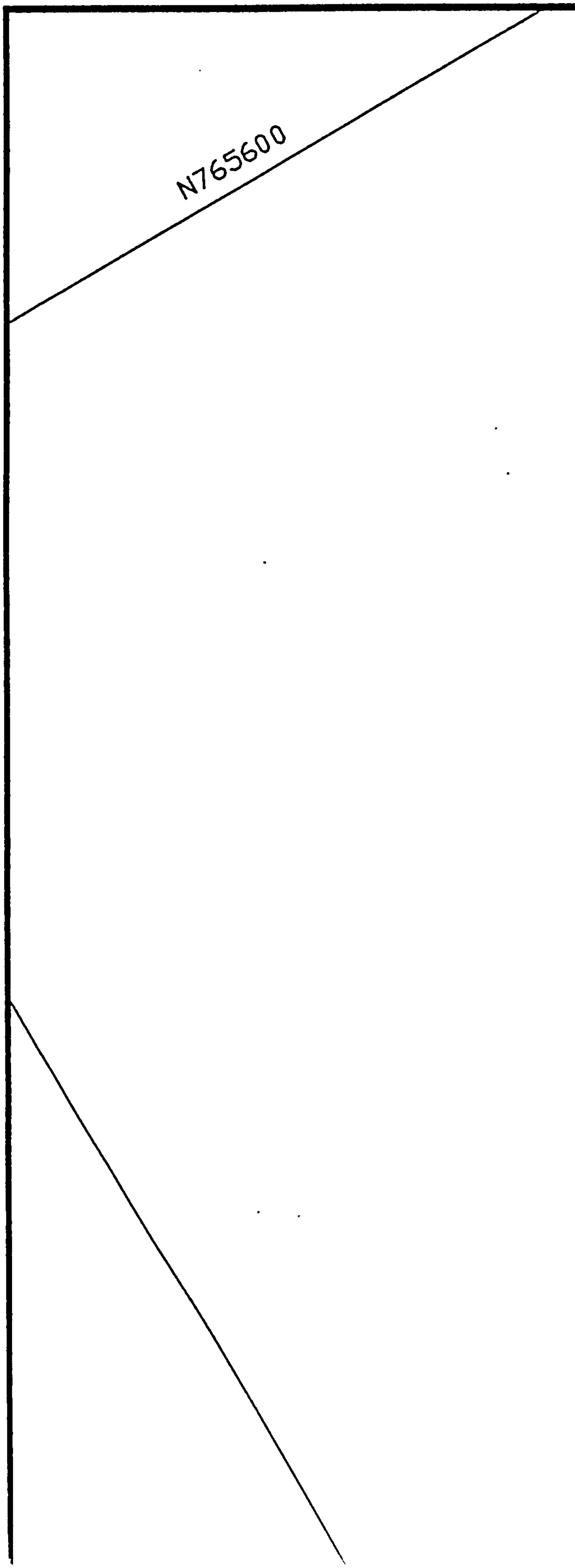


Figure A-1. Flow chart showing data collection activities and traceability of data in scientific notebooks.

Appendix B
NRST Plan Map with Instrument Locations

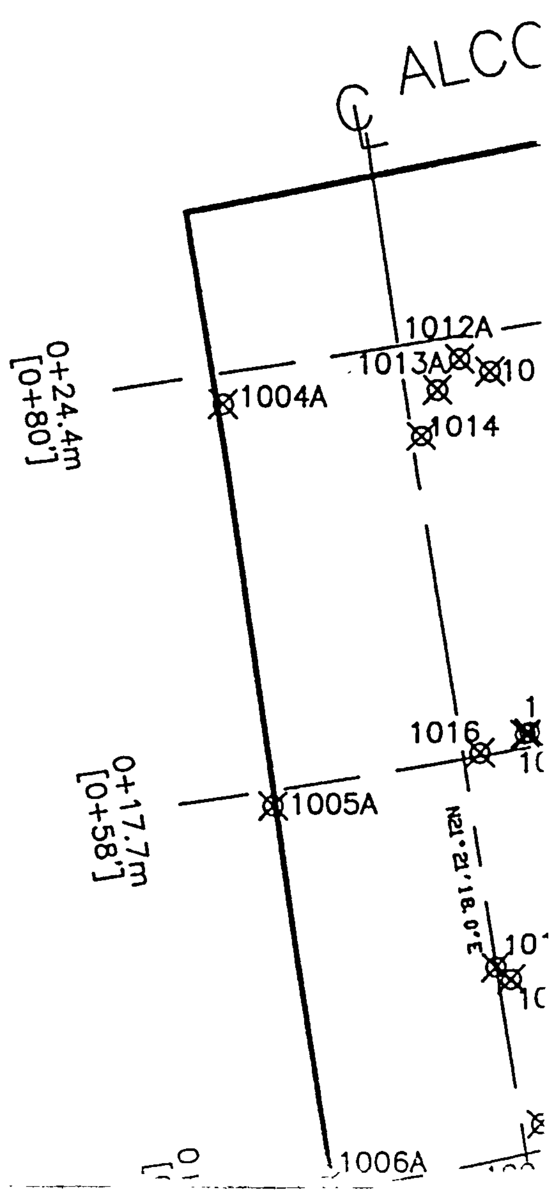


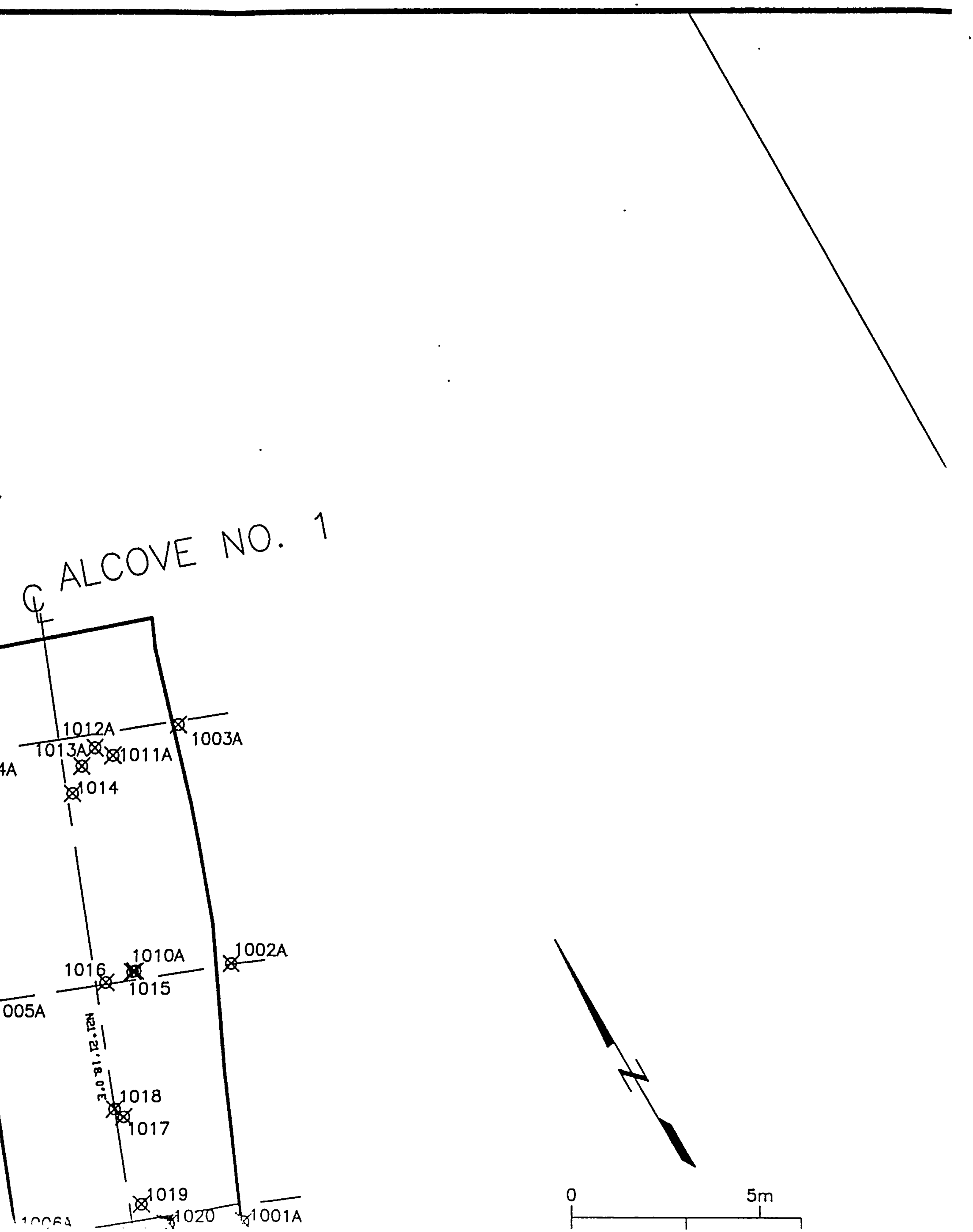
Point ID	Northing (ft)	Easting (ft)	Elev. (ft)	
200	765359.44	569802.38	3682.71	Convergenc
201	765345.67	569795.02	3701.25	Convergenc
202	765351.26	569798.12	3713.05	Convergenc
203	765367.02	569806.72	3713.03	Convergenc
204	765372.93	569809.83	3700.66	Convergenc
205	765366.75	569789.07	3683.22	Convergenc
206	765353.25	569781.59	3701.62	Convergenc
207	765358.56	569784.57	3712.87	Convergenc
208	765374.29	569793.57	3713.25	Convergenc
209	765379.99	569796.69	3700.78	Convergenc
210	765380.02	569765.24	3682.55	Convergenc
211	765367.12	569757.97	3698.90	Convergenc
212	765371.95	569760.65	3712.02	Convergenc
215	765386.77	569771.01	3712.01	Convergenc
216	765391.75	569773.04	3699.77	Convergenc
217	765383.39	569727.67	3699.02	Convergenc
218	765387.04	569731.52	3712.78	Convergenc
219	765387.52	569731.10	3712.62	Rock Bolt Lc
220	765396.92	569736.90	3715.97	Rock Bolt Lc
221	765405.12	569738.39	3712.81	Convergenc
222	765407.59	569740.30	3708.59	Rock Bolt Lc
223	765393.12	569710.92	3699.30	Convergenc
224	765410.23	569720.66	3716.99	Rock Bolt Lc
225	765415.93	569723.24	3711.88	Rock Bolt Lc
226	765406.21	569685.78	3699.07	Convergenc
228	765410.89	569689.74	3711.84	Convergenc
229	765426.38	569694.23	3712.99	Extensomet
230	765428.75	569696.41	3711.24	Convergenc
232	765427.54	569646.87	3700.76	Convergenc
233	765434.13	569650.41	3713.12	Convergenc
234	765434.41	569649.89	3712.94	Rock Bolt Lc
235	765396.58	569734.78	3682.52	Convergenc
236	765406.23	569717.56	3689.03	Convergenc
237	765420.79	569690.77	3689.11	Convergenc
238	765441.16	569654.44	3716.91	Rock Bolt Lc
239	765449.45	569658.90	3711.96	Rock Bolt Lc
240	765449.09	569659.50	3711.93	Convergenc
244	765441.57	569653.62	3683.32	Convergenc
245	765453.82	569661.26	3698.39	Convergenc
247	765397.80	569714.61	3715.81	Rock Bolt Lc
248	765398.78	569716.21	3718.04	Convergenc
249	765413.36	569722.09	3716.54	Convergenc
250	765421.05	569726.88	3699.31	Convergenc
251	765409.97	569740.88	3698.66	Convergenc

Point ID	Northing (ft)	Easting (ft)	Elev. (ft)	
301 A	765463.24	569645.25	3698.62	Borehole Pre
302 A	765460.38	569648.04	3698.41	Borehole Pre
303 A	765464.48	569644.86	3698.68	Borehole Pre
304 A	765462.64	569646.55	3698.54	Borehole Pre
305 A	765467.01	569644.03	3698.72	Borehole Pre
306 A	765465.10	569645.64	3698.60	Borehole Pre

Elev. (ft)	Descriptor
3682.71	Convergence Pin 0+04.6 F
3701.25	Convergence Pin 0+04.6 SLL
3713.05	Convergence Pin 0+04.6 UL
3713.03	Convergence Pin 0+04.6 UR
3700.66	Convergence Pin 0+04.6 SLR
3683.22	Convergence Pin 0+09.1 F
3701.62	Convergence Pin 0+09.1 SLL
3712.87	Convergence Pin 0+09.1 UL
3713.25	Convergence Pin 0+09.1 UR
3700.78	Convergence Pin 0+09.1 SLR
3682.55	Convergence Pin 0+16.8 F
3698.90	Convergence Pin 0+16.8 SLL
3712.02	Convergence Pin 0+16.8 UL
3712.01	Convergence Pin 0+16.8 UR
3699.77	Convergence Pin 0+16.8 SLR
3699.02	Convergence Pin 0+27.4 SLL
3712.78	Convergence Pin 0+27.4 UL
3712.62	Rock Bolt Load Cell 0+27.4 L
3715.97	Rock Bolt Load Cell 0+27.4 C
3712.81	Convergence Pin 0+27.4 UR
3708.59	Rock Bolt Load Cell 0+27.4 R
3699.30	Convergence Pin 0+33.5 SLL
3716.99	Rock Bolt Load Cell 0+33.5 C
3711.88	Rock Bolt Load Cell 0+33.5 R
3699.07	Convergence Pin 0+42.7 SLL
3711.84	Convergence Pin 0+42.7 UL
3712.99	Extensometer (MPBX) 0+42.7V
3711.24	Convergence Pin 0+42.7 UR
3700.76	Convergence Pin 0+56.3 SLL
3713.12	Convergence Pin 0+56.3 UL
3712.94	Rock Bolt Load Cell 0+56.3 L
3682.52	Convergence Pin 0+27.4 F
3689.03	Convergence Pin 0+33.5 F
3689.11	Convergence Pin 0+42.7 F
3716.91	Rock Bolt Load Cell 0+56.3 C
3711.96	Rock Bolt Load Cell 0+56.3 R
3711.93	Convergence Pin 0+56.3 UR
3683.32	Convergence Pin 0+56.3 F
3698.39	Convergence Pin 0+56.3 SLR
3715.81	Rock Bolt Load Cell 0+33.5 L
3718.04	Convergence Pin 0+33.5 UL
3716.54	Convergence Pin 0+33.5 UR
3699.31	Convergence Pin 0+33.5 SLR
3698.66	Convergence Pin 0+27.4 SLR

Elev. (ft)	Descriptor
3698.62	Borehole Pressure Cell (BHPC) V1
3698.41	Borehole Pressure Cell (BHPC) H1
3698.68	Borehole Pressure Cell (BHPC) H2
3698.54	Borehole Pressure Cell (BHPC) V2
3698.72	Borehole Pressure Cell (BHPC) H3
3698.60	Borehole Pressure Cell (BHPC) V3





ALCOVE NO. 1
1012A
1013A
1011A
1003A
4A
1014
1010A
1002A
1016
1015
005A
N21°21'18.0"E
1018
1017
1019
1020
1001A
0
5m

Point ID	Northing (ft)	Easting (ft)	Elev. (ft)	Descriptor
1000 A	765432.79	569707.74	3692.32	Convergence Pin A1 0+04.6 A1R
1001 A	765450.01	569713.84	3693.01	Convergence Pin A1 0+11.3 SLR
1002 A	765470.21	569724.23	3692.42	Convergence Pin A1 0+17.7 SLR
1003 A	765490.89	569730.75	3693.84	Convergence Pin A1 0+24.4 SLR
1004 A	765497.03	569711.59	3693.12	Convergence Pin A1 0+24.4 SLL
1005 A	765477.52	569703.20	3692.69	Convergence Pin A1 0+17.7 SLL
1006 A	765459.06	569695.61	3692.85	Convergence Pin A1 0+11.3 SLL
1007 A	765440.66	569688.89	3693.07	Convergence Pin A1 0+04.6 A1L
1008 A	765434.96	569699.86	3683.86	Convergence Pin A1 0+04.6 F
1009 A	765453.41	569707.75	3684.51	Convergence Pin A1 0+11.3 F
1010 A	765474.00	569716.41	3684.37	Convergence Pin A1 0+17.7 F
1011 A	765491.45	569724.50	3685.02	Convergence Pin A1 0+24.4 F
1012 A	765492.81	569723.46	3702.94	Convergence Pin A1 0+24.4 U
1013 A	765492.02	569721.66	3703.31	Instrumented Rock Bolt A1 0+23.8 U
1014	765490.35	569719.75	3703.49	Extensometer (MPBX) A1 0+24.1 V
1015	765473.94	569716.64	3701.56	Convergence Pin A1 0+17.7 U
1016	765474.37	569713.90	3702.21	Instrumented Rock Bolt A1 0+17.7 U
1017	765463.30	569709.35	3702.53	Instrumented Rock Bolt A1 0+14.0 U
1018	765464.31	569709.00	3702.67	Extensometer (MPBX) A1 0+17.7 V
1019	765455.88	569706.83	3702.64	Instrumented Rock Bolt A1 0+11.9 U
1020	765453.22	569707.99	3701.95	Convergence Pin A1 0+11.3 U
1021	765452.26	569704.15	3701.58	Extensometer (MPBX) A1 0+11.3 V
1022	765434.86	569699.94	3702.33	Convergence Pin A1 0+04.6 U
1023	765433.57	569704.93	3699.42	Instrumented Rock Bolt A1 0+04.6 SLR
1024	765435.45	569698.76	3702.09	Instrumented Rock Bolt A1 0+04.6 U

Point ID	Northing (ft)	Easting (ft)	Elev. (ft)	Descriptor
106	765429.22	569643.71	3697.37	Extensometer (MPBX) 0+56.3 H

Point ID	Northing (ft)	Easting (ft)	Elev. (ft)	Descriptor
1000 B	765447.66	569654.42	3713.95	Extensometer (MPBX) 0+56.3 V
1001 B	765408.81	569687.53	3707.41	Rock Bolt Load Cell 0+42.7 L
1002 B	765419.72	569692.55	3714.52	Rock Bolt Load Cell 0+42.7 C
1003 B	765428.02	569697.14	3711.24	Rock Bolt Load Cell 0+42.7 R
1004 B	765388.28	569770.71	3711.43	Rock Bolt Load Cell 0+16.8 R
1005 B	765374.40	569761.99	3714.19	Rock Bolt Load Cell 0+16.8 L
1006 B	765379.15	569767.88	3715.38	Rock Bolt Load Cell 0+16.8 C
1007 B	765345.91	569806.63	3727.93	Rock Bolt Load Cell 0+00.0 L
1008 B	765357.53	569806.93	3731.13	Rock Bolt Load Cell 0+00.0 C
1009 B	765364.06	569816.42	3722.05	Rock Bolt Load Cell 0+00.0 R
1010 B	765369.69	569783.27	3683.46	Convergence Pin 0+10.7 F
1011 B	765373.11	569786.58	3715.76	Convergence Pin 0+10.7 UR
1012 B	765356.87	569774.39	3708.10	Convergence Pin 0+10.7 SLL
1013 B	765363.22	569779.87	3715.88	Convergence Pin 0+10.7 UL

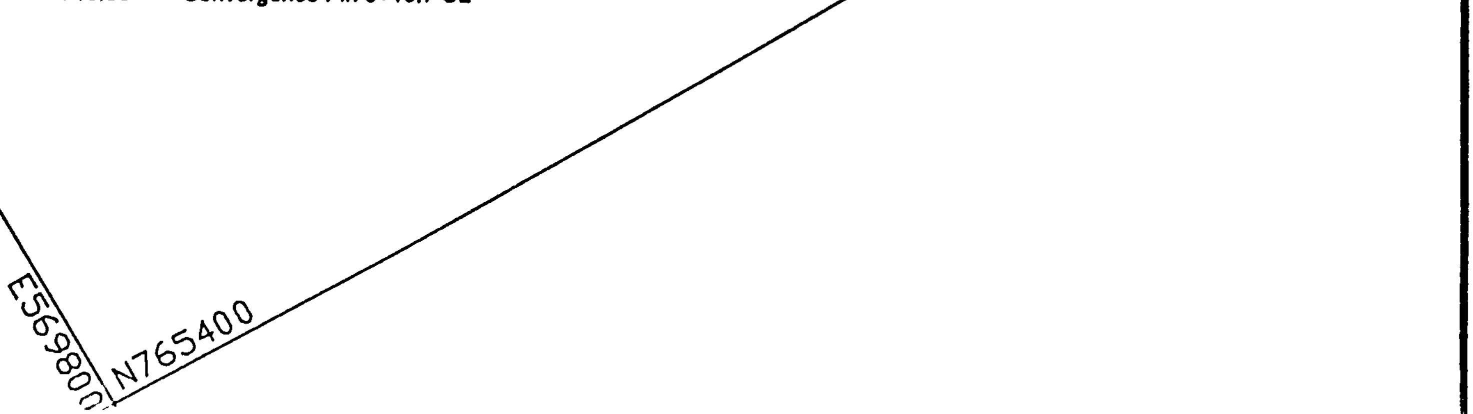
E569800

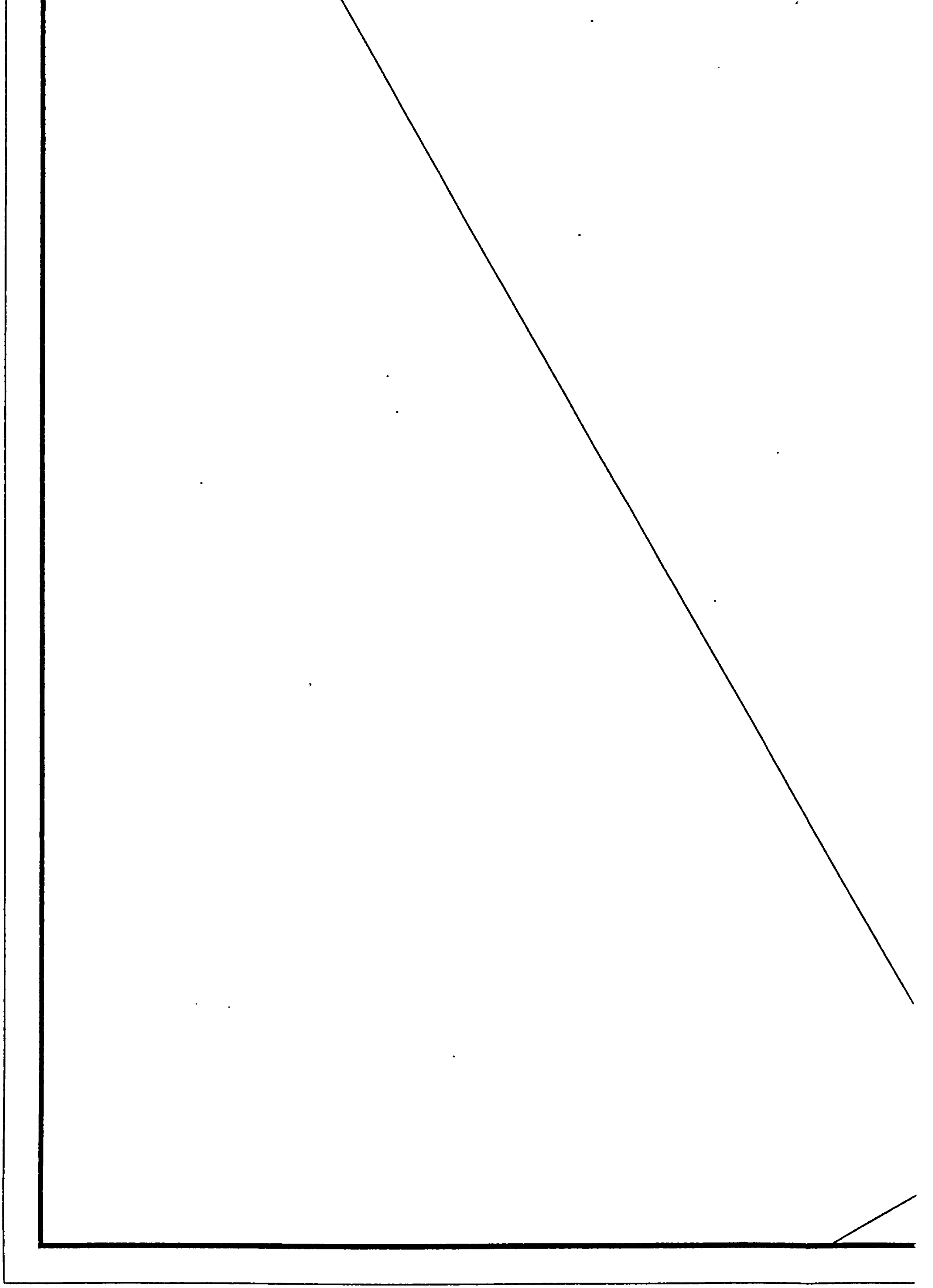
N765400

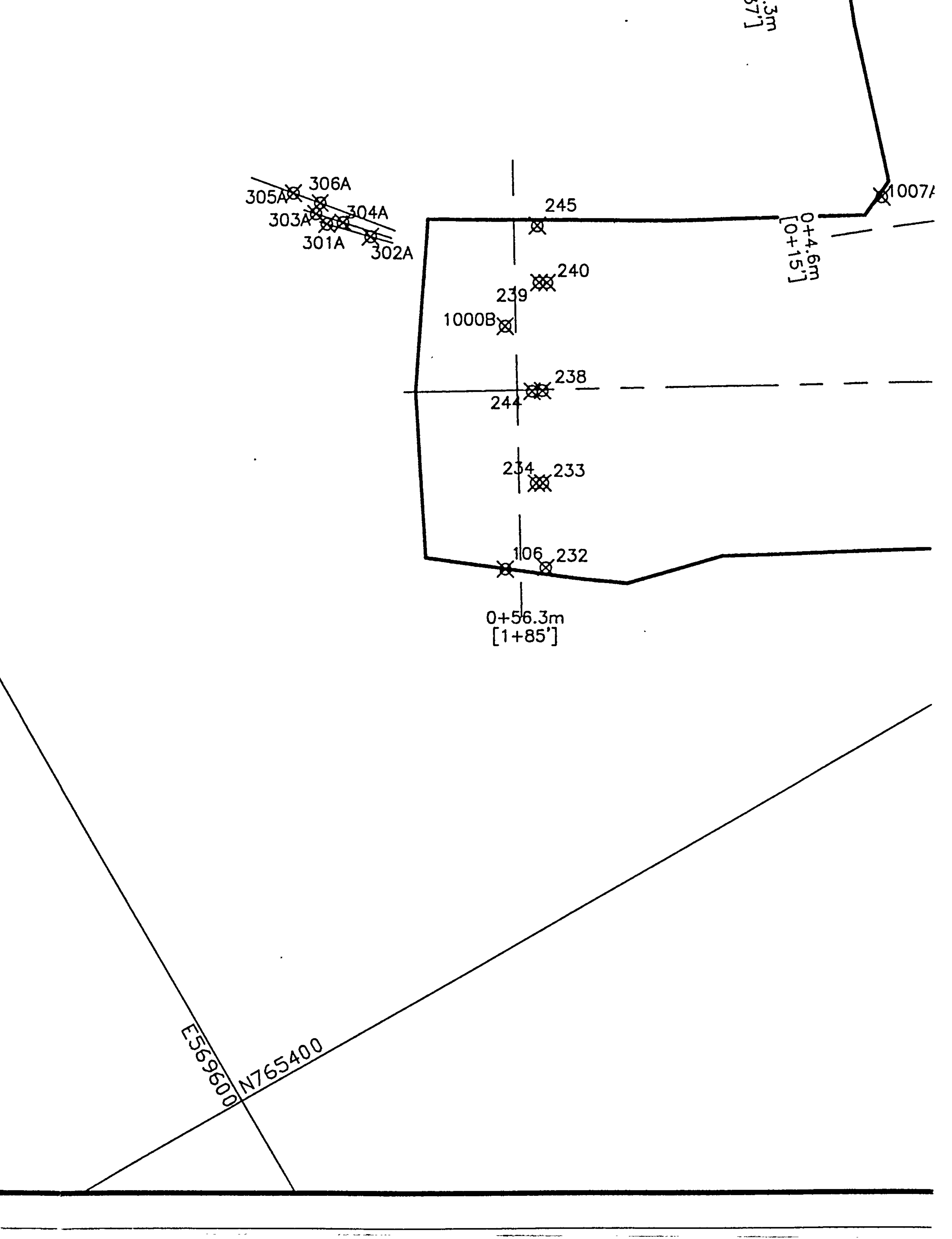
)	Elev. (ft)	Descriptor
4	3692.32	Convergence Pin A1 0+04.6 A1R
4	3693.01	Convergence Pin A1 0+11.3 SLR
3	3692.42	Convergence Pin A1 0+17.7 SLR
5	3693.84	Convergence Pin A1 0+24.4 SLR
9	3693.12	Convergence Pin A1 0+24.4 SLL
0	3692.69	Convergence Pin A1 0+17.7 SLL
1	3692.85	Convergence Pin A1 0+11.3 SLL
9	3693.07	Convergence Pin A1 0+04.6 A1L
3	3683.86	Convergence Pin A1 0+04.6 F
5	3684.51	Convergence Pin A1 0+11.3 F
1	3684.37	Convergence Pin A1 0+17.7 F
0	3685.02	Convergence Pin A1 0+24.4 F
3	3702.94	Convergence Pin A1 0+24.4 U
3	3703.31	Instrumented Rock Bolt A1 0+23.8 U
5	3703.49	Extensometer (MPBX) A1 0+24.1 V
1	3701.56	Convergence Pin A1 0+17.7 U
0	3702.21	Instrumented Rock Bolt A1 0+17.7 U
5	3702.53	Instrumented Rock Bolt A1 0+14.0 U
0	3702.67	Extensometer (MPBX) A1 0+17.7 V
3	3702.64	Instrumented Rock Bolt A1 0+11.9 U
9	3701.95	Convergence Pin A1 0+11.3 U
5	3701.58	Extensometer (MPBX) A1 0+11.3 V
1	3702.33	Convergence Pin A1 0+04.6 U
3	3699.42	Instrumented Rock Bolt A1 0+04.6 SLR
3	3702.09	Instrumented Rock Bolt A1 0+04.6 U

)	Elev. (ft)	Descriptor
	3697.37	Extensometer (MPBX) 0+56.3 H

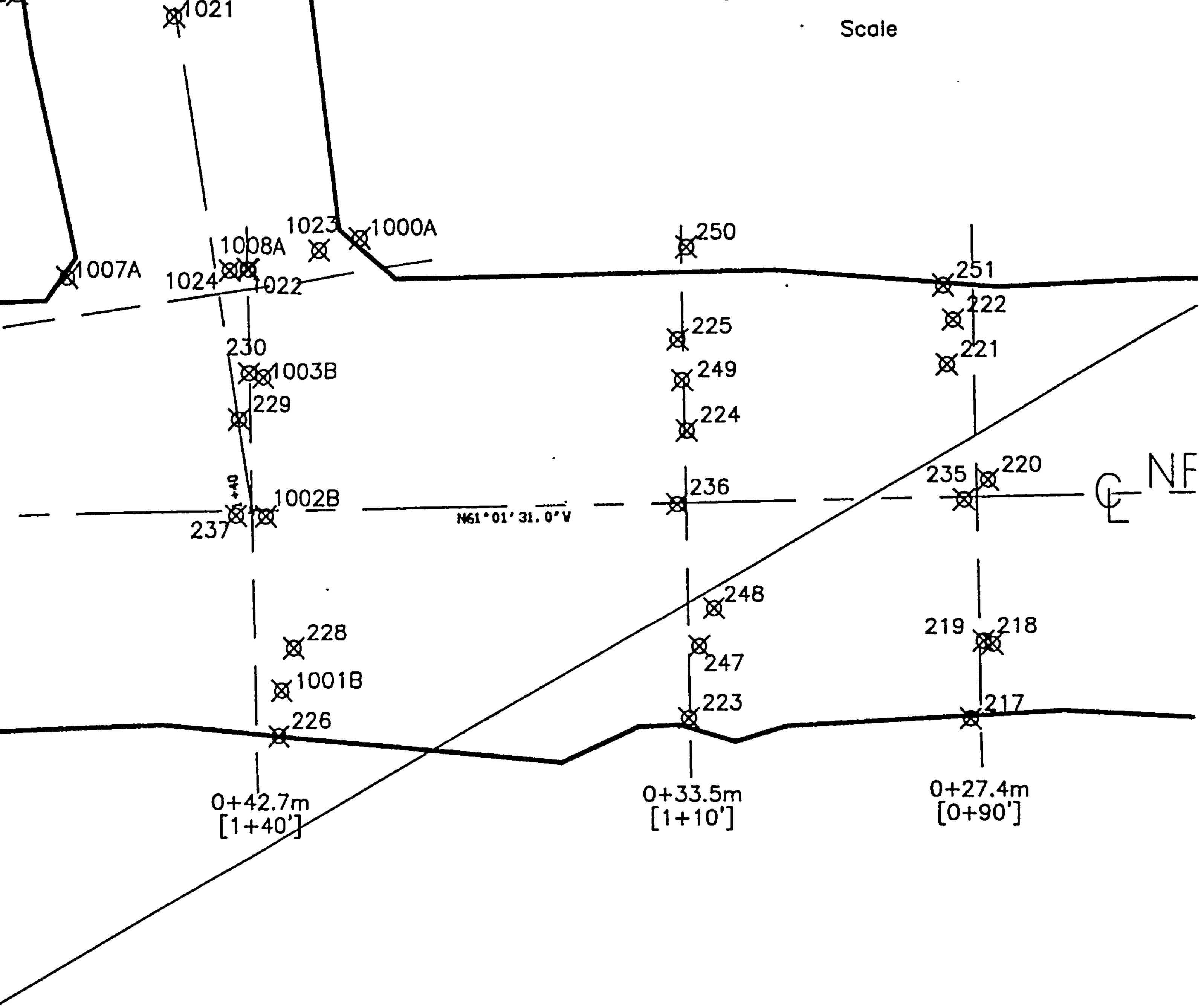
)	Elev. (ft)	Descriptor
2	3713.95	Extensometer (MPBX) 0+56.3 V
1	3707.41	Rock Bolt Load Cell 0+42.7 L
5	3714.52	Rock Bolt Load Cell 0+42.7 C
5	3711.24	Rock Bolt Load Cell 0+42.7 R
	3711.43	Rock Bolt Load Cell 0+16.8 R
0	3714.19	Rock Bolt Load Cell 0+16.8 L
3	3715.38	Rock Bolt Load Cell 0+16.8 C
1	3727.93	Rock Bolt Load Cell 0+00.0 L
1	3731.13	Rock Bolt Load Cell 0+00.0 C
2	3722.05	Rock Bolt Load Cell 0+00.0 R
7	3683.46	Convergence Pin 0+10.7 F
3	3715.76	Convergence Pin 0+10.7 UR
0	3708.10	Convergence Pin 0+10.7 SLL
7	3715.88	Convergence Pin 0+10.7 UL



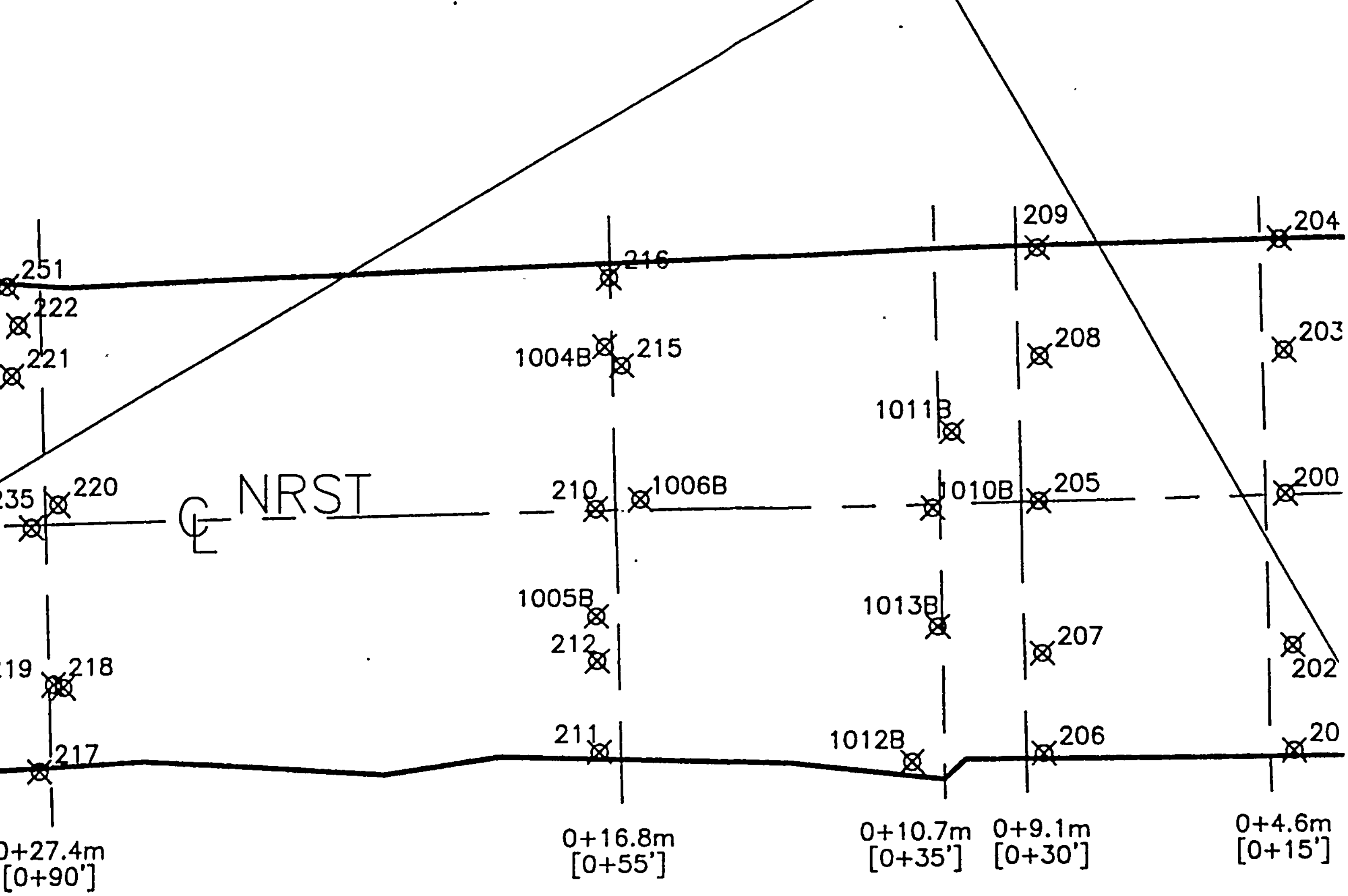




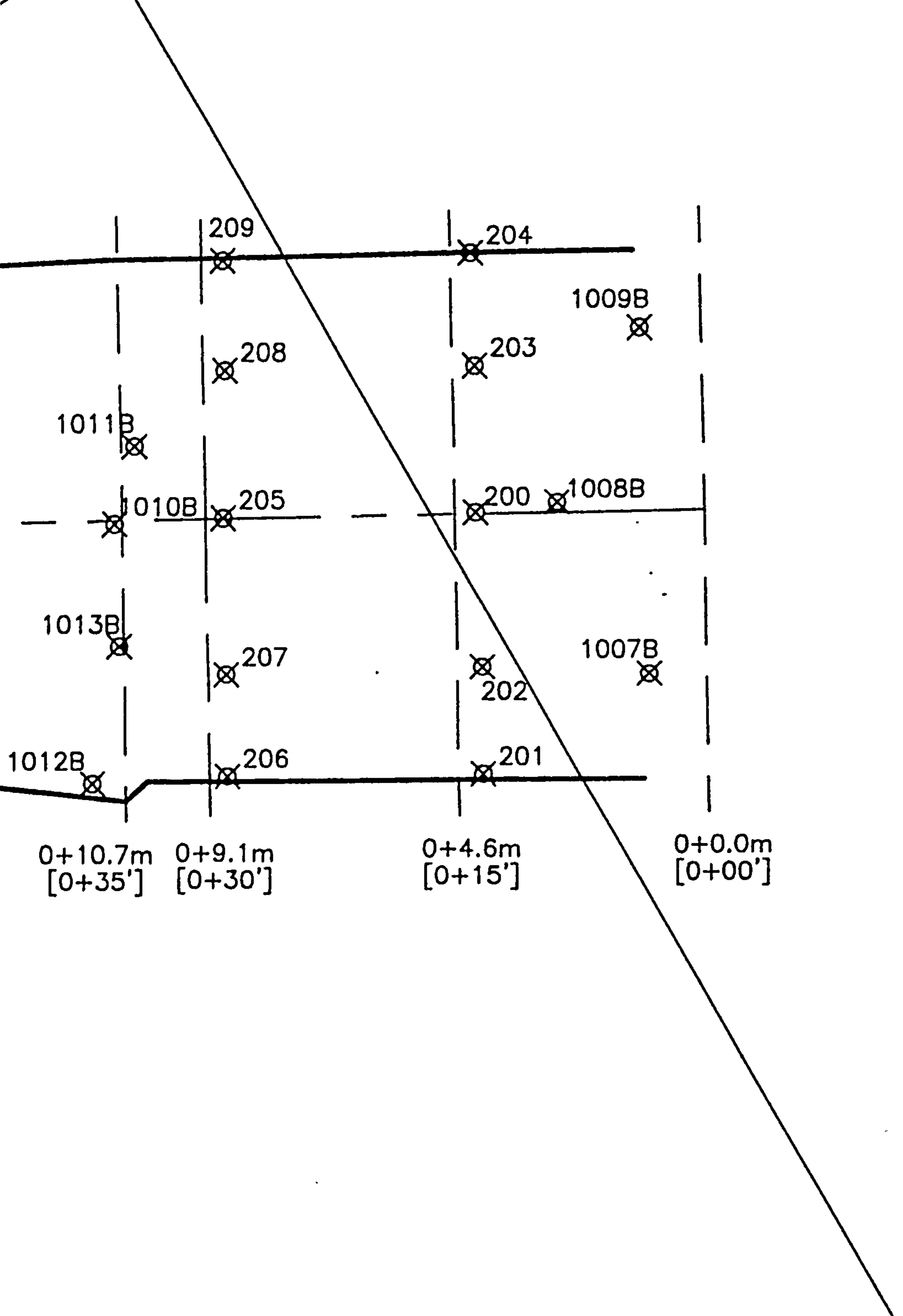
306A
305A
303A
304A
301A
302A
245
0+4.6m
[0+15']
1007A
240
239
1000B
238
244
234
233
106
232
0+56.3m
[1+85']
E569600
N765400



Scale
1021
1023
1000A
1008A
1007A
1024
1022
250
251
222
221
225
249
224
230
1003B
229
236
235
220
NF
1002B
237
1+40
N61°01'31.0"W
248
228
1001B
247
219
218
223
217
226
0+42.7m
[1+40']
0+33.5m
[1+10']
0+27.4m
[0+90']



NOTE: Tunnel outlines and grid from Raytheon Services Nevada – File ST.DXF



d grid from Raytheon Services
DXF

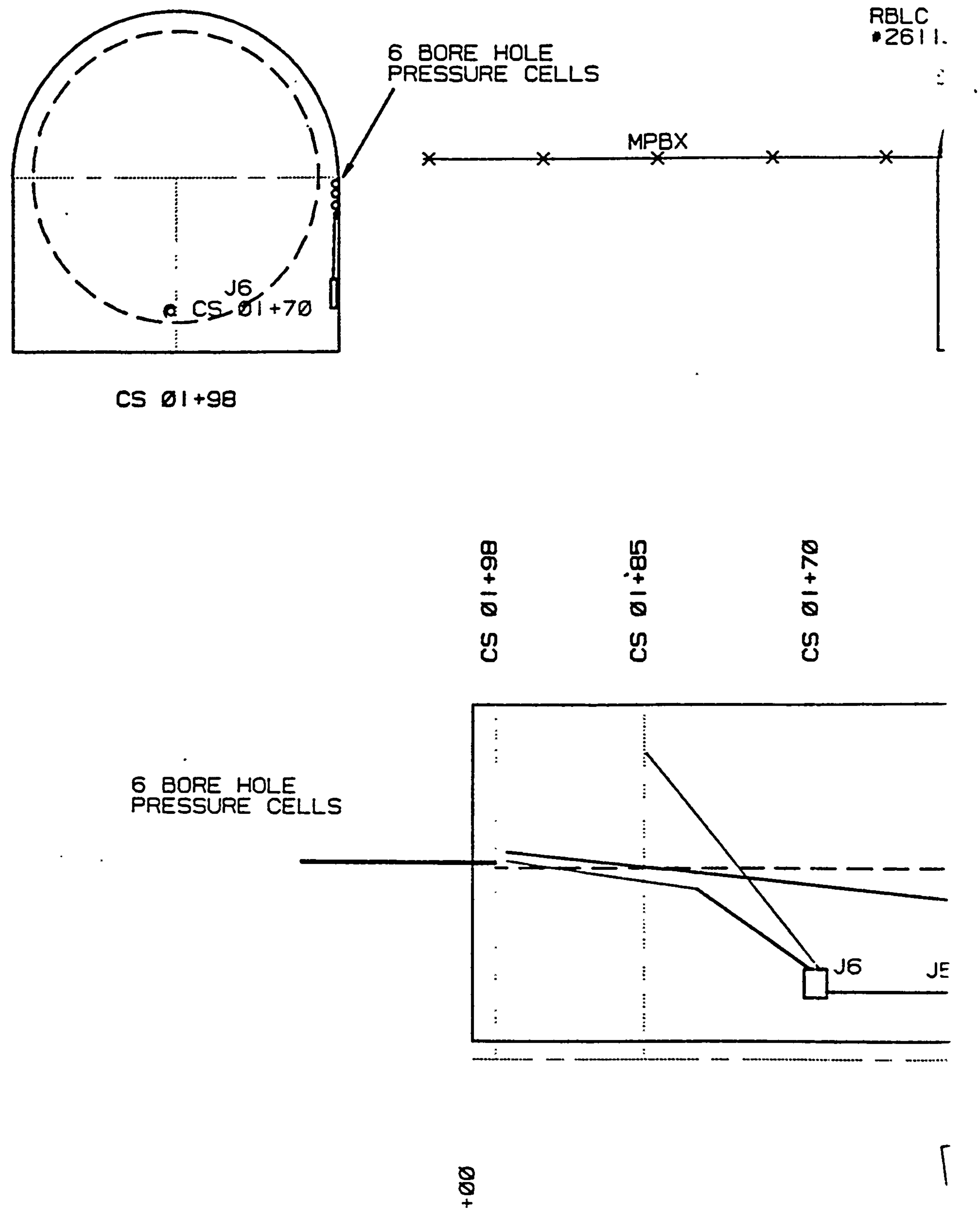
PLAN VIEW OF NORTH RAMP
STARTER TUNNEL AND ALCOVE NO. 1
AT 4 FEET ABOVE FLOOR

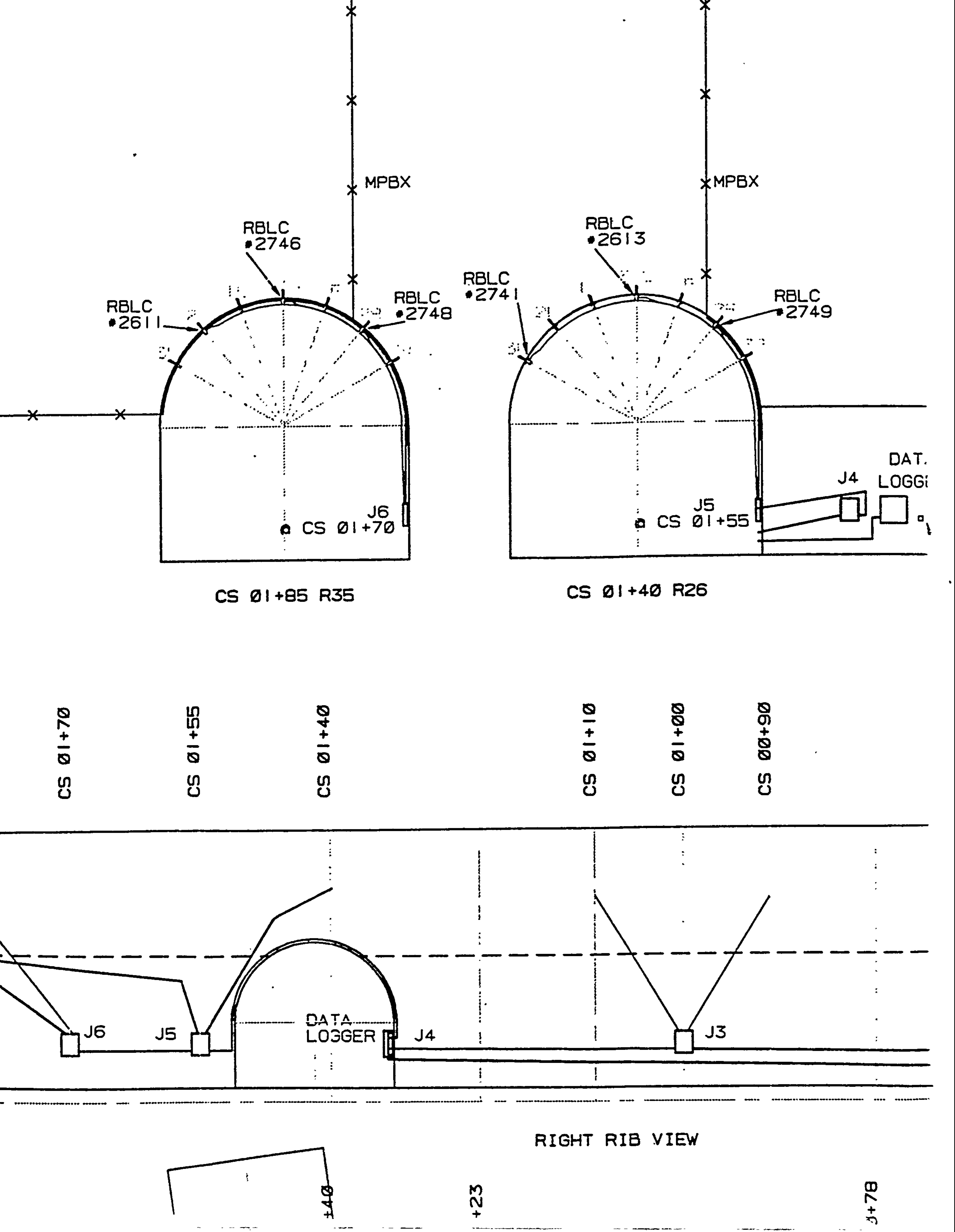
Sandia National Laboratories

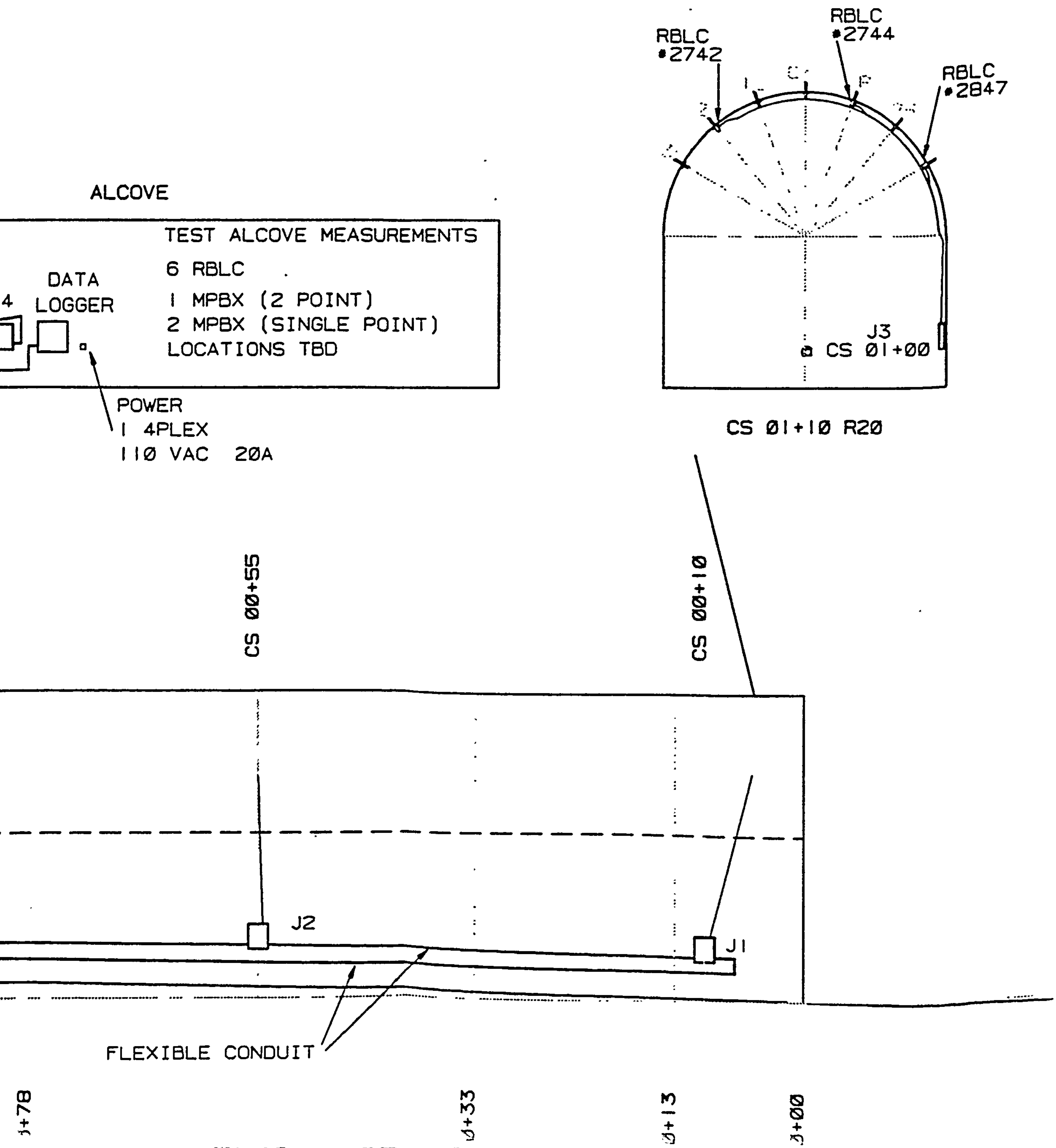
APPROVED BY: CEB DRAWN BY: RJL	DATE: 8-22-95 SCALE: AS SHOWN
AGAPITO ASSOCIATES, INC. GRAND JUNCTION, COLORADO, USA LAS VEGAS, NEVADA, USA	REV: 0 SHEET 1 OF 1

ACADR13 FILE: WIN\ST-DATA\P-INSTR.DWG

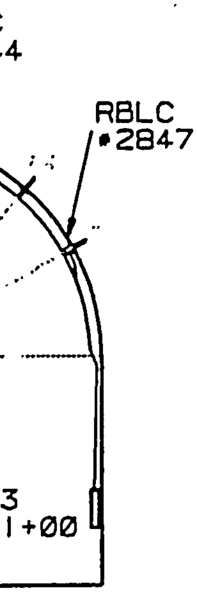
Appendix C
NRST Instrument Stations and Data Acquisition System


6 BORE HOLE
PRESSURE CELLS
J6
CS Ø1+70
CS Ø1+98
MPBX
RBLC
CS Ø1+98
CS Ø1+85
CS Ø1+70
6 BORE HOLE
PRESSURE CELLS
J6

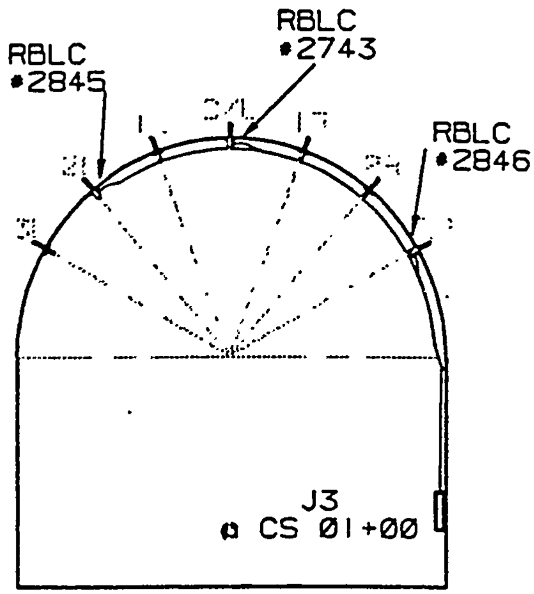

MPBX
MPBX
RBLC #2746
RBLC #2613
RBLC #2611
RBLC #2748
RBLC #2741
RBLC #2749
J6
CS Ø1+70
J5
CS Ø1+55
J4
DAT. LOGG
CS Ø1+85 R35
CS Ø1+40 R26
CS Ø1+70
CS Ø1+55
CS Ø1+40
CS Ø1+1Ø
CS Ø1+ØØ
CS ØØ+9Ø
J6
J5
DATA LOGGER
J4
J3
RIGHT RIB VIEW
+40
+23
+78



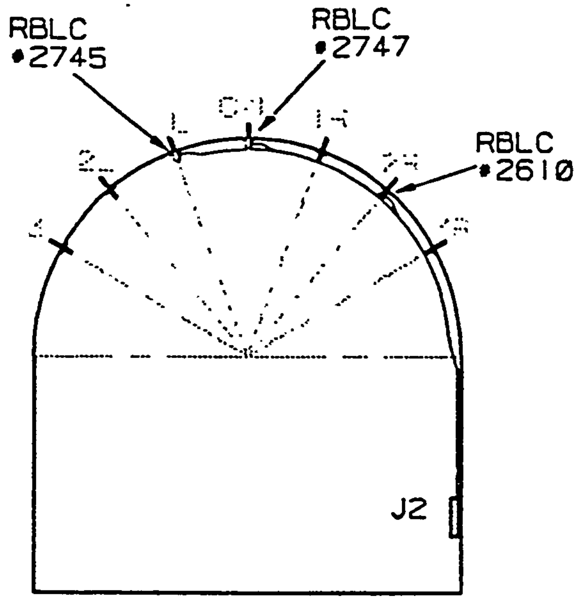
ALCOVE
TEST ALCOVE MEASUREMENTS
6 RBLC
1 MPBX (2 POINT)
2 MPBX (SINGLE POINT)
LOCATIONS TBD
DATA LOGGER
4
POWER
1 4PLEX
110 VAC 20A
RBLC #2742
RBLC #2744
RBLC #2847
J3
CS 01+00
CS 01+10 R20
CS 00+55
CS 00+10
J2
J1
FLEXIBLE CONDUIT
+78
+33
+13
+00



Ø



CS ØØ+9Ø R16



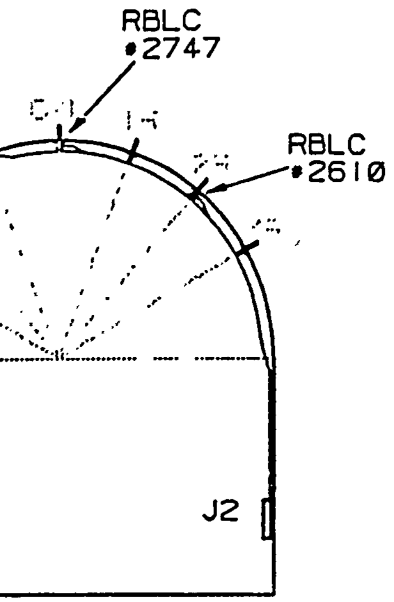
CS ØØ+55 R9

NOTES.

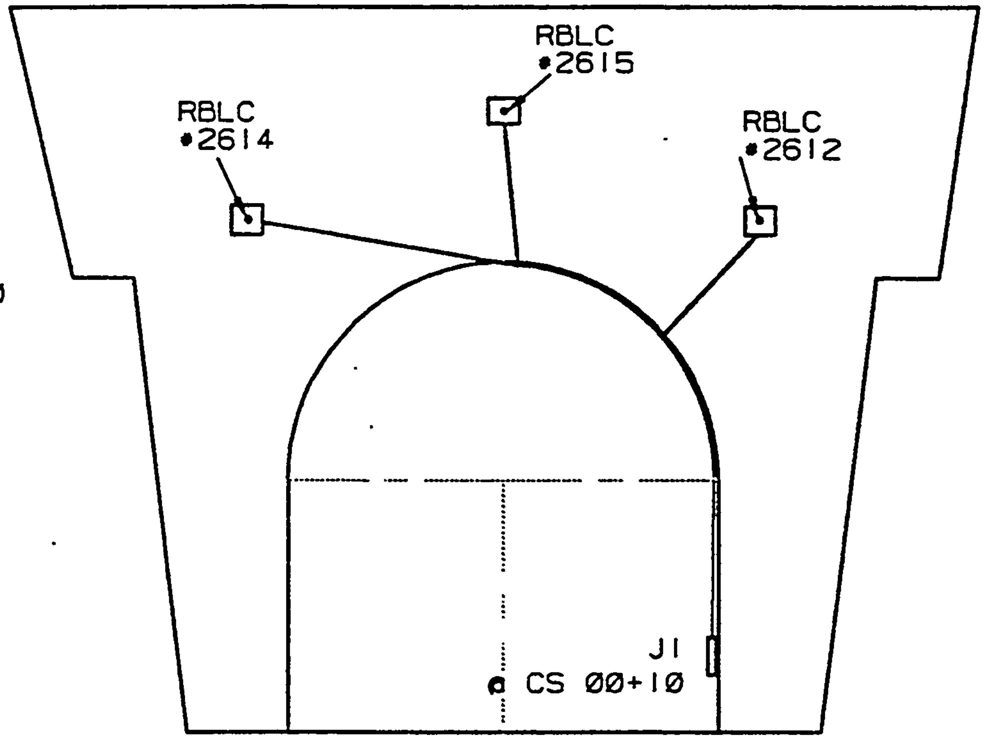
1. ALL NEMA CANS ARE TO BE MOUNTED
   ABOVE POURED INVERT.
2. NEMA CANS J1 THRU J6 AND THE DAT.
3. NEMA CANS J1 THRU J6 ARE 3Ø"X24"
4. DATA LOGGER NEMA CAN IS 36"X36"X
5. CABLE SIZES AND TYPES.

TRANSDUCER	CABLE TYPE
MPBX	7 PAIR
RBLC	3SPS •16
PRESSURE CELL	QUAD
	2Ø TSP

6. CONDUIT AND ASSOCIATED HARDWARE
7. THIS COLOR REPRESENTS REECo RESP



00+55 R9



CS 00+00

A CANS ARE TO BE MOUNTED TO THE RIB WITH CENTER OF CAN AT 5'
OURED INVERT.

NS J1 THRU J6 AND THE DATA LOGGER CAN ARE FURNISHED BY SANDIA.

NS J1 THRU J6 ARE 30"X24"X6".

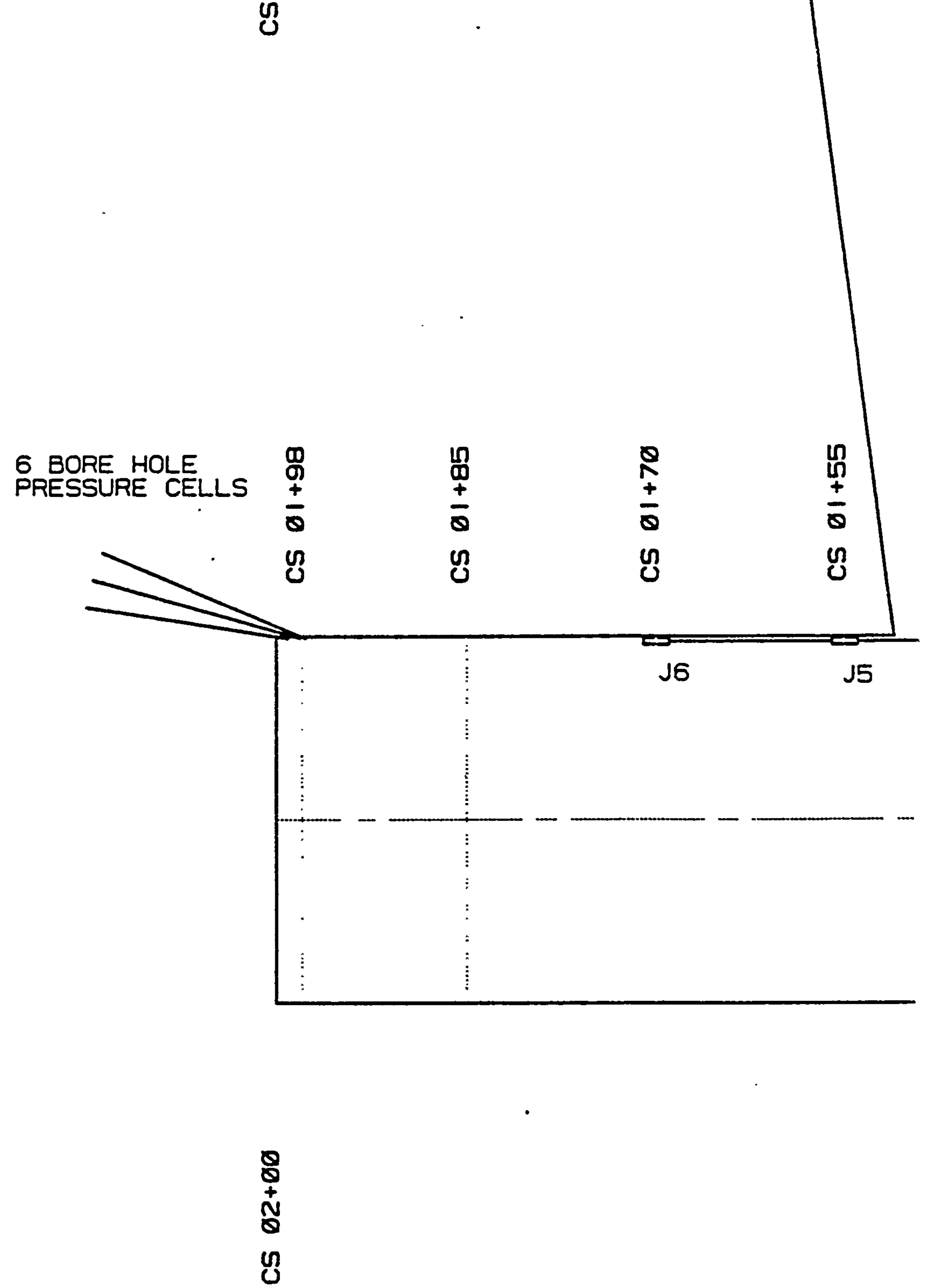
GGER NEMA CAN IS 36"X36"X12.

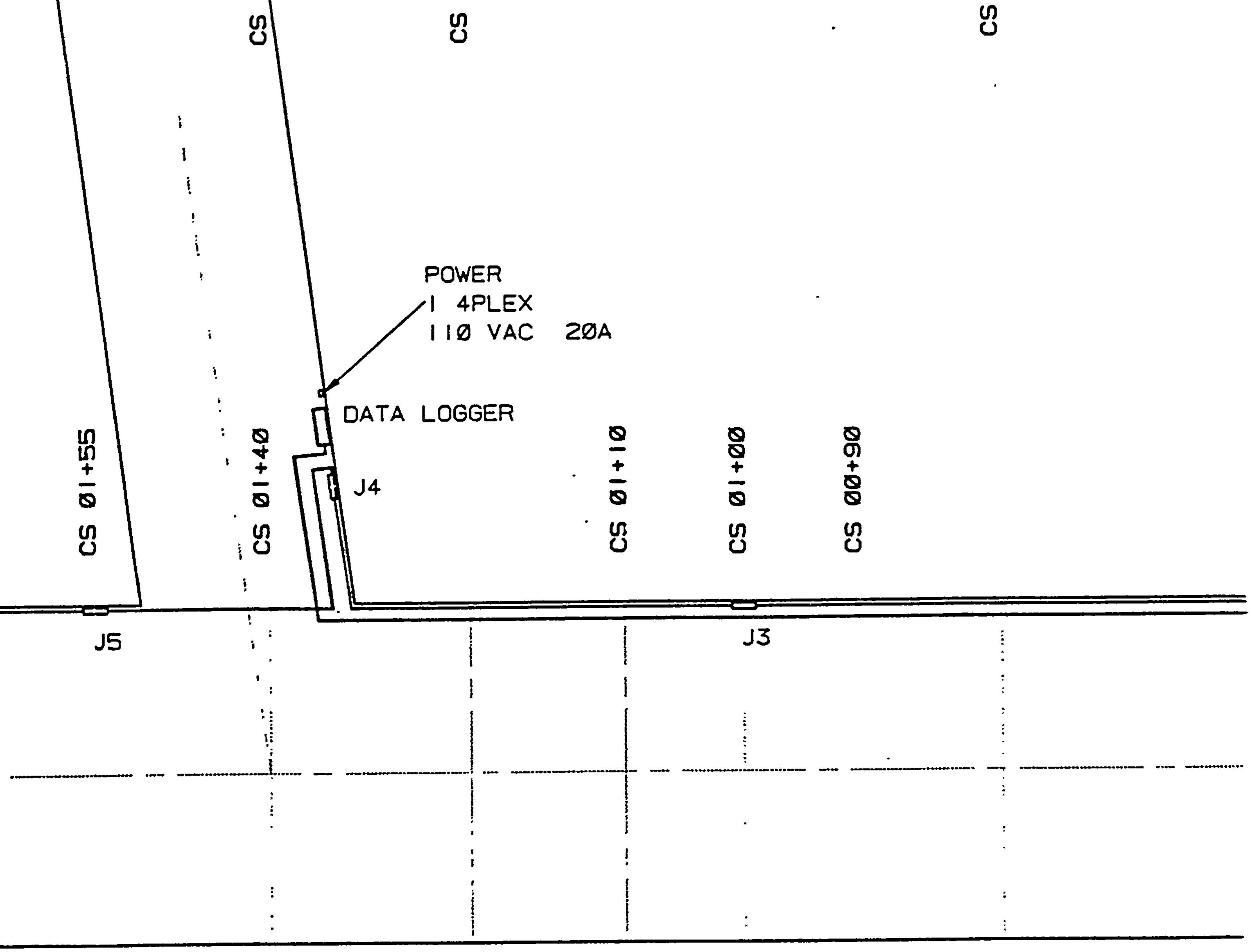
IZES AND TYPES.

TRANSDUCER	CABLE TYPE	CABLE O/D SIZE
MPBX	7 PAIR	1/2"
RBLC	3SPS #16	5/8"
PRESSURE CELL	QUAD	1/4"
	20 TSP	3/4"

AND ASSOCIATED HARDWARE SUPPLIED AND INSTALLED BY REECo.

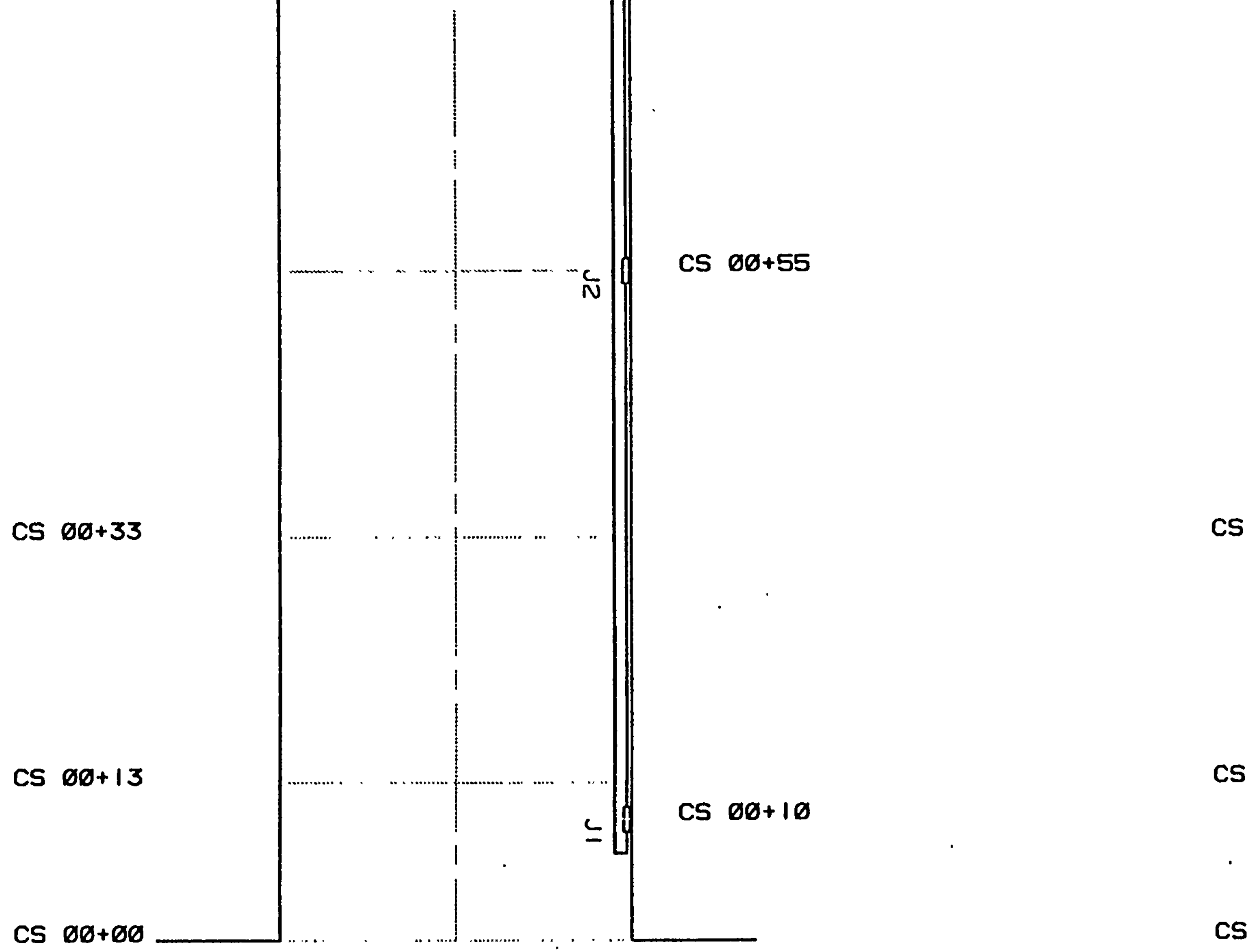
LCR REPRESENTS REECo RESPNSIBILITY.


CS
6 BORE HOLE PRESSURE CELLS
CS Ø1+98
CS Ø1+85
CS Ø1+70
CS Ø1+55
J6
J5
CS Ø2+ØØ


CS
CS
CS
POWER
1 4PLEX
110 VAC 20A
DATA LOGGER
J4
CS 01+55
CS 01+40
CS 01+10
CS 01+00
CS 00+90
J5
J3

PLAN VIEW


CS 01+40
CS 01+23
CS 00+78


CS 00+55
J2
CS 00+33
CS
CS 00+13
CS
CS 00+10
J1
CS 00+00
CS

8. THIS COLOR REPRESENTS SANDIA/AGAPITO F

9. 20 TSP CABLE IS RUN BETWEEN NEMA CANS

10. MPBX CABLES INSTALLED IN FLEXIBLE CON
CAN.

11. PRESSURE CELL CABLES INSTALLED IN FLE
TO NEMA CAN.

12. ROCK BOLT LOAD CELL CABLES DO NOT REQ

Sandia N

YUCCA MOUN

INSTRUMENT

DRAWING #E

12-1-93

COLOR REPRESENTS SANDIA/AGAPITO RESPONSIBILITY.

SP CABLE IS RUN BETWEEN NEMA CANS IN FLEXIBLE CONDUIT.

CABLES INSTALLED IN FLEXIBLE CONDUIT FROM DRILL HOLE TO NEMA

SURE CELL CABLES INSTALLED IN FLEXIBLE CONDUIT FROM DRILL HOLE
EMA CAN.

BOLT LOAD CELL CABLES DO NOT REQUIRE CONDUIT TO NEMA CAN.

Sandia National Laboratories

YUCCA MOUNTAIN PROJECT

INSTRUMENTATION LAYOUT

DRAWING #ESFDEF

12-1-93 JERRY CHAEL ORG 9324

PHONE (702) 295-4911

Appendix D
U.S. Bureau of Reclamation
Full Periphery Map/Starter Tunnel
DTN: GS940208314224.002

D

NOTES

1. Distinct shear zone intersects crown centerline at S
continues into right wall as intensely fractured zor
surfaces coated with up to 3 cm of opal and calcite.

2. Shear zone with breccia, no observed displacement ne
terminates in possible cooling fracture Sta. 0+57.

3. Sta. 0+05 to 0+20: Lithophysae aspect ratios range
to 3:1 in upper half of tunnel.

4. Cooling fracture with bedded sand infilling, exhibi-
on fracture surfaces consisting of elongate, anastor
subparallel channels extending 5mm into the wall roc
bounds a shear zone striking 235°, dipping 70-83° SV

5. Shear zone with crushed rock and breccia.

6. Zone of intensely fractured rock intersecting tunnel
in the left wall and Sta. 1+15 in the right wall.

7. Brownish gray to gray, densely welded, rhyolitic, as
Lithophysae comprise approximately 5-10% of the rock
average diameter 7-20 cm; lithophysae less than 1.5
filled with drusy quartz and opal.

8. Foliation more prominent in crown, and combined with th
fracture causes fallout resulting in small wedge-sha
crown.

9. Fault with crushed wall rock and sandy infilling. F
is offset approximately 1.5 ft.

10. Lithophysae: oblate to spheroidal; average size 5 c
from Sta. 0+92 to Sta. 1+00; from Sta. 1+00 to 1+05
20 cm, maximum size is 45 cm.

C

e at Sta. 0+17 and
ed zone. Fracture
lcite.

ent near crown,
+57.

ange from 1:4 (L:H)

xhibits decorations
nastomosing to
l rock. The fracture
33° SW.

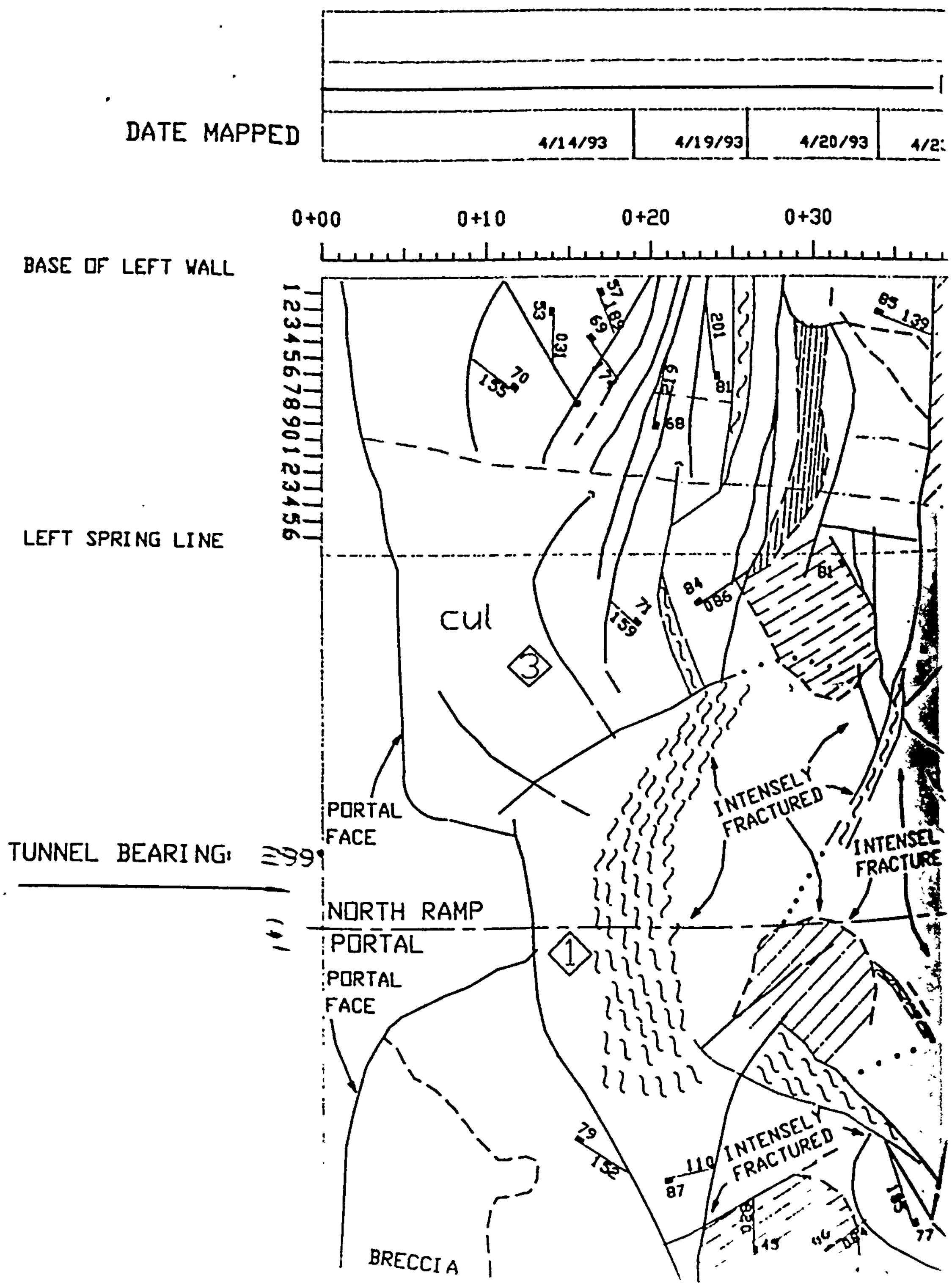
tunnel at Sta. 0+50
l.

c, ash-flow tuff.
e rock by volume;
n 1.5 cm are typically

d with high-angle
ge-shaped casts in the

ng. Foliation trace

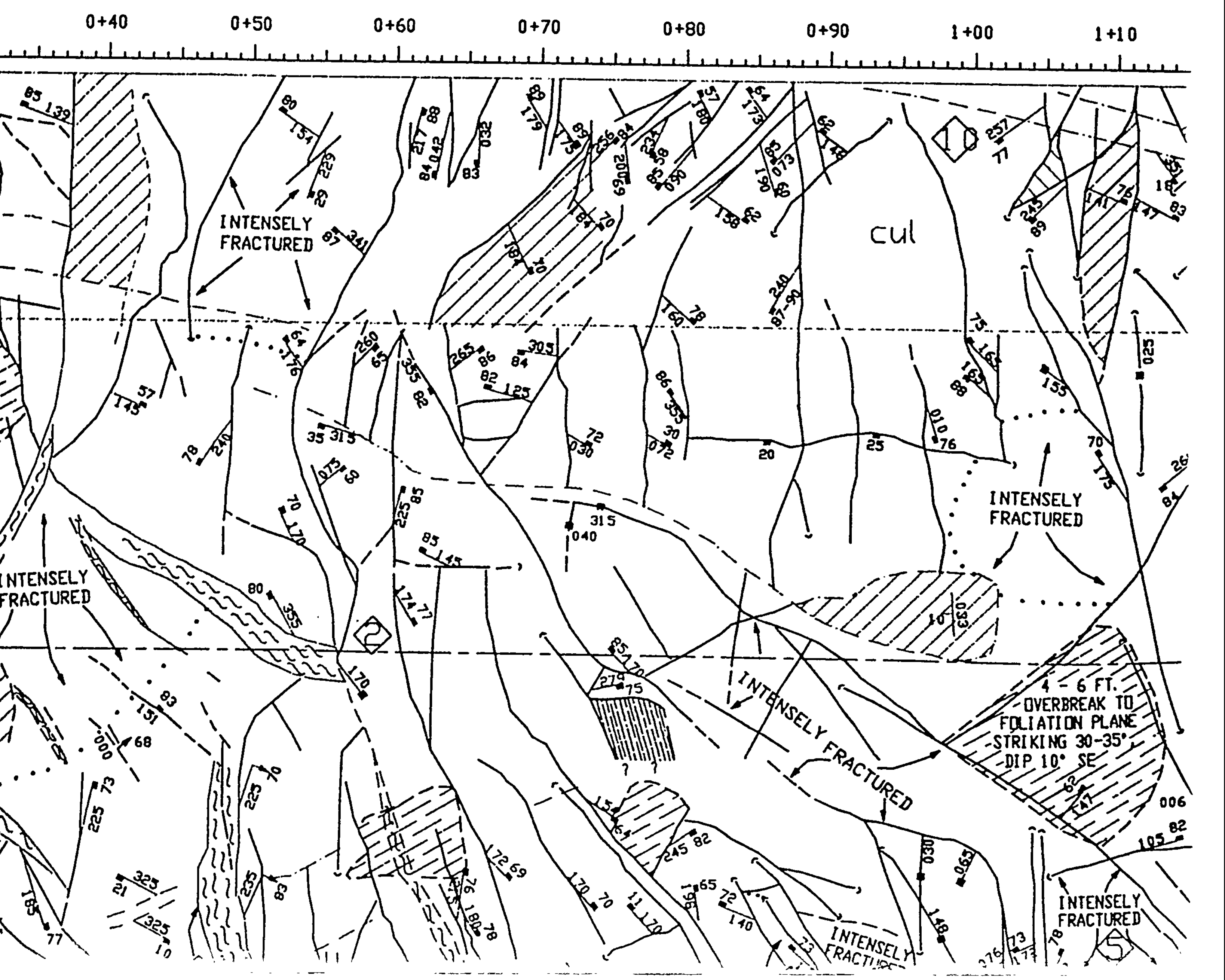
ze 5 cm diameter
1+05 average size is



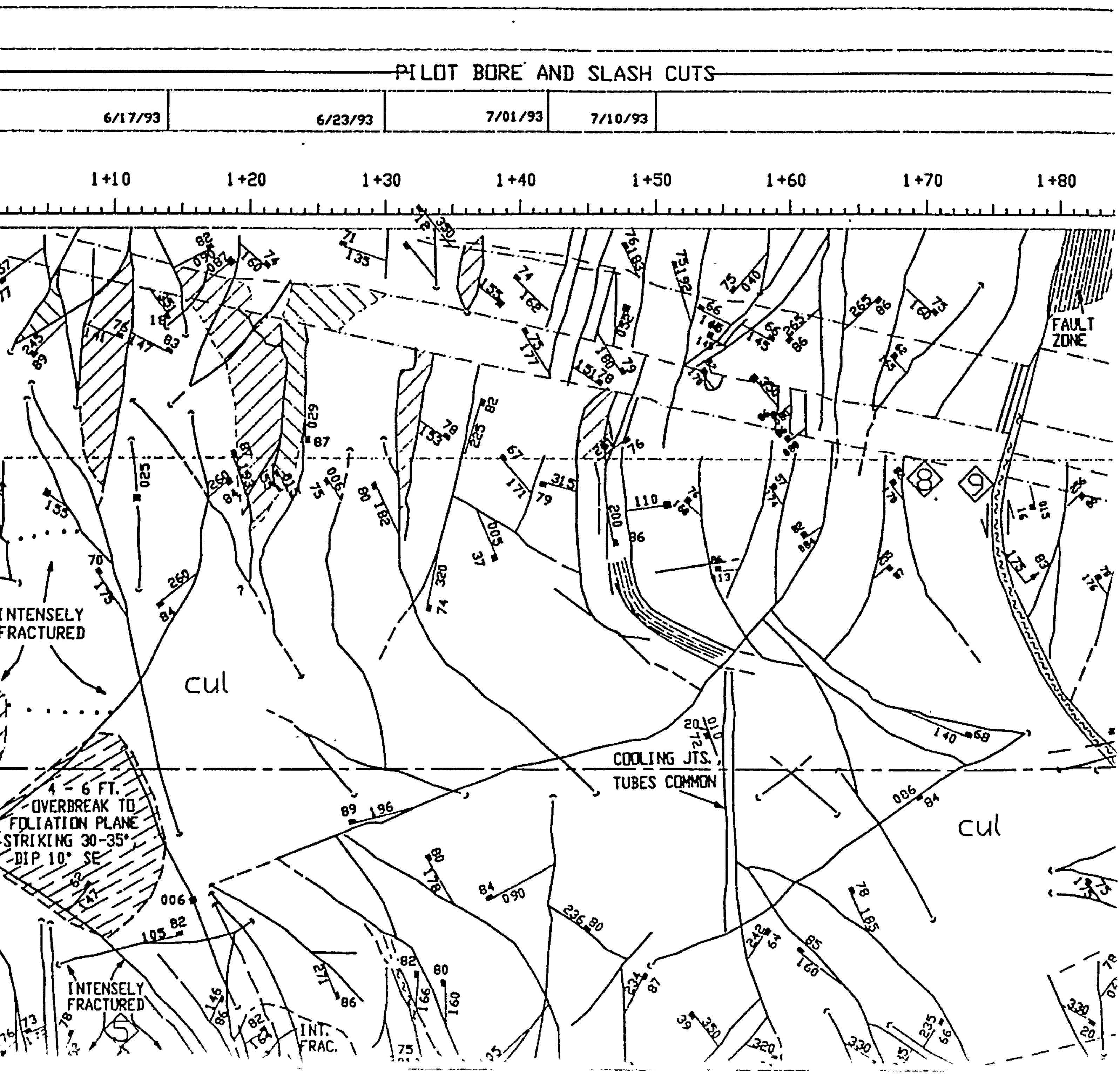
3

—PILOT BORE—

93	4/23/93	4/26/93	4/28/93	4/30/93	5/03/93	5/13/93	5/17/93	6/17/93

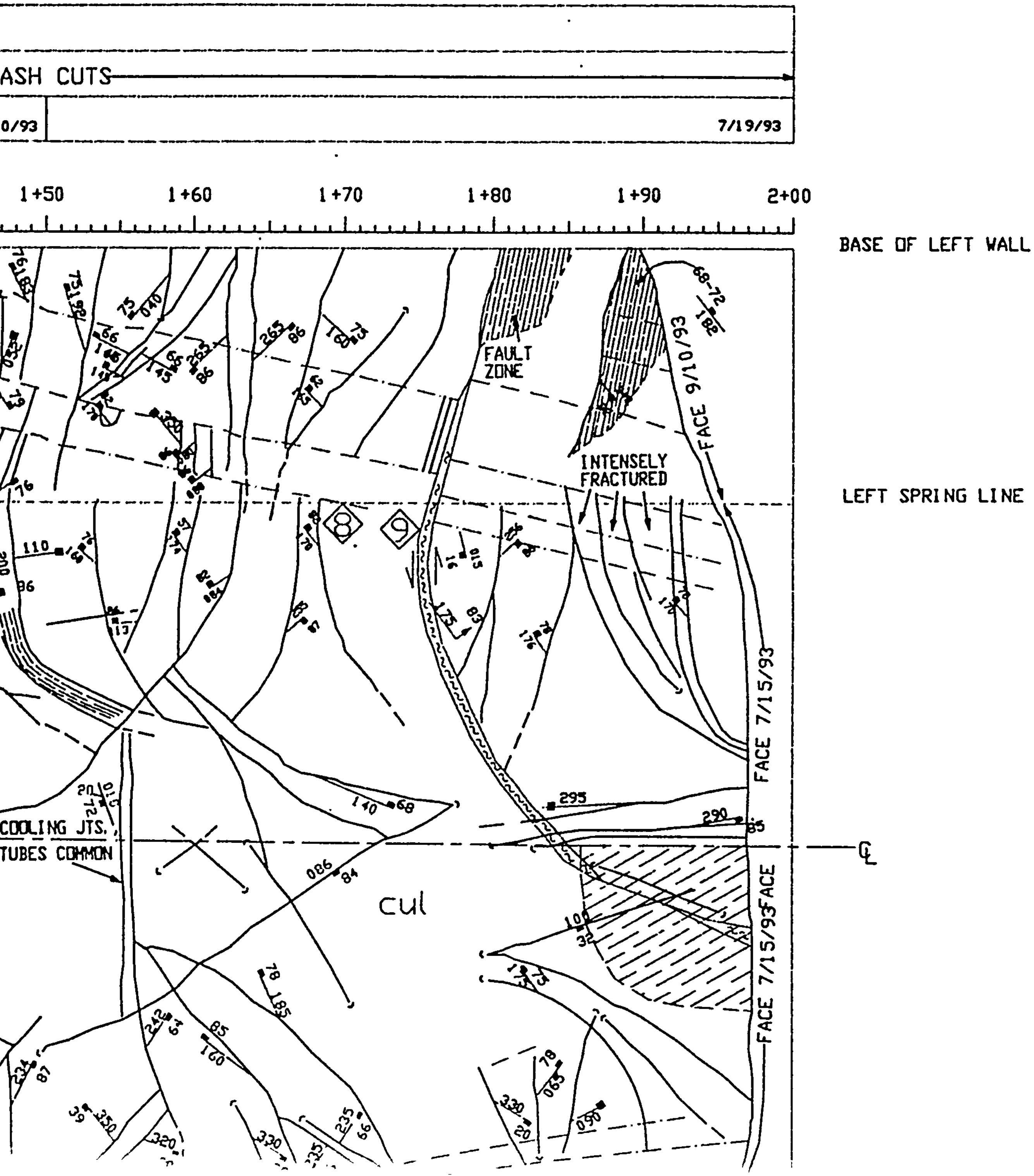


2


PILOT BORE AND SLASH CUTS
6/17/93
6/23/93
7/01/93
7/10/93
1+10
1+20
1+30
1+40
1+50
1+60
1+70
1+80
FAULT ZONE
INTENSELY FRACTURED
cul
4 - 6 FT. OVERBREAK TO FOLIATION PLANE STRIKING 30-35° DIP 10° SE
COOLING JTS. TUBES COMMON
cul
INTENSELY FRACTURED
INT. FRAC.

1

D

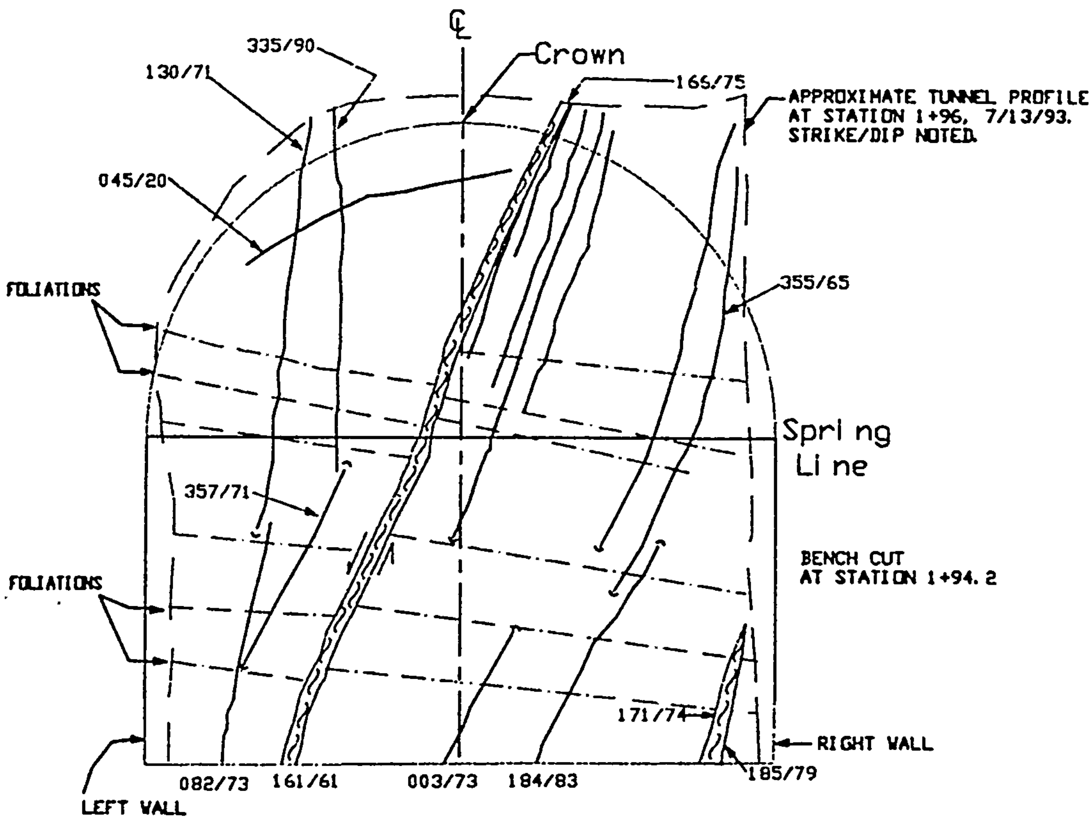

ASH CUTS
0/93
7/19/93
1+50
1+60
1+70
1+80
1+90
2+00
BASE OF LEFT WALL
FAULT ZONE
FACE 9/10/93
INTENSELY FRACTURED
LEFT SPRING LINE
FACE 7/15/93
COOLING JTS.
TUBES COMMON
℄
FACE 7/15/93 FACE
FACE 7/15/93
cul

C

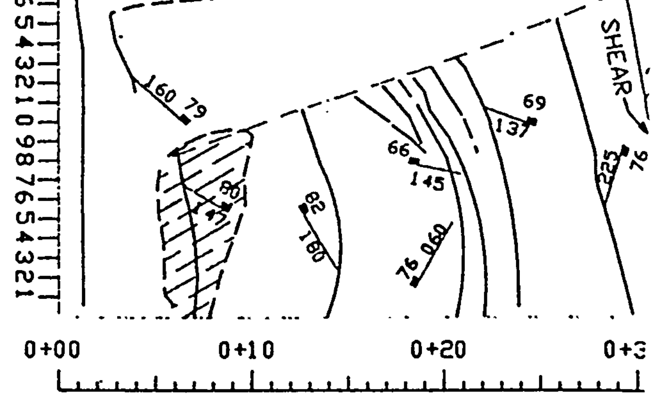
B

SKETCH OF TUNNEL FACE
STATION 1+96

A



BASE OF RIGHT WALL



L F A C E

9 6

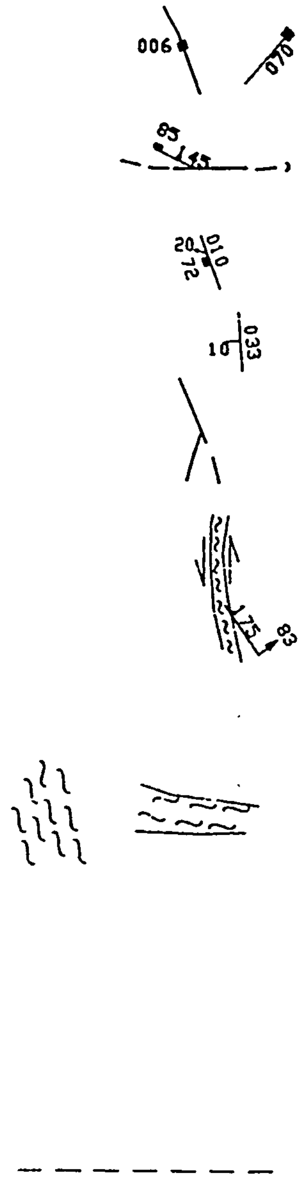
OXIMATE TUNNEL PROFILE
TATION 1+96, 7/13/93.
KE/DIP NOTED.

5

l ng
ne

CH CUT
STATION 1+94.2

GHT WALL



STRIKE

FRACTUR
STRIKE
CAPPED
UNCAPPE

STRIKE
PLUNGE

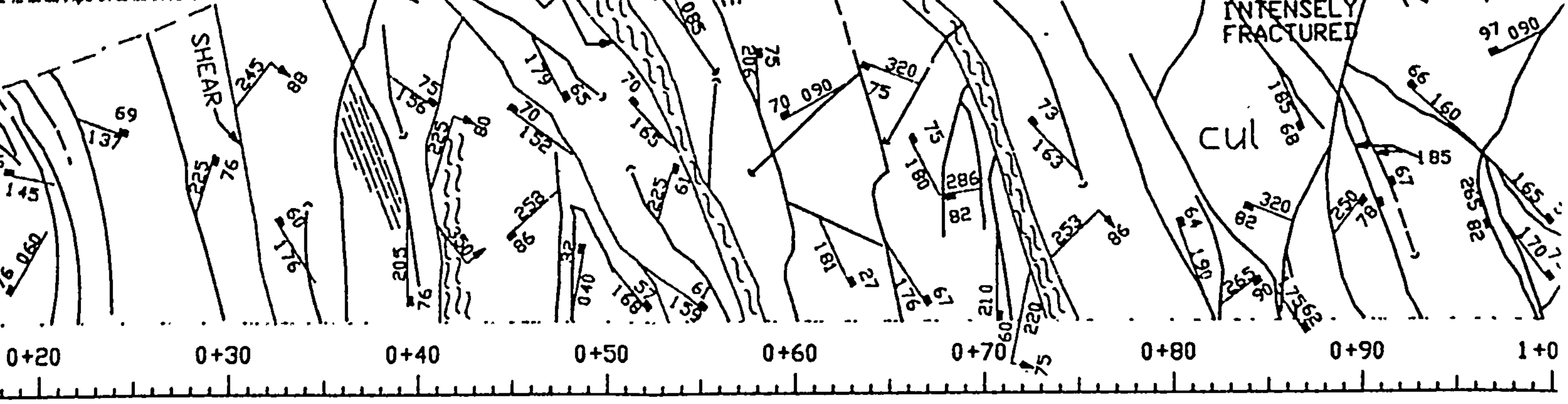
STRIKE A
BEDDING

FRACTUR
(DASHED

FAULT C
ARROWS

SHEAR ZC
SHEAR ZO
SUPPORTE
THE BREC
OF CLAY
WHERE TH
TYPICALL
CLASTS (
SHEARS D
ADJACENT
ARE SIMI
COMPOSIT
DISPLAY (
ARE COMM

TRACE OF



EXPLANATION

STRIKE OF VERTICAL FRACTURE.

FRACTURE TRACE WITH SYMBOL SHOWING STRIKE AND DIP; DASHED WHERE APPROXIMATE; CAPPED WHERE TERMINATION OBSERVED. UNCAPPED WHERE TERMINATION NOT OBSERVED.

STRIKE AND DIP OF FRACTURE, ARROW SHOWING PLUNGE OF FRACTURE DECORATIONS.

STRIKE AND DIP OF BEDDING OR FOLIATION.

FRACTURE TRACE, ORIENTATION NOT RECORDED. (DASHED WHERE INDISTINCT.)

FAULT OR SHEAR - SHOWING STRIKE AND DIP; ARROWS INDICATE DISPLACEMENT.

SHEAR ZONE; LIMITS SHOWN BY LINES WHERE BOUNDED BY FRACTURES. SHEAR ZONE FILLINGS ARE COMPOSED OF BOTH CLAST- AND MATRIX-SUPPORTED BRECCIAS. WHERE THE ZONES WIDEN TO 30 CM (1 FT.) OR MORE, THE BRECCIA IS USUALLY CLAST-SUPPORTED, WITH THE MATRIX COMPOSED OF CLAY TO GRAVEL-SIZE MATERIALS, AND CLAST SIZES TO >30CM (1 FT.). WHERE THE ZONES ARE LESS THAN 30 CM (1FT.) THICK, THE SHEARS ARE TYPICALLY MATRIX-SUPPORTED, WITH SMALL LENTICULAR AND TABULAR CLASTS (<15 CM) IN A MATRIX OF CLAY TO SAND-SIZE MATERIALS. SOME SHEARS DISPLAY THIN FILLINGS OF GOUGE PROBABLY DERIVED FROM THE ADJACENT WALL ROCK. ALL MATERIALS OBSERVED IN THE SHEAR ZONES ARE SIMILAR IN APPEARANCE TO THE WALL ROCK IN COLOR AND COMPOSITION. THE BRECCIAS ARE UNCEMENTED AND FREQUENTLY DISPLAY OPEN VOIDS ALONG THE CLAST BOUNDARIES. THE SHEAR ZONES ARE COMMONLY BOUNDED BY DISTINCT FRACTURE PLANES.

TRACE OF VAPOR PHASE PARTING.

6 LOCATION OF NOTES.

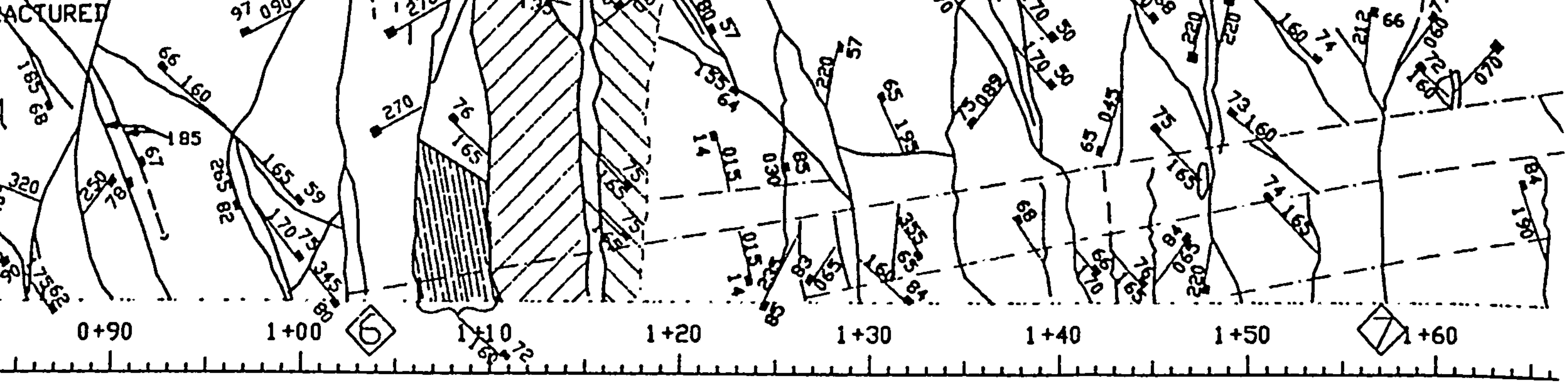
cul UPPER LITHOPHYSAL
PAINTBRUSH TUFF.
ASH-FLOW TUFF. THI
WITH LIGHT-GRAY, FI
LITHOPHYSAE ARE PRI
TUNNEL. THE LITHO
ROCK MASS, HAVING
SIZE NEAR THE PORT

FRACTURE FACES;
HAS BROKEN TO A
LARGE AREA (GREA
WHERE FACES ADJO
LIMITS DASHED WH

CLOSELY SPACED FI
APPROXIMATE TRAC
FRACTURES HAVE S

FALLOUT OR OVERBI
DASHED WHERE APPI

APPROXIMATE LIMI
FRACTURED ZONE.



D E F

LOCATION OF NOTES.

UPPER LITHOPHYSAL ZONE OF THE TIVA CANYON MEMBER OF THE PAINTBRUSH TUFF. THE ROCK IS A RHYOLITIC, DENSELY WELDED, ASH-FLOW TUFF. THE ROCK IS BROWNISH-GRAY TO GRAY, DEVITRIFIED, WITH LIGHT-GRAY, FLATTENED PUMICE FRAGMENTS TO 5 CM IN SIZE. LITHOPHYSAE ARE PRESENT THROUGHOUT THE ROCK IN THIS SECTION OF TUNNEL. THE LITHOPHYSAE VARY IN SIZE AND PERCENT VOLUME OF THE ROCK MASS, HAVING BOTH THE HIGHEST PERCENT VOLUME AND LARGEST SIZE NEAR THE PORTAL IN THE UPPER HALF OF THE TUNNEL.

FRACTURE FACES; OCCURS WHERE TUNNEL WALL HAS BROKEN TO A FRACTURE SURFACE OVER A LARGE AREA (GREATER THAN 1 SQUARE METER) WHERE FACES ADJOIN, HATCHING IS REVERSED. LIMITS DASHED WHERE APPROXIMATE.

CLOSELY SPACED FRACTURES SHOWING APPROXIMATE TRACES WHERE NUMEROUS FRACTURES HAVE SIMILAR ORIENTATIONS.

FALLOUT OR OVERBREAK BOUNDARY, DASHED WHERE APPROXIMATE.

APPROXIMATE LIMITS OF INTENSELY FRACTURED ZONE.

<u>INTENSELY FRACTURED</u> - CEMENTED OR UNC
FRAGMENTS CREATED BY MULTI-INTER
FRACTURED AREAS ARE INDICATED BY

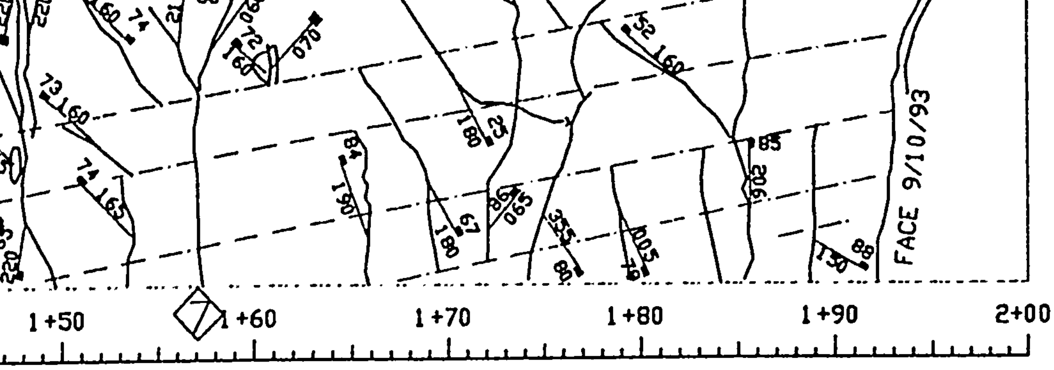
<u>FOLIATION</u> - 0.5 TO 15 CM THICK, PLANA
OF PUMICE FRAGMENTS.

<u>BRECCIA</u> - AT THIS LOCATION ONLY - A C
PRIMARILY SAND SIZE, NOT ASSOCIA

6/6
D-361

S

GEOLO
DRAWN
CHECKE
CADD
AUTOC
DENVEF



BASE OF RIGHT WALL

B

D E F I N I T I O N S

ACTURED - CEMENTED OR UNCEMENTED, PREDOMINATELY ANGULAR (MAY BE PLATY), ROCK
TS CREATED BY MULTI-INTERSECTING FRACTURES. APPROXIMATE LIMITS OF INTENSELY
ED AREAS ARE INDICATED BY DOTTED LINES AND/OR FRACTURE TRACES.

0.5 TO 15 CM THICK, PLANAR STRUCTURES THAT RESULT FROM FLATTENING OF CONCENTRATIONS
CE FRAGMENTS.

THIS LOCATION ONLY - A CLAST-SUPPORTED BRECCIA, WITH SLIGHT CEMENTATION, MATRIX IS A
LY SAND SIZE, NOT ASSOCIATED WITH A DISTINCT SHEAR.

DTN: GS940208314224.002

6/6/94 *D-3610*	*MINOR REVISIONS TO EXPLANATION, AND LOCATED NOTE #2*

ALWAYS THINK SAFETY

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
FOR
DEPARTMENT OF ENERGY

YUCCA MOUNTAIN PROJECT
STARTER TUNNEL-EXPLORATORY STUDIES FACILITY
FULL-PERIPHERY GEOLOGY MAP

A

GEOLOGY USBR Mapping Team *PRINCIPAL INVESTIGATOR* Beason
DRAWN D. L. Lowther *T. P. O.* J. Krulik
CHECKED C. Sullivan *ADMIN. APPROVED* R. Throner
CHIEF, GEOLOGY BRANCH

CADD SYSTEM	*CADD FILENAME*	*DATE AND TIME PLOTTED*
AUTOCAD/R12	*... YUCCA/171*	*6-6-94/2:45*
DENVER, COLORADO	*JANUARY 21, 1994*	0A-46-171

Appendix E
NRST Top Heading and Alcove No. 1
Blast Monitoring Data

Table E-1. Peak Particle Velocity, Distance, and Explosive Charge Weight—Top Heading

PPV (in/sec)	Distance (R) (ft)	Explosive Charge Weight (W) (lbs)	$R/W^{1/2}$ ($ft/lb^{1/2}$)	$R/W^{1/3}$ ($ft/lb^{1/3}$)	Blasting Seismograph
0.06	200	1	200	200	EV 6000 PDs
0.55	76	4	38	48	
0.43	81	6	33	44	
0.41	85	3	49	59	
0.47	95	3	55	66	
0.33	102	6	42	56	
0.36	105	2	75	84	
0.30	111	4	56	70	
0.18	118	12	34	51	
0.20	122	29	23	40	
0.10	127	1	127	127	
1.54	46	2	33	37	
0.56	51	1	51	51	
0.41	138	5	62	81	
0.49	145	30	27	47	
0.45	153	2	108	122	
0.53	158	10	50	73	
0.36	165	7	62	86	
0.19	172	13	48	73	
0.22	172	15	45	70	
0.24	181	4	91	114	
0.19	188	11	57	85	
0.20	203	6	83	112	
0.16	213	24	43	74	
0.28	220	7	83	115	
0.08	228	5	102	133	
0.11	235	9	78	113	
0.18	243	7	92	127	
0.16	254	7	96	133	
0.25	204	4	102	128	EV II PDs
0.07	210	6	86	115	
0.13	214	3	124	149	
0.86	41	3	24	28	
0.86	45	6	19	25	
0.51	48	2	34	38	
0.37	53	4	27	33	
0.33	59	12	17	26	
0.26	63	29	12	21	
0.15	67	1	67	67	

Table E-1 *continued*

PPV (in/sec)	Distance (R) (ft)	Explosive Charge Weight (W) (lbs)	$R/W^{1/2}$ ($ft/lb^{1/2}$)	$R/W^{1/3}$ ($ft/lb^{1/3}$)	Blasting Seismograph
0.35	68	2	48	54	
0.24	73	1	73	73	
0.45	77	5	35	45	
0.27	85	30	15	27	
0.33	92	2	65	73	
0.18	104	7	39	54	
0.19	110	13	31	47	
0.22	110	15	29	45	
0.15	120	4	60	75	
0.16	134	5	60	79	
0.21	141	6	58	78	
0.18	151	24	31	52	
0.88	72	7	27	38	EV 6000 NSs
0.96	76	4	38	48	
0.50	76	8	27	38	
1.16	85	21	19	31	
1.15	92	12	27	40	
0.49	93	14	25	39	
0.81	100	20	22	37	
0.47	108	6	44	60	
0.47	119	33	21	37	
0.84	128	5	57	75	
0.31	136	60	18	35	
0.67	144	19	33	54	
0.37	153	20	34	56	
0.18	158	7	60	83	
0.18	165	18	39	63	
0.29	172	4	86	108	
0.31	202	15	52	82	
0.24	210	32	37	66	
0.32	220	18	52	84	
0.23	228	16	57	90	
0.25	235	22	50	84	
0.18	254	5	114	149	
0.13	254	24	52	88	
3.36	32	4	16	20	EV II NSs
2.33	32	8	11	16	
3.71	35	21	8	13	
2.05	39	12	11	17	
1.85	40	14	11	16	
1.24	44	20	10	16	
0.53	51	6	21	28	

Table E-1 *continued*

PPV (in/sec)	Distance (R) (ft)	Explosive Charge Weight (W) (lbs)	$R/W^{1/2}$ ($ft/lb^{1/2}$)	$R/W^{1/3}$ ($ft/lb^{1/3}$)	Blasting Seismograph
0.67	60	33	10	19	
0.40	68	5	31	40	
0.29	76	60	10	19	
0.35	83	19	19	31	
0.39	92	20	21	34	
0.26	97	7	37	51	
0.18	104	18	24	40	
0.22	110	4	55	69	
0.18	114	9	38	55	
0.18	127	11	38	57	
0.19	134	11	41	60	
0.23	140	15	36	57	
0.28	148	32	26	47	
0.56	74	1	74	74	EV 6000 SSs
0.70	32	1	32	32	EV II SSs

Table E-2. Near-Field Blast Data from Alcove No. 1 (data sorted by PPV)

PPV (in/sec)	Distance (R) (ft)	Explosive Charge Weight (W) (lbs)	$R/W^{1/2}$ ($ft/lb^{1/2}$)	$R/W^{1/3}$ ($ft/lb^{1/3}$)	Blasting Seismograph
43.30	12	6	4.90	6.64	TA 3 Near
43.30	8	3	4.62	5.57	TA 3 Near
43.30	3	24	0.61	1.05	TA 4 Near
29.18	11	3	6.35	7.65	TA 3 Near
27.34	9	3	5.20	6.26	TA 4 Near
25.37	13	3	7.51	9.05	TA 3 Far
24.60	8	24	1.63	2.80	TA 4 Far
24.23	17	6	6.94	9.41	TA 3 Far
22.77	14	3	8.08	9.74	TA 4 Far
22.44	11	12	3.18	4.84	TA 3 Far
20.84	8	1	8.00	8.00	TA 1 Near
20.12	12	10	3.79	5.61	TA 1 Near
19.99	12	3	6.93	8.35	TA 3 Near
17.79	6	12	1.73	2.64	TA 3 Near
16.25	16	3	9.24	11.13	TA 3 Far
15.73	17	10	5.38	7.95	TA 1 Far
12.62	17	3	9.81	11.83	TA 3 Far
11.91	5	7	1.89	2.63	TA 1 Near
11.04	13	1	13.00	13.00	TA 1 Far
11.00	13	21	2.73	4.58	TA 4 Far
10.93	3	19	0.69	1.14	TA 3 Near
10.88	8	21	1.64	2.75	TA 4 Near
10.62	8	19	1.84	3.03	TA 3 Far
8.39	13	11	3.92	5.89	TA 3 Near
7.95	8	1	8.00	8.00	TA 1 Far
7.37	10	6	4.08	5.54	TA 4 Near
7.19	18	11	5.43	8.16	TA 3 Far
6.81	12	3	6.93	8.35	TA 4 Near
6.19	12	6	4.90	6.64	TA 4 Near
6.06	3	1	3.00	3.00	TA 1 Near
5.10	17	6	6.94	9.41	TA 4 Far
5.10	15	6	6.12	8.30	TA 4 Far
4.98	12	3	6.93	8.35	TA 4 Near
4.98	12	3	6.93	8.35	TA 4 Near
4.88	12	6	4.90	6.64	TA 4 Near
4.78	11	9	3.67	5.33	TA 4 Near
4.32	5	3	2.89	3.48	TA 4 Near
4.06	10	7	3.78	5.26	TA 1 Far
4.06	17	3	9.81	11.83	TA 4 Far
3.68	17	3	9.81	11.83	TA 4 Far

Table E-2 *continued*

PPV (in/sec)	Distance (R) (ft)	Explosive Charge Weight (W) (lbs)	$R/W^{1/2}$ ($ft/lb^{1/2}$)	$R/W^{1/3}$ ($ft/lb^{1/3}$)	Blasting Seismograph
3.68	17	3	9.81	11.83	TA 4 Far
3.26	10	3	5.77	6.96	TA 4 Far
2.89	16	9	5.33	7.75	TA 4 Far
2.83	17	6	6.94	9.41	TA 4 Far

Appendix F
Empirical Ground Support Design Category
(from Barton et al. 1974)

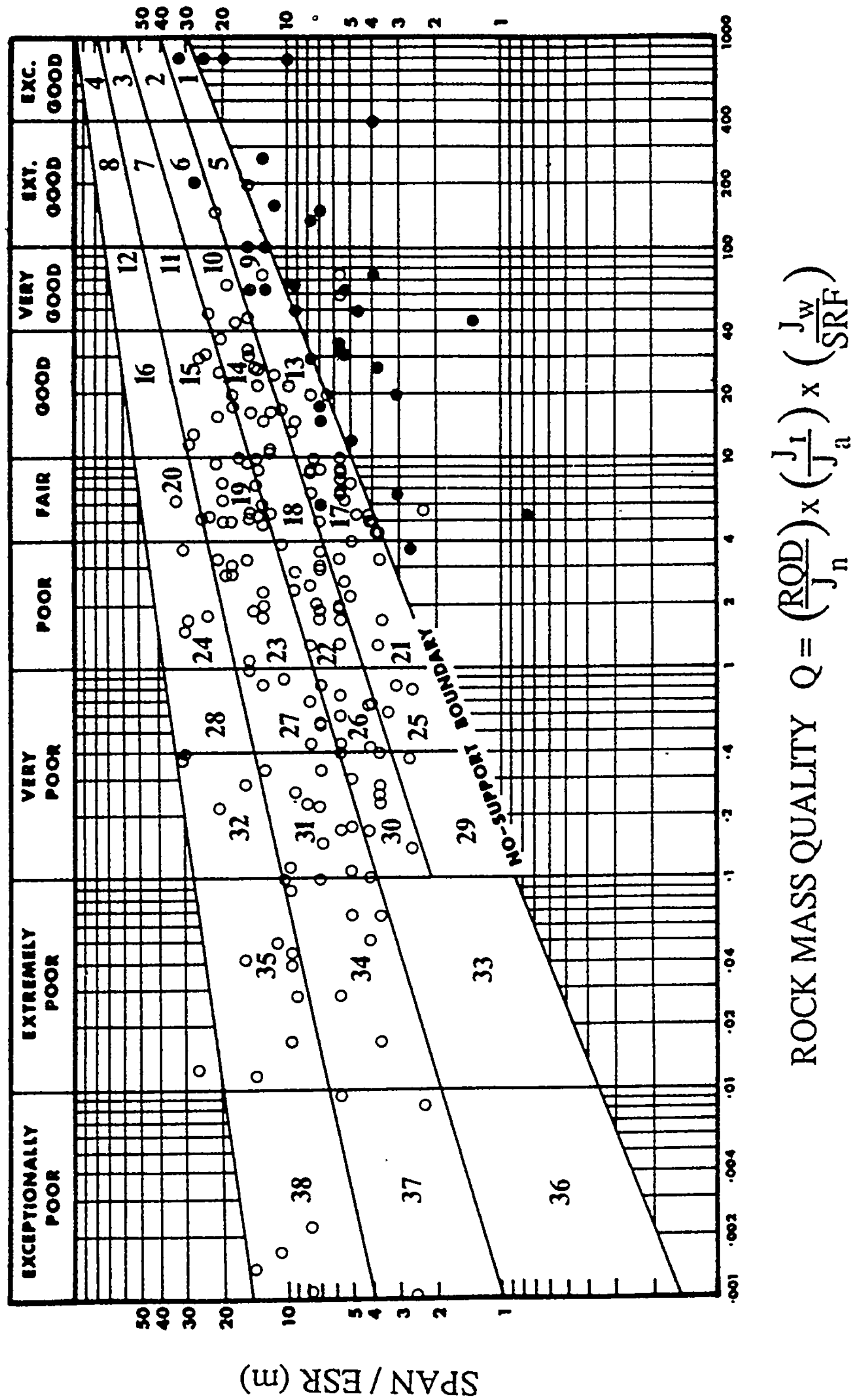


Figure F-1. Rock support category corresponding with rock mass quality.

Key to Support Tables:

sb = spot bolting
B = systematic bolting
(utg) = untensioned, grouted
(tg) = tensioned, (expanding shell type for competent rock masses, grouted post-tensioned in very poor quality rock masses; see Note XI)
S = shotcrete
(mr) = mesh reinforced
clm = chain link mesh
CCA = cast concrete arch
(sr) = steel reinforced

Bolt spacingsare given in metres (m). Shotcrete, or cast concrete arch thickness is given in centimetres (cm).

Table 8. Support Measures for Rock Masses of "Exeptional","Extremely Good","Very Good" and "Good" Quality (Q range: 1000-10)

Support category	Conditional factors RQD/J_n	J_r/J_a	SPAN/ESR	Type of support	Notes
1*	-	-	-	sb(utg)	-
2*	-	-	-	sb(utg)	-
3*	-	-	-	sb(utg)	-
4*	-	-	-	sb(utg)	-
5*	-	-	-	sb(utg)	-
6*	-	-	-	sb(utg)	-
7*	-	-	-	sb(utg)	-
8*	-	-	-	sb(utg)	-
9	≥20	-	-	sb(utg)	
	<20	-	-	B(utg) 2.5-3 m	-
10	≥30	-	-	B(utg) 2-3 m	-
	<30	-	-	B(utg) 1.5-2 m +clm	-
11*	≥30	-	-	B(tg) 2-3 m	-
	<30	-	-	B(tg) 1.5-2 m +clm	-
12*	≥30	-	-	B(tg) 2-3 m	-
	<30	-	-	B(tg) 1.5-2 m +clm	-
13	≥10	≥1.5	-	sb(utg)	I
	≥10	<1.5	-	B(utg) 1.5-2 m	I
	<10	≥1.5	-	B(utg) 1.5-2 m	I
	<10	<1.5	-	B(utg) 1.5-2 m +S 2-3 cm	I
14	≥10	-	≥15	B(tg) 1.5-2 m +clm)	I,II
	<10	-	≥15	B(tg) 1.5-2 m +S(mr) 5-10 cm	I,II
	-	-	<15	B(utg) 1.5-2 m +clm	I,III
15	>10	-	-	B(tg) 1.5-2 m +clm	I,II,IV
	≤10	-	-	B(tg) 1.5-2 m +S(mr) 5-10 cm	I,II,IV
16* See note XII	>15	-	-	B(tg) 1.5-2 m +clm	I,V,VI
	≤15	-	-	B(tg) 1.5-2 m +S(mr) 10-15 cm	I,V,VI

*Authors' estimates of support. Insufficient case records available for reliable estimation of support requirements.

Note: The type of support to be used in categories 1 to 8 will depend on the blasting technique. Smooth wall blasting and thorough barring-down may remove the need for support. Rough-wall blasting may result in the need for single applications of shotcrete, especially where the excavation height is >25 m. Future case records should differentiate categories 1 to 8.

Table 9. Support Measures for Rock Masses of "Fair" and "Poor" quality (Q range: 10-1).

Support category	Conditional factors RQD/J_n	J_r/J_a	SPAN/ESR	Type of support	Note
17	>30	-	-	sb(utg)	I
	(≥10, ≤30)	-	-	B(utg) 1-1.5 m	I
	<10	-	≥6 m	B(utg) 1-1.5 m +S 2-3 cm	I
	<10	-	<6 m	S 2-3 cm	I
18	>5	-	≥10 m	B(tg) 1-1.5 m +clm	I,III
	>5	-	<10 m	B(utg) 1-1.5 m +clm	I
	≤5	-	≥10 m	B(tg) 1-1.5 m +S 2-3 cm	I,III
	≤5	-	<10 m	B(utg) 1-1.5 m +S 2-3 cm	I
19	-	-	≥20 m	B(tg) 1-2 m +S(mr) 10-15 cm	I,II,IV
	-	-	<20 m	B(tg) 1-1.5 m +S(mr) 5-10 cm	I,II
20* See note XII	-	-	≥35 m	B(tg) 1-2 m +S(mr) 20-25 cm	I,V,VI
	-	-	<35 m	B(tg) 1-2 m +S(mr) 10-20 cm	I,II,IV
21	≥12.5	≤0.75	-	B(utg) 1 m +S 2-3 cm	I
	<12.5	≤0.75	-	S 2.5-5 cm	I
	-	>0.75	-	B(utg) 1 m	I
22	(>10, <30)	>1.0	-	(B(utg) 1 m +clm	I
	≤10	>1.0	-	S 2.5-7.5 cm	I
	<30	≤1.0	-	B(utg) 1 m +S(mr) 2.5-5 cm	I
	≥30	-	-	B(utg) 1 m	I
23	-	-	≥15 m	B(tg) 1-1.5 m +S(mr) 10-15 cm	I,II,IV, VII
	-	-	<15 m	B(utg) 1-1.5 m +S(mr) 5-10 cm	I
24* See note XII	-	-	≥30 m	B(tg) 1-1.5 m +S(mr) 15-30 cm	I,V,VI
	-	-	<30 m	B(tg) 1-1.5 m +S(mr) 10-15 cm	I,II,IV

*Authors' estimates of support. Insufficient case records available for reliable estimation of support requirements.

Table 10. Support Measures for Rock Masses of "Very poor" Quality (Q range: 1.0-0.1)

Support category	Conditional factors RQD/J_n	J_r/J_a	SPAN/ESR	Type of support	Note
25	>10	>0.5	-	B(utg) 1 m +mr or clm	I
	≦10	>0.5	-	B(utg) 1 m +S(mr) 5 cm	I
	-	≦0.5	-	B(tg) 1 m +S(mr) 5 cm	I
26	-	-	-	B(tg) 1 m +S(mr) 5-7.5 cm	VIII,X, XI
	-	-	-	B(utg) 1 m +S 2.5-5 cm	I,IX
27	-	-	≧12m	B(tg) 1 m +S(mr) 7.5-10cm	I,IX
	-	-	<12m	B(utg) 1 m +S(mr) 5-7.5 cm	I,IX
	-	-	>12m	CCA 20-40 cm +B(tg) 1 m	VIII,X, XI
	-	-	<12m	S(mr) 10-20 cm +B(tg) 1 m	VIII,X, XI
28* See note XII	-	-	≧30m	B(tg) 1 m +S(mr) 30-40 cm	I,IV,V, IX
	-	-	(≧20, <30m)	B(tg) 1 m +S(mr) 20-30 cm	I,II,IV, IX
	-	-	<20m	B(tg) 1 m +S(mr) 15-20 cm	I,II,IX
	-	-	-	CCA(sr)30-100cm +B(tg) 1 m	IV,VIII, X,XI
29*	>5	>0.25	-	B(utg) 1 m +S 2-3 cm	-
	≦5	>0.25	-	B(utg) 1 m +S(mr) 5 cm	-
	-	≦0.25		B(tg) 1 m +S(mr) 5 cm	-
30	≧5	-	-	B(tg) 1 m +S 2.5-5 cm	IX
	<5	-	-	S(mr) 5-7.5 cm	IX
	-	-	-	B(tg) 1 m +S(mr) 5-7.5 cm	VIII,X, XI
31	>4	-	-	B(tg) 1 m +S(mr) 5-12.5cm	IX
	≦4,≧1.5	-	-	S(mr) 7.5-25 cm	IX
	<1.5	-	-	CCA 20-40 cm +B(tg) 1 m	IX
	-	-	-	CCA(sr)30-50 cm +B(tg) 1 m	VII,X, XI
32 See note XII	-	-	≧20m	B(tg) 1 m +S(mr) 40-60 cm	II,IV, IX
	-	-	<20m	B(tg) 1 m +S(mr) 20-40 cm	III,IV, IX
	-	-	-	CCA(sr)40-120cm +B(tg) 1 m	IV,VIII, X,XI

*Authors' estimates of support. Insufficient case records available for confident prediction of support requirements.

Table 11. Support Measures for Rock Masses of "Extremely Poor" and "Exceptionally Poor" Quality (Q range: 0.1-0.001)

Support category	Conditional factors RQD/J_n	J_r/J_a	SPAN/ESR	Type of support	Note
33*	≧2	-	-	B(tg) 1 m +S(mr) 2.5-5 cm	IX
	<2	-	-	S(mr) 5-10 cm	IX
	-	-	-	S(mr) 7.5-15 cm	VIII,X
34	≧2	≧0.25	-	B(tg) 1 m +S(mr) 5-7.5 cm	IX
	<2	≧0.25	-	S(mr) 7.5-15 cm	IX
	-	<0.25	-	S(mr) 15-25 cm	IX
	-	-	-	CCA(sr)20-60 cm +B(tg) 1 m	VIII,X XI
35 See note XII	-	-	≧15m	B(tg) 1 m +S(mr) 30-100cm	II,IX
	-	-	≧15m	CCA(sr)60-200cm +B(tg) 1 m	VIII,X, XI,II
	-	-	<15m	B(tg) 1 m +S(mr) 20-75 cm	IX,III
	-	-	<15m	CCA(sr)40-150cm +B(tg) 1 m	VIII,X, XI,III
36*	-	-	-	S(mr) 10-20 cm	IX
	-	-	-	S(mr) 10-20 cm +B(tg) 0.5-1.0m	VIII,X, XI
37	-	-	-	S(mr) 20-60 cm	IX
	-	-	-	S(mr) 20-60 cm +B(tg) 0.5-1.0m	VIII,X, XI
38 See note XIII	-	-	≧10m	CCA(sr)100-300cm	IX
	-	-	≧10m	CCA(sr)100-300cm +B(tg) 1 m	VIII,X, II,XI
	-	-	<10m	S(mr) 70-200 cm	IX
	-	-	<10m	S(mr) 70-200 cm +B(tg) 1 m	VIII,X, III,XI

*Authors' estimates of support. Insufficient case records available for confident prediction of support requirements.

Supplementary Notes for Support Tables (from Barton et al. 1974)

I. For cases of heavy rock bursting or "popping", tensioned bolts with enlarged bearing plates often used, with spacing of about 1 m (occasionally down to 0.8 m). Final support when "popping" activity ceases. (Selmer-Olsen, 1970)

II. Several bolt lengths often used in same excavation, i.e. 3, 5 and 7 m.

III. Several bolt lengths often used in same excavation, i.e. 2, 3 and 4 m.

IV. Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 2-4 m.

V. Several bolt lengths often used in same excavations, i.e. 6, 8 and 10 m.

VI. Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 4-6 m.

VII. Several of the older generation power stations in this category employ systematic or spot bolting with areas of chain link mesh, and a free span concrete arch roof (25-40 cm) as permanent support.

VIII. Cases involving swelling, for instance montmorillonite clay (with access of water). Room for expansion behind the support is used in cases of heavy swelling. See Selmer-Olsen (1970). Drainage measures are used where possible.

IX. Cases not involving swelling clay or squeezing rock.

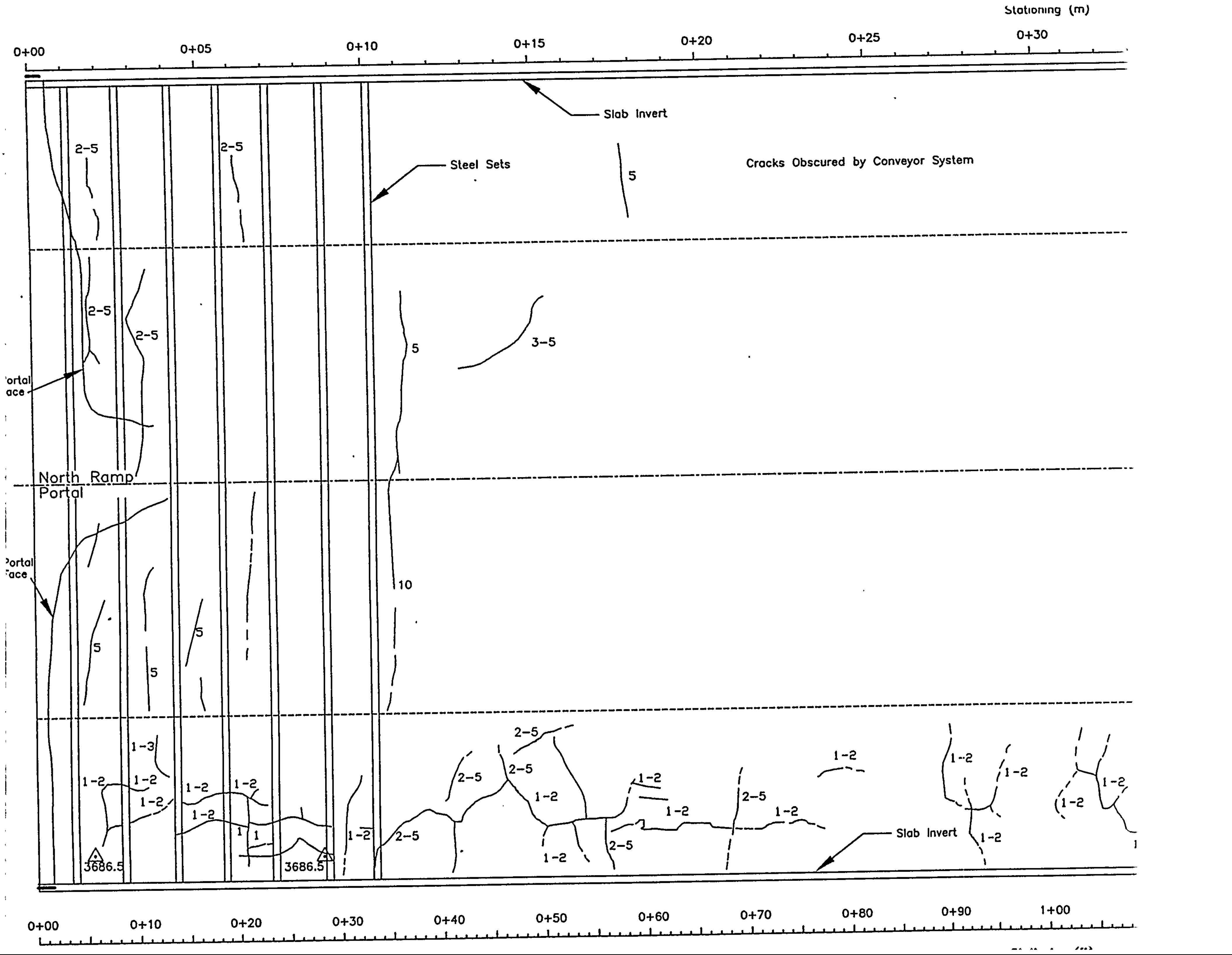
X. Cases involving squeezing rock. Heavy rigid support is generally used as permanent support.

XI. According to the authors' experience, in cases of swelling or squeezing, the temporary support required before concrete (or shotcrete) arches are formed may consist of bolting (tensioned shell-expansion type) if the value of RQD/J_n is sufficiently high (i.e. <1.5), possibly combined with shotcrete. If the crushed rock mass is very heavily jointed or crushed (i.e. RQD/J_n <1.5, for example a "sugar cube" shear zone in quartzite), then the temporary support may consist of up to several applications of shotcrete. Systematic bolting (tensioned) may by added after casting the concrete, but it may not be effective when RQD/J_n <1.5, or when a lot of clay is present, unless the bolts are grouted before tensioning. A sufficient length of anchored bolt might also be obtained using quick setting resin anchors in these extremely poor quality rock-masses. Serious occurrences of swelling and/or squeezing rock may require that the concrete arches are taken right up to the face, possibly using a shield as temporary shuttering. Temporary support of the working face may also be required in these cases.

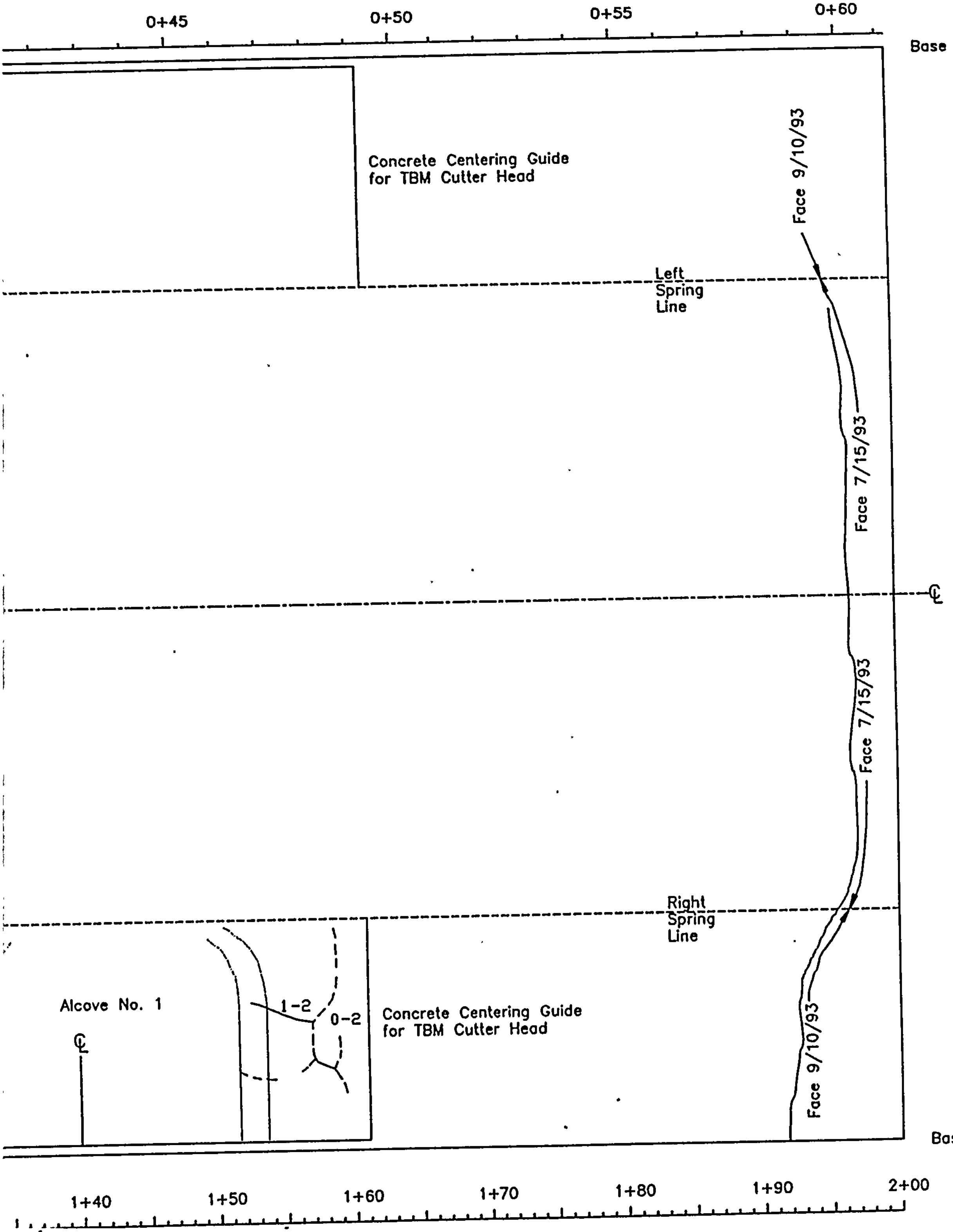
XII. For reasons of safety, the multiple drift method will often be needed during excavation and supporting of roof arch. Categories 16, 20, 24, 28, 32, 35 (SPAN/ESR <15 m only).

XIII. Multiple drift method usually needed during excavation and support of arch, walls and floor in cases of heavy squeezing. Category 38 (SPAN/ESR <10 m only).

Appendix G
North Ramp Starter Tunnel
Fibercrete Crack Mapping



Stationing (m)
0+00
0+05
0+10
0+15
0+20
0+25
0+30
Slab Invert
Steel Sets
Cracks Obscured by Conveyor System
North Ramp Portal
Portal Face
Portal Face
2-5
3-5
5
10
1-2
1-3
1
3686.5
3686.5
Slab Invert
0+00
0+10
0+20
0+30
0+40
0+50
0+60
0+70
0+80
0+90
1+00



SHOTCRETE CRACK MAP OF STARTER TUNNEL

Mapped by K. Donaldson, A.A.I., 7/31/95
Checked by H. Perry, A.A.I., 8/21/95

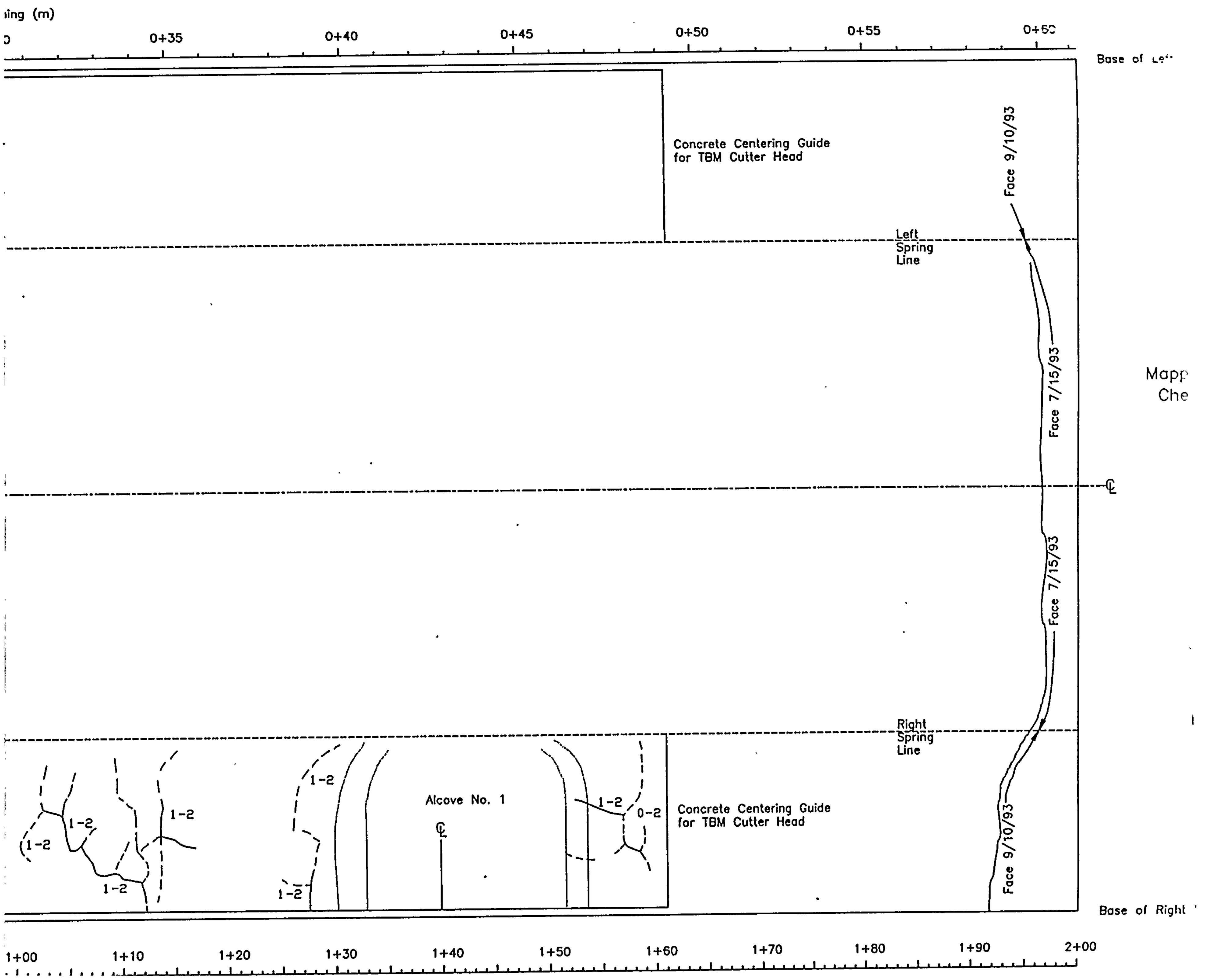
EXPLANATION

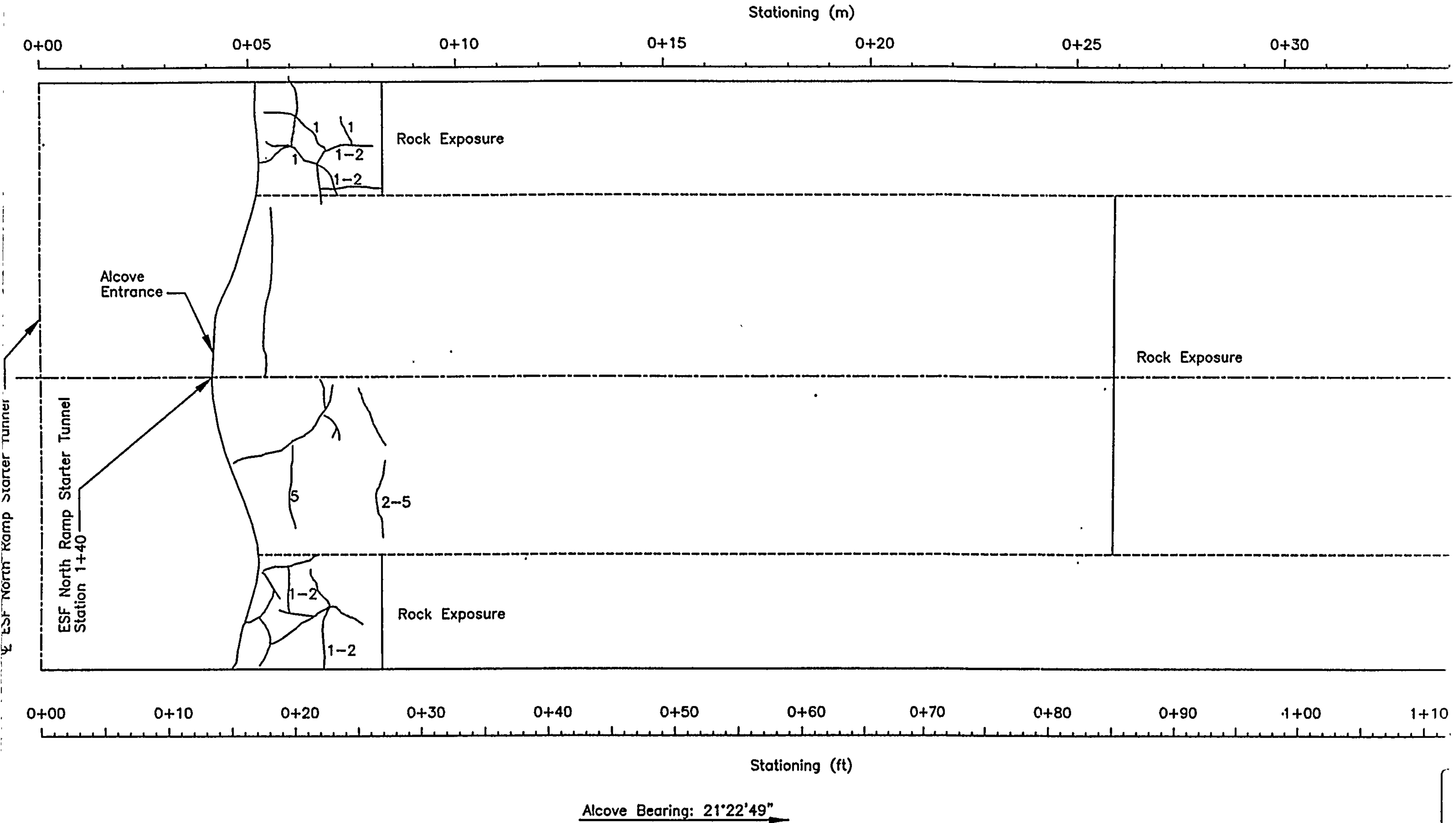
1-2 Shotcrete crack, (dashed where discontinuous). Aperature width (mm)

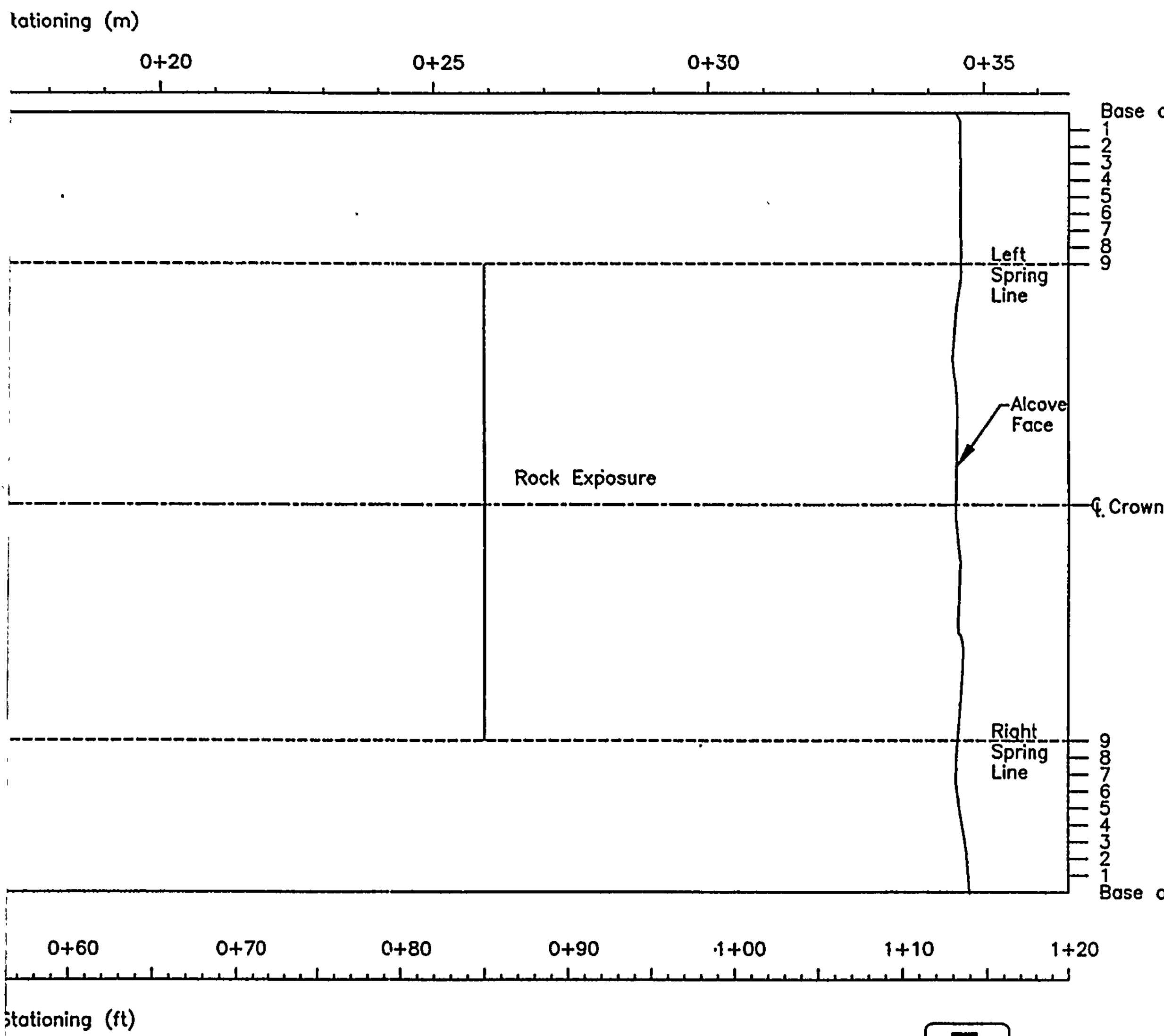
3686.5 Elevation Station (ft – Mean Sea Level)

Note: Base Map from U.S.B.R. DTN:GS940208314224.002

\ST-DATA\CRACK2.DWG
Rev. 0


ing (m)
0+35
0+40
0+45
0+50
0+55
Base of Left
Concrete Centering Guide for TBM Cutter Head
Face 9/10/93
Left Spring Line
Face 7/15/93
Mapp
Che
Face 7/15/93
Right Spring Line
Alcove No. 1
1-2
0-2
Concrete Centering Guide for TBM Cutter Head
Face 9/10/93
Base of Right
1+00
1+10
1+20
1+30
1+40
1+50
1+60
1+70
1+80
1+90
2+00

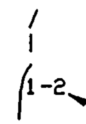

Stationing (m)
0+00
0+05
0+10
0+15
0+20
0+25
0+30
Rock Exposure
Rock Exposure
Rock Exposure
Alcove Entrance
ESF North Ramp Starter Tunnel Station 1+40
1
1
1
1
1-2
1-2
1-2
1-2
5
2-5
0+00
0+10
0+20
0+30
0+40
0+50
0+60
0+70
0+80
0+90
1+00
1+10
Stationing (ft)
Alcove Bearing: 21°22'49"



SHOTCRETE CRACK MAP OF ALCOVE NO. 1

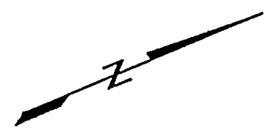
Mapped by K. Donaldson, A.A.I., 7/31/95
Checked by H. Perry, A.A.I., 8/21/95

EXPLANATION



Shotcrete crack, (dashed where discontinuous).
Aperature width (mm)

Note: Base Map from U.S.B.R.



\ST-DATA\CRACK1.DWG
Rev. 0

Sandia National Laboratories

YUCCA MOUNTAIN SITE CHARACTERIZATION PROJECT

UC814 - DISTRIBUTION LIST

1 D. A. Dreyfus (RW-1)
Director
OCRWM
US Department of Energy
1000 Independence Avenue SW
Washington, DC 20585

1 L. H. Barrett (RW-2)
Acting Deputy Director
OCRWM
US Department of Energy
1000 Independence Avenue SW
Washington, DC 20585

1 S. Rousso (RW-40)
Office of Storage and Transportation
OCRWM
US Department of Energy
1000 Independence Avenue SW
Washington, DC 20585

1 R. A. Milner (RW-30)
Office of Program Management
and Integration
OCRWM
US Department of Energy
1000 Independence Avenue SW
Washington, DC 20585

1 D. R. Elle, Director
Environmental Protection Division
DOE Nevada Field Office
US Department of Energy
P.O. Box 98518
Las Vegas, NV 89193-8518

1 T. Wood (RW-14)
Contract Management Division
OCRWM
US Department of Energy
1000 Independence Avenue SW
Washington, DC 20585

4 Victoria F. Reich, Librarian
Nuclear Waste Technical Review Board
1100 Wilson Blvd., Suite 910
Arlington, VA 22209

5 Wesley Barnes, Project Manager
Yucca Mountain Site Characterization
Office
US Department of Energy
P.O. Box 98608--MS 523
Las Vegas, NV 89193-8608

1 Director
Office of Public Affairs
DOE Nevada Operations Office
US Department of Energy
P.O. Box 98518
Las Vegas, NV 89193-8518

8 Technical Information Officer
DOE Nevada Operations Office
US Department of Energy
P.O. Box 98518
Las Vegas, NV 89193-8518

1 P. K. Fitzsimmons, Technical Advisor
Office of Assistant Manager for
Environmental Safety and Health
DOE Nevada Operations Office
US Department of Energy
P.O. Box 98518
Las Vegas, NV 89193-8518

1 J. A. Blink
Deputy Project Leader
Lawrence Livermore National Laboratory
101 Convention Center Drive
Suite 820, MS 527
Las Vegas, NV 89109

2 J. A. Canepa
Technical Project Officer - YMP
N-5, Mail Stop J521
Los Alamos National Laboratory
P.O. Box 1663
Los Alamos, NM 87545

1 Repository Licensing & Quality
Assurance
Project Directorate
Division of Waste Management
US NRC
Washington, DC 20555

1 Senior Project Manager for Yucca
Mountain
Repository Project Branch
Division of Waste Management
US NRC
Washington, DC 20555

1 NRC Document Control Desk
Division of Waste Management
US NRC
Washington, DC 20555

1 Chad Glenn
NRC Site Representative
301 E Stewart Avenue, Room 203
Las Vegas, NV 89101

1 E. P. Binnall
Field Systems Group Leader
Building 50B/4235
Lawrence Berkeley Laboratory
Berkeley, CA 94720

1 Center for Nuclear Waste
Regulatory Analyses
6220 Culebra Road
Drawer 28510
San Antonio, TX 78284

2 W. L. Clarke
Technical Project Officer - YMP
Attn: YMP/LRC
Lawrence Livermore National Laboratory
P.O. Box 5514
Livermore, CA 94551

1 V. R. Schneider
Asst. Chief Hydrologist -- MS 414
Office of Program Coordination
and Technical Support
US Geological Survey
12201 Sunrise Valley Drive
Reston, VA 22092

1 J. S. Stuckless, Chief
Geologic Studies Program
MS 425
Yucca Mountain Project Branch
US Geological Survey
P.O. Box 25046
Denver, CO 80225

1 N. Z. Elkins
Deputy Technical Project Officer
Los Alamos National Laboratory
Mail Stop 527
101 Convention Center Drive, #820
Las Vegas, NV 89109

2 Michaele C. Brady
Technical Project Officer - YMP
Sandia National Laboratories
Organization 6302, MS 1399
101 Convention Center Drive, Suite 880
Las Vegas, NV 89109

1 Ray Wallace
US Geological Survey
106 National Center
12201 Sunrise Valley Drive
Reston, VA 22092

1 L. R. Hayes
Technical Project Officer
Yucca Mountain Project Branch
MS 425
US Geological Survey
P.O. Box 25046
Denver, CO 80225

1 A. L. Flint
US Geological Survey
MS 721
P.O. Box 327
Mercury, NV 89023

1 R. E. Lewis
Yucca Mountain Project Branch
MS 425
US Geological Survey
P.O. Box 25046
Denver, CO 80225

1 D. Zesiger
US Geological Survey
101 Convention Center Drive
Suite 860, MS 509
Las Vegas, NV 89109

2 L. D. Foust
Nevada Site Manager
TRW Environmental Safety Systems
101 Convention Center Drive
Suite P-110, MS 423
Las Vegas, NV 89109

1 C. E. Ezra
YMP Support Office Manager
EG&G Energy Measurements Inc.
MS V-02
P.O. Box 1912
Las Vegas, NV 89125

1 E. L. Snow, Program Manager
Roy F. Weston, Inc.
955 L'Enfant Plaza SW
Washington, DC 20024

1 Technical Information Center
Roy F. Weston, Inc.
955 L'Enfant Plaza SW
Washington, DC 20024

1 Technical Project Officer - YMP
US Bureau of Reclamation
Code D-3790
P.O. Box 25007
Denver, CO 80225

1 B. T. Brady
Records Specialist
US Geological Survey
MS 421
P.O. Box 25046
Denver, CO 80225

1 M. D. Voegele
Technical Project Officer - YMP
M&O/SAIC
101 Convention Center Drive
Suite 407
Las Vegas, NV 89109

1 Paul Eslinger, Manager
PASS Program
Pacific Northwest Laboratories
P.O. Box 999
Richland, WA 99352

1 A. T. Tamura
Science and Technology Division
OSTI
US Department of Energy
P.O. Box 62
Oak Ridge, TN 37831

1 P. J. Weeden, Acting Director
Nuclear Radiation Assessment Div.
US EPA
Environmental Monitoring Sys. Lab
P.O. Box 93478
Las Vegas, NV 89193-3478

1 ONWI Library
Battelle Columbus Laboratory
Office of Nuclear Waste Isolation
505 King Avenue
Columbus, OH 43201

1 C. H. Johnson
Technical Program Manager
Agency for Nuclear Projects
State of Nevada
Evergreen Center, Suite 252
1802 N. Carson Street
Carson City, NV 89710

1 John Fordham, Deputy Director
Water Resources Center
Desert Research Institute
P.O. Box 60220
Reno, NV 89506

1 The Honorable Cyril Schank
Chairman
Churchill County Board of
Commissioners
190 W. First Street
Fallon, NV 89406

1 T. Hay, Executive Assistant
Office of the Governor
State of Nevada
Capitol Complex
Carson City, NV 89710

3 R. R. Loux
Executive Director
Agency for Nuclear Projects
State of Nevada
Evergreen Center, Suite 252
1802 N. Carson Street
Carson City, NV 89710

1 Brad R. Mettam
Inyo County Yucca Mountain
Repository Assessment Office
P. O. Drawer L
Independence, CA 93526

1 Lander County Board of Commissioners
315 South Humbolt Street
Battle Mountain, NV 89820

1 Vernon E. Poe
Office of Nuclear Projects
Mineral County
P.O. Box 1600
Hawthorne, NV 89415

1 Les W. Bradshaw
Program Manager
Nye County Nuclear Waste Repository
Project Office
P.O. Box 1767
Tonopah, NV 89049

1 Florindo Mariani
White Pine County Coordinator
P. O. Box 135
Ely, NV 89301

1 Judy Foremaster
City of Caliente Nuclear Waste
Project Office
P.O. Box 158
Caliente, NV 89008

1 Philip A. Niedzielski-Eichner
Nye County Nuclear Waste
Repository Project Office
P.O. Box 221274
Chantilly, VA 22022-1274

1 Dennis Bechtel, Coordinator
Nuclear Waste Division
Clark County Department of
Comprehensive Planning
301 E. Clark Avenue, Suite 570
Las Vegas, NV 89101

1 Juanita D. Hoffman
Nuclear Waste Repository
Oversight Program
Esmeralda County
P.O. Box 490
Goldfield, NV 89013

1 Eureka County Board of Commissioners
Yucca Mountain Information Office
P.O. Box 714
Eureka, NV 89316

1 Economic Development Dept.
City of Las Vegas
400 E. Stewart Avenue
Las Vegas, NV 89101

1 Community Planning & Development
City of North Las Vegas
P.O. Box 4086
North Las Vegas, NV 89030

1 Community Development & Planning
City of Boulder City
P.O. Box 61350
Boulder City, NV 89006

1 Commission of European Communities
200 Rue de la Loi
B-1049 Brussels
BELGIUM

2 Librarian
YMP Research & Study Center
MS 407
P.O. Box 98521
Las Vegas, NV 89193-8521

1 Amy Anderson
Argonne National Laboratory
Building 362
9700 S. Cass Avenue
Argonne, IL 60439

1 Glenn Van Roekel
Director of Community Development
City of Caliente
P.O. Box 158
Caliente, NV 89008

1 G. S. Bodvarsson
Head, Nuclear Waste Department
Lawrence Berkeley Laboratory
1 Cyclotron Road, MS 50E
Berkeley, CA 94720

1 Michael L. Baughman
Intertech Services Corp.
P.O. Box 93537
Las Vegas, NV 89193

3 Agapito Associates
Attention: Carl Brechtel
3841 W. Charleston, Suite 203
Las Vegas, Nevada 89102

1 University of Nevada, Reno
Attention: J. J. K. Daemen
Mackay School of Mines
Dept. of Mining Engineering
Mail Stop 173
Reno, Nevada 89557-0139

2	MS-1330	C. B. Michaels, 6752 100/WBS 1.2.4.2.1.4, 1.2.3.2.7.3.1, 1.2.3.2.7.3.4 SAND95-1675/QA
20	MS-1330	WMT Library, 6752
5	MS 1325	R.E. Finley, 6313
1	MS 1325	L. Costin, 6302
1	MS 1325	J. Pott, 6313
1	MS 1325	J.T. George, 6313
1	MS 1325	E.E. Ryder, 6313
1	MS 1325	S.R. Sobolik, 6313
1	MS 1325	R.H. Price, 6313
1	MS 1325	N.S. Brodsky, 6313
1	MS 1399	M.C. Brady, 6314
1	MS 1399	D.S. Kessel, 6314
1	MS 1399	M. Riggins, 6314
1	MS 1399	J.E. Grant, 6314
1	MS 1399	M. Lee, 6314
1	MS 1399	R.S. Taylor, 6314
1	MS-9018	Central Technical Files, 8523-2
5	MS-0899	Technical Library, 4414
1	MS-0619	Print Media, 12615
2	MS-0100	Document Processing, 7613-2 for DOE/OSTI

☆ U.S. GOVERNMENT PRINTING OFFICE: 1996–774–125–42007