DOE/WIPP-95-2100

Geotechnical Analysis Report for July 1993 - June 1994

August 1995

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Waste Isolation Pilot Plant

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FOREWORD AND ACKNOWLEDGMENTS

This report provides an assessment of the geotechnical status of the Waste Isolation Pilot Plant (WIPP). During the construction of the principal underground access and experimental areas, reporting was on a quarterly basis. Since 1987, reports have been published annually because additional excavations are taking place gradually, over extended periods. This report presents and analyzes data collected from July 1, 1993 to June 30, 1994.

The format of the Geotechnical Analysis Report was selected to meet the needs of several audiences. This report focuses on the geotechnical performance of the various underground facilities including the shafts, shaft stations, access drifts, experimental rooms, and waste storage areas. The results of excavation effects, investigations, stratigraphic mapping, and other geologic studies are also included. The report provides an evaluation of the geotechnical aspects of performance in the context of the relevant design criteria and also describes the techniques used to acquire the data and the performance history of the instruments. The depth and breadth of the evaluation for the different underground facilities varies according to the types and quantities of data that are available, and the complexity of the recorded geotechnical responses.

Documentation of data in graph format and instrument history in tabular format can be provided upon request at nominal cost.

The Geotechnical Analysis Report is a multi-author report that was prepared by Westinghouse, Waste Isolation Division for the DOE, Carlsbad Area Office, Carlsbad, New Mexico. Work was supported by the DOE under Contract No. DE-AC04-86AL31950.

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Table of Contents

Table of Contents (Continued)

Table of Contents (Continued)

List of Tables

List of Figures

List of Acronyms/Abbreviations

C&SH	Construction and Salt Handling
CH	Contact Handled
CMR	Central Monitoring Room
DOE	U.S. Department of Energy
EEP	Excavation Effects Program
GIS	Geomechanical Instrumentation System
LTC	Local Termination Cabinets
SH	Salt Handling
SPDV	Site Preliminary Design Validation
WIPP	Waste Isolation Pilot Plant

1.0 Introduction

The geotechnical data from the underground excavations at the WIPP are interpreted and presented in this Geotechnical Analysis Report. The data are used to characterize conditions, assess design assumptions, and understand and predict the performance of the underground excavations during operations. The data are obtained as part of a regular monitoring program. They do not include data from tests performed by Sandia National Laboratories, the Scientific Advisor to the project in support of performance assessment studies.

Geotechnical Analysis Reports have been prepared routinely and made available to the public since 1983. During the Site and Preliminary Design Validation Program, the Architect/Engineer for the project produced the reports on a quarterly basis to document the geomechanical performance during and immediately after construction of the underground. Upon completion of the construction phase of the project in 1987, the reports have been prepared annually by the Management and Operating Contractor for the facility. This report describes the performance and conditions of selected areas from July 1, 1993, to June 30, 1994.

1.1 Location and Description

The WIPP is located in southeastern New Mexico, about 30 miles (50 km) east of Carlsbad (Figure 1-1). The surface facilities have been built on the flat to gently rolling hills that are characteristic of the Los Medanos (sand dunes) area. The underground facilities are being excavated approximately 2,150 feet (655 m) beneath the surface, in the Salado Formation. A plan view of the underground facilities at the WIPP site is shown in Figure 1-2.

1.2 Mission

The WIPP was authorized by Congress in 1979 (Public Law 96-164) to provide "...a research and development facility to demonstrate the safe disposal of radioactive wastes resulting from the defense activities and programs of the United States exempted from regulation by the Nuclear Regulatory Commission." The WIPP is intended to receive, handle, and permanently dispose of transuranic mixed waste. To fulfill this mission, the DOE is constructing a full scale facility to demonstrate both technical and operational principles of the permanent storage/disposal of transuranic mixed waste. Technical aspects are those concerned with the

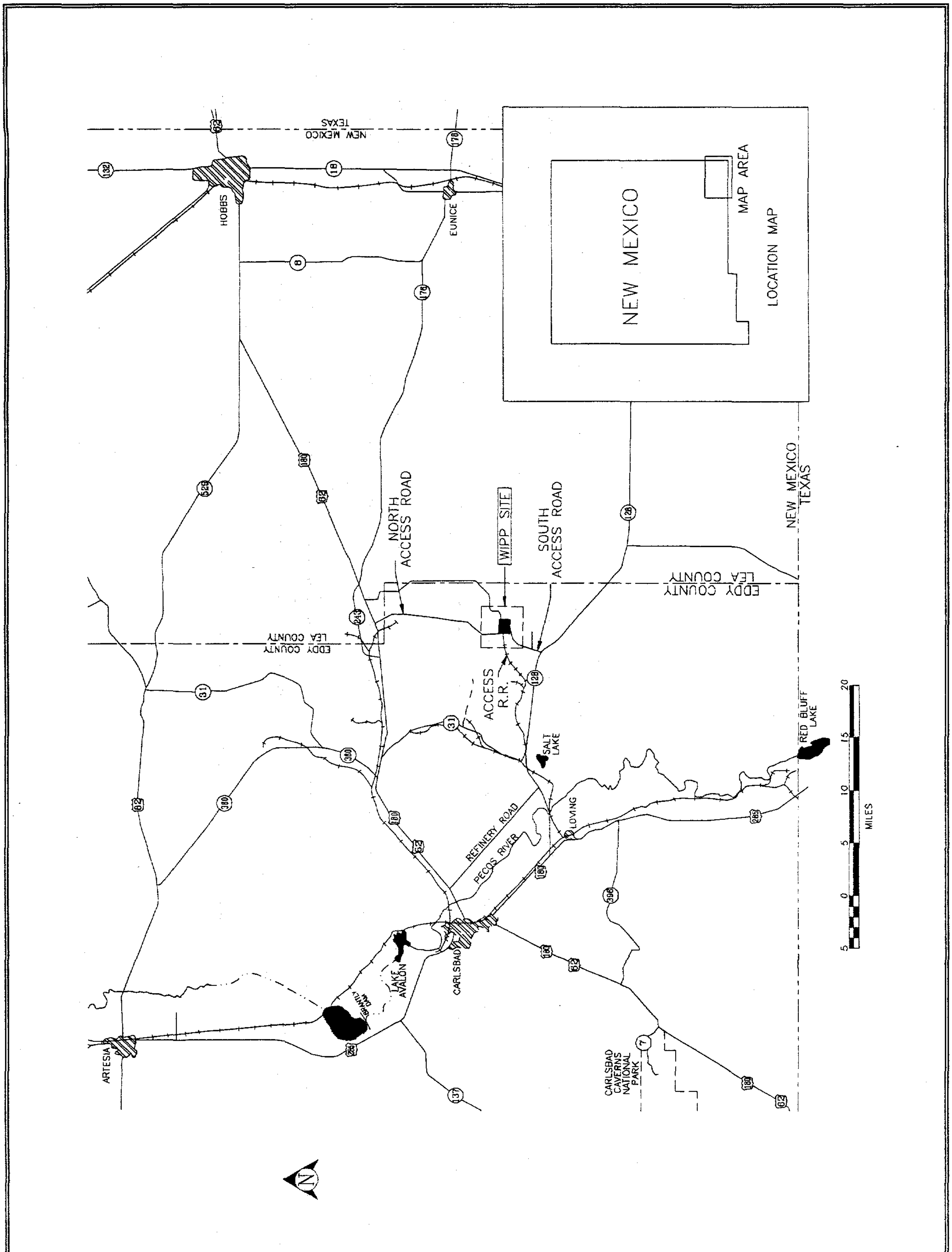


Figure 1-1
General Location of WIPP Facility

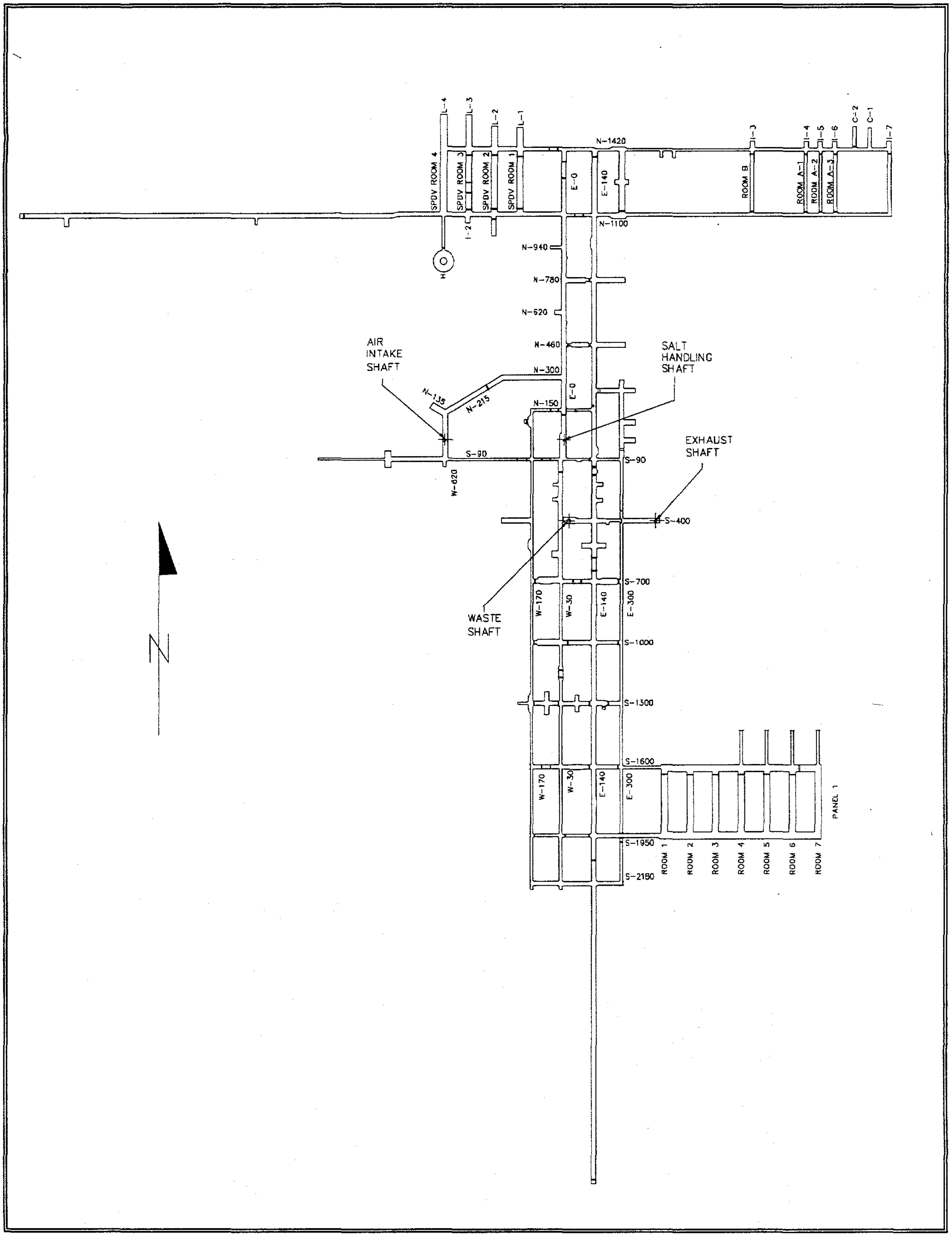


Figure 1-2
Schematic of Underground Facilities

design, construction, and performance of the subsurface structures. Operational aspects refer to the receiving, handling, and emplacement of transuranic mixed waste in salt. The facility is also designed for in-situ studies and nonradioactive experiments in salt.

1.3 Development Status

To fulfill its mission, the DOE is developing the WIPP in a phased manner. The SPDV phase began in 1980 with the purpose of characterizing the site and obtaining geotechnical data to determine whether site characteristics and design were suitable for a permanent disposal facility. During this phase, the Salt Handling Shaft, a Ventilation Shaft, a drift to the southernmost extent of the proposed waste storage area, a four-room experimental panel, and access drifts were excavated. Surface-based geological and hydrological investigations were also conducted. The data obtained from the SPDV investigations were reported in the Summary of the Results of the Evaluation of the WIPP Site and Preliminary Design Validation Program (DOE, 1983).

Based on the favorable results of the SPDV investigations, additional activities were started. These included construction of surface structures, conversion of the Ventilation Shaft for use as the Waste Shaft, excavation of the Exhaust Shaft, development of additional access drifts to the waste disposal area, and excavation of additional experimental rooms to support research and development activities. Geotechnical data acquired during this phase were used to evaluate the performance of the excavations in the context of established design criteria (DOE, 1984). Results of these evaluations were reported in Geotechnical Field Data and Analysis Reports (DOE, 1985; DOE, 1986a) and were summarized in the Design Validation Final Report (DOE, 1986b).

The Design Validation Final Report concluded that the facilities including waste disposal areas could be developed and operated to fulfill the long-term mission of the WIPP. However, some modifications to the reference design were proposed in order to meet the requirements for the life of the openings during an initial demonstration phase when retrievability of the waste remained an option. Current modifications are completed in accordance with current design descriptions.

The original design for the waste disposal rooms at the WIPP provided a limited time in which to mine the salt and emplace waste. Each panel, consisting of seven disposal rooms, was scheduled to be mined, filled with waste containers, and backfilled in less than five years before being closed. Field studies, as part of the SPDV Program, showed that unsupported openings of a typical disposal room configuration at the WIPP would remain stable, and that closure from creep would not impact equipment clearances during the five year period following excavation. The information from these studies provided the validation of the design of openings for the permanent disposal of waste under routine operations.

Panel 1 was developed to receive waste for an operations demonstration and test phase that were scheduled to start in October 1988. This original plan consisted of drums of contact-handled transuranic waste stored in rooms for a period of five years. The option to re-enter the waste-filled rooms were to be inaccessible, but the option to re-enter was to be maintained so that the waste could be removed, if required. To assist with the possible re-entry, rockbolts were installed in the rooms in Panel 1 to enhance roof stability.

The operations demonstration was deferred, and the test phase was modified to use contact-handled transuranic waste in bin-scale tests, planned for Room 1, Panel 1. This program was to investigate the repository conditions that would result from the different types of waste to be disposed of underground. The decision to use Room 1, Panel 1 for these bin-scale tests was made in June 1989, when initial waste receipt was anticipated for 1990. For the planned on-site bin-scale tests, an additional seven years of useful life was required for the test rooms in Panel 1. Panel 1 test rooms were required to provide essentially uninterrupted accessibility throughout the test phase. These changing requirements led to more stringent criteria for roof stability. In late 1993, the DOE decided to conduct the test phase off site, with a new date for first waste receipt at the WIPP in 1998. Although the test program has be moved off site, Panel 1 will be maintained and monitoring will continue.

1.4 Purpose and Scope of Geomechanical Monitoring Program

The purpose of the geomechanical monitoring program is to provide in- situ data to support:

- early detection of conditions that could compromise operational safety
- evaluation of room closure
- design modifications and remedial actions and,
- interpretation of the in-situ behavior of underground openings, for comparison with established design criteria.

The geomechanical instrumentation used in the WIPP facility provides data which are collected, processed, and stored for analysis. The following discussion briefly describes the major components of the GIS.

1.4.1 Instrumentation

Instruments installed for measuring the geomechanical response of the shafts and other underground openings include convergence points, convergence meters, extensometers, rockbolt load cells, pressure cells, strain gauges, piezometers, and joint meters. A summary of the geomechanical instrumentation specifications is presented in Table 1-1.

1.4.2 Data Acquisition

The individual instruments that comprise the GIS are either read manually, using portable devices, or remotely, by polling devices located on the surface. Remotely read instruments are connected to one of ten dataloggers located underground, and readings are collected by referencing the appropriate identifier code. Upon completion of a verification process, as described in WIPP procedures, the data are transferred to a computer database. The manual reading devices are taken to the instrument locations underground, and the instruments are read. The data are recorded on an appropriate data sheet and later entered into database files where they are combined with the remotely acquired readings.

The underground data acquisition system consists of instruments, polling devices, and a communications network. The polling devices are installed in underground LTC's to query each individual instrument. The polling devices are connected by datalink cables and modems to the surface datalogging computer. Each polling device consists of a remote switching station and readout unit that is connected to the instruments throughout the facility. The surface datalogging computer utilizes datalink cables with modem interfaces to receive the instrument readings from the polling units.

Whether acquired manually or automatically, GIS data are entered into the database files of the GIS data processing system. The processing system is programmed to enter, reduce, and transfer the instrumentation data to permanent storage files. Using dBase IV® software, the datalogging programs and the GIS data processing system programs are accessed by a personal computer located on the surface at the WIPP site. Additional programming allows access to these permanent storage files for tabular reporting and graphical plotting.

Table 1-1
Geomechanical Instrumentation System

Instrument Type	Measures	Range	Resolution
Sonic Probe Borehole Extensometer	Cumulative Deformation	0-2 inches	0.001 in.
Convergence Points	Cumulative Deformation	2-50 feet	0.001 in.
Wire Convergence Meters	Cumulative Deformation	0-3 feet	0.001 in.
Sonic Probe Convergence Meters	Cumulative Deformation	0-2.5 feet	0.001 in.
Embedded Strain Gauges	Cumulative Strain	0-3000 µin./in.	1 µin./in
Spot-Welded Strain Gauges	Cumulative Strain	0-2500 µin./in.	1 µin./in.
Rockbolt Load Cells	Load	0-50 tons	40 lb.
Earth Pressure Cells	Pressure	0-1000 psi	1 psi
Piezometers	Fluid Pressure	0-500 psi	0.5 psi
Joint Meters	Cumulative Deformation	0-4 inches	0.001 in.
Vibrating Wire Borehole Extensometer	Cumulative Deformation	0-4 inches	0.001 in.

1.4.3 Data Evaluation

Copies of the instrumentation data base and data plots are available upon request and can be found in previous Geotechnical Field Data and Analysis Reports.

Closure measurements are acquired manually from convergence point anchors and remotely with convergence meters. The plots are presented as ground displacement monitored over time and plotted as either surface displacement or closure versus time.

Extensometers provide relative displacement data acquired from sonic probe and vibrating wire extensometers. The displacement is the measure of movement in the rock strata, intercepted by the extensometer borehole at various depths. The displacement is then compared to a fixed point providing a measured displacement relative to a fixed point or relative displacement. The ground displacement is monitored over time and plotted as either surface displacement or closure versus time.

The extensometers consist of rods that are anchored in a borehole at various depths. The deepest anchor is assumed to be fixed in undisturbed ground and is considered the reference point. Typically, the plots will show greater ground movement near the opening, i.e., the collar of the hole.

Rockbolt load cells are used to determine the bolt loading and provide an indication of the condition of rockbolts. Plots consist of load in pounds, versus time for each instrumented bolt.

Earth pressure cells and strain gauges are used to determine the loads in and around the shaft liners and are depicted in time-based plots. Monitoring of these instruments indicates whether there is any stress buildup in the shaft lining systems.

Piezometers are used to measure the gauge pressure of ground water. They are installed in the shafts at varying elevations to monitor the hydraulic head acting on the shaft liners. Plots from piezometers are presented as pressure versus time.

Joint meters are currently installed underground in the S1950 drift and the E0 drift. They are installed perpendicular to a crack and monitor the separation of the crack over time. Plots are presented as displacement versus time.

1.4.4 Data Errors

As described above, GIS data are processed through a comprehensive database management system. These steps are performed according to approved quality control procedures. Whether acquired manually or remotely, GIS data are processed and permanently stored according to approved procedures. On occasion, when an erroneous reading occurs; there are several possible explanations for this.

- the measuring device was misread
- the reading was recorded incorrectly or,
- the measuring device was not functioning within specifications.

When an erroneous reading is discovered, an immediate evaluation of the previous readings is performed and a second reading is collected. If the second reading falls in line with the instrument trend, the first reading is discarded and the second reading is entered in the database. If the second reading and other readings taken thereafter remain out of the instrument trend, an assessment of ground conditions in the vicinity of the instrument is performed to determine the reason for the discrepancy. In addition, reading frequency may be increased.

All steps performed to correct erroneous readings are documented and filed for future reference.

2.0 Geology

This section provides a generalized description of the stratigraphy of the region surrounding the WIPP. Detailed descriptions of the units can be found in Geologic Mapping of the Air Intake Shaft at the Waste Isolation Pilot Plant (DOE, 1990a). A generalized stratigraphy of the geology of the region surrounding WIPP is given in Figure 2-1.

2.1 Castile Formation

The Castile Formation formed from the precipitation of evaporites within the confines of a great Permian reef complex. At the WIPP, the Castile is composed chiefly of halite with a few interbeds of anhydrite and some limestone. The formation is approximately 1250 feet (380 m) thick in the WIPP vicinity.

2.2 Salado Formation

After deposition of the Castile Formation, a shallow saline lagoon environment remained which then progressed through numerous desiccation and inflow cycles, resulting in the deposition of nearly 2000 feet (610 m) of evaporites (primarily halite) of the Salado Formation. The desiccation and inflow cycles are reflected in cyclic bed sequences. Each cycle begins with a layer consisting predominantly of claystone. This lowermost zone is followed by a layer of anhydrite/polyhalite, which is in turn followed by a layer of halite. The entire sequence is capped by a bed of argillaceous (clay-rich) halite.

The Salado Formation is subdivided into three informal members as shown in Figure 2-1, each of which contains similar amounts of halite, anhydrite, and polyhalite, and which are differentiated based on the presence of other minerals. The WIPP storage and experimental horizons are located within the lower Salado member, 2,150 feet (655 m) below the surface.

The excavations lie within a 40-foot (12 m) laterally continuous stratigraphic unit as described above. Within this unit, layers of clay and anhydrite are locally designated as shown in Figure 2-2. The layers of clay and anhydrite have a significant impact on the mechanical performance of the excavations. Clay layers provide surfaces along which slip and separation can occur, whereas anhydrite acts as a brittle unit that does not deform plastically.

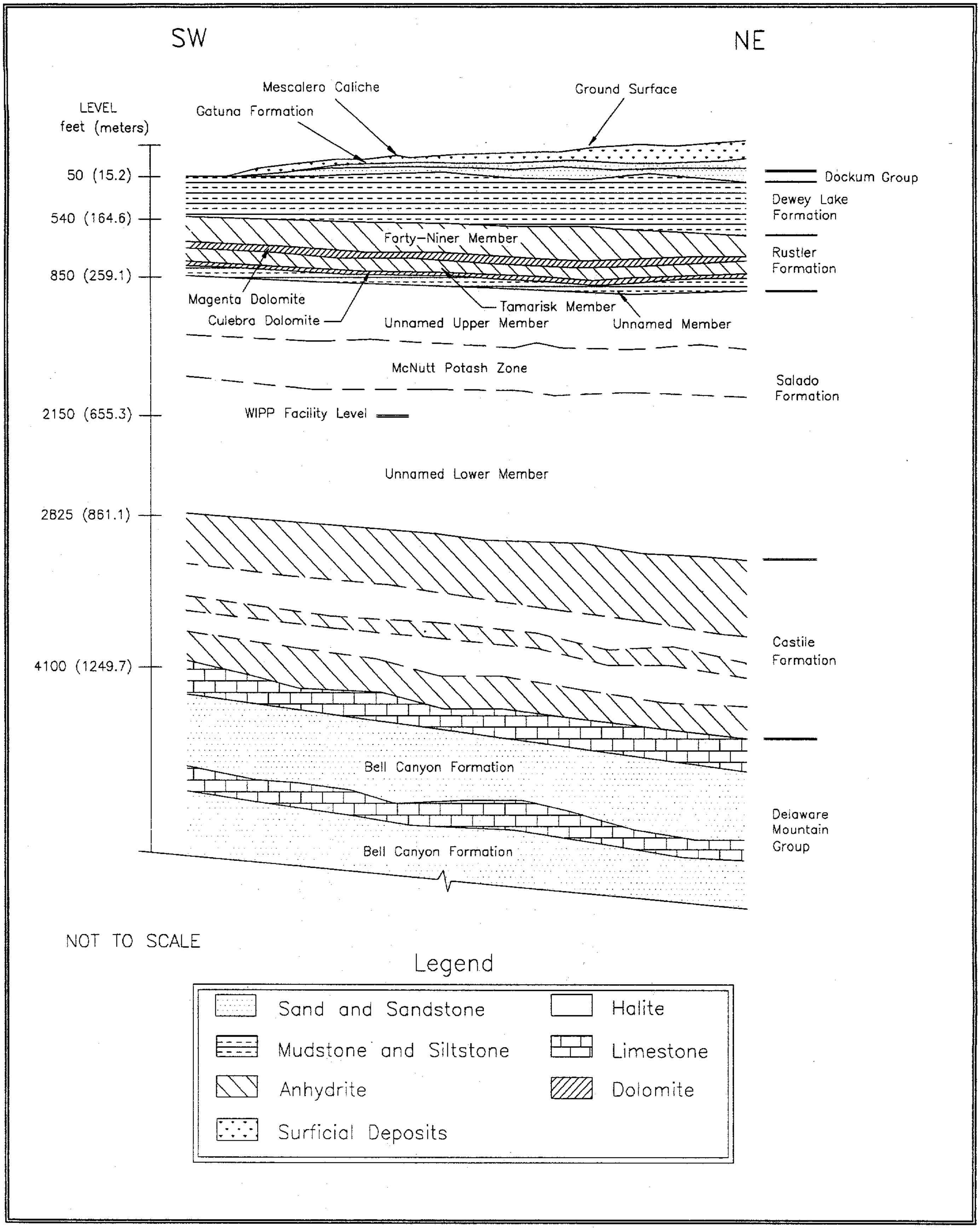
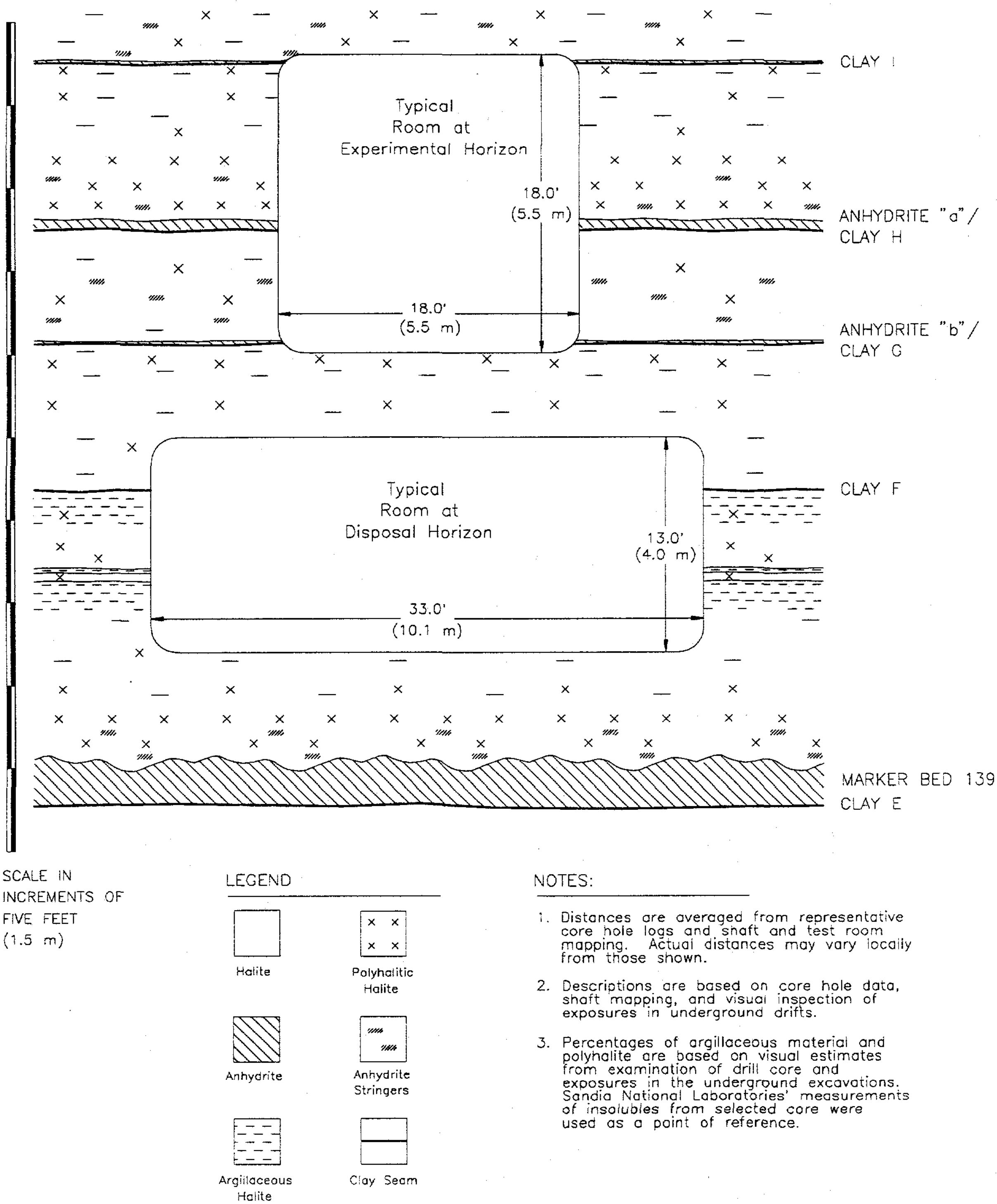


Figure 2-1
General Geologic Cross Section



NOTES:

1. Distances are averaged from representative core hole logs and shaft and test room mapping. Actual distances may vary locally from those shown.
2. Descriptions are based on core hole data, shaft mapping, and visual inspection of exposures in underground drifts.
3. Percentages of argillaceous material and polyhalite are based on visual estimates from examination of drill core and exposures in the underground excavations. Sandia National Laboratories' measurements of insolubles from selected core were used as a point of reference.

Figure 2-2
General Stratigraphic Column

2.2.1 Disposal Horizon

At the disposal horizon, Anhydrite "a", is underlain by Clay H, is located about 13 feet (4 m) above the roof of the excavations. Anhydrite "b", underlain by Clay G, is located about 6.5 feet (2 m) above the roof, and a diffuse clay, Clay F, is found in the ribs about two feet (0.6 m) below the roof.

A 20- to 32-inch (50 to 80 cm) thick, bed of anhydrite, identified as Marker Bed 139 (MB139), typically lies about five feet (1.5 m) below the floor throughout the storage horizon. MB139 shows lateral variability in thickness, of up to six inches (15 cm), at both repository and regional scale. The top of MB139 is undulatory, while the bottom is subhorizontal and underlain by Clay E.

2.2.2 Experimental Horizon

Excavations in the experimental horizon lie 18 feet (6 m) above the storage horizon. The floor is at Anhydrite "b", and Anhydrite "a" is visible in the ribs or roof. Clay I typically lies 4.5 to 9 feet (1.4 to 2.7 m) above the roof in the access drifts, and near the roof in the rooms. As in the disposal horizon, the clay layers have a significant impact on the mechanical deformation of the experimental excavations.

2.3 Rustler Formation

Overlying the Salado Formation is the Rustler Formation. The Rustler is about 300 feet (91 m) thick and contains proportionately the largest amount of clastic material of the three evaporite formations. The Rustler is subdivided into five members as follows: an unnamed lower member, the Culebra Dolomite Member, the Tamarisk Anhydrite Member, the Magenta Dolomite Member and the Forty-niner Anhydrite Member.

At the WIPP, the Rustler is lithologically divided. The lower portion consists primarily of sandstone, with lesser amounts of anhydrite, polyhalite and halite. The anhydritic upper portion contains interbeds of mudstone and dolomite. The Culebra and Magenta dolomite members are found within this upper portion and are areally persistent, serving as important marker units. The Culebra is the most transmissive hydrologic unit.

2.4 Dewey Lake Redbeds

The Dewey Lake Redbeds are the uppermost of the Permian formations in the WIPP area. Within the series, the Dewey Lake represents a transition from the lower, marine-influenced evaporite deposition to deposition on a broad, low-relief, fluvial plain. The redbeds, about 475 feet (145 m) thick, consist of interbedded fine sandstone, siltstone, mudstone, and claystone with locally abundant greenish-gray reduction spots and gypsum-filled fractures.

2.5 Dockum Group

The Dockum Group consists of about 25 feet (7.6 m) of fine-grained flood plain sediments and coarse alluvial debris of late Triassic age. At the WIPP, the Dockum Group forms an erosional wedge, pinching out near the center of the site. Some authors (DOE, 1990a) use the term Santa Rosa to refer to local rocks of Triassic age.

2.6 Gatuña Formation and Mescalero Caliche

The Gatuña Formation and Mescalero caliche are of Miocene to Pleistocene age. The Gatuña, which overlies the Dockum Group in the WIPP vicinity, consists of about 13 feet (4 m) of poorly consolidated sand, gravel, and silty clay which is light red and mottled with dark stains. The unit contains abundant calcium carbonate but is poorly cemented.

The Mescalero caliche, about 4 feet (1.2 m) thick in the WIPP vicinity, is a hard, resistant soil horizon that lies beneath a cover of wind-blown sand. The horizon is petrocalcic, or very strongly cemented with calcium carbonate. Petrocalcic horizons form slowly beneath a stable landscape at the maximum depth of infiltration of soil moisture. Many of the surface structures at WIPP are founded on top of the Mescalero caliche.

3.0 Performance of Shafts and Keys

3.1 Salt Handling Shaft

The first construction activity undertaken during the SPDV work was the excavation of the Exploratory Shaft, subsequently designated the Construction and Salt Handling (C&SH) Shaft, and currently designated the Salt Handling Shaft (Figure 1-2). The shaft was drilled from July 4 to October 24, 1981, and geologic mapping was conducted in the spring of 1982 (DOE, 1983). Figure 3-1 presents the generalized stratigraphy of the Salt Handling Shaft.

The Salt Handling Shaft is lined with steel casing and has a 10-foot (3-meter) inside diameter from the ground surface to a depth of 846 feet (258 meters). The 10-foot (3-meter) diameter extends through the concrete shaft key to a depth of 880 feet (268 meters). The nominal shaft diameter is 12 feet (4 m) from the key to the bottom of the shaft at 2,298 feet (700 meters) and is unlined. The steel liner thickness increases with depth, starting at 0.62-inch (1.6-centimeter) thick at the top and increasing to 1.5 inches (3.8 centimeters) thick at the key. Cement grout surrounds the liner. The shaft key is a 37.5-foot (11.4 meter)-long reinforced-concrete structure at the base of the steel liner.

3.1.1 Shaft Performance

During this reporting period no modifications were made to the Salt Handling Shaft. Shaft inspections continued to be performed on a weekly basis. The shaft is in satisfactory condition.

3.1.2 Instrumentation

Installation of geomechanical instrumentation in the Salt Handling Shaft took place in April and July of 1982. Instrumentation consists of extensometers, piezometers, and radial convergence points at various levels in the shaft as shown in Figure 3-2. In the shaft key, instrumentation includes strain gauges, pressure cells, and piezometers (Figure 3-3).

Seventy-two percent of the instrumentation in the Salt Handling Shaft remain functional. Instruments that are still in operation include one extensometer at level 2057' (627 m), all twelve piezometers, three earth pressure cells, and 31 strain gauges.

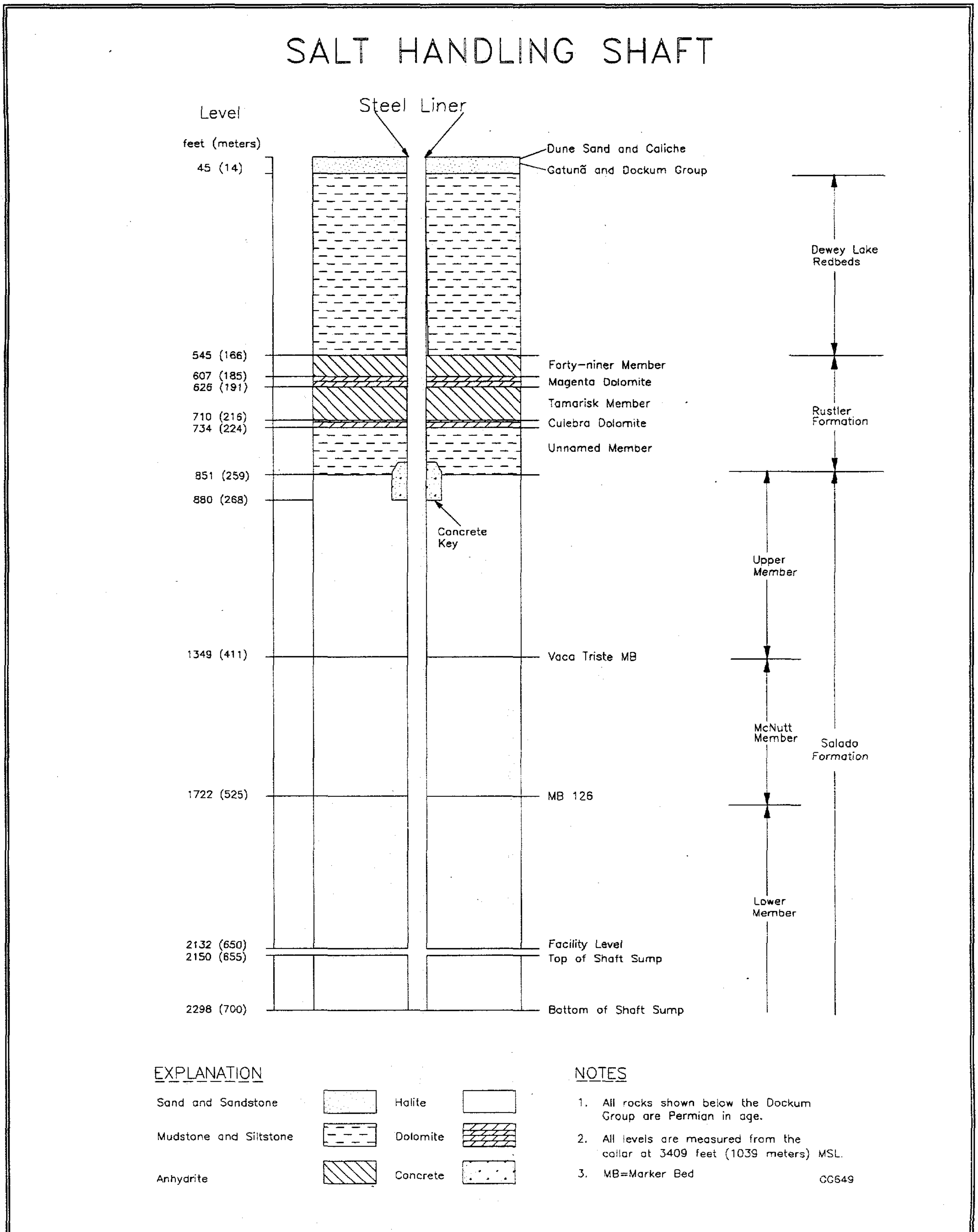


Figure 3-1
Generalized Salt Handling Shaft Statigraphy

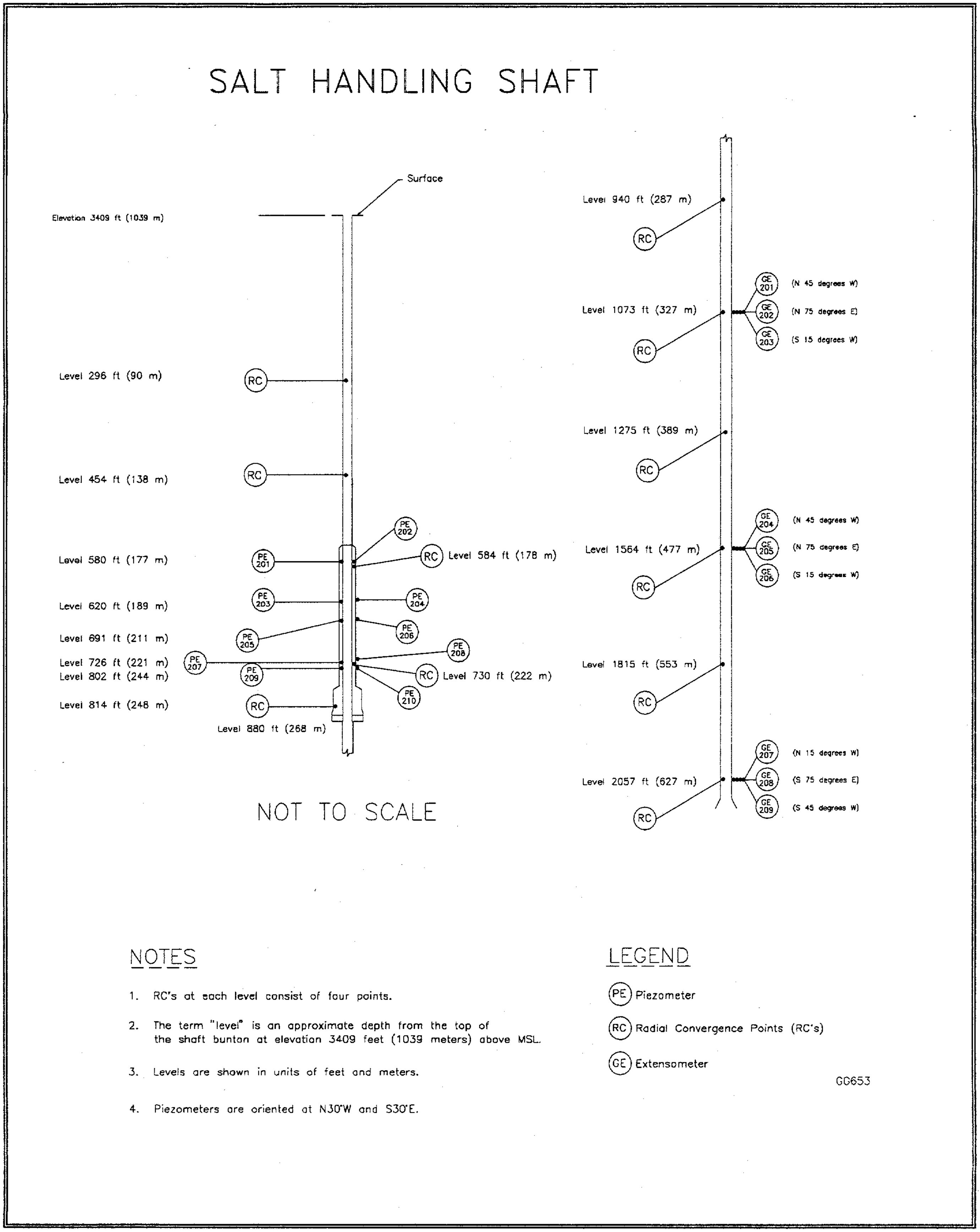


Figure 3-2
Salt Handling Shaft Instrumentation (without Shaft Key)

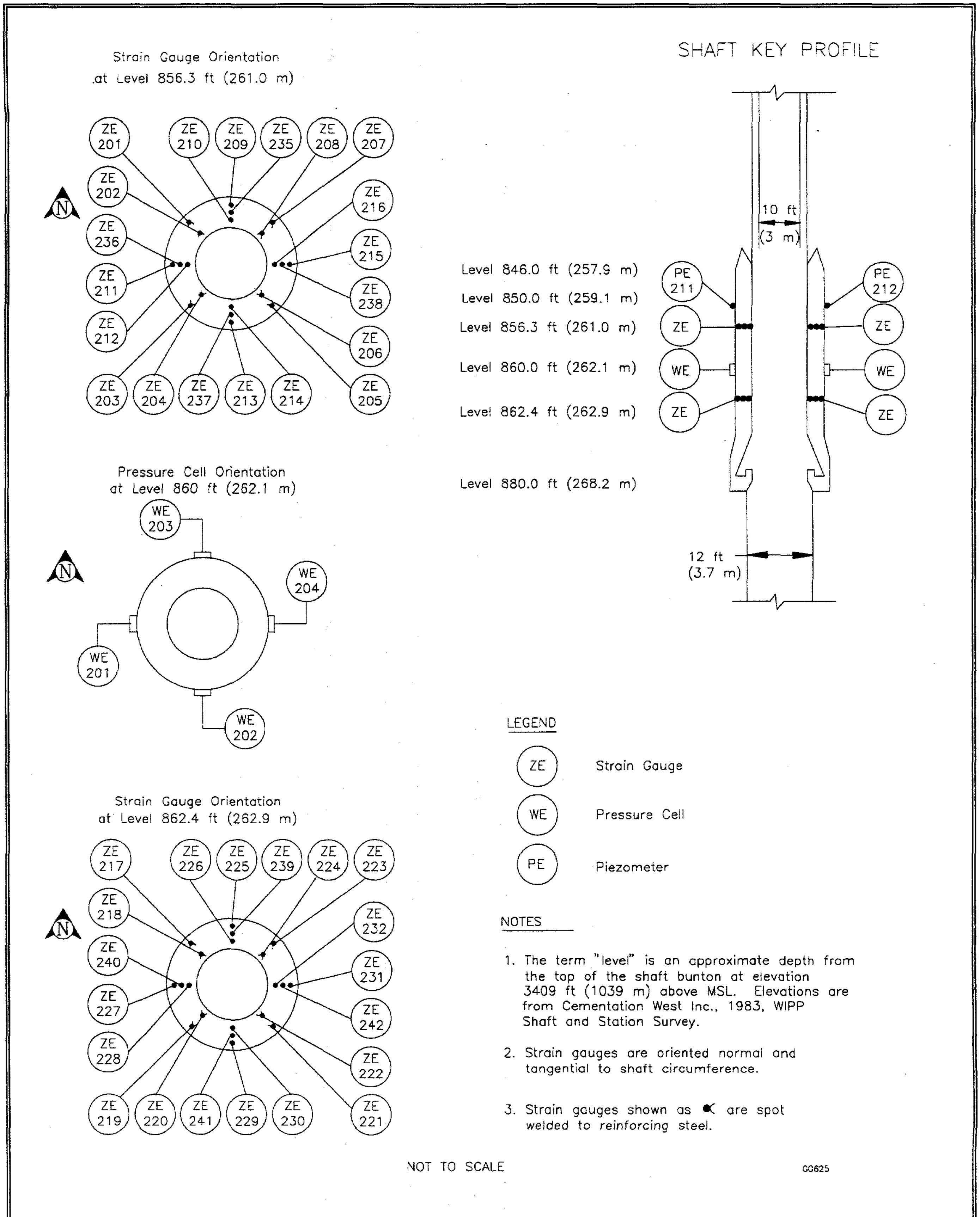


Figure 3-3
Salt Handling Shaft Key Instrumentation

3.2 Waste Shaft

As part of the SPDV program, a six-foot (2-meter) diameter ventilation shaft was excavated from December 1981 through February 1982 in order to allow a two-shaft underground air circulation scheme to function. From October 11, 1983 through June 11, 1984, this shaft was enlarged to 20- to 23-foot (6- to 7- meter) diameter and lined, then renamed the Waste Shaft. Geologic mapping was conducted during shaft enlargement from December 9, 1983 through June 5, 1984 (Holt and Powers, 1984). The generalized stratigraphy of the Waste Shaft is presented in Figure 3-4.

The Waste Shaft is lined with unreinforced concrete and has a 19-foot (6-meter) inside diameter from the ground surface to 837 feet (255 meter). The liner thickness increases with depth from 10 inches (25 cm) at the surface to 20 inches (51 cm) at the key. The Waste Shaft key is 63 feet (19 m) long, 4.25 feet (1.3 m) thick, and constructed of reinforced concrete. The bottom of the key is at 900 feet (274 meter). The section of the shaft below the key is 20 feet (6 m) in diameter and increases to 23 feet (7 m) just above the shaft station. The 23-foot (7-meter) diameter extends to a depth of approximately 2,286 feet (697 meters).

3.2.1 Shaft Performance

Shaft inspections are conducted on a weekly basis by mining operations personnel. The condition of the shaft is satisfactory. During this reporting period no modifications have been made to the Waste Shaft. The cable-support ground control system installed in the east brow of the shaft station was detensioned twice during this reporting period.

3.2.2 Instrumentation

Extensometers, piezometers, earth pressure cells, and radial convergence points were installed in the Waste Shaft from August 27 through September 10, 1984. The instrumentation configuration is presented in Figures 3-5 and 3-6.

Nine multi-position borehole extensometers were installed in arrays at levels 1071' (326 m), 1566' (477 m), and 2059' (628 m) as shown in Figure 3-5. Each array consists of three extensometers. Currently, five extensometers remain functional.

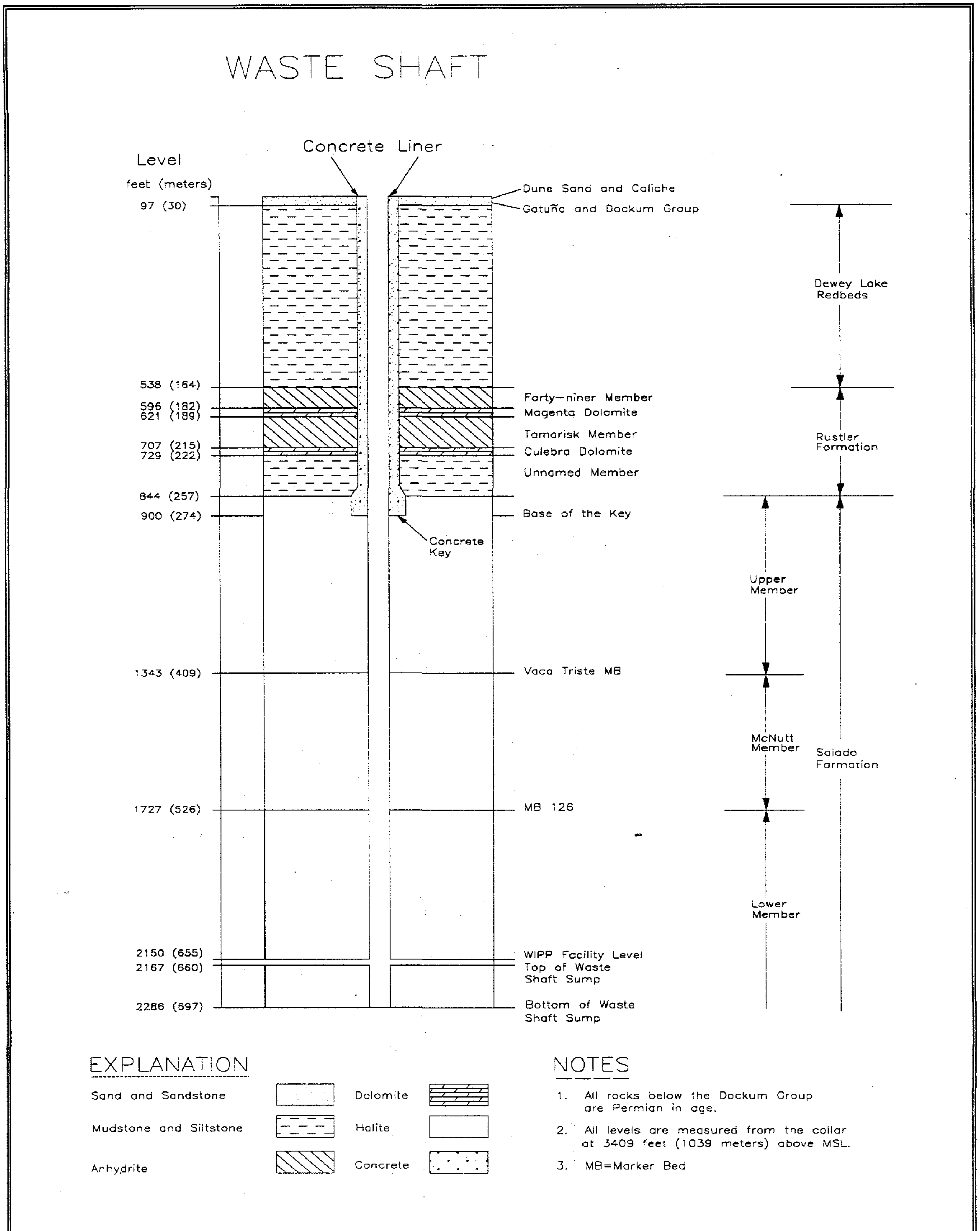


Figure 3-4
Generalized Waste Shaft Stratigraphy

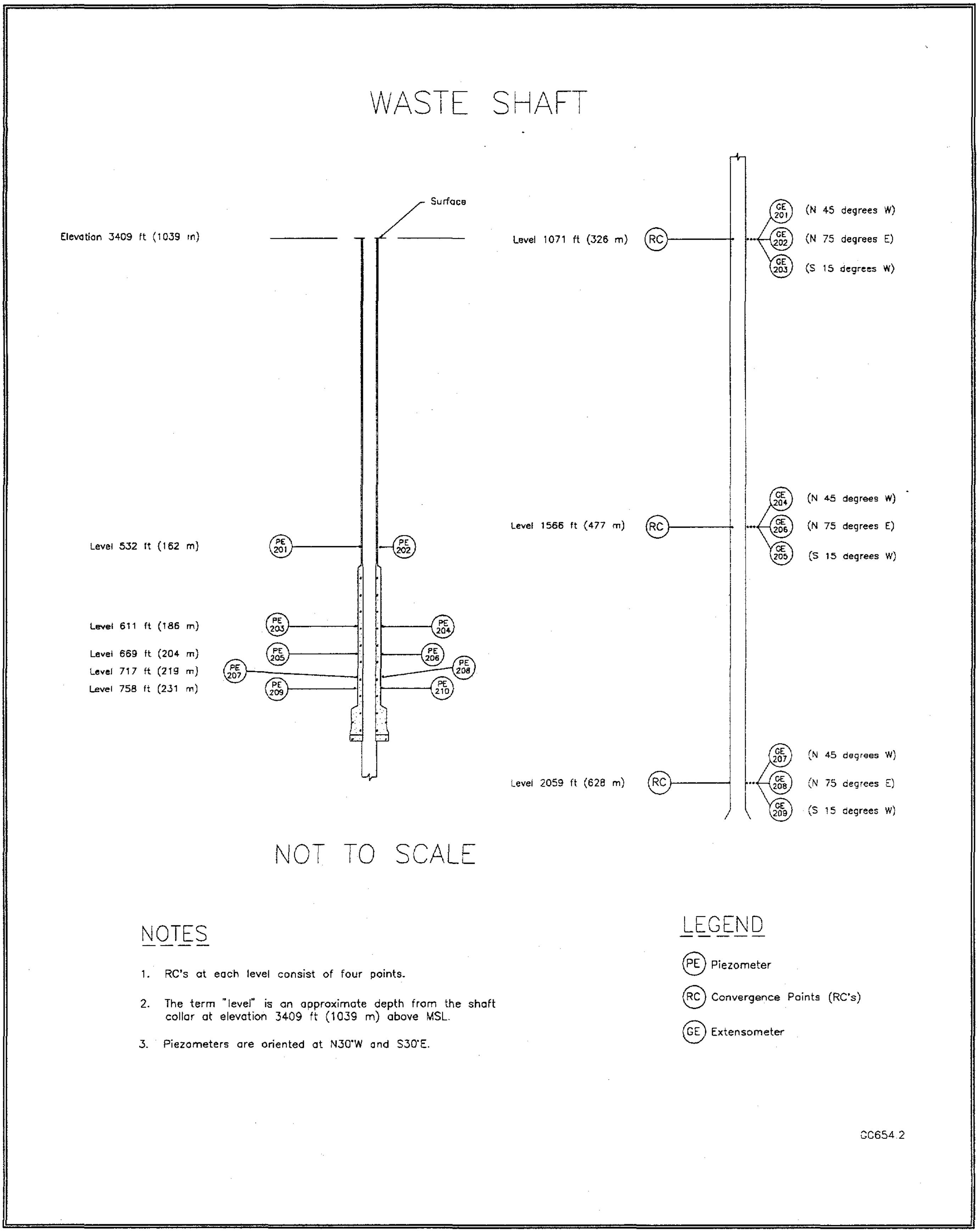


Figure 3-5
Waste Shaft Instrumentation (without Shaft Key)

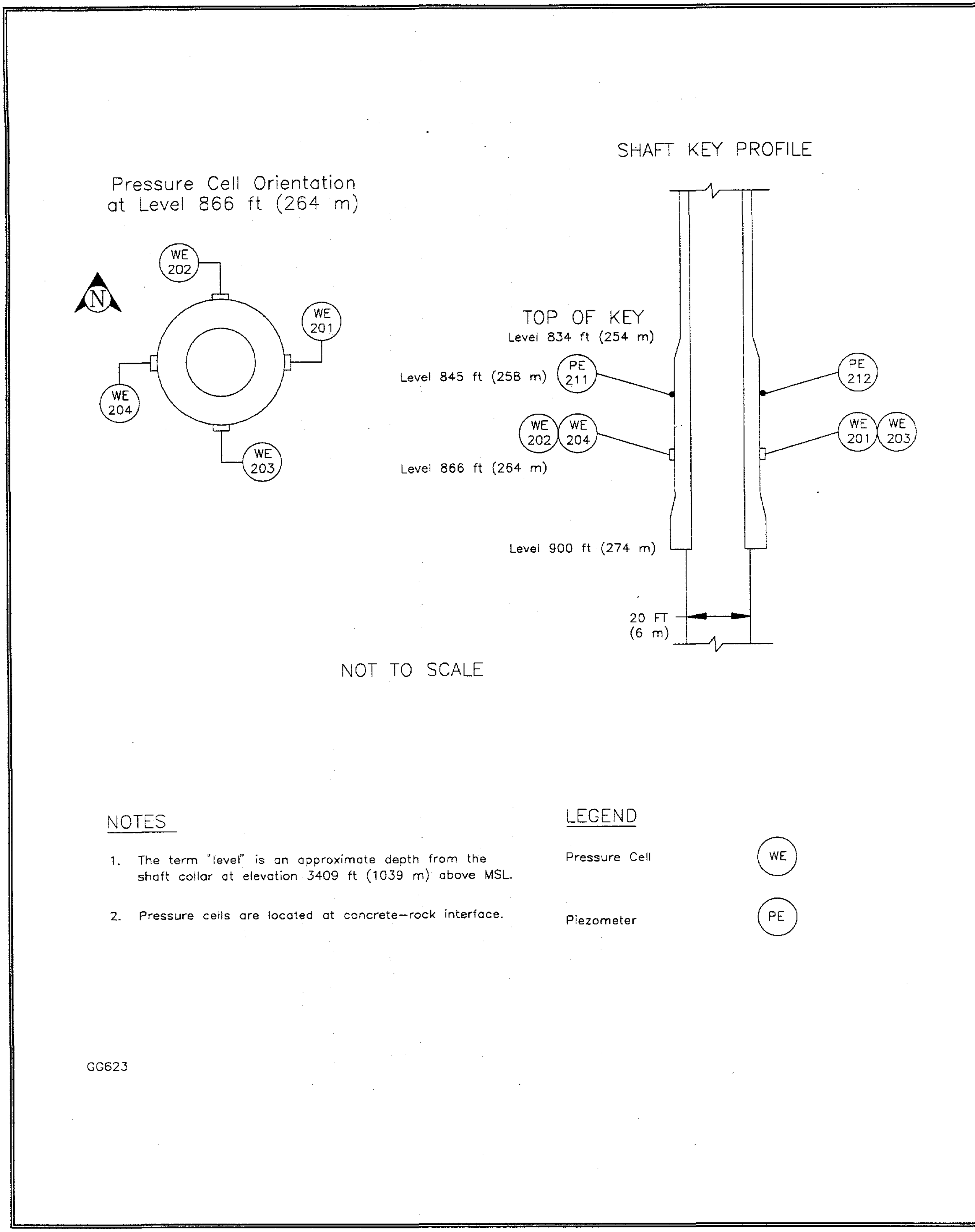


Figure 3-6
Waste Shaft Key Instrumentation

Twelve piezometers were installed in the lined section of the Waste Shaft on September 7, and 8, 1984 to monitor pressure behind the shaft liner and key section in the shaft. All twelve piezometers remain in operation

Four earth pressure cells were installed in the key section of the Waste Shaft during concrete emplacement from March 23 through April 3, 1984. These instruments measure the normal stress between the concrete key and the Salado Formation as the creep effects load the key structure. All earth pressure cells remain in working condition.

3.3 Exhaust Shaft

The Exhaust Shaft was excavated from September 22, 1983, through November 29, 1984, to establish a dedicated route for exhaust air from the underground facility to the surface. Comprehensive geologic mapping was conducted from July 16, 1984, through January 18, 1985, (DOE, 1986c). The generalized Exhaust Shaft stratigraphy is presented in Figure 3-7.

The Exhaust Shaft is lined with unreinforced concrete from the surface to the top of the shaft key at a depth of 844 feet (257 m). The liner thickness increases from 10 to 16 inches (25 to 41 cm) over that distance. The Exhaust Shaft key is 63 feet (19 m) long and 3.5 feet (1 meter) thick.

The diameter below the shaft key is 15 feet (5 m). There, the shaft is lined with wire mesh anchored by rockbolts. The shaft terminates at the facility horizon, 2150 feet (655 m), since there is no shaft sump.

3.3.1 Shaft Performance

Semi-annual video inspections continue to be performed and most recent inspection indicates that the shaft remains in satisfactory condition.

3.3.2 Instrumentation

The Exhaust Shaft was equipped with geomechanical instrumentation in two stages. Earth pressure cells were installed behind the liner key during November 1984, and piezometers and multi-position borehole extensometers were installed during November and December 1985. The instrumentation configuration is presented in Figures 3-8 and 3-9.

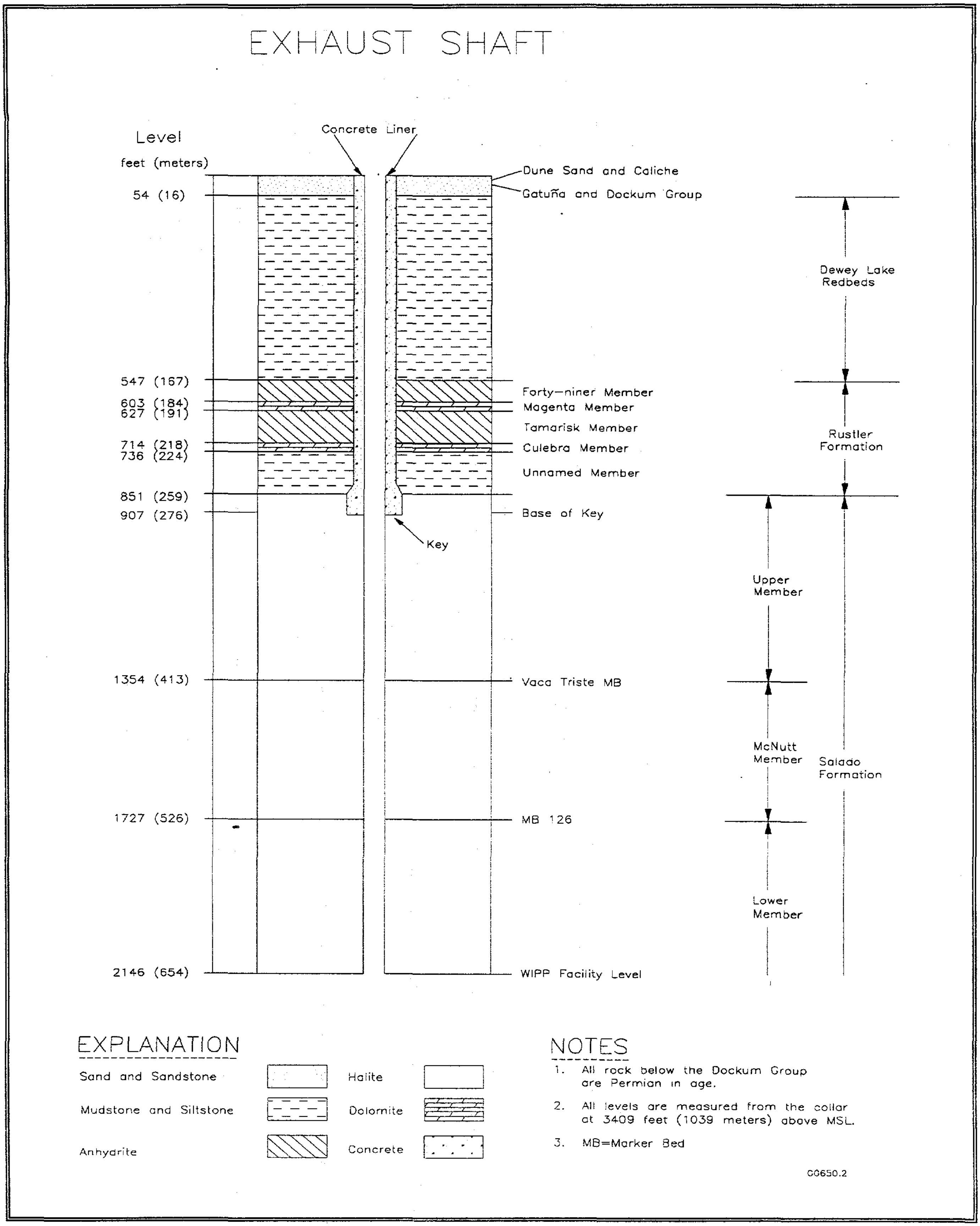
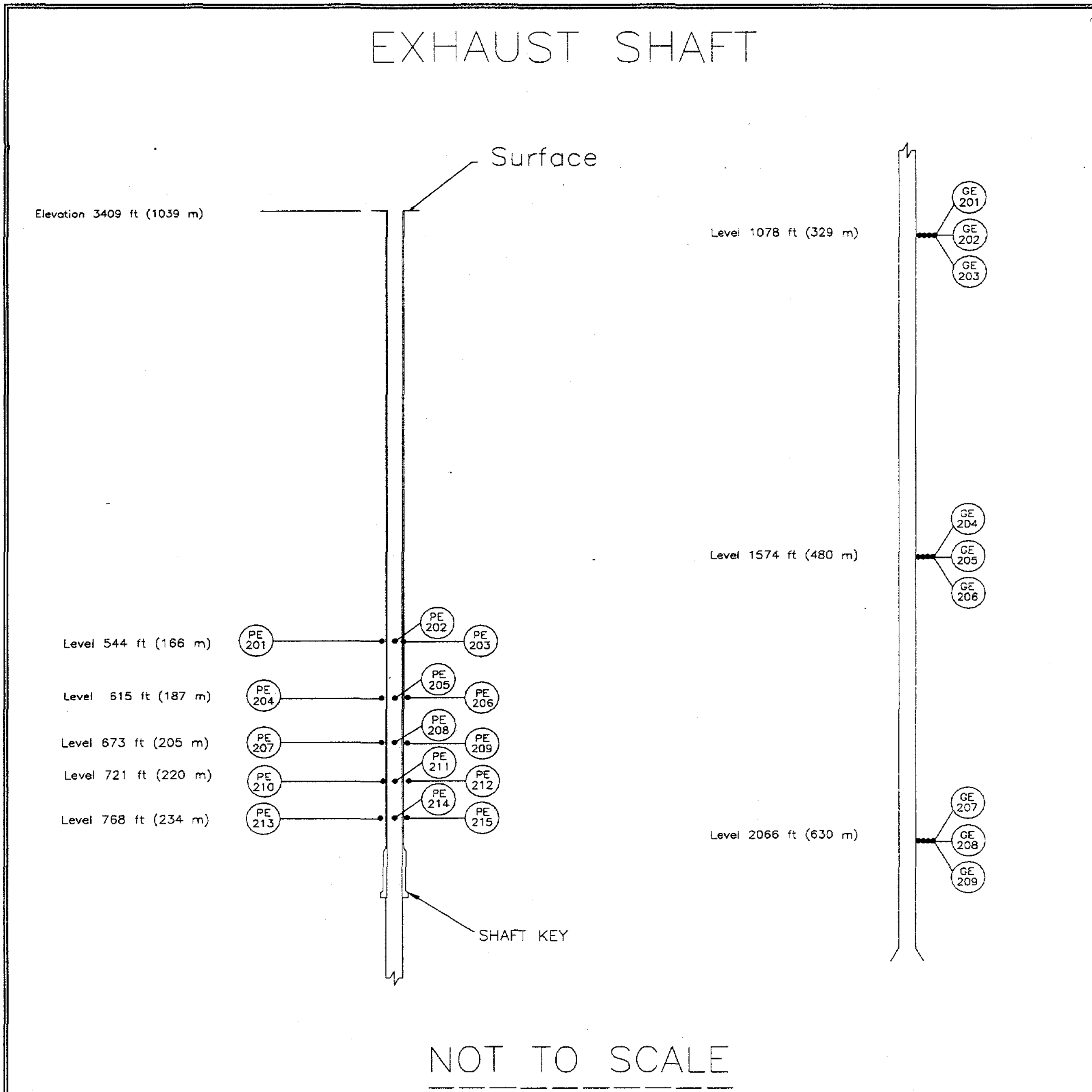


Figure 3-7
Generalized Exhaust Shaft Stratigraphy



NOTES

1. The term "level" is an approximate depth from the shaft collar at elevation 3409 ft (1039 m) above MSL.
2. Piezometers and extensometers are oriented at N75°E, N45°W and S15°W.

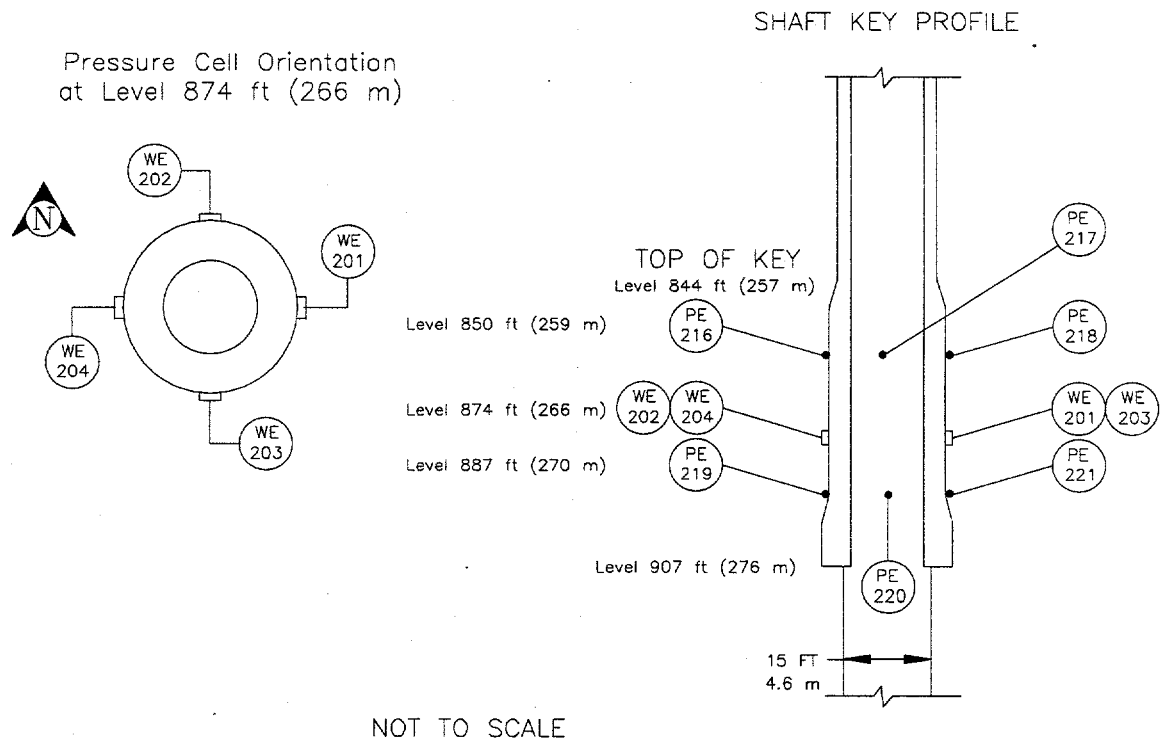
LEGEND

(PE) Piezometer

(GE) Extensometer

CG655

Figure 3-8
Exhaust Shaft Instrumentation (without Shaft Key)



LEGEND

(WE) Pressure Cell

(PE) Piezometer

NOTES

1. The term "level" is an approximate depth from the shaft collar at elevation 3409 ft (1039 m) above MSL.
2. Pressure cells are located at concrete–rock interface and oriented at azimuths of 0°, 90°, 180°, and 270°.
3. Piezometers are oriented at azimuths of 75°, 195°, and 315°.

CG624

Figure 3-9
Exhaust Shaft Key Instrumentation

Nine multi-position borehole extensometers were installed in the Exhaust Shaft, and only one at level 1078' (329 m) remains operational. Of the 21 piezometers installed, 20 remain in working condition.

3.4 Air Intake Shaft

The Air Intake Shaft was excavated from December 4, 1987 through August 31, 1988 to establish a dedicated route for surface air to enter the underground facility. Comprehensive geologic mapping was conducted from September 14, 1988 through November 14, 1989 (DOE, 1990a). The generalized Air Intake Shaft stratigraphy is presented in Figure 3-10.

The Air Intake Shaft was lined with unreinforced concrete from the surface to a depth of 903 feet (275 m), the bottom of the shaft key. The Air Intake Shaft key is 81 feet (25 m) long with an inside diameter of 16 feet (5 m). The diameter below the shaft key is 20 feet (6 m), and the shaft is unlined from 81 feet (25 m) to depth of 2,150 feet (655 m).

3.4.1 Shaft Performance

The Air Intake Shaft remains in excellent condition and weekly inspections continue to be performed.

3.4.2 Instrumentation

At the present time, the Air Intake Shaft is monitored by Sandia National Laboratories, and the data collected is not included in this report.

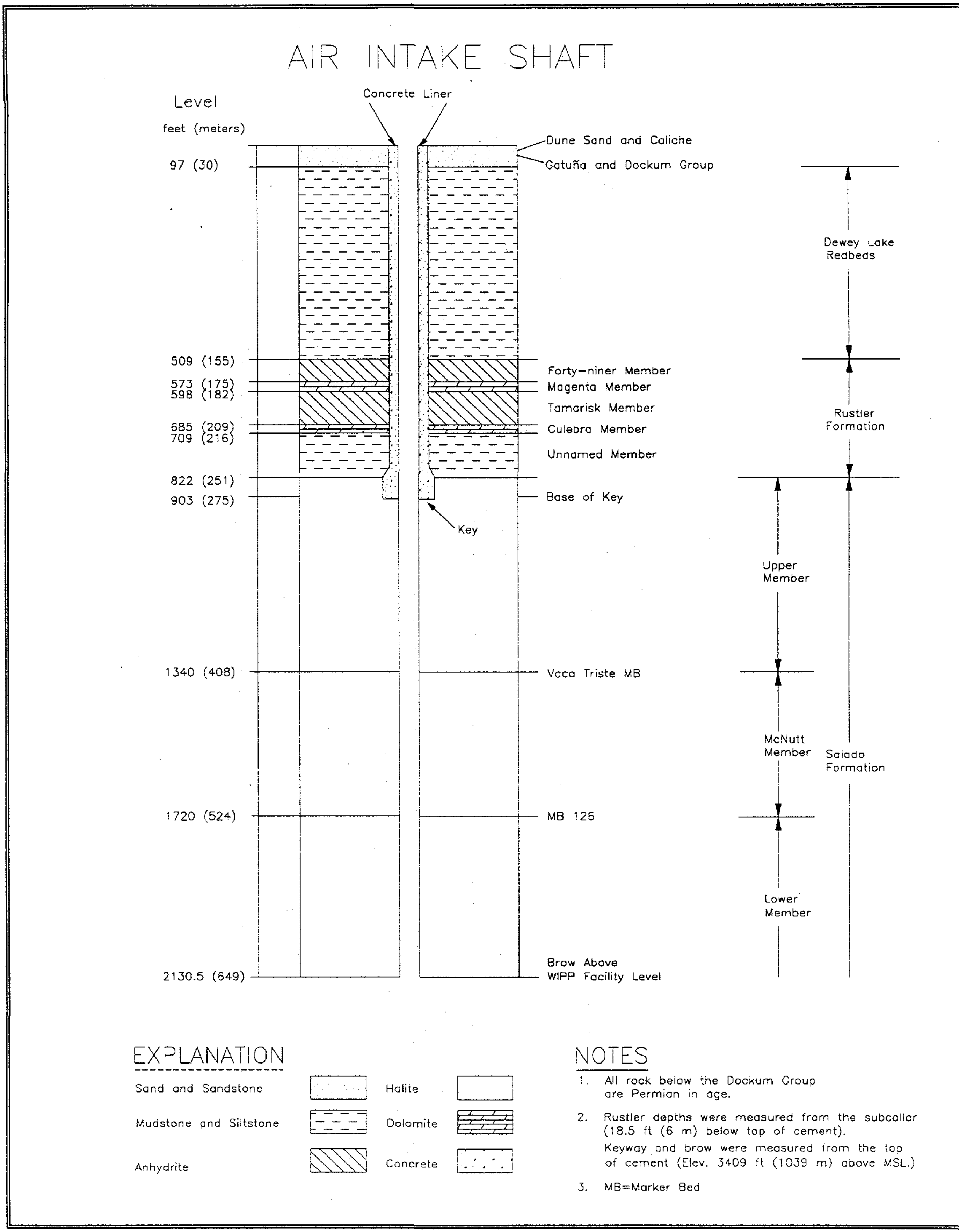


Figure 3-10
Generalized Air Intake Shaft Stratigraphy

4.0 Performance of Shaft Stations

4.1 Salt Handling Shaft Station

The Salt Handling Shaft Station, Figure 1-2, was excavated from May 2 to June 3, 1982 by drilling and blasting. In 1987, the station was trimmed, removing the roof beam up to Anhydrite "b" between S90 and N20 using a continuous miner. The station area south of the shaft is 90 feet (27 m) long and 32 to 38 feet (10 to 12 m) wide. The height of the station south of the shaft is 18 feet (5.5 m) for a distance of 54 feet (16.5 m), and 14 feet (4.3 m) for the remaining 36 feet (11 m). The station dimensions north of the shaft are approximately 30 feet (9 m) long, 32 to 35 feet (10 to 11 m) wide and 12 feet (4 m) high. The shaft extends approximately 140 feet (43 m) below the facility horizon to accommodate the skip loading equipment and to act as a sump to collect any water that might enter the shaft. A generalized cross section of the station is shown in Figure 4-1.

4.1.1 Modifications to Excavation

Modifications made during this reporting period include local scaling of roof and ribs. No major modifications were made to the salt shaft station during this reporting period.

4.1.2 Instrumentation

Geomechanical instrumentation was installed in the Salt Handling Shaft Station from June 1982 through February 1983, with subsequent reinstallation of extensometers and convergence points as necessary. Figure 4-2 details the instrument locations in the Salt Handling Shaft Station before the roof was removed. Most of the instruments were either removed, or readings were suspended prior to the mining of the roof. Figure 4-3 details the instrument locations after the roof was taken down.

Currently, there are two functioning extensometers in the Salt Handling Shaft Station, they are located in the roof at S30, and S65. These extensometers were installed in September 1988, replacing two extensometers mined out during trimming activities. In addition, five convergence point arrays are monitored at N39, S18, S30, S65, and W12.

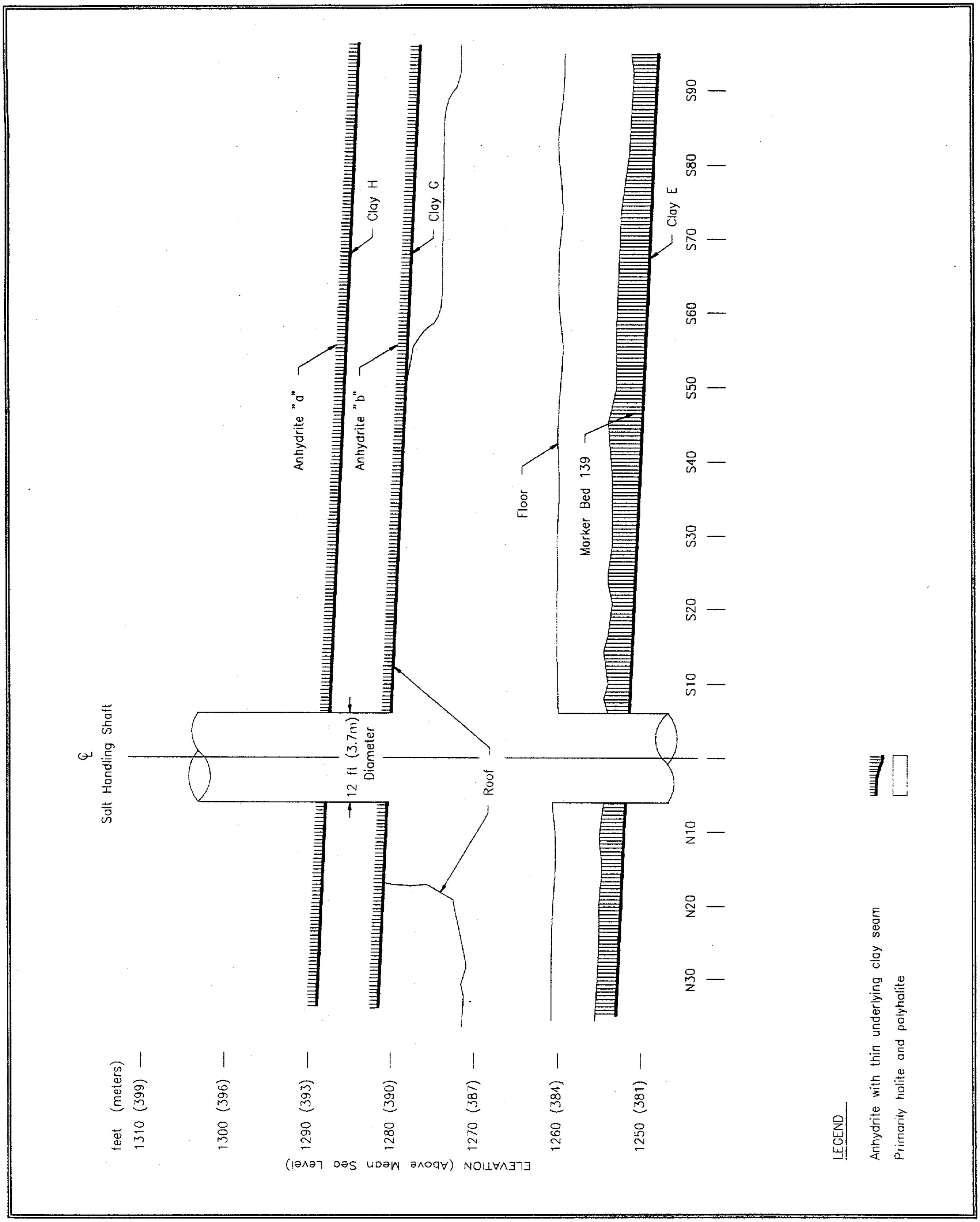


Figure 4-1
Generalized Salt Handling Shaft Station Stratigraphy

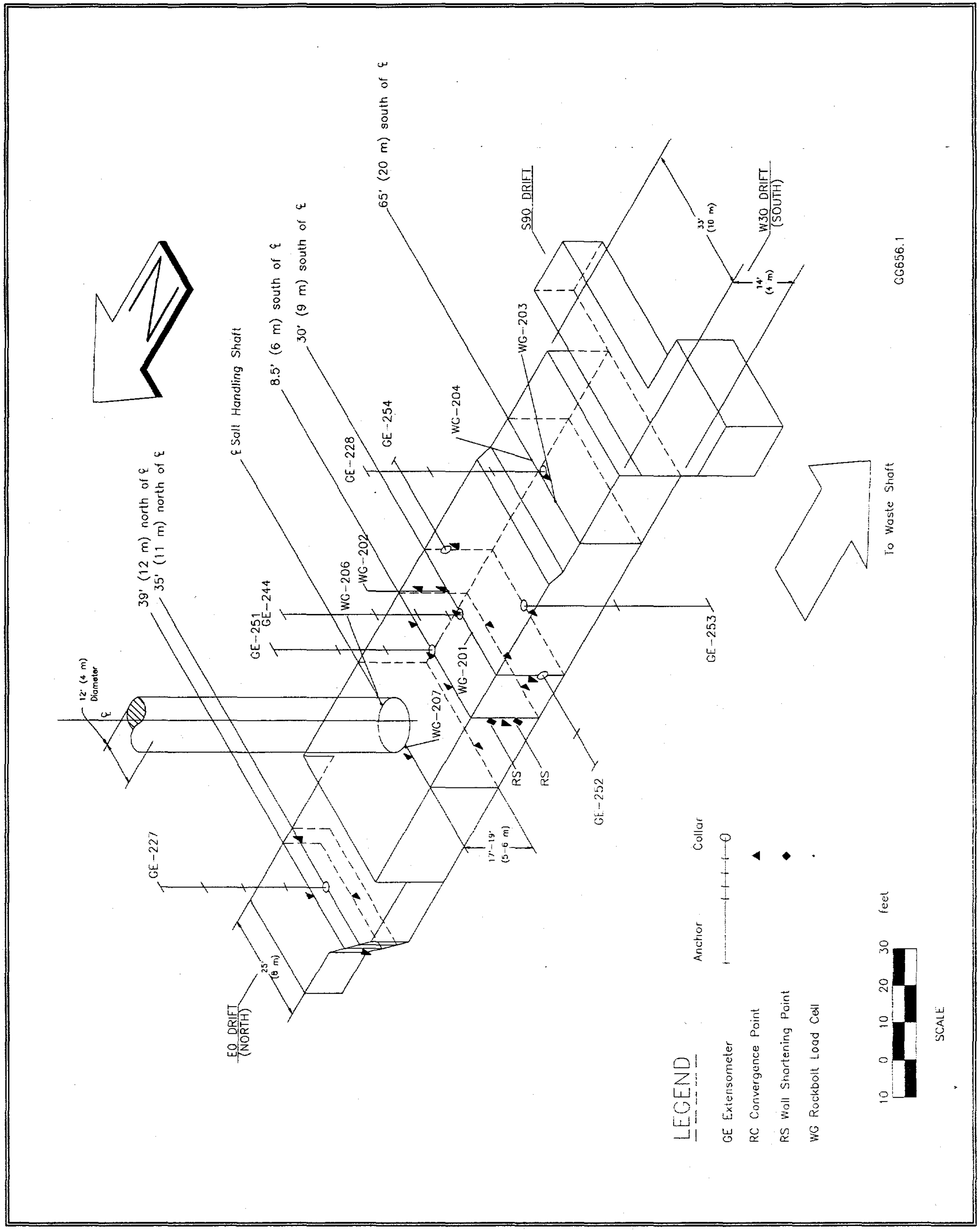


Figure 4-2
Salt Handling Shaft Station Instrumentation before Roof Excavation

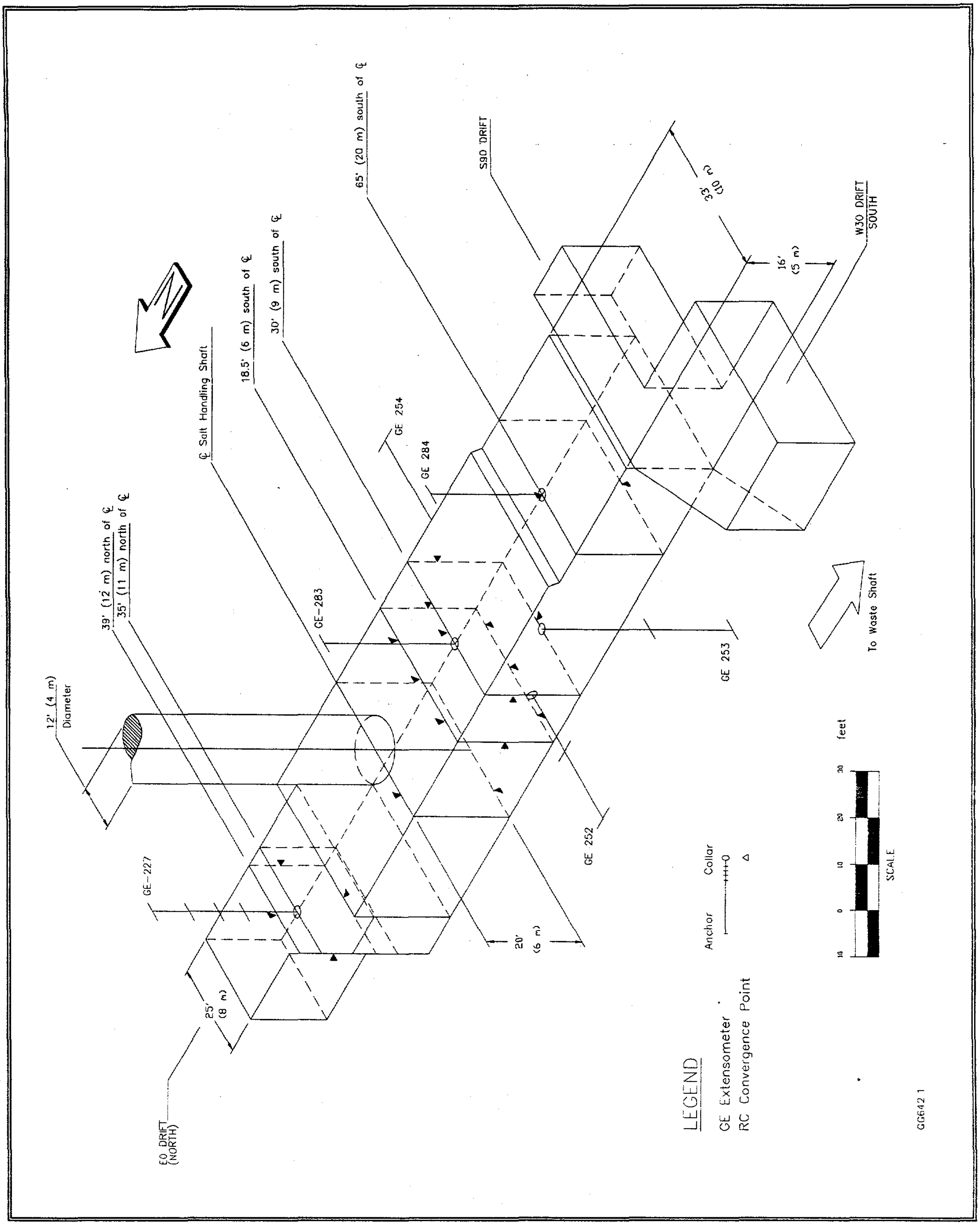


Figure 4-3
Salt Handling Shaft Station Instrumentation after Roof Excavation

4.2 Waste Shaft Station

The Waste Shaft Station was initially excavated with a continuous miner as a ventilation connection to a six foot (1.8 m) diameter exhaust shaft in November 1982. In 1984, the shaft station was enlarged to a height of 15 to 19 feet (4.5 to 6 m), and a width of 20 to 30 feet (6 to 9 m); the station is approximately 150 feet (46 m) long. In 1987, the station was trimmed, and concrete was placed on the floor. In February 1991, a portion of the concrete slab was removed. The portion of the concrete slab removed was approximately 53 feet (16 m) long, 23 feet (7 m) wide, and 18 inches (46 cm) thick. A generalized cross section of the station is shown in Figure 4-4.

4.2.1 Modifications to Excavation

During this reporting period no modifications were made to the Waste Shaft Station. However, routine maintenance was performed.

4.2.2 Instrumentation

Initial instrumentation was installed between November 12 and December 2, 1982. Figure 4-5 details the instrument locations in the Waste Shaft Station before enlargement of the station, and Figure 4-6 details the locations after enlargement.

Currently there are three functioning extensometers in the Waste Shaft Station, they are located in the roof at W30, E35, and E140. In addition, convergence points are monitored at E30, E90, and E140.

Six rockbolt load cells were installed in February 1988, after trimming in the station was completed. Monitoring of the six rockbolt load cells continues. Twelve rockbolt load cells were installed in the brow of the Waste Shaft in March 1992. The rockbolt load cells are used to monitor the load on the rockbolts installed in the brow as part of the cable support system and determine when detensioning of the cable anchors is required. During this reporting period, the cable-support ground control system was detensioned twice.

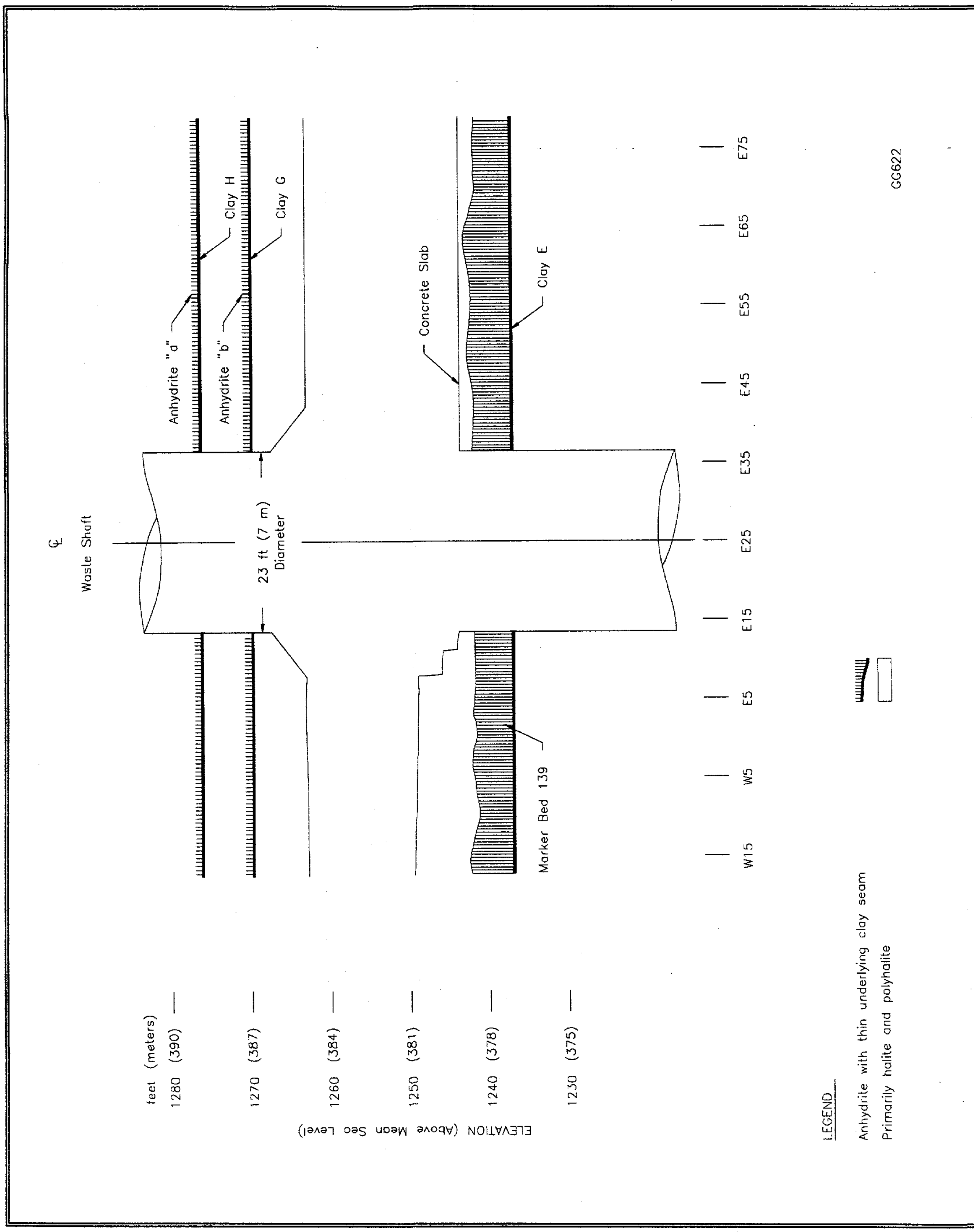


Figure 4-4
Generalized Waste Shaft Station Stratigraphy

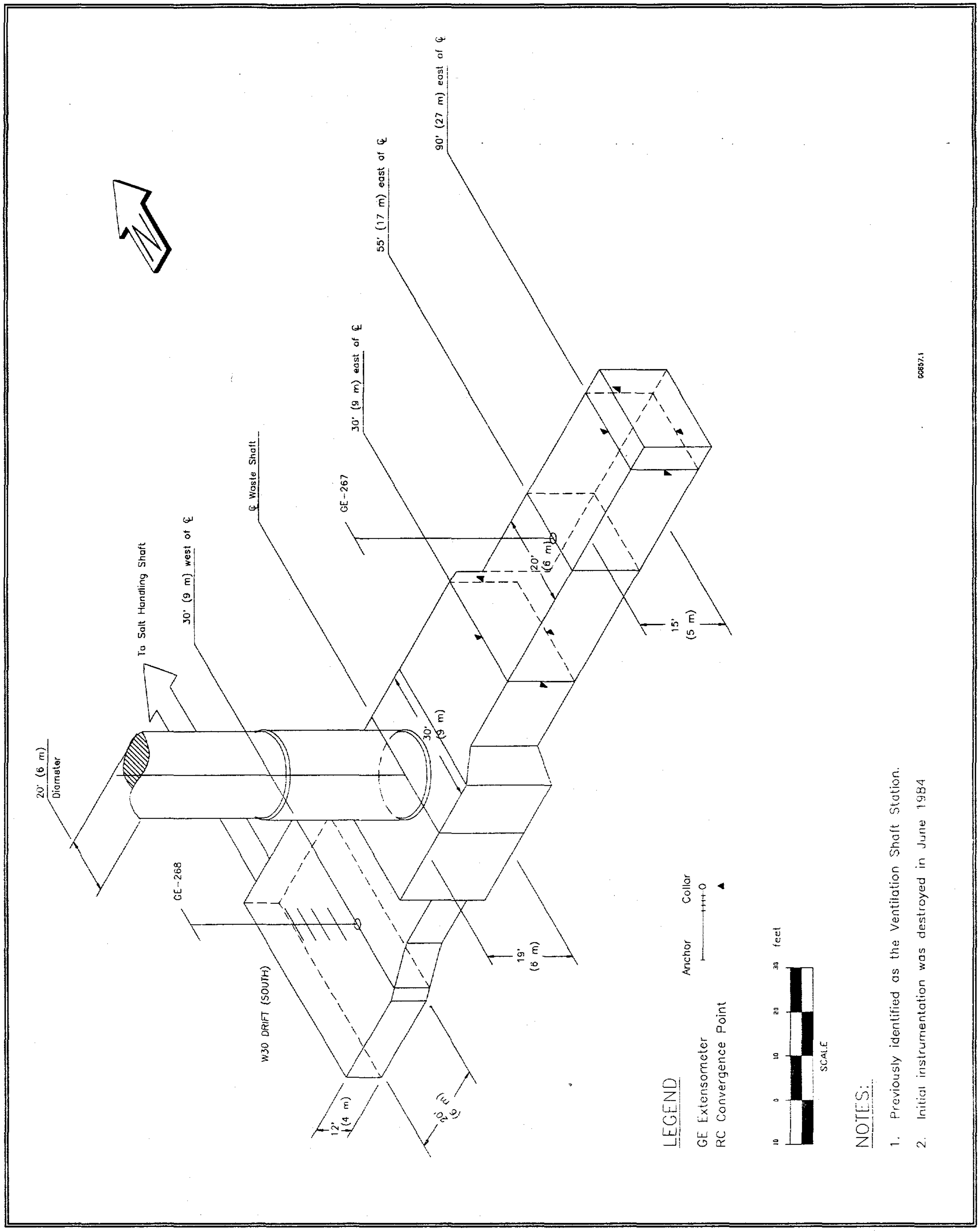


Figure 4-5
Waste Shaft Station Instrumentation before Trimming

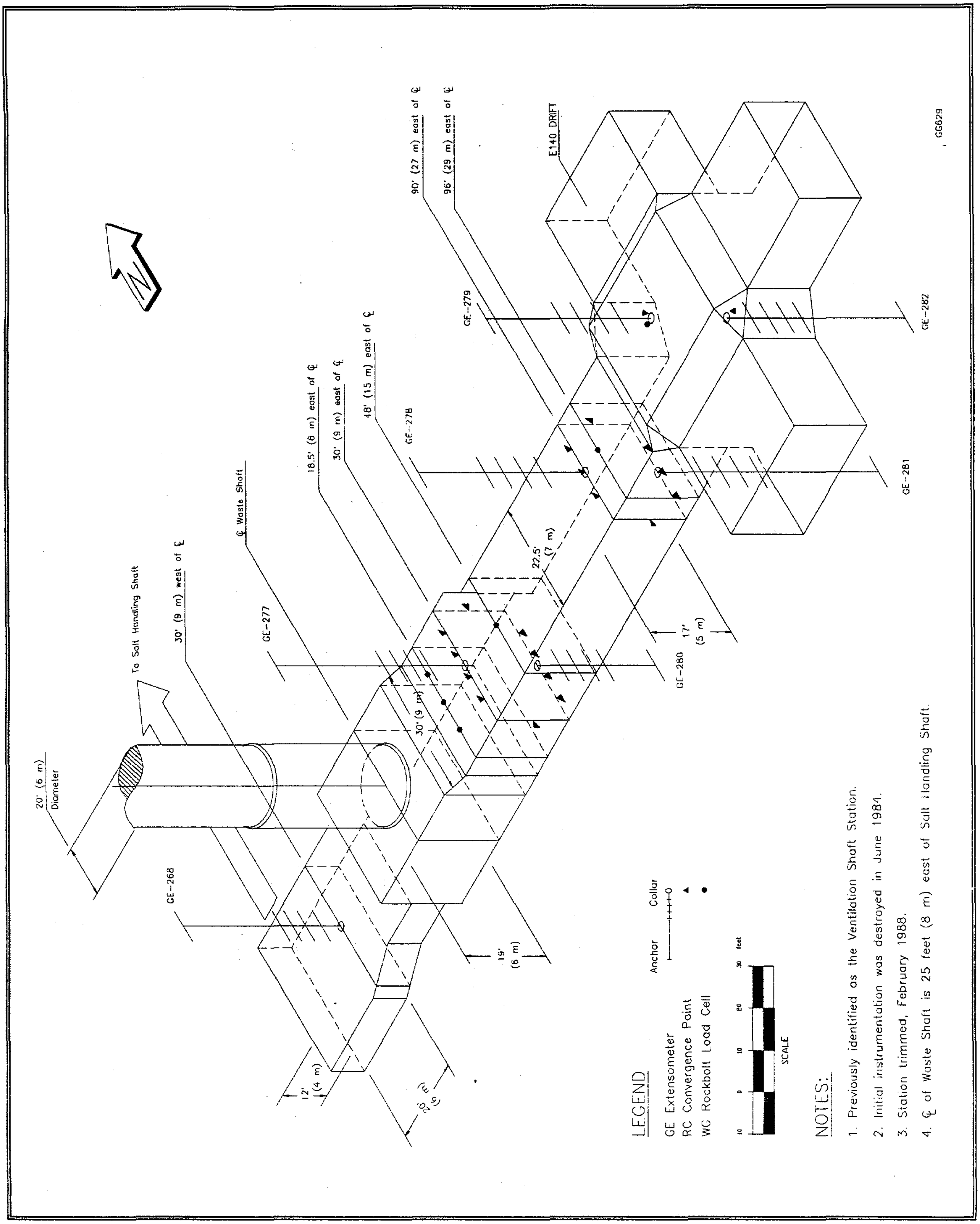


Figure 4-6
Waste Shaft Station Instrumentation after Trimming

5.0 Performance of Access Drifts

This section describes the in situ geomechanical performance of the underground access drifts. Both routine and special studies are conducted and documented to evaluate changes observed during this reporting period. The routine studies include the evaluation of convergence data to confirm that openings continue to behave in accordance with predictions. In addition, this section presents a comparison of the performance as defined by field data with the requirements that must be met by the design for the underground excavations (DOE, 1984).

5.1 Modifications/Maintenance

No major modifications were performed during this reporting period, however, strategic roof bolting was performed in locations throughout the E140 and N1420 drifts where ground conditions warranted. Localized roof bolting also occurred in other areas of the underground, such as Room D, SPDV Room 4 and the L-4 alcove.

In September 1993, trimming was performed on the north and south side of the E0 overcast in order to accommodate the cable support system installed over the E0 overcast.

5.2 Instrumentation

During this reporting period, new extensometers were installed in the roof of the E140 drift and the E300 maintenance shop.

Four new sonic probe extensometers were installed in the roof of the E140 drift at S1100, S1200, S1725, and S1825. Extensometers 51X-GE-00302 and 51X-GE-00303 were installed July 16, 1993, at S1200 and S1725. The deepest rod for each extensometer was positioned at 24 feet (7.3 m). Extensometer 51X-GE-00304 was installed at S1825 on July 23, 1993, with the deepest anchor at 23 feet (7 m). The fourth extensometer was installed at S1100 on October 14, 1993, with the deepest anchor at 24 feet (7.3 m). These extensometers are intended to monitor expansion of the immediate roof beam.

Monitoring of the four vibrating wire extensometers located in the E300 maintenance shop and four joint meters at the S1950 overcast continues. The joint meters monitor fracturing in the brows at the S1950 overcast.

During this reporting period, new convergence stations were installed at E140-S1400 and

S700-E58. Other installations included replacement of convergence points that had been destroyed by mine maintenance activities.

5.3 Analysis of Convergence Data

Measurements of room closure are evaluated as a primary means of identifying areas where conditions may be deteriorating. Closure measurements are made throughout the facility approximately every two months. They are used in conjunction with other observations to assess the geomechanical performance of the excavations. Points that significantly vary from a closure model are evaluated more closely to determine the cause of the variance.

Closure and closure rates are determined by using radial convergence points which measure the reduction in distance between opposing surfaces of the excavations. Radial convergence points are: 1, accurate; 2, easy to install and read; and 3, analyzed with simple engineering techniques. Closure rates indicate how an excavation is performing; rates that slow down with time generally indicate stable excavations, whereas increasing closure rates, or rates that are higher than anticipated, may indicate potential instability. Previously reported results (DOE, 1990b) indicate that closure rates generally decline with time and may also show cyclic variations that can be attributed to seasonal temperature changes.

Closure rates are routinely plotted against time. In addition, actual rates are compared to predicted values. The predictions are based on statistical evaluation of selected data from openings of various sizes and ages that provide an empirical relationship between closure rates, room dimensions, and the age of the excavation. The relationship is updated each year as additional data become available. This approach provides an equation for the closure rate as a transient function of time and opening dimensions (Table 5-1). Predicted values are assessed at an upper bound based on a 95% prediction limit from the statistical analysis. Parameters A, b, c, and d and the 95% prediction limit are determined using nonlinear regression techniques that are in the commercial software package Statgraphics (Version 4.0). A similar analysis was performed to obtain a predictive equation for closure rates at intersections (Table 5-2).

Measured closure rates from July 1, 1993, to June 30, 1994, from locations throughout the underground are compared to the predicted values (Tables 5-3 and 5-4). Actual closure rates are generally in agreement with predicted rates. At locations where predictions have been exceeded, additional investigations were conducted and explanations for the differences are provided in the table. If the cause of a high closure rate cannot be adequately explained (i.e., due to recent mining nearby, instrument malfunction, location in an intersection, etc.), the location is monitored

Table 5-1
Analyses of Underground Closure Rates
(From convergence data through June 30, 1994)

Closure	Parameter:				
	A	b	c	d	R^2
Roof to Floor	0.0026	1.0063	1.1896	-0.2170	0.875
Wall to Wall	0.0105	0.5810	1.0600	-0.2702	0.812

Notes:

1. Parameters A, b, c, d, are determined using nonlinear regression techniques by means of STATGRAPHICS (Version 4.0).

2. Parameters used in the following relationship:

$$C(t) = A * w^{b} * h^{c} * t^{d} * (365/200)$$

Where:
- C(t) = closure rate (inches/year)
- w = room width (feet)
- h = room height (feet)
- t = age of excavation (years)

3. R^2 is the coefficient of determination.

Table 5-2
Analyses of Underground Closure Rates at Intersections
(From convergence data through June 30, 1994)

Closure	Parameter:				
	A	b	c	d	R^2
Roof to Floor	0.0024	0.7327	0.7458	-0.2295	0.863

Notes:

1. Parameters A, b, c, d, are determined using nonlinear regression techniques by means of STATGRAPHICS (Version 4.0).

2. Parameters used in the following relationship:

$$C(t) = A * a^{b} * h^{c} * t^{d} * (365/200)$$

Where:
- C(t) = closure rate (inches/year)
- a = intersection area ($feet^2$)
- h = room height (feet)
- t = age of excavation (days)

3. R^2 is the coefficient of determination.

more frequently, and further field studies may be initiated. The results of these analyses indicate that the underground is generally performing within the bounds of the empirical analyses with the exception of sections in the E140 drift, the E0 drift, and Rooms 1, 4, and 6 of Panel 1.

These areas are currently monitored more frequently and (where required) supplemental roof support has been installed. In addition, physical inspections are performed on a regular basis.

It could be argued that many of the higher than predicted closure rates could be attributed to the nature of the empirical equation. That is, the analysis is more appropriate for the time period from approximately 300 days following excavation through the steady state phase -- the steady state phase being the approximate 2-8 year period following initial excavation, when closure rates remain fairly constant. This, obviously, does not necessarily apply in areas where physical observations, such as significant fracturing and rockbolt failure, may indicate increasing instability.

The primary purpose of these analyses is to locate areas where closure rates are statistically higher than expected, and not to serve as a specific predictor of excavation performance beyond the period of steady state. As the excavations continue to age, however, consideration of excavation performance beyond the period of steady state will ultimately be required in the analyses.

Table 5-3
Mid-Room Roof to Floor Closure Rates

Fieldtag	Location	Date of Excavation	Closure Rate 1988-89 (in/yr)	Closure Rate 1989-90 (in/yr)	Closure Rate 1990-91 (in/yr)	Closure Rate 1991-92 (in/yr)	Closure Rate 1992-1993 (in/yr)	Closure Rate 1993-1994 (in/yr)	Upper Bound of Predicted Closure 1993-1994 (in/yr)	Comments
W170-S1150 RC:A-E	W170 drift-S1150	21-Aug-84	1.03	0.79	0.79	0.78	0.64	0.73	1.23	
W170-S1000 RC:A-C	W170 drift-S1000 intersection	19-Aug-84	1.24	0.88	0.81	0.80	0.64	0.73	1.22	
W170-S700 RC:A-C	W170-S700 drifts intersection	14-Aug-84	1.03	0.74	0.72	0.71	0.63	0.65	1.22	
W170-S5 RC:A-C	W170 drift-S5	28-Jul-87	0.87	0.58	0.64	0.62	0.50	0.56	1.23	
W30-S120 RC:A-C	W30 drift-S120	29-Jul-87	1.23	0.97	0.92	0.91	0.96	0.92	1.45	
W30-S1000 RC-3:A-C	W30-S1000 drifts intersection	21-Aug-84	2.66	1.81	1.30	1.28	1.09	0.46	1.25	
W30-S850 RC-2:A-E	W30 drift-S850	15-Aug-84	1.06	0.61	0.78	0.68	0.67	0.66	1.26	
W30-S700 RC:A-C	W30-S700 drifts intersection	08-Aug-84	1.98	0.74	1.08	1.04	0.92	0.92	1.78	
W30-S500 RC:A-C	W30 drift-S500	08-Aug-84	1.29	0.78	0.89	0.83	0.73	0.76	1.57	
W30-S250 RC-3:A-C	W30 drift-S250	28-Sep-84	1.32	0.82	0.87	0.96	1.01	1.00	1.57	
E0-N626 RC-3:A-C	E0 drift-N626	20-Feb-85	1.61	2.81	1.96	1.98	2.02	1.98	1.71	High rate due to floor heave.
E0-N940 RC-3:A-C	E0 drift-N940	22-Feb-85	1.75	2.53	1.95	2.03	1.88	2.03	1.37	High rate due to floor heave.
E0-N1110 RC-3:A-C	E0-N1100 drifts intersection	04-Mar-85	1.60	1.51	1.40	1.38	1.24	1.36	1.54	
E0-N1266 RC-3:A-C	E0 drift-N1266	16-Jan-85	1.66	1.82	1.80	1.79	1.70	1.81	1.75	High rate due to floor heave.
E140-S460 RC-3:A-C	E140 drift-S460	15-Jun-84	1.56	2.02	2.07	2.02	1.99	2.01	2.49	
E140-S550 RC-3:A-C	E140 drift-S550	01-Mar-88	2.01	1.62	1.61	1.53	1.39	1.41	2.21	
E140-S850 RC-5:A-C	E140 drift-S850	07-Jun-84	1.50	1.78	1.69	1.60	1.45	2.30	2.33	
E140-S1000 RC:A-C	E140-S1000 drifts intersection	01-Mar-88	1.89	1.62	1.36	1.35	1.20	1.27	2.01	
E140-S1150 RC:A-C	E140 drift-S1150	14-Jun-84	1.62							
E140-S1150 RC-2:A-C	E140 drift-S1150	28-Dec-87	2.55	2.20	2.06	2.10	2.24	2.04	2.30	
E140-S1246 RC-2:A-C	E140 drift-S1246	11-Jun-84								
E140-S1246 RC-4:A-C	E140 drift-S1246	01-Mar-88	2.86	2.37	2.01	1.84	1.63	1.81	2.20	
E140-S1300 RC-2:A-C	E140-S1300 drifts intersection	01-Mar-88	2.29	1.83	1.55	1.49	1.52	1.19	1.49	
E140-S1450 RC-4:A-E	E140 drift-S1450	18-Jun-84		2.72	2.61	2.33	2.07	2.30	2.21	Fracturing in immediate roof beam.
E140-S1600 RC-3:A-C	E140-S1600 drifts intersection	01-Mar-88		1.97	1.71	1.66	1.53	1.51	1.65	
E140-S1879 RC-2:A-C	E140 drift-S1879	06-Jan-88	3.16	2.69	2.79	2.92	2.19	2.22	2.22	
E140-S1950 RC-3:A-C	E140-S1950 drifts intersection	01-Mar-88		2.08	1.72	1.66	2.10	1.37	1.59	

Table 5-3
Mid-Room Roof to Floor Closure Rates (continued)

Fieldtag	Location	Date of Excavation	Closure Rate 1988-89 (in/yr)	Closure Rate 1989-90 (in/yr)	Closure Rate 1990-91 (in/yr)	Closure Rate 1991-92 (in/yr)	Closure Rate 1992-93 (in/yr)	Closure Rate 1993-94 (in/yr)	Upper Bound of Predicted Closure 1993-1994 (in/yr)	Comments
E140-N5 RC-3:A-C	E140 drift-N5	24-May-84	1.33	1.00	1.11	1.28	1.37	1.27	1.89	
E140-N250 RC:A-C	E140 drift-N250	22-Feb-83						2.36	2.88	
E140-N626 RC-2:A-C	E140 drift-N626	24-Feb-83	0.92	0.20		3.20	2.63	2.76	2.19	High rate due to fracturing along west rib.
E140-N1266 RC-2:A-C	E140 drift-N1266	28-Feb-83	0.97	0.67	3.10	2.71	2.39	2.46	2.29	High rate due to floor heave.
E300-S1150 RC-3:A-E	E300 drift-S1150	26-Jul-84	0.80	0.66	0.60	0.56	0.46	0.51	1.35	
E300-S1000 RC:A-C	E300 drift-S1000	25-Jul-84	0.89	0.62	0.64	0.60	0.50	0.55	1.22	
E300-S850 RC:A-E	E300 drift-S850	25-Jul-84	0.74	0.44	0.52	0.49	0.39	0.45	1.30	
E300-S700 RC:A-C	E300-S700 drifts intersection	24-Jul-84	0.92	0.65	0.69	0.66	0.55	0.63	1.24	
S90-W100 RC:A-C	S90 drift-W100	28-Jul-87	0.85	0.54	0.58	0.56	0.45	0.50	1.30	
S90-W400 RC:A-C	S90 drift-W400	03-Dec-87	1.35	1.02	0.76	0.64	0.53	0.62	1.43	
N140-E90 RC:A-C	N150 drift-E90	29-Jul-87	0.77	0.56	0.68	0.76	0.79	0.70	1.22	
N1110-E80 RC-3:A-C	N1100 drift-E80	12-Jan-84	0.62	1.06	0.96	0.90	0.89	0.91	1.11	
TR2-N1110 RC-2:A-C	N1100 drift-Test Rm. 2 Intersection	19-Apr-84	1.81	1.61	1.43	1.29	1.16	1.37	1.79	
N1110-W783 RC-2:A-C	N1100 drift-W783	04-Feb-86	0.84	0.86	0.78	0.74	0.66	0.72	1.24	
N1110-W951 RC-2:A-C	N1100 drift-W951	04-Feb-86	0.71	0.74	0.62	0.61	0.57	0.63	1.25	
N1110-W1159 RC-2:A-C	N1100drift-W1159	04-Feb-86	0.83	0.85	0.73	0.71	0.62	0.71	1.21	
N1110-W1347 RC-2:A-C	N1100 drift-W1347	04-Feb-86	0.79	0.84	0.74	0.80	0.64	0.71	1.27	
N1420-W258 RC:A-E	N1420 drift-W258	07-Apr-84	1.17	1.29	1.19	1.12	1.09	1.15	2.11	
N1420-W365 RC-3-C	N1420 drift-Test Rm. 2 intersection	07-Apr-84	2.37	2.51	2.30	2.18	2.61	2.22	2.03	High rate due to floor heave.
N1420-W391 RC:A-E	N1420 drift-W391	07-Apr-84	1.31	1.38	1.24	1.17	1.13	1.11	2.10	
N1455-W365 RC:A-E	Room L2	30-Apr-84	1.79	1.91	1.69	1.63	1.48	1.63	2.18	
E520-S1802 RC-6:A-E	Panel 1 Room 1	30-May-86	4.44	3.47	3.82	3.21	2.76	2.93	2.65	High rate due to floor heave along east rib.
E660-S1775 RC-5:A-C	Panel 1 Room 2	23-Jan-87	4.64	2.95	2.76	2.48	2.17	2.26	2.64	
E790-S1775 RC-3:A-C	Panel 1 Room 3	19-Feb-87	4.68	3.18	2.56	2.42	2.22	2.41	2.57	
E920-S1775 RC-5:A-F	Panel 1 Room 4	03-Feb-88	4.54	3.89	2.45	2.35	2.32	2.69	2.60	High rate due to floor heave.
E1050-S1775 RC-4:A-F	Panel 1 Room 5	12-Feb-88	4.00	2.60	2.17	2.12	2.07	2.42	2.53	
E1190-S1775 RC-4:A-F	Panel 1 Room 6	26-Feb-88	4.74	3.25	2.61	2.48	2.41	2.70	2.52	High rate due to floor heave.
E1320-S1775 RC-3A-F	Panel 1 Room 7	10-Mar-88	4.53	3.12	2.54	2.48	2.31	2.57	2.51	Fracturing in immediate roof along east rib.

Note: Fieldtag represents the most recent installation.

Table 5-4
Mid-Room Wall to Wall Closure Rates

Fieldtag	Location	Date of Excavation	Closure Rate 1988-89 (in/yr)	Closure Rate 1989-90 (in/yr)	Closure Rate 1990-91 (in/yr)	Closure Rate 1991-92 (in/yr)	Closure Rate 1992-93 (In/yr)	Closure Rate 1993-94 (In/yr)	Upper Bound of Predicted Closure 1993-1994 (in/yr)	Comments
W170-S1150 RC:C-G	W170 drift-S1150	21-Aug-84	1.11	0.81	0.87	0.79	0.70	0.75	1.14	
W170-S5 RC:B-D	W170 drift-S5	28-Jul-87	0.96	0.66	0.72	0.65	0.56	0.65	1.15	
W30-S850 RC:C-G	W30 drift-S850	15-Aug-84	1.07	0.73	0.83	0.86	0.72	0.73	1.15	
W30-S500 RC:B-D	W30 drift-S500	08-Aug-84	1.29	0.82	0.88	0.78	0.73	0.76	1.30	
W30-S250 RC-5:B-D	W30 drift-S250	28-Sep-84	1.22			1.00	0.76	0.77	1.31	
E0-N1266 RC-4:B-D	E0 drift-N1266	16-Jan-85	1.14	1.07	1.16		1.00	1.09	1.38	
E140-S460 RC-2:B-D	E140 drift-S460	09-Mar-88	1.56	1.22	1.22	1.20	1.14	1.15	1.76	
E140-S550 RC-4:B-D	E140 drift-S550	16-Mar-88	1.60	1.28	1.25	1.17	1.03	1.09	1.58	
E140-S850 RC-3:B-D	E140 drift-S850	07-Jun-84	1.50	1.08	1.02	0.94	0.83	0.86	1.66	
E140-S1150 RC-2:B-D	E140 drift-S1150	14-Jun-84	0.62	1.27	1.14	1.07	1.06	1.12	1.68	
E140-S1246 RC-3:B-D	E140 drift-S1246	01-Mar-88	1.25	1.13	1.54	1.06	0.95	1.00	1.58	
E140-S1450 RC-2:B-D	E140 drift-S1450	18-Jun-84	1.77	0.97	0.96	0.90	0.84	0.85	2.30	
E140-S1879 RC-2:B-D	E140 drift-S1879	06-Jan-88	1.44	1.30	1.60	1.29	1.14	1.13	1.64	
E140-N5 RC-2:B-D	E140 drift-N5	24-May-84	1.03	0.72	0.85	1.00	1.13	1.02	1.38	
E140-N626 RC-2:B-D	E140 drift-N626	24-Feb-83	0.81	0.09		2.14	1.64	1.64	1.60	Rate higher than predicted.
E140-N1266 RC-3:B-D	E140 drift-N1266	28-Feb-83	0.67	0.73	1.87	1.47		1.33	1.70	
E300-S1150 RC:C-G	E300 drift-S1150	26-Jul-84	0.89	0.73	0.71	0.68	0.54	0.63	1.15	
E300-S850 RC:C-G	E300 drift-S850	25-Jul-84	0.82	0.54	0.60	0.58	0.42	0.52	1.16	
S90-W100 RC:B-D	S90 drift-W100	28-Jul-87	0.55	0.58	1.11	0.55	0.47	0.53	1.17	
S90-W400 RC:B-D	S90 drift-W400	03-Dec-87	1.32	1.03	0.85	0.63	0.50	0.58	1.34	
N140-E90 RC:B-D	N150 drift-E90	29-Jul-87	0.55	0.68	1.08	0.79	0.85	0.77	1.15	
N140-W50 RC:B-D	N150 drift-W50	31-Jul-87	1.06	1.01	1.59	1.03	0.83	0.88	1.60	
N1110-E80 RC-4:B-D	N1100 drift-E80	12-Jan-84	0.60	1.06	0.95	0.81		0.92	1.12	
N1110-W783 RC-2:B-D	N1100 drift-W783	04-Feb-86	0.58	0.67	0.58	0.55	0.47	0.55	1.10	
N1110-W951 RC-2:B-D	N1100 drift-W951	04-Feb-86	0.53	0.58	0.49	0.47	0.41	0.49	1.10	
N1110-W1159 RC-2:B-D	N1100 drift-W1159	04-Feb-86	0.60	0.61	0.52	0.50	0.43	0.54	1.11	
N1110-W1347 RC-2:B-D	N1100 drift-W1347	04-Feb-86	0.58	0.63	0.54	0.51	0.46	0.39	1.10	
N1420-W258 RC-4:C-G	N1420 drift-W258	07-Apr-84	1.17	1.45	1.37	0.69	0.86	0.89	1.94	
N1420-W391 RC-3:C-G	N1420 drift-W391	07-Apr-84	1.55		1.32	0.76	0.95	1.00	1.34	
N1455-W365 RC-3:C-G	Room L2	30-Apr-84	1.28	1.41	1.27	1.22	1.17	1.19	1.68	
E520-S1802 RC-3:C-G	Panel 1 Room 1,	30-May-86	2.56	1.89	1.78	1.46	1.26	1.35	1.74	
E660-S1775 RC-5:B-D	Panel 1 Room 2	23-Jan-87	2.94	1.93	1.55	1.51	1.14	1.25	1.79	
E790-S1775 RC-5:B-D	Panel 1 Room 3	19-Feb-87	3.27	2.13	1.65		1.35	1.48	1.79	
E920-S1775 RC-4:D-G	Panel 1 Room 4	03-Feb-88	3.32	1.99	1.57	1.50	1.29	1.47	1.83	
E1050-S1775 RC-4:D-G	Panel 1 Room 5	12-Feb-88	3.09	1.93	1.50	1.37	1.28	1.50	1.84	
E1190-S1775 RC-3:D-G	Panel 1 Room 6	26-Feb-88	2.88	1.97	1.55	1.35	1.17	1.31	1.74	
E1320-S1775 RC-5:D-G	Panel 1 Room 7	10-Mar-88	2.94	1.87	1.43	1.39	1.16	1.24	1.85	

Note: Fieldtag represents most recent installation.

6.0 Performance of SPDV Rooms and Northeast Experimental Area

The Site and Preliminary Design Validation (SPDV) Rooms, located in the northwest section of the underground facility, were excavated as a four-room panel in early 1983 (Figure 1-2). The rooms were equipped with geomechanical instrumentation almost immediately after excavation. To validate the disposal room design, the SPDV Rooms were mined to the same specifications as the disposal rooms. The dimensions are 13 feet (4 m) high, 33 feet (10 m) wide, and 300 feet (90 m) long, with 100-foot (30-meter) pillars.

The design criteria for the disposal rooms required total vertical and horizontal closure to remain below certain limits. However, closure in the SPDV Rooms was found to be higher than the anticipated values based on laboratory test results.

After design validation was considered complete, geomechanical monitoring of the SPDV Rooms continued primarily to observe the long-term behavior of the disposal rooms. While the behavior of the SPDV Rooms is relevant to the disposal rooms, it must be remembered that variability is inherent in natural materials such as salt. Although the SPDV rooms have behaved differently, they still provide a good analogue to the waste disposal rooms.

6.1 Modifications to Excavations

No modifications were made during this reporting period; however, routine maintenance was performed on the ribs to remove loose rock.

6.2 Instrumentation

Geotechnical instrumentation was installed in the SPDV Rooms in March and April 1983. Instrument locations for SPDV Room 4 are shown in Figure 6-1.

Currently, three extensometers have remained operational at the midpoint of SPDV Room 4 since April 1983. Extensometer 51X-GE-00206, located in the east rib, has a total displacement of 8.3 inches (21 cm). Extensometer 51X-GE-00207, located in the roof, has a total displacement of 11.3 inches (29 cm). Extensometer 51X-GE-00208, located in the west rib, has a total displacement of 6.5 inches (16.5 cm).

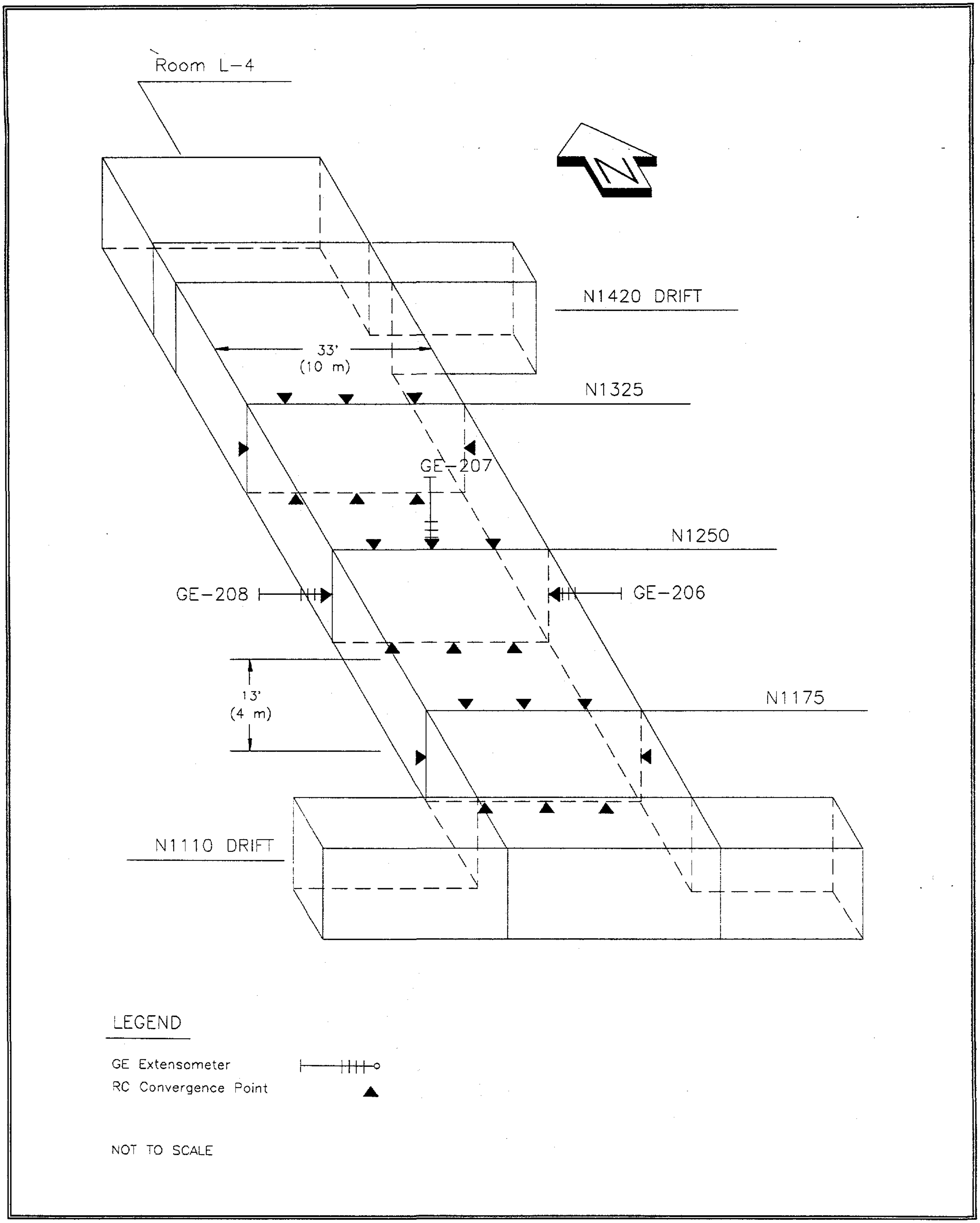


Figure 6-1
SPDV Room 4 Instrumentation

The convergence points located in SPDV Rooms 1, 2, and 3 remain inaccessible. All convergence points in SPDV Room 4 continue to be read regularly. Arrays of convergence points in SPDV Room 4 are located at the north end of the room, at the midpoint, and at the south end of the room. The arrays located at the north and south ends of the room were installed in April 1989. The vertical closure rates at the room centerline for the north and south end of the room are 2.2 in/yr (5.6 cm/yr) and 2.0 in/yr (5.1 cm/yr), respectively. The vertical closure rate at midpoint of the room, centerline is 2.7 in/yr (6.9 cm/yr). The midpoint array was installed in May 1983 and has been read continuously since installation.

Due to the roof fall in SPDV Room 2 on June 12, 1994, eleven out of twelve wire convergence meters located in the room were destroyed. Currently, one wire convergence meter located at the north end of the room remains functional.

6.3 Excavation Performance

Only SPDV Room 4 is currently open and in use. Gradual deterioration in ground conditions and roof falls caused the other rooms to be barricaded. The rooms were barricaded rather than remedied so that the long term performance of unsupported, unmaintained rooms could be studied. Allowing one or two excavations to safely deform to the point of roof failure gives ground control engineers a reference against which conditions in other drifts can be compared. It should be noted, however, that all the rooms can be restored to safe operating conditions, if desired, using common mining practices. The status of the SPDV rooms are summarized below:

SPDV 1: Closed to all but geotechnical monitoring activities in May 1989. Closed to all activities in November 1989. A substantial roof fall occurred in February 1991. This event was predicted with increasing confidence since October 1989.

SPDV 2: Closed in November 1989. Monitored since then only with remote instrumentation. A substantial roof fall occurred in June 1994. The roof fall was predicted in October 1993.

SPDV 3: Closed in June 1990. Due to inaccessibility and the absence of remotely read instruments in the room, no roof fall estimate is available.

SPDV 4: Open and in use. Receives routine maintenance. Current life expectancy is long, but indefinite.

6.3.1 SPDV Room 2 Stability and Roof Fall Prediction

On June 12, 1994, at approximately 11:15 PM, six years after the room was barricaded, a roof fall occurred as predicted. The size, shape, timing and manner of collapse of the roof are consistent with predictions made in The Current Bases for Roof Fall Prediction at WIPP and a Preliminary Prediction for SPDV Room 2 (DOE, 1993a). Figures 6-2 through 6-4 show the convergence data up to the time of the roof fall. Figure 6-5 compares the convergence data in SPDV Rooms 1 and 2.

By visual inspection, the slab appears to be approximately 200 feet (61 m) long, 30 feet (9.1 m) wide. The cross-section appears to be roughly triangular and seven to eight feet (2.1 to 2.4 m) thick at the apex. These dimensions indicate that the weight of the fallen rock is about 1400 to 1600 tons. Examination of the video tapes of the roof fall shows that the slab fell almost instantly in one large block with no warning. The fall appears to encompass the entire roof of the room from about 37.5 feet (11.4 m) north of the N1100 drift to between 37.5 feet (11.4 m) and 75 feet (22.9 m) south of the N1420 drift.

Beginning on the evening of June 10, 1994, Geotechnical Engineering staff had continuously monitored the SPDV Room 2 datalogger. Convergence rates increased rapidly and steadily from about 70 in/yr (177 cm/yr) on the morning of June 10th to about 100 in/yr (254 cm/yr) on the morning of the 11th and then to about 200 in/yr (508 cm/yr) on the morning of the 12th. The last reading before the roof fell was 1098 $\pm$ 28 in/yr (2789 cm/yr $\pm$ 71 cm/yr).

An alarm system attached to the SPDV Room 2 datalogger performed as intended and automatically notified the Central Monitoring Room operator that the fall had occurred. The CMR operator then notified the appropriate Engineering, Operations, and Safety personnel who immediately reported to the site. Operations performed an underground re-entry checkout, the video tapes were retrieved from the VCRs, and the room was visually examined through the bulkhead at N1100 drift. The video recording system provided two different views of the roof fall.

In the months preceding the SPDV Room 2 roof fall, an empirical curve fitting analysis was used to estimate the date of the fall. The primary assumption of this analysis was that the deterioration of the slab in SPDV Room 2 would be similar to that of SPDV Room 1, especially in the final months preceding the roof fall. Data from each of the four convergence meters located in SPDV Room 1 were fit to the equation:

$$\log(t) = \begin{cases} m_1\dot{C} + b_1 & \text{for} \quad \dot{C} > \dot{C}_c \\ m_2\dot{C} + b_2 & \text{for} \quad \dot{C} < \dot{C}_c \end{cases}$$

where t is the number of days between the time of the reading and the roof fall, $\dot{C}$ is the convergence rate in inches per year, and m, b, and $\dot{C}_c$ are curve fitting parameters. The curve fitting results for each of the SPDV Room 1 convergence meters are presented in Table 6-1.

Table 6-1
SPDV Room 1 Curve Fitting Results

Instrument	m_1	b_1	m_2	b_2	$\dot{C}_c$
51X CW-001	-0.01224	1.484	-0.2238	3.631	10.15
51X CW-002	-0.01475	1.638	-0.2050	3.639	10.52
51X CW-003	-0.02044	1.488	-0.3392	4.174	8.43
51X CW-004	N/A	N/A	-0.2958	3.599	N/A

There are no values for m_1, b_1, and $\dot{C}_c$ for 51X CW-004, because the closure rate for that instrument did not accelerate until shortly before the roof fell. Table 6-2 and Figure 6-6 summarize the predictions that have been made using the curve fitting analysis.

Table 6-2
Historical Summary of SPDV Room 2 Roof Fall Predictions

Publication/Letter	Date Prediction Was Made	Predicted Range of Dates of Roof Fall	Most Probable Predicted Date of Roof Fall
DOE/WIPP 93-033	4/93	10/93 - 4/95	4/94
Addendum 1	10/06/93	10/93 - 4/95	2/09/94
Addendum 2	11/16/93	12/13/93 - 04/05/94	2/24/94
Addendum 3	01/11/94	01/26/94 - 04/18/94	3/19/94
Addendum 4	02/14/94	02/22/94 - 04/16/94	3/26/94
Addendum 5	03/10/94	03/16/94 - 05/01/94	4/13/94
Addendum 6	04/04/94	04/09/94 - 05/17/94	5/01/94
Addendum 7	05/02/94	05/18/94 - 05/29/94	5/22/94
Addendum 8	05/23/94	06/03/94 - 06/13/94	6/08/94
Addendum 9	06/06/94	N/A *	N/A *

* The roof fall was considered imminent at this time.

6.3.2 SPDV Room 3

SPDV Room 3 was barricaded in June of 1990, and ten-foot (3-meter) rockbolts were installed in the north half of the room in November 1990. Fly ash block walls were installed at both ends of the room in 1993, allowing only limited visual inspection. Installed instruments have not functioned for several years; there is no remote geotechnical monitoring of this room. At the time the room was barricaded, the condition of the roof was comparable to that of SPDV Room 2.

6.3.3 SPDV Room 4

SPDV Room 4 is the only SPDV Room still accessible. It is primarily used as a warehouse and for access to the N1420 drift. The roof of the room is pattern bolted with ten-foot (3-meter) mechanical rockbolts. Geomechanical conditions in SPDV Room 4 have always been better than in the other SPDV Rooms. Convergence and borehole extensometer data are collected from SPDV Room 4 and are supplemented with visual observations. Convergence rates are relatively low, less than three inches per year (7.6 cm/yr). Roof displacement rates measured by the extensometers are stable. Roof fracturing in the north half of the room is beginning to form a wedge-shaped pattern. The convergence at the north array is slightly asymmetric across the width of the room, with the east side moving about twenty percent faster than the west side. These data are precursors indicative of roof instability or floor lift, but their magnitude is minimal and not cause for concern at this time.

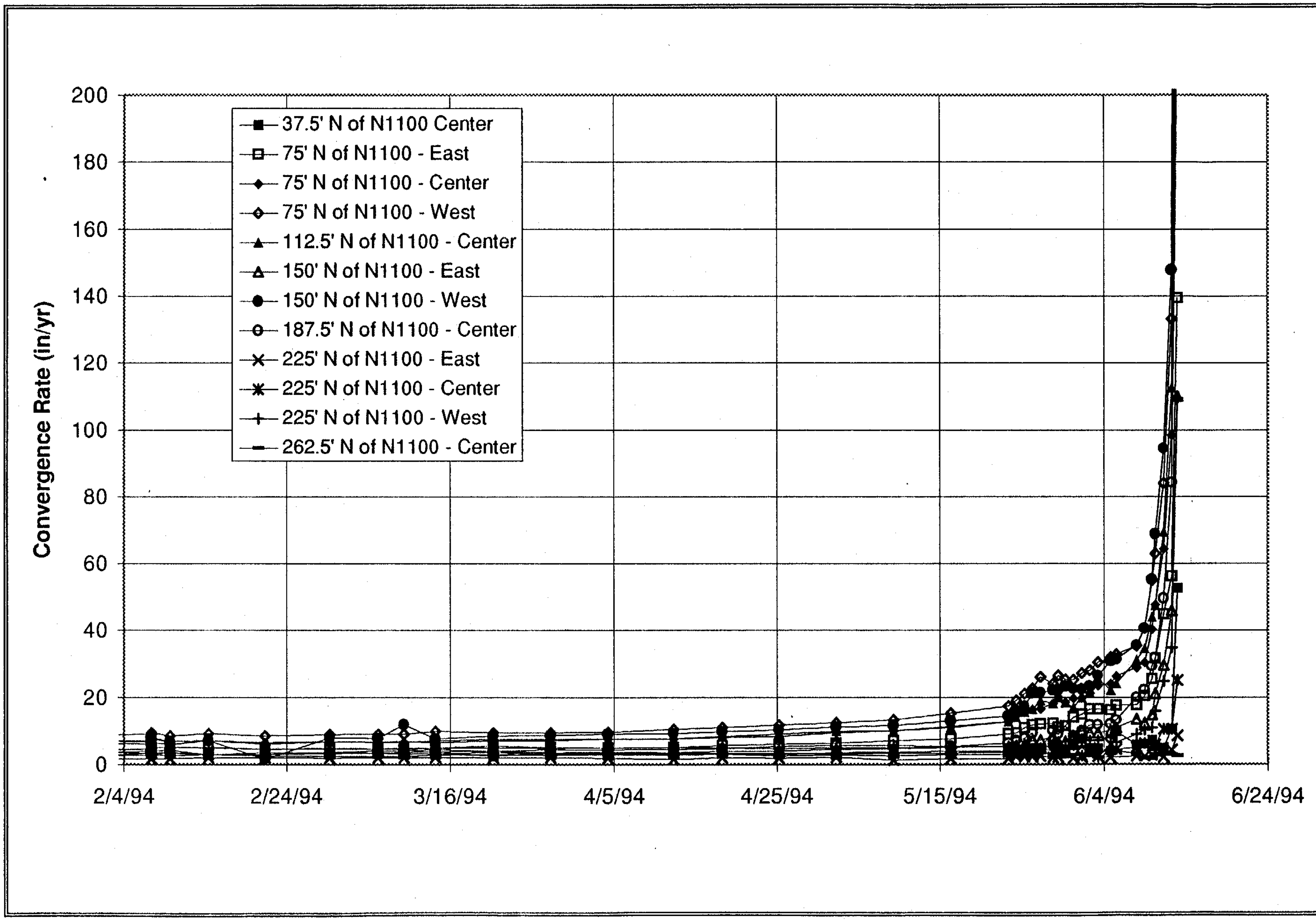


Figure 6-2
SPDV Room 2 Convergence Rates (for 2/4/94 through 6/12/94)

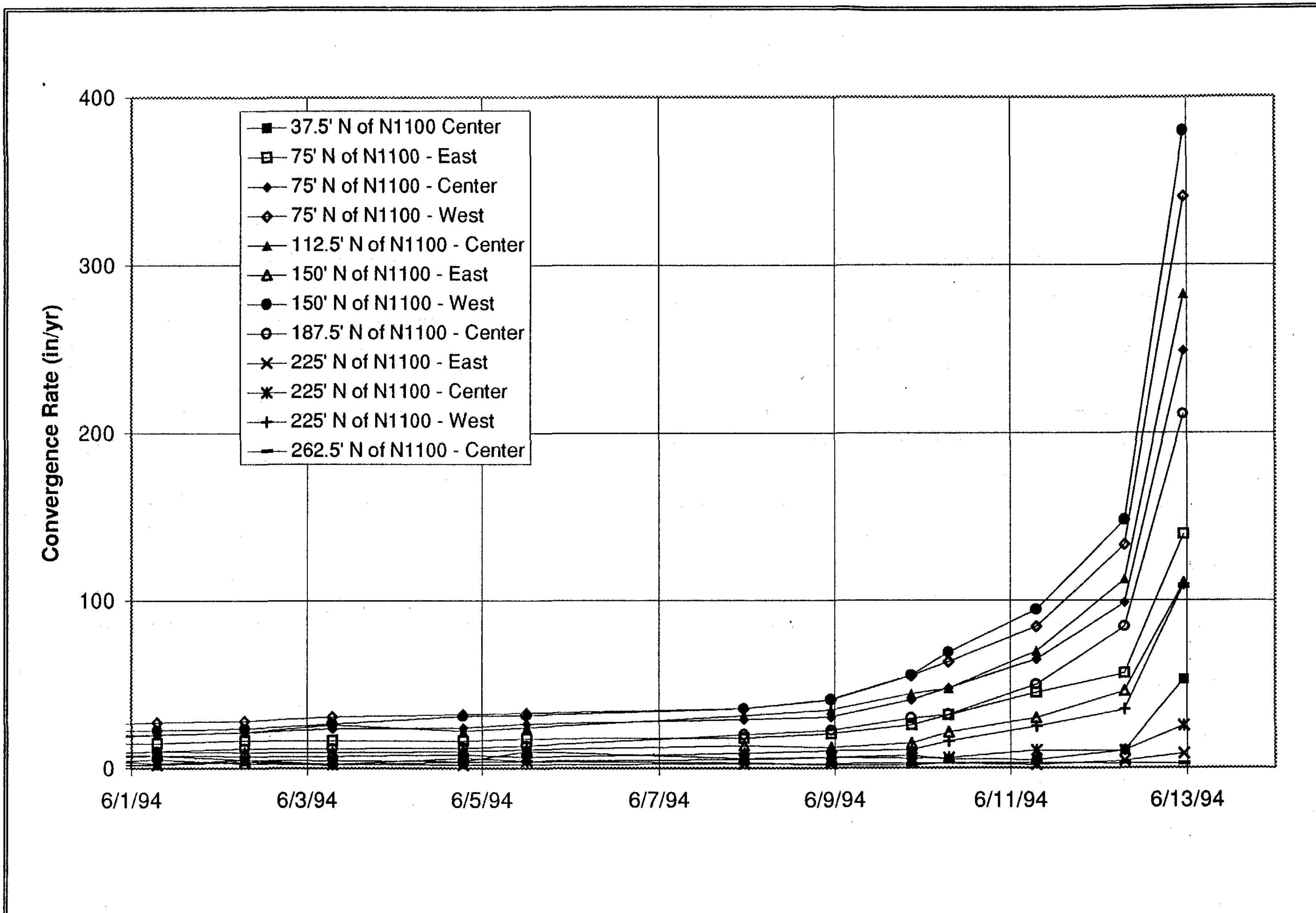


Figure 6-3
SPDV Room 2 Convergence Rates (for 6/1/94 through 6/12/94)

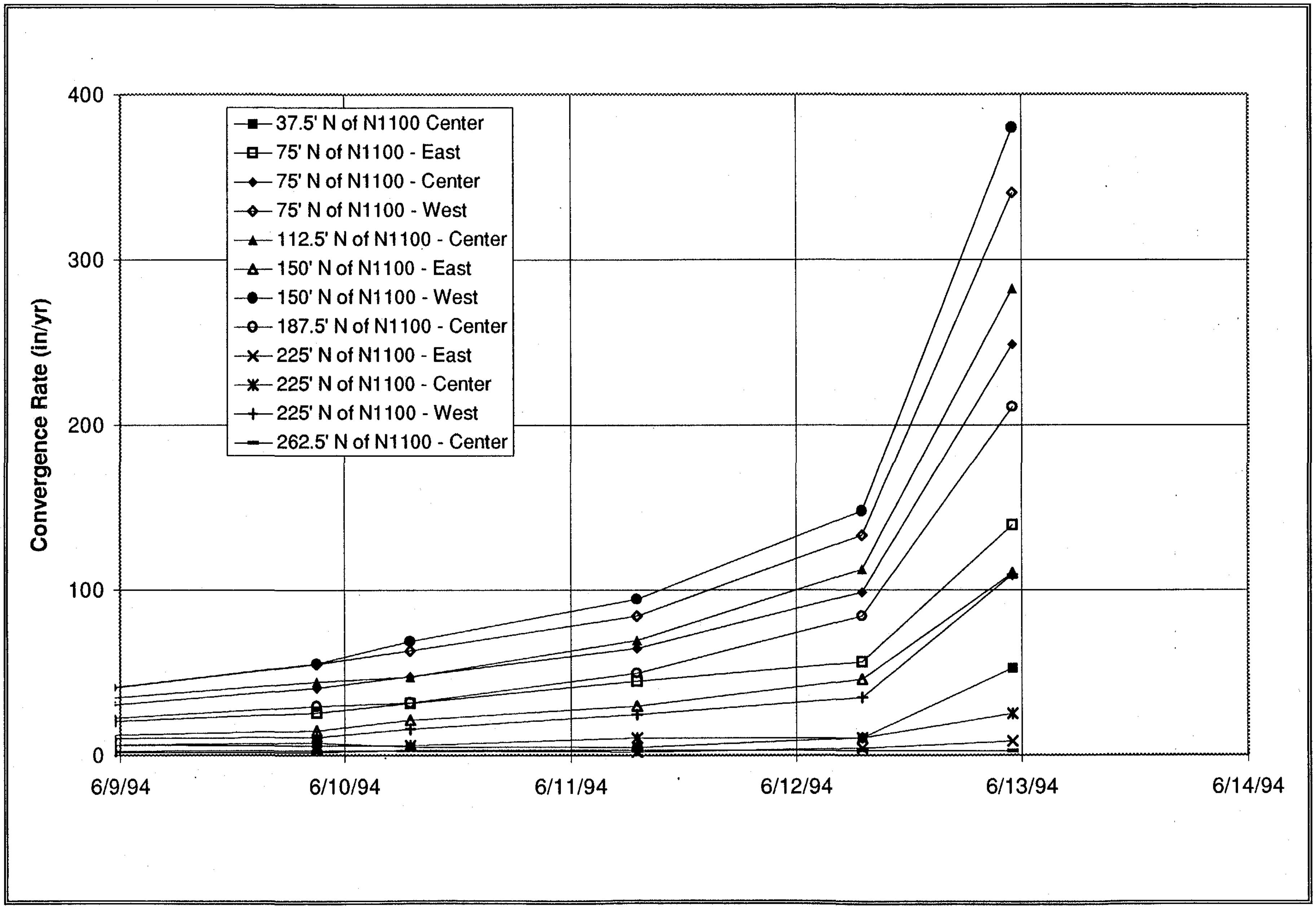


Figure 6-4
SPDV Room 2 Convergence Rates (for 6/9/94 through 6/12/94)

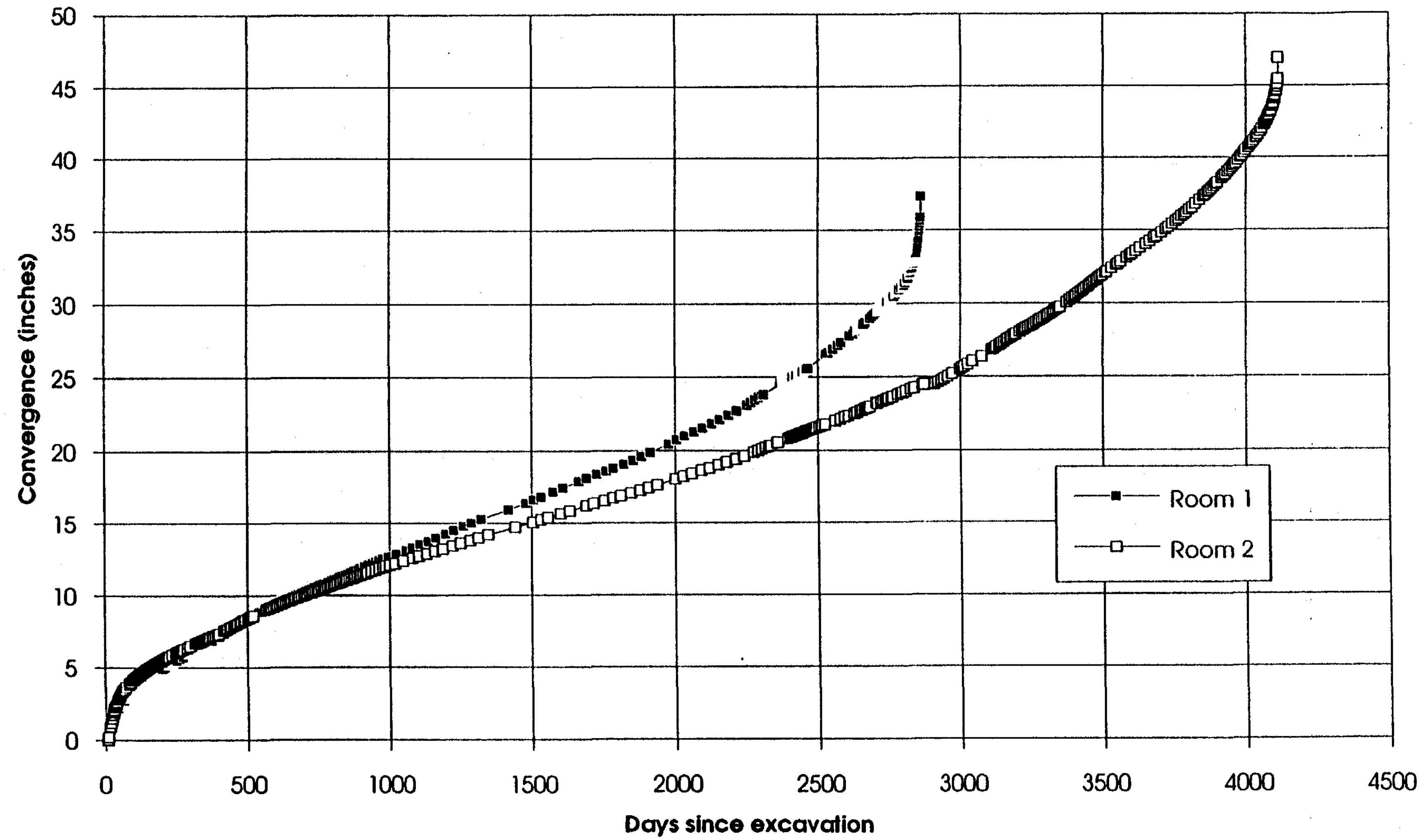


Figure 6-5
SPDV Rooms 1 and 2 Convergence Data

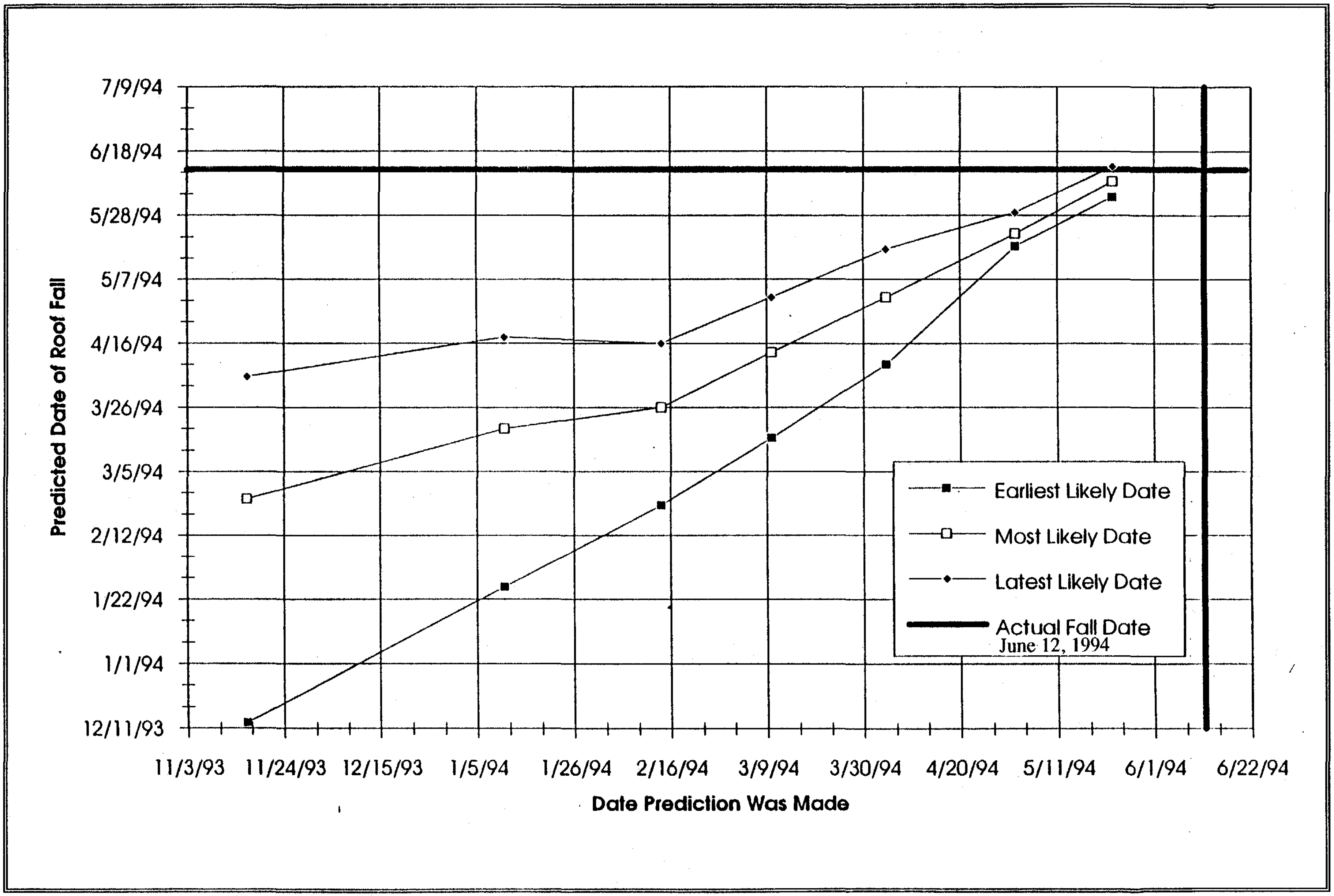


Figure 6-6
Historical Summary of SPDV Room 2
Roof Fall Predictions

7.0 Performance of Waste Disposal Area

Excavation of the waste disposal area began in May 1986, with the mining of entries to Panel 1. Initially, the disposal rooms and drifts were developed as pilot drifts that were later excavated to 13 feet (4 m) high, 33 feet (10 m) wide, and 300 feet (91 m) long. Room 1 was excavated to near full dimensions in August 1986, and pilot drifts for disposal rooms 2 and 3 were excavated in January and February of 1987. Rooms 4 through 7 were completed between March and May of 1988.

In 1991, a supplementary roof support system was designed and installed in Room 1, in order to facilitate a bin-scale test program. At that time, the existing geotechnical monitoring system in the room was upgraded to meet the needs of evaluating the performance of both Room 1 and the support system. A detailed description of the installation is presented in the Waste Isolation Pilot Plant Supplementary Roof Support System Underground Storage Area Room 1, Panel 1 (DOE, 1991). Detailed analyses of the system performance, maintenance activities, and procedure development are presented in performance summaries (DOE, 1992; DOE, 1993b; DOE, 1993c).

7.1 Modifications to Excavations

No new excavations were mined in the Waste Disposal Area during the reporting period. However, routine maintenance was performed on ribs, floor, and back. In addition, Room 7 was pattern bolted in March 1994.

7.2 Instrumentation

Installation of convergence points immediately followed the completion of each mining pass. Multi-point borehole extensometers were installed after completion of the final room trimming and drilling of the boreholes.

No new extensometers were installed during this reporting period. Monitoring continues on all extensometers.

No new convergence points were installed during this reporting period; however, reinstallations were performed as a result of floor trimming in the S1600 and S1950 drifts.

Monitoring of four convergence meters in Room 6 continued during this reporting period. The convergence meters were placed at the midpoints of Room 6 to monitor horizontal and vertical closure of the room.

The 286 rockbolt load cells in the Room 1 yielding roof support system are monitored on a weekly basis and determine when the bolts are to be detensioned, based on the load indicated by the rockbolt load cells. By detensioning, the loads were reduced from approximately 20,000 lbs down to 5,000 lbs.

7.3 Excavation Performance

Convergence points were installed at selected locations immediately following excavation in order to collect early closure data. Tables 7-1 and 7-2 compare the closure rate for the Panel 1 rooms and the SPDV rooms. Although the history of the Panel 1 rooms is shorter than that of the SPDV rooms, a similar pattern of closure is emerging. The vertical closure rates in Room 1, Panel 1 have shown a slight increase from the last reporting period. Room 1, Panel 1 data will continue to be compared with data from the SPDV rooms in order to determine the effect of rockbolts and the supplemental roof support system on the life of the room. Roof to floor convergence data indicate that the closure in the waste disposal areas is comparable to that of the SPDV rooms at a similar stage of development.

During this reporting period minor modifications were made to the supplementary roof support system in Room 1, Panel 1. Documentation of the modifications can be found in the SRSAG reports (DOE, 1993c and DOE, 1994).

The support system has performed in accordance with design expectations with no major problems encountered. The monitoring system is performing well and is able to adequately measure the performance of the room and support system. Room 1 continues to exceed performance criteria.

Table 7-1
Closure Rates by Time Since Excavation
Vertical Convergence Data (inches/year)

ROOM	DATE OF EXCAVATION AT INSTRUMENT	EXCAVATION COMPLETED TO FINAL DIMENSIONS	YEAR							
			0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8
PANEL 1:										
ROOM 1	June 1986	August 1986	4.13	4.26	4.60	3.24	3.56	3.08	2.76	2.90
ROOM 2	January 1987	March 1988	3.02	6.71	3.33	2.91	2.60	2.32	2.32	
ROOM 3	February 1987	March 1988	1.60	9.85	3.34	2.71	2.46	2.32	2.40	
ROOM 4	February 1988	March 1988	8.33	3.14	2.56	2.37	2.36	2.54		
ROOM 5	February 1988	March 1988	9.19	2.76	2.28	2.17	2.10	2.27		
ROOM 6	February 1988	May 1988	8.36	3.41	2.76	2.49	2.44	2.16		
ROOM 7	March 1988	March 1988	9.26	3.22	2.70	2.51	2.41	2.61		
SPDV:										
ROOM 1	April 1983	April 1983	6.79	3.75	2.93	2.88	2.85	3.32	4.46	12.40
ROOM 2	March 1983	March 1983	7.10	3.18	2.39	2.19	2.14	2.33	2.74	2.87
ROOM 3	March 1983	March 1983	7.89	3.75	2.73					
ROOM 4	April 1983	April 1983	5.40	3.18	2.65	2.33	2.13	2.30	2.45	2.31

<u>NOTES:</u> Year is determined from date of excavation at instrument.

Table 7-2
Closure Rates by Time Since Excavation
Horizontal Convergence Data (inches/year)

ROOM	DATE OF EXCAVATION AT INSTRUMENT	EXCAVATION COMPLETED TO FINAL DIMENSIONS	YEAR							
			0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8
PANEL 1:										
ROOM 1	June 1986	August 1986	4.24	3.00	2.72	1.80	1.78	1.52	1.30	1.32
ROOM 2	January 1987	March 1988	1.67	5.29	2.03	1.64	1.43	1.26	1.28	
ROOM 3	February 1987	March 1988	1.63	8.14	2.26	1.76	1.60	1.41	1.50	
ROOM 4	February 1988	March 1988	6.53	2.16	1.67	1.55	1.34	1.44		
ROOM 5	February 1988	March 1988	6.92	2.10	1.66	1.50	1.36	1.48		
ROOM 6	February 1988	May 1988	7.37	2.09	1.64	1.39	1.28	1.31		
ROOM 7	March 1988	March 1988	6.53	1.93	1.50	1.37	1.16	1.22		
SPDV:										
ROOM 1	April 1983	April 1983	5.56	2.47	1.95	1.79	1.56	1.89		
ROOM 2	March 1983	March 1983	5.06	2.16	1.63	1.52	1.42	1.55	(1.85)	
ROOM 3	March 1983	March 1983	6.46	2.28	1.75	1.50	2.08	1.77	1.77	
ROOM 4	April 1983	April 1983	4.55	1.79	1.53	1.29	1.17	1.31	1.36	1.28

NOTES: Year is determined from date of excavation at instrument.
( ) Estimate based on less than one full year of data.

8.0 Excavation Effects Program

Excavation effects, which are the structural responses of the rock mass to excavations, have been observed and monitored since mining of the underground facility began at the WIPP. A historical summary of excavation effects activities is presented in Table 8-1. During this reporting period, observations of excavation effects include:

- subsurface fractures in boreholes
- fractures on excavation surfaces
- rockbolt failures.

These observations quantify, and assist in the interpretation of fractures and bedding plane separations that result from the underground excavations. The size and orientation of fracture apertures or of bedding plane separations are recorded. When possible, indirect measurement or visual estimates are made of the distance that a fracture or separation extends into the rock.

8.1 Borehole Fracture Observations

Systematic borehole observations were started in mid-1986 to study fractures and separations in the roof and the floor of the repository. This program consists of an annual inspection of boreholes that are arranged in arrays throughout the underground as shown in Figure 8-1. Variation in the field data resulting from interpretation by individual inspectors must be taken into consideration when evaluating the results presented here. Graphical representation of the arrays is available upon request.

8.1.1 Interpretation of Fracture Observations

The results of the 1994 survey are compared with the results of previous surveys in Table 8-2. The fractures in roof and floor boreholes are interpreted in terms of their distribution. The main conclusions from the data are that over time:

- fracture frequency increase
- width of fractures and separations increases
- relative displacement at horizontal offsets increases.

Table 8-1
Historical Summary of Excavation Effects

Date	Event	Comments
April 1983	Excavation of SPDV Rooms completed.	None.
July 1983	Sounding survey of all underground openings.	All drummy or slabby areas were removed by hand or with continuous miner or rock bolted.
August 1985	Floor fracturing identified in SPDV Room 3.	None.
December 1985	Observation holes drilled in Waste Shaft Station.	27 holes drilled in Waste Shaft Station to determine condition of MB139.
May - June, 1986	Supplemental drilling program for newly established Excavation Effects Program to investigate conditions in the roof and floor.	30 borehole arrays established throughout the underground (Figure 8-1).
1986	Ground-penetrating radar performed in test rooms as referenced in Borns and Stormont (1989).	Technique was effective in locating first reflector, such as fracture or clay layer, but structures below this were not easily distinguished.
1987	Feasibility study of electromagnetic methods for detection of fluid and fracture zones underground at WIPP.	Study successfully demonstrated use of a direct current resistivity measurement method.
February 1987	Recommendation by Underground Maintenance Committee to cease normal ground maintenance activities in SPDV Room 2 to allow observation of unrestricted deformation.	Scaling and rockbolting prohibited in Room 2.

Table 8-1 (continued)
Historical Summary of Excavation Effects

Date	Event	Comments
September 1, 1987 to February 12, 1988	The section from Clay G downward in Salt Handling Shaft station removed by scaling machine.	Action taken because of observed displacements, separations, and fractures in boreholes in shaft station roof, and minor separations in some holes drilled into the station floor.
January to May, 1989	Detailed fracture surveys completed in each of the SPDV rooms.	Surveys performed in response to increased fracturing in SPDV Rooms 1 and 2.
April 5, 1989	South half of SPDV Room 1 evacuated and barricaded.	Extensive continuous fracture system in roof confirmed by drilling operations.
May 4, 1989	North half of SPDV Room 1 and all of SPDV Room 2 evacuated and barricaded.	Intensive fracture mapping in these two rooms indicated that they should be barricaded.
May 1989	Time Domain Reflectometry cables installed in SPDV Rooms 2 and 4.	Time Domain Reflectometry cables measure rock displacements based on electromagnetic signals caused by changes in cable geometry.
August 1989	Ground probing radar performed in SPDV rooms and Room 6, Panel 1.	Improved processing techniques and equipment identify fractures and stratigraphy within the first two meters of an excavated surface.
March and June 1990	Ground probing radar performed in SPDV rooms, the underground core library, and Room 6, Panel 1.	Enhanced processing enables differentiation between brine and air filled velocities fractures.
June 1990	SPDV Room 3 barricaded.	Radar and roof soundings indicated incompetent ground.
June 19, 1990	Rock fall in experimental Room A2.	Slab is estimated to be 1-2' thick and has a triangular plan view, estimated to be 70' long and 18' wide.

Table 8-1 (continued)
Historical Summary of Excavation Effects

Date	Event	Comments
September 1990	Cribbing in N1100 and N1420 drifts.	Cribs were installed in the entrances to SPDV Rooms 1,2, and 3, and experimental Rooms A1 and A3.
October 1990	Cribbing at experimental Room A2.	Cribs were installed in the entrances to the room.
February 4, 1991	Rock fall in SPDV Room 1.	Size of slab is estimated to be 150'L X 33'H X 6'(ave.) H.
March 1991	Cribbing in N1100 and N1420 drifts.	Cribs were installed in the entrances to experimental Rooms A, B, and D.
April 1991	Ground support at S-1950, E-300 overcast.	Straps and rockbolts were emplaced on both sides of the overcast to provide additional ground support for the area.
May 3, 1991	Rock fall in experimental Room A2.	Slab dimensions are estimated to be 18' X 30" X 18".
May 23, 1991	Rock falls in experimental Room A1.	Two separate falls occurred within 15 minutes of one another. Combined size estimated to be 50' X 18" X 18".
March - May 1991	Northern section of E140 enlarged.	E140, N1420 to S90, was enlarged to 14' X 25'.
August 1991	N460 enlarged.	N460, E0 to E140, was enlarged to 13' X 25'.
September 1991	Roof maintenance in E140 drift.	Extensive roof maintenance was performed on the west side of the E140 drift at approximately S1450.
December 1991	Supplementary ground support installed in Room 1, Panel 1.	A supplementary roof support system was installed in Room 1, Panel 1 to address project Test Phase requirements.
March 1992	Additional ground support installed in Waste Shaft Station.	A cable-support ground control system was installed in the eastern brow of the Waste Shaft.

Table 8-1 (continued)
Historical Summary of Excavation Effects

Date	Event	Comments
June 1992	Additional ground support installed in E140 drift.	An enhanced ground control system approximately 50 feet in length and consisting of wire mesh, cable lacing and mechanical anchored rockbolts was installed in E140 drift at approximately S1500.
October 1992	Extension of E300 drift and excavation of N250 drift.	A N250 drift was excavated between E140 and E300. The E300 drift was extended to N250. Various alcoves were mined to the north and east from E300.
January 1993	Ground support installed in Air Intake Shaft station.	A ground control system consisting of cables and resin-anchored rockbolts was installed on both station brows.
February 1993	Mesh installed on roof of new E300/N250 shop.	Installation of short resin-anchored suspension bolts and welded wire mesh on the roof of the new E300 maintenance shop.
September 1993	Cable support system installed over E0 overcast brow.	Brow trimmed and supported with cables installed using cable shoes and resin-anchored bolts.
February 1994	Pattern bolts installed in Room 7, Panel 1 from S1600 to S1950.	The bolts are anchored above the first major clay seam.
May 1994	Supplementary roof support installed in N1400 from Room C2 to Room A3.	This installation provides continued safe access to data collection facilities.
June 12, 1994	Rock fall in SPDV Room 2.	Size of slab is estimated to be 200'L X 30'W X 4'(ave.)H.

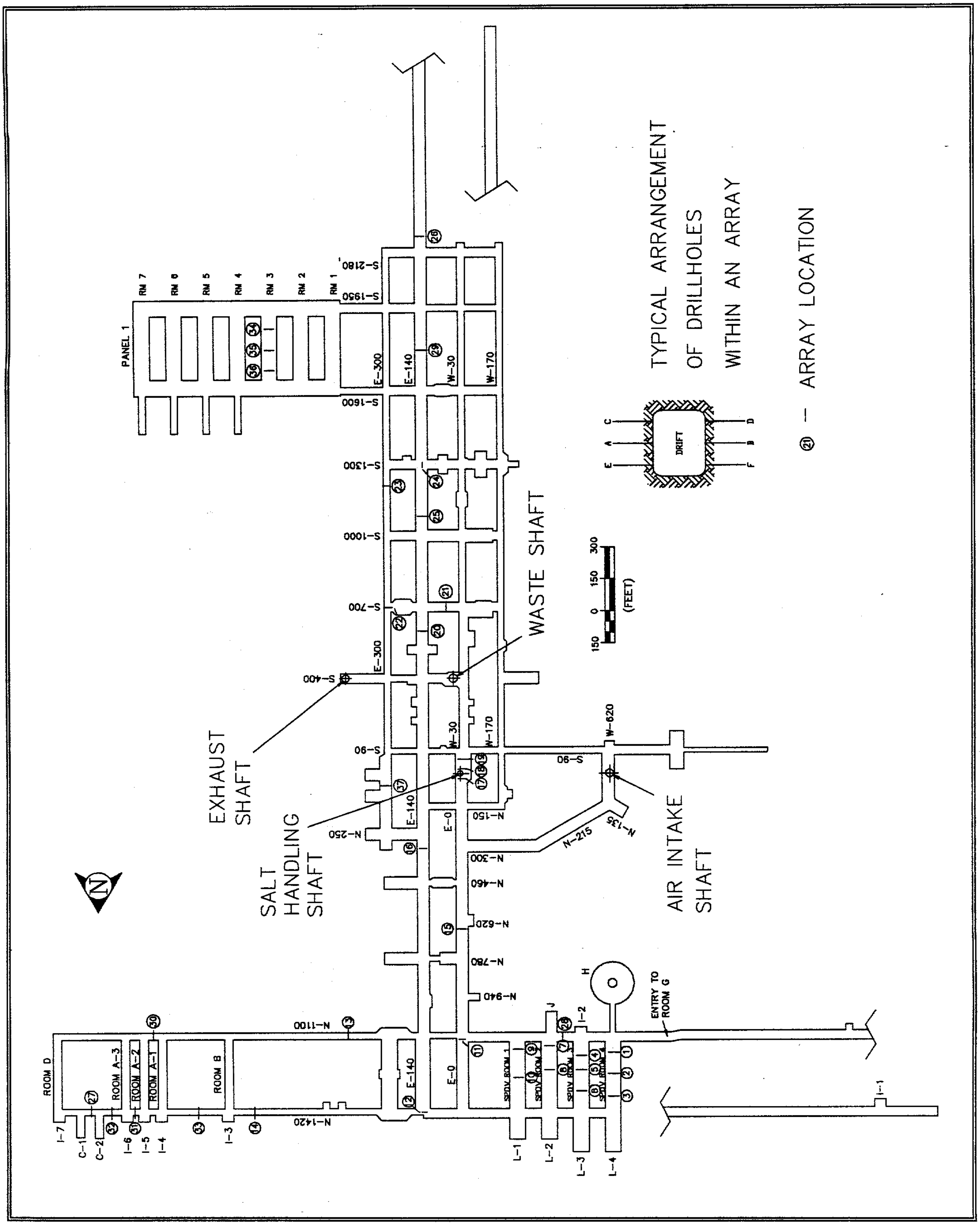


Figure 8-1
Borehole Locations for the Excavation Effects Program

Table 8-2
Cumulative Data from the Excavation Effects Program

Fracturing									
Year	Percent of E Holes (Roof)	Percent of A Holes (Roof)	Percent of C Holes (Roof)	Percent of F Holes (Roof)	Percent of B Holes (Floor)	Percent of D Holes (Floor)	Percent of All Roof Holes	Percent of All Floor Holes	Percent of All Holes
1986	22	28	22	29	46	36	24	37	30
1987	41	52	41	46	62	64	44	57	51
1988	52	64	52	54	65	71	56	63	60
1989	63	80	67	61	69	71	70	67	68
1990	78	88	78	71	77	79	81	76	78
1991	81	88	78	71	77	79	82	76	79
1992	81	92	89	71	77	79	87	76	81
1993	81	92	89	71	81	79	87	77	82
1994	81	96	89	75	81	79	89	78	83

Horizontal Offsetting									
Year	Percent of E Holes (Roof)	Percent of A Holes (Roof)	Percent of C Holes (Roof)	Percent of F Holes (Roof)	Percent of B Holes (Floor)	Percent of D Holes (Floor)	Percent of All Roof Holes	Percent of All Floor Holes	Percent of All Holes
1987	63	32	63	21	35	21	53	26	39
1988	78	44	85	32	46	43	70	40	55
1989	85	60	89	46	54	46	78	49	63
1990	89	64	93	54	58	54	82	55	68
1991	89	80	93	54	62	54	87	56	71
1992	93	80	93	57	62	54	89	57	73
1993	93	84	93	57	62	54	90	57	73
1994	93	84	93	57	62	54	90	57	73

Horizontal offsets in the roof usually occur at clay seams or within the first few inches of the roof as shallow spalls. Horizontal offsets in the floor are generally associated with fractures that define large, shallow, dish-shaped slabs. Direction of offset is observed as the movement of the side of the offset nearest the observer relative to the far side. Direction of offset is typically toward the center of the excavation at clay seams and toward the ribs at shallow fractures. The number and magnitude of offsets is greater in boreholes located near ribs.

8.1.2 Results

Results from the survey are presented in two groups to separate offset information for boreholes drilled at different times. In 1986, 161 roof and floor boreholes (Arrays 1-30) were drilled. Offsets were first recorded for many of these holes in 1987. Eighteen additional roof holes (Arrays 31-36) were drilled in 1991, and offsets were first recorded for these holes in 1992. Array 37, containing six roof and floor holes, was drilled in 1993, with offsets first recorded in the same year.

Table 8-3 lists arrays that are no longer inspected. Thirty-five percent of all roof and floor boreholes that were inspected in 1994 are blocked by horizontal offsets. Access into some floor holes has been lost due to infilling with salt debris.

Arrays 1-30

Since 1986, fractures have been recorded in 89 percent of all roof holes in Arrays 1-30. During this reporting period, fractures were recorded in 82 percent of inspected roof holes. Access to the remainder is blocked by equipment, cables, or location in barricaded areas. Twenty percent of all roof fractures recorded this year were at least 1/8 inch (3 mm) wide with a maximum width of 1-3/16 inch (3 cm). Eleven percent of the roof fractures occurred at clay seams. In general, floor fractures in 1994 have vertical openings no wider than 3/16 inch (5 mm). The maximum floor fracture opening, six inches (15 cm), was found adjacent to the Salt Shaft grizzly. Openings wider than six inches have been observed in the floor near the ribs in the SPDV rooms in past inspections. They are associated with shallow, dish-shaped slabs.

Horizontal offsets have occurred in 90 percent of roof holes and in 57 percent of floor holes in Arrays 1-30 since 1986. The maximum annual relative displacement at any roof offset was two inches (5 cm), and at any floor offset 3/4 inches (2 cm). Fifty-eight percent of the offsets occurred at clay seams.

Table 8-3
Inaccessible Arrays

Array	Last Survey Date	Comment
4	1990	Not accessible
5	1986	Not accessible
6	1990	Not accessible
7	1989	Not accessible
8	1989	Not accessible
9	1989	Not accessible
10	1989	Not accessible
16	1989	Holes are filled with muck
26	1989	Not accessible

Arrays 31-36

Arrays 31 and 33 located in the a barricaded area, therefore were not inspected during the 1994 survey. Ninety-two percent of the boreholes in the remaining arrays contained fractures, with maximum width of 3/8 inch (1 cm) within the first foot (0.3 m) of two boreholes. Ten percent of the fractures occurred at clay seams.

The maximum total relative displacement at any offset was 1-1/4 inch (3 cm). The maximum annual displacement was 3/8 inch (1.6 cm).

Array 37

Array 37 is located in the youngest WIPP excavation (completed in 1992). The largest offset was 1 inch (2.5 cm) relative displacement. The maximum annual displacement was 7/8 inch (2.2 cm). Tight fractures were found in five of the six holes.

8.2 Fracture Mapping of Excavation Surfaces

Maps of surficial fractures in the WIPP excavations are generated for assessments. During this reporting period, fracture maps were produced to assist in ground stability assessments for the following areas:

- N1420 Drift - Roof
- Rooms C1, C2 - Brows
- E140 Drift South - Roof
- SPDV Room 4 - Roof

Unless otherwise noted, fracture apertures are 1/16 inch (2 mm) or less for the following discussion.

8.2.1 N1420 Roof Fractures

This section summarizes changes in fracture conditions in the roof of N1420 from Room D to Substation 2A. Figure 8-2 shows the overall pattern of fracturing as of February 1994. This was compared to fractures documented in January 1993.

The majority of the mapped area lies at the experimental horizon, with Clay I approximately four to five feet (1.2 to 1.5 meters) above the roof. Anhydrite "a" lies approximately eight feet (2.4 meters) above the floor, while Anhydrite "b" lies about one foot (0.3 meter) above the floor.

West of Room B, the excavation ramps down to the disposal horizon. The roof intersects with anhydrites "a" and "b" respectively as it descends. Brows occurring at these intersections terminate at each overlying anhydrite layer.

At the base of the ramp, the drift lies at the facility horizon, where anhydrite "b" typically lies about 7 feet (2.1 meters) above the roof.

Boreholes located east of Room B were drilled in March 1991. Boreholes west of Room B were drilled in June 1986. Where boreholes were inaccessible during the 1994 survey, information from the previous survey is provided.

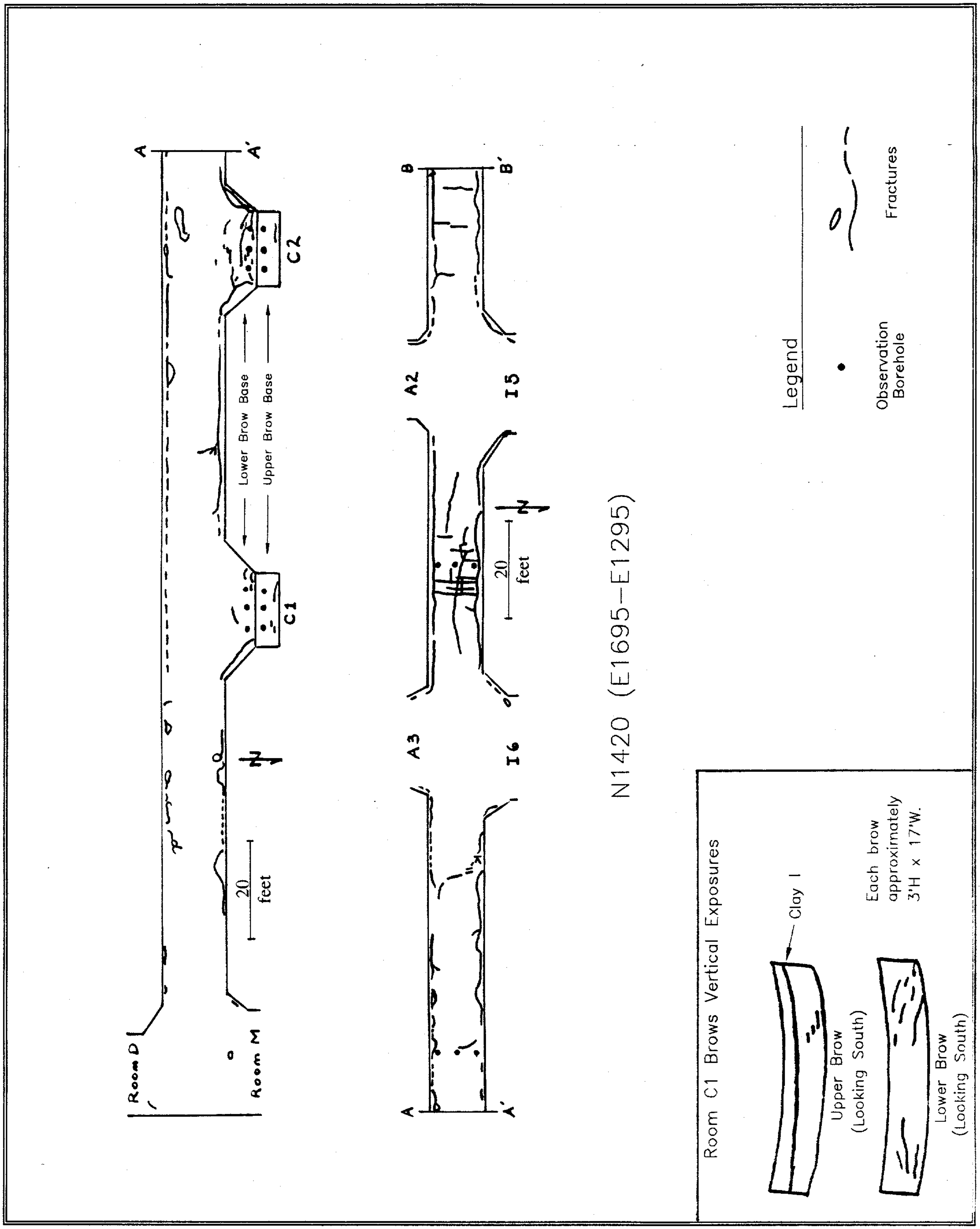


Figure 8-2
N1420 Roof Fracture Patterns

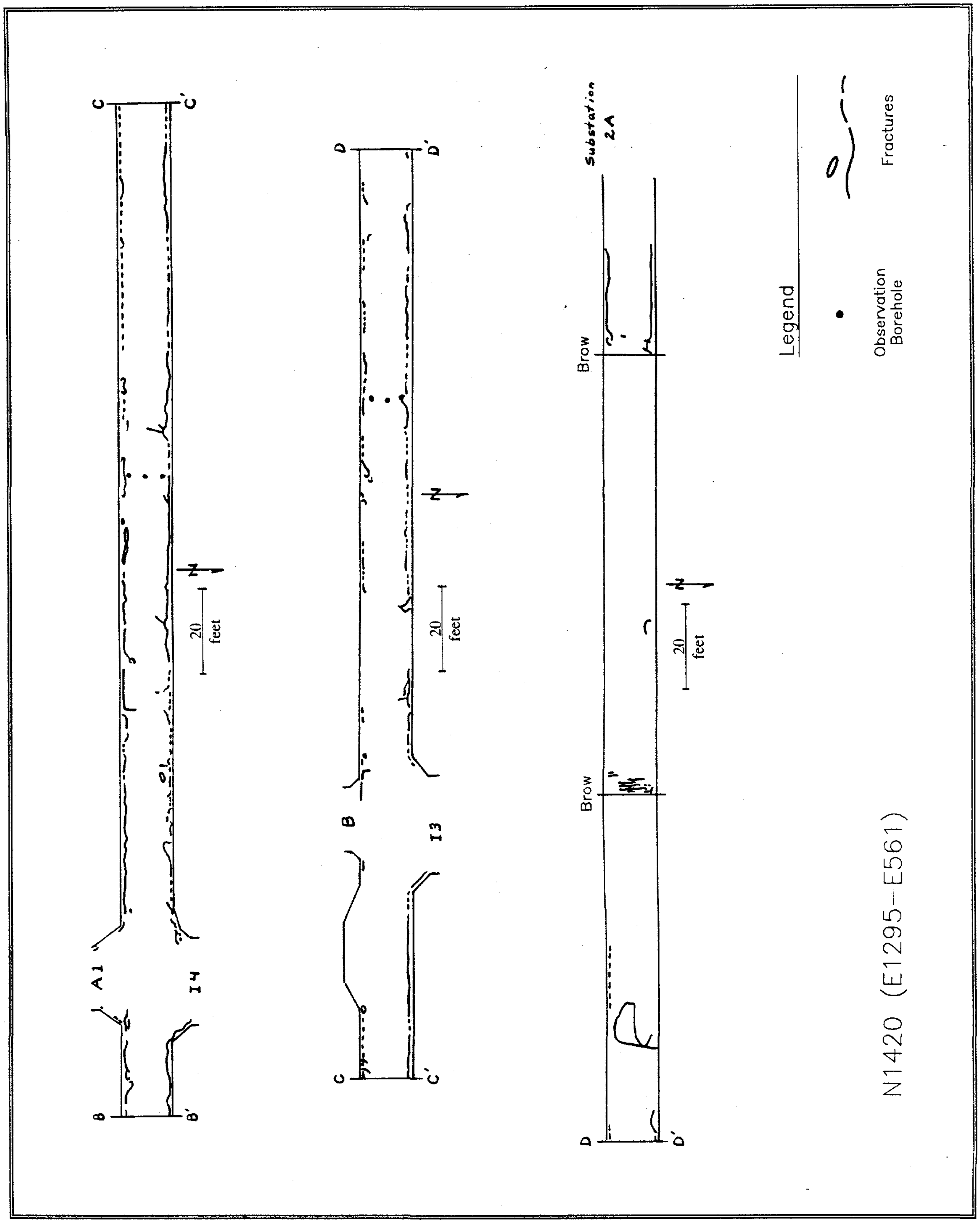


Figure 8-2 (continued)
N1420 Roof Fracture Patterns

Fracturing in the roof along the ribs is virtually continuous from Room D to the ramp. The fractures typically start about one foot (0.3 meter) from the ribs and propagate at low-angles into the roof beam. Apertures are typically about 1/4 inch (6 mm), with a maximum of two inches (5 cm). The fractures often curve around miters at intersections, but rarely continue into the intersections. Near-rib fractures are absent in the roof between the brows in the ramp area, but resume west of the lower brow.

Vertical fractures propagating across the drift have appeared between rooms C2 and A3. Observation boreholes in the area reveal fractures concentrated within 1.9 feet (0.6 meter) of the roof. Displacements have occurred at half of the fractures. Displacement of the roof beam toward the drift centerline increased 1/4 inch (6 mm).

Between rooms A3 and A2, previously mapped vertical fractures propagating across the drift remain relatively unchanged, and similar new fractures have appeared. A previously mapped vertical fracture propagating along the drift increased in length, and several similar new fractures have appeared. Between Rooms A1 and Room B, new vertical fractures have appeared, propagating from the ribs; however, none cross the drift centerline. Observation boreholes in these areas previously identified fractures, often concentrated within about 2 feet (0.6 m) of the roof, and revealed displacement of the roof beam toward the drift centerline.

Fracturing decreases west from Room B and is absent near the upper brow. This can be attributed to the location of Clay H (exposed in the ribs), which confines the horizontal load to the ribs below the clay and allows them to offset into the excavation. Observation boreholes in the area previously revealed displacement of the roof beam toward the drift centerline.

The brow in the upper portion of the ramp terminates at approximately the top of Anhydrite "a". The brow is displaced east along Clay H, and horizontal convergence is apparent from numerous low-angle fractures in the vertical exposure. Some fractures have lengthened (and in some cases interconnected), and several new fractures have appeared.

Numerous vertical fractures have appeared in the underside of the upper brow within ten feet (3 meters) of the vertical exposure. These fractures are oriented parallel to the plane of the vertical exposure. Excluding the former, fractures are virtually absent between the upper and lower brows. This is attributed to the location of Clay G (exposed in the ribs) which confines the horizontal load to the ribs below the clay and allows them to offset into the excavation.

The vertical exposure of the lower brow terminates at Anhydrite "b". The brow is displaced east along Clay G. Fractures in the vertical exposure remain unchanged in number and length; however, apertures have reached a maximum of one inch (2.5 cm). Fractures in the drift from the base of the brow to Substation 2A are relatively unchanged.

8.2.2 Rooms C1, C2 - Brow Fractures

In this summary, the bases of the lower brows are the portions of the roof between the mitered corners at the openings of the rooms (Figure 8-2). Vertical fractures in these areas are typically parallel to the centerline of N1420, while low-angle fractures occur along the miters.

Previously mapped fractures in the base of the lower brow of Room C1 remained unchanged, and one new fracture has appeared. Vertical fractures have appeared in the base of the upper brow. The vertical exposures of the lower and upper brows (see inset Figure 8-2) contain low-angle fractures near the ribs.

Previous inspections of observation boreholes in the brows revealed no separation at and no offsetting. The center hole in the lower brow was rubblized at the clay level.

Previously mapped fractures in the base of the lower brow of Room C2 remain relatively unchanged and several new fractures have appeared. Vertical fractures remain unchanged in the base of the upper brow. Access to inspect the vertical exposures was blocked at the time of inspection. Previous inspections of observation boreholes in the brows revealed no separation at Clay I. One hole in the upper brow revealed a 1/4 inch (6 mm) displacement at the clay, and in the center hole in the lower brow, the salt was broken up at the clay level.

8.2.3 E140 Drift South - Roof Fractures

Fractures in the roof in E140 from S640 to S2180 (Figure 8-3) were mapped in February 1994. Information on subsurface fracturing and displacement was obtained from observation boreholes (drilled in arrays of three), located in the mapped area. Two arrays were drilled in July 1986, between S700 and S1950; twelve arrays were added in this area in May 1992. Three more arrays were drilled in August and September 1993, between S1950 and S2180.

The occurrence of surface fractures from S640 to S1300 is infrequent in comparison to those mapped further south. The maximum fracture length is seven feet (2.1 meters). Between S700

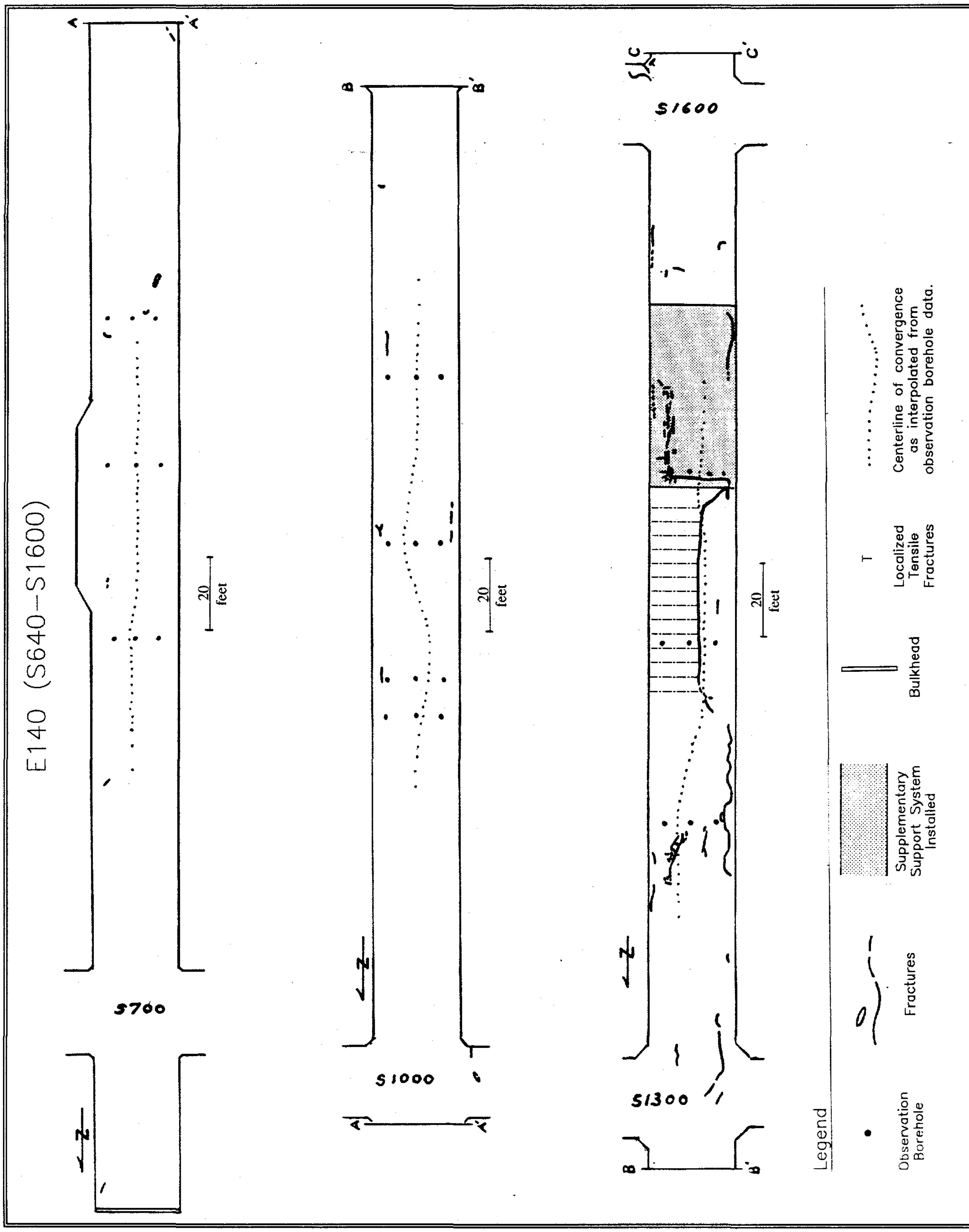


Figure 8-3
South E140 Drift Roof Fractures

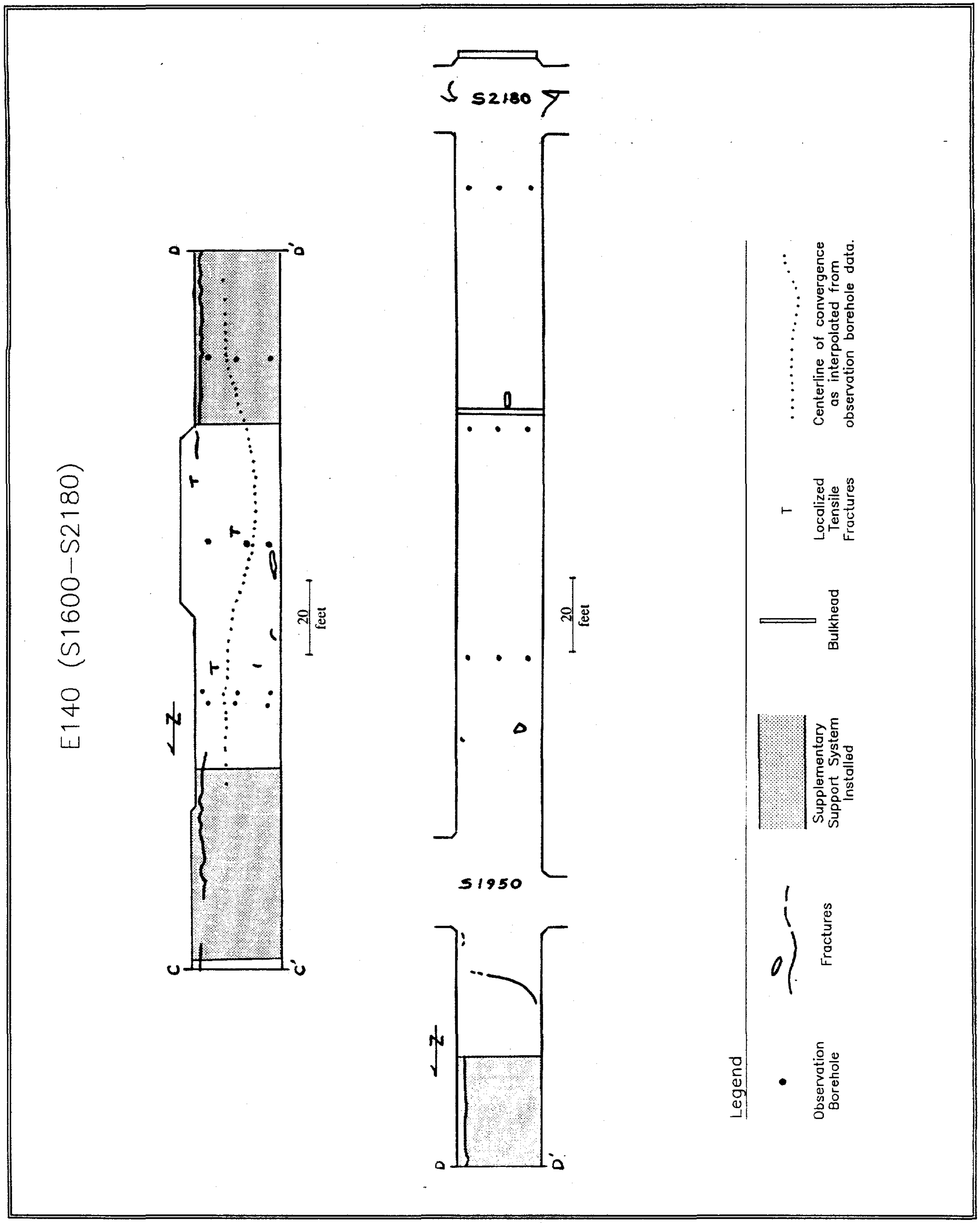


Figure 8-3 (continued)
South E140 Drift Roof Fractures

and S1000, observation boreholes reveal a maximum roof beam displacement of 1/2 inches (1.3 cm) and symmetry of roof beam convergence toward the drift centerline. Between S1000 and S1300, the maximum observable roof beam displacement was 2 inches (5.1 cm), and the center of roof beam convergence moves from west to east to the center of the drift as observed from north to south.

Between S1300 and S1600, low-angle fractures dominate the west side of the drift midway between the intersections. Concentrated areas of vertical fracturing appear on the east side of the drift, as fracturing on the west side diminishes. Roof beam displacement of up to 2-1/4 inches (5.7 cm) has occurred, and the center of convergence, as observed from S1300 east to S1600 west, moves from east to west of the drift centerline.

Between S1600 and S1950, low-angle fracturing dominates the east rib, breaking at a cut-out in the east rib located midway between the intersections. Localized areas of vertical fracturing occur in the vicinity of the cut-out. Roof beam displacement of at least 1-7/8 inches (4.8 cm) has occurred, and the center of convergence, as observed from S1600 east to S1950 east, moves from east to west and then returns east of the drift centerline.

From S1950 to S2180, the drift is virtually devoid of fractures. Observation boreholes in the area were new at the time of inspection and contained no observable displacements.

8.2.4 SPDV Room 4 - Roof Fractures

Fractures in the roof of SPDV Room 4 (Figure 8-4) were mapped in January 1994. Low-angle fractures occur within ten feet (3.1 meters) of the ribs through most of the room. Fracturing along the west rib tends to be continuous. Near the east rib, fractures are fairly evenly spaced and tend to propagate diagonally from the rib. Fracturing along both ribs decreases toward the ends of the room. Vertical fracturing is prominent along the center of the roof and is concentrated in three areas. The pattern of vertical fracturing does not appear to correlate with the presence or absence of low-angle fracturing along the ribs, except that this fracturing also decreases toward the ends of the room. Observation boreholes in the roof reveal that the centerline of convergence lies west of the drift centerline.

8.3 Observations of Rockbolt Failures

Rockbolt failures are documented as close to the time of failure as possible and recorded in a database. Each failure record includes, when possible, the type of failure, location, bolt length

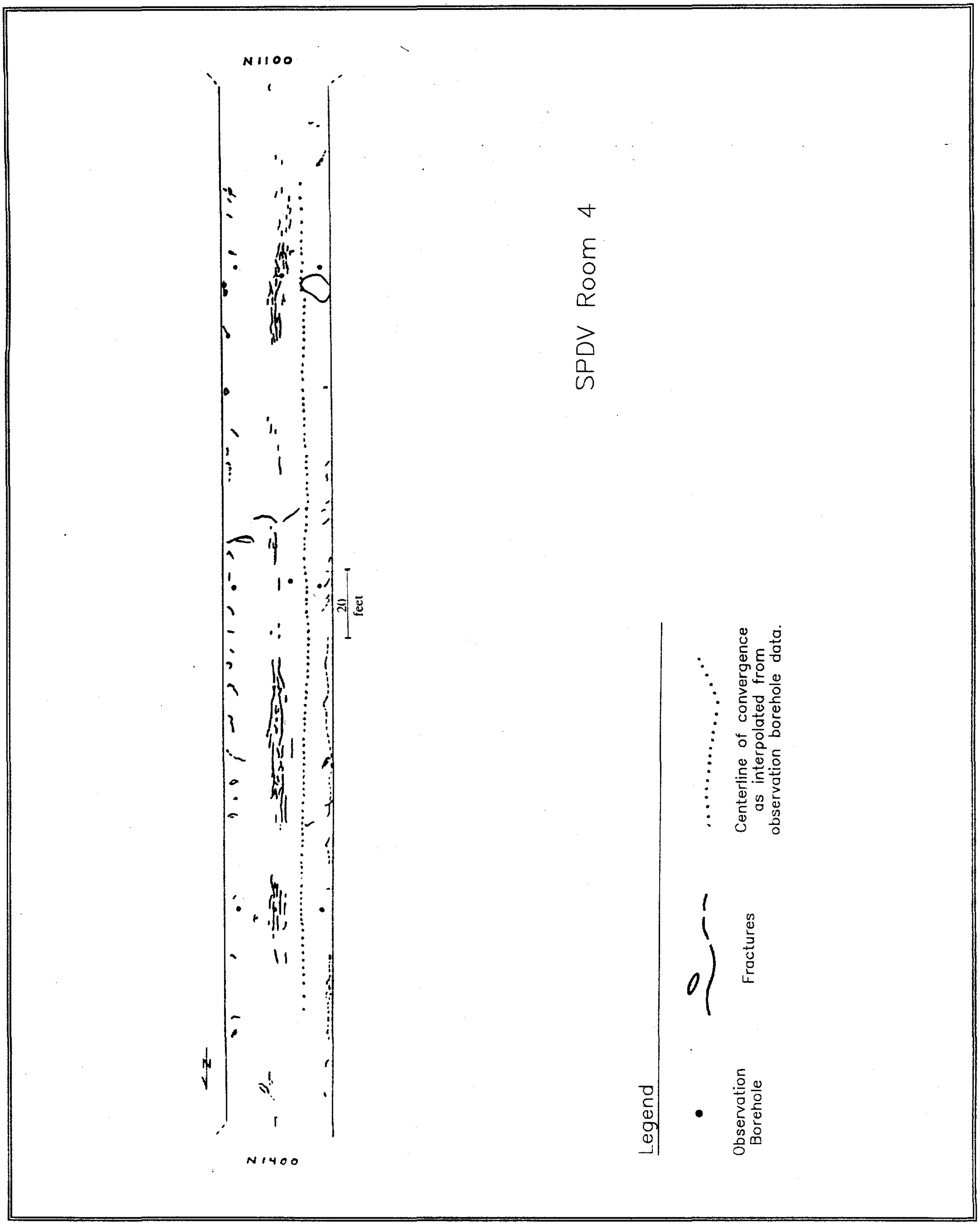


Figure 8-4
SPDV Room 4 Roof Fractures

and diameter, plate type, and any unusual condition associated with the installation.

8.3.1 Failure Mechanisms

Bolt head failure occurs when the head of the bolt separates from the bolt shaft. Head failures probably can be attributed to conditions causing the bearing plate to be placed at an angle not perpendicular to the bolt axis, which results in the bolt head becoming unevenly loaded as the salt deforms (Figure 8-5). The induced bending moment causes the bolt head to break off as the bolt takes load. Over-torquing may impart high and uneven initial bolt loads, exacerbating other conditions which might lead to failure.

Shaft failure occurs when the bolt breaks along the shaft. Bolts that fail in this manner typically exhibit some degree of bending near the broken ends, and are termed offset failures, indicating that lateral forces were exerted on the bolts. Inspections of observation boreholes located near many of these failures often reveal slip-planes (usually clay layers) lying at approximately the same level as the depth of bolt breakage. These generally have some associated corrosion exhibited on the failure surface.

Almost half of the rockbolts that failed in this recording period are suspected shaft failures. These bolts are typically held in place by wire mesh or silicone sealant, or are located in inaccessible areas. Since these bolts were not available for physical observation, they were assumed to have broken along the shaft.

A relatively small number of bolts experience failure of the anchor or plate. Anchor failures to date occur when the anchor assemblies fail or slip down the bolt hole and are probably associated with the installation procedure or conditions at the anchor position. Plate failures typically exhibit cracking or tearing of the plate around the bolt hole and bolt pull-through.

Figure 8-5 depicts the numbers of failure types that occurred each month of the reporting period.

8.3.2 Failure Locations

The vast majority of failures occurring during this reporting period were located in access drifts and rooms which were excavated to a width of 25 feet (7.6 meters) or greater (see Table 8-4). The remainder occurred at various locations throughout the underground. In most cases, new bolts were installed very near the failures.

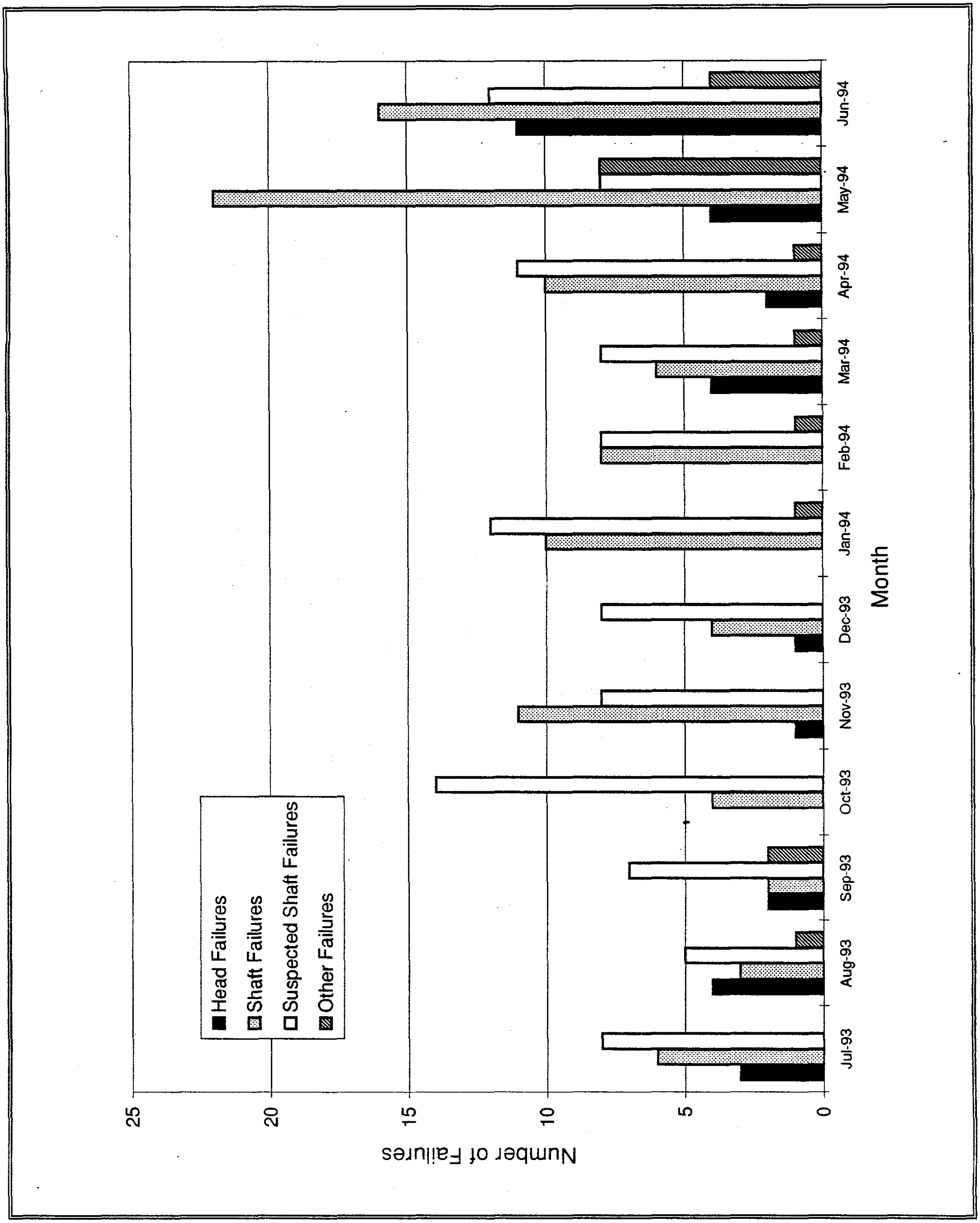


Figure 8-5
Bolt Failures by Type and Month

Table 8-4 - Bolt Failures July 1, 1993 through June 30, 1994 - Location and Type

Location	Width:	Shaft Failures Recovered:	Shaft Failures Suspected:	Head Failures	Other Failures	Total Failures in Location	Percent of total :
Exp. Ops. Shop	33' (10m)	11	81	0	3	95	36%
Panel 1-Rooms	33' (10m)	17	24	1	0	42	16%
L-Rooms	33' (10m)	7	2	0	2	11	4%
E140-North	25' (7.6m)	33	0	0	3	36	14%
E140-South	25' (7.6m)	8	0	10	1	19	7%
E0	25' (7.6m)	6	1	3	7	17	6%
Other		20	1	18	3	42	16%
Total Failures of Type:		102	109	32	19	262 Total	
Percent of total:		39%	42%	12%	7%		

8.3.2.1 E300 Shop (Experimental Operations Shop)

Bolt failures in this area account for approximately one third of the total bolt failures in the underground this year. Because these bolts were installed almost immediately after excavation, they were greatly affected by the early high deformation of the rock, whereas most pattern bolts in other areas of the facility were installed some years after excavation. The effective life of the rockbolts has been reduced due to the early excessive deformation.

The shaft failures typically occurred about three feet (0.9 m) from the ribs. The recovered shaft failures were typically broken about 6.5 feet (2 m) from the heads, and 91 percent exhibit the characteristic offset-induced bending at the breaks. In the E300 shop, the first stratigraphic contact has been shown by core logs to lie approximately 6.5 feet (2 m) above the roof. Wire mesh is installed on the roof to prevent broken bolts from falling. Bolts that cannot be recovered through the mesh are termed suspected shaft failures and account for 74 percent of the total suspected shaft failures this year.

8.3.2.2 Panel 1 - Rooms 2 through 7

The pattern bolts in rooms 2 through 6 of the panel (Panel 1) are ten-feet (3-meter) long and penetrate Clay G, which lies at an average of 6.7 feet (2 m) above the back. Shaft failures from these rooms are typically at 6.5 feet (2 m). Pattern bolts installed in Room 7 are six-feet (1.8 m) long and have been thought to not penetrate the clay, however, offset failures occurred this year suggesting that Clay G lies somewhat lower than six feet (1.8 m) in areas of the room. The bolts were typically broken about 5.4 feet (1.6 m) from the heads. New ground control installations contain bolts which are anchored above the clay layer. Lack of reported bolt failures in Room 1, Panel 1 is due primarily to containment by the meshes of the Supplementary Roof Support System.

The increased number of reported shaft failures in the panel rooms is the result of a torque-testing program to determine whether silicon sealant present in the installations was preventing broken bolts from falling from the holes. The tests indicated several suspected shaft failures, and, of those that were recovered, the vast majority were shaft failures with characteristic offset induced bending at the breaks. It is unknown how long some of the bolts were broken prior to discovery. The shaft failures typically occurred about eight feet from the ribs.

8.3.2.3 E140 Drift - North

The E140 drift north of N300 (inclusive) produced approximately one third of the total reported recovered shaft failures for the year. The onset of failures in this area are, as in the E300 Shop, probably due to bolt installation immediately after excavation activities to enlarge the drift. Installation of the rockbolts immediately after excavation activities subject the bolts to early high deformation of the rock. This area was pattern bolted soon after the drift was enlarged.

The failures typically occurred about 7.5 feet (2.3 m) from the ribs and 6.4 feet (2 m) into the roof. Over ninety percent exhibit evidence of offsetting at the breaks. The first stratigraphic contact in the area of these failures has been shown by core logs to lie at an average of 7.1 feet (2.2 m) above the roof.

8.3.2.4 E140 Drift - South

The E140 drift south of S900 (inclusive) contains approximately one third of the total recorded head failures for the year. The relatively small number of shaft failures averaged about 6.5 feet (2 m) from the ribs and 4.4 feet (1.3 m) into the roof. The zone of offsetting in the area averages 4.8 feet (1.5 m) above the roof.

8.3.2.5 E0 Drift

The E0 drift shaft failures were an average 8.6 feet (2.6 m) into the roof and typically occurred 7.5 feet (2.3 m) from the ribs. Shaft failures in this area started in April 1994.

8.3.2.6 General

The remainder of failures this year represent installations ranging from mesh support to pattern bolts. These failures occurred at various locations, with several failures occurring in the S1300 Shop, the Waste Shaft Station, and the E300/N1100 and E300/N1420 alcoves. Bolts are replaced, as necessary, to maintain the integrity of the support patterns.

The database of rockbolt failures is updated as bolts fail. Investigations into the causes and impacts of rockbolt failures in the underground are made as additional data is collected.

9.0 Summary

At the start of the project, criteria were developed which address the requirements for the design of the WIPP. These criteria (DOE, 1984) cover all aspects of the facility and its operation as a pilot plant for the demonstration of technical and operational methods for permanent isolation of contact- and remote-handled transuranic waste. These criteria are compared with conditions actually observed in the underground from July 1993 to June 1994 (Table 9-1).

Fracture development in the roof is primarily caused by the concentration of compressive stresses in the roof beam and is influenced by the size and shape of the excavation and the stratigraphy in the immediate vicinity of the opening. Pillar deformations induce lateral compressive stresses into the immediate roof and floor. The buildup of stress, with time, causes differential movement along stratigraphic boundaries. Stratigraphic boundaries allow differential movements which become manifest as horizontal offsets in Excavation Effects Program boreholes and as bending deformations in failed rockbolts. Large strains, associated with lateral movements in the roof, can induce fracturing in the roof which is frequently seen near the ribs. This scenario of roof deterioration, combining a buildup of compressive stresses over time, horizontal offsetting, and large strains associated with lateral movements, substantiated by the shape and size of the roof fall in SPDV Room 2.

The in situ performance of the excavations generally continues to satisfy the appropriate design criteria, although specific areas are being identified where deterioration due to aging must be addressed through routine maintenance and implementation of engineered systems. This deterioration has been identified through the analysis of data acquired from geomechanical instrumentation and the Excavation Effects Program. The expected life of some of the openings may need to be extended, which may require redesigning the access drifts or additional ground control effort.

Through data analysis, a roof fall was predicted in SPDV Room 2. Using a curve fitting method of prediction, the rate of deterioration of SPDV Room 2 was found to be slower than that of SPDV Room 1. However, care had to be taken due to the very small variations in closure rates that could cause a large over- or under-estimation of the remaining stand-up time for the excavation. As a result of the roof fall in SPDV Rooms 1 and 2, understanding of the mechanism leading to roof falls and the ability to predict them has been greatly improved. The database of instrumentation data from roof falls has quadrupled from four to sixteen instruments, which will provide significantly more confidence in future roof fall predictions.

In addition to underground instrumentation, qualitative assessments of fracture development are documented through mapping of the underground repository and inspections of the excavation effects boreholes and other observation boreholes. The information acquired from these programs provides early detection of ground deterioration and contributes to the design of effective ground control and support systems.

The latest evaluation of the yielding supplementary roof support system in Room 1, Panel 1 concluded that the system continues to function in accordance with design specifications, with minor modifications as described in DOE, 1993c and DOE, 1994. The evaluation is based on an investigation of observation boreholes and geomechanical instrumentation located in the room. In addition, the rock stratum in which the support system is anchored appears to be stable.

Table 9-1
Comparison of Excavation Performance to Design Criteria

Criterion (DOE, 1984)	Description	Comments
Chapter 5: Design Criteria, Underground Item 2: Ground Control a: Shaft Design	"Shafts shall be designed to be structurally stable throughout the operating life of the underground facility and for the period of time thereafter required for decommissioning of the facility."	Water pressure observed on piezometers located behind the shaft keys remains below design levels. Geomechanical data indicate that the shafts are structurally stable.
	"The time dependent closure of shafts due to salt creep shall be considered in the design of shafts. Shafts shall be dimensioned so that the minimum dimensions required for shaft functions are maintained for the entire design life."	Closure of the shafts remains within design requirements.
	"Groundwater inflow to the shaft shall be controlled so that no uncontrolled groundwater reaches the storage horizon via the shafts."	The small amount of groundwater inflow into the shafts have been controlled through grouting.

Table 9-1 (continued)
Comparison of Excavation Performance to Design Criteria

c. Mine Design	"...deformation of excavations and pillars shall remain within the limits required for structural function, ventilation, and safety."	Pillars remain stable. Deformation of excavations remains within the required limits. A roof fall occurred in SPDV Room 2 in June 1994. SPDV Rooms 1, 2, and 3 remain barricaded.
	"Rock bolts shall be used where necessary to provide positive support of roofs and walls."	Rock bolts have been installed throughout the underground to aid in roof and wall support. Failures have been documented and will be studied further as additional data become available.
d. Emplacement Criteria	"The underground storage rooms and access drifts shall be compatible with the waste transport vehicles and with the waste container sizes, shapes, weights, stacking configurations, and the handling and backfilling equipment requirements... . The storage rooms shall be sized for efficient handling and stacking of the CH waste containers."	**Access Drifts** Geomechanical instrument data and visual observations indicate that the current design is compatible with emplacement criteria. Closure rates indicate that access drifts will require periodic trimming in order to provide clearances for equipment. The disposal rooms and access drifts have been sized to permit transportation and disposal of the CH TRU waste. **Disposal Rooms** Geomechanical instrument data and visual observations indicate that the current design is compatible with emplacement criteria.
e. Retrievability	"All wastes placed into the WIPP are retrievable, with retrievability to be demonstrated, until such time as the pilot plant is converted to an operational repository for permanent disposal of wastes. The storage room shall allow for salt creep and shall be sized to minimize breaching of the CH waste containers for a period of ten years."	Ground control was implemented in Panel 1 to ensure access into rooms during the Demonstration/Test Phase. Delays in using Panel 1 and changes in its use have required additional remedial measures to be taken. These include trimming of the floors, backfilling of the floor fractures, and the installation of supplementary roof support in some areas.
f. Instrumentation	"Underground instrumentation is required to measure phenomena important to the performance of the facility or which cannot be otherwise quantified. Instruments shall be provided to measure such phenomena as rock behavior, gases, seismic activity, and groundwater pressures in order to confirm or revise design assumptions."	Geotechnical instrumentation is operated and maintained to meet the requirements of this criteria. Geotechnical experts agree that the monitoring program at the WIPP has been proven adequate, specifically with regard to the instrumentation in Room 1, Panel 1.

10.0 References

DOE - See U.S. Department of Energy

Borland International, Inc., 1992, dBase IV (Version 4.0).

Borns, D.J., and J.C. Stormont, 1989, An Interim Report on Excavation Effect Studies at the Waste Isolation Pilot Plant: The Delineation of the Disturbed Rock Zone, SAND87-1375, Sandia National Laboratories, Albuquerque, New Mexico

Holt, R.M., and D.W. Powers, 1984, Geotechnical Activities in the Waste Handling Shaft, Waste Isolation Pilot Plant (WIPP) Project, Southeastern New Mexico, WTSD-TME 038, Waste Isolation Pilot Plant, Carlsbad, New Mexico.

Statistical Graphics Corporation, 1990, Statgraphics (Version 4.0).

U.S. Department of Energy, 1983, Summary of the Results of the Evaluation of the WIPP Site and Preliminary Design Validation Program, WIPP-DOE-161; Waste Isolation Pilot Plant, Carlsbad, New Mexico.

U.S. Department of Energy, 1984, Design Criteria, Waste Isolation Pilot Plant (WIPP) Revised Mission Concept-IIA, WIPP-DOE-071; Waste Isolation Pilot Plant, Carlsbad, New Mexico.

U.S. Department of Energy, 1985, Quarterly Geotechnical Field Data Report, DOE-WIPP-221, Waste Isolation Pilot Plant, Carlsbad, New Mexico.

U.S. Department of Energy, 1986a, Interim Geotechnical Field Data Report, DOE-WIPP-86-012, Waste Isolation Pilot Plant, Carlsbad, New Mexico.

U.S. Department of Energy, 1986b, Design Validation Final Report, DOE-WIPP-86-010; Waste Isolation Pilot Plant, Carlsbad, New Mexico.

U.S. Department of Energy, 1986c, Geotechnical Activities in the Exhaust Shaft, DOE-WIPP 86-008, Waste Isolation Pilot Plant, Carlsbad, New Mexico.

U.S. Department of Energy, 1990a, Geologic Mapping of the Air Intake Shaft at the Waste Isolation Pilot Plant, DOE/WIPP 90-051, Waste Isolation Pilot Plant, Carlsbad, New Mexico.

U.S. Department of Energy, 1990b, Geotechnical Field Data and Analysis Report, DOE/WIPP 90-006, Waste Isolation Pilot Plant, Carlsbad, New Mexico.

U.S. Department of Energy, 1991, Waste Isolation Pilot Plant Supplementary Roof Support System Underground Storage Area Room 1, Panel 1, DOE/WIPP 91-057, Waste Isolation Pilot Plant, Carlsbad, New Mexico.

U.S. Department of Energy, 1992, WIPP Supplementary Roof Support System Room 1, Panel 1 Geotechnical Field Data Analysis Bi-Annual Report, DOE/WIPP 92-024, Waste Isolation Pilot Plant, Carlsbad, New Mexico.

U.S. Department of Energy, 1993a, Current Bases for Roof Fall Prediction at WIPP and a Preliminary Prediction for SPDV Room 2, DOE/WIPP 93-033, Waste Isolation Pilot Plant, Carlsbad, New Mexico.

U.S. Department of Energy, 1993b, WIPP Supplementary Roof Support System, Room 1, Panel 1 Geotechnical Field Data Analysis Report March 1993, DOE/WIPP 93-012, Waste Isolation Pilot Plant, Carlsbad, New Mexico.

U.S. Department of Energy, 1993c, WIPP Supplementary Roof Support System, Room 1, Panel 1 Biannual Performance Summary October 1993, DOE/WIPP 93-067, Waste Isolation Pilot Plant, Carlsbad, New Mexico.

U.S. Department of Energy, 1994, WIPP Supplementary Roof Support System, Room 1, Panel 1 Biannual Performance Summary September 1994, DOE/WIPP 94-2025, Waste Isolation Pilot Plant, Carlsbad, New Mexico.