RI 9360

REPORT OF INVESTIGATIONS/1991

# Evaluation of In Situ Cemented Backfill Performance

By D. R. Tesarik, J. D. Vickery, and J. B. Seymour

UNITED STATES DEPARTMENT OF THE INTERIOR

BUREAU OF MINES

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**Library of Congress Cataloging in Publication Data:**

**Tesarik, D. R. (Douglas R.)**
Evaluation of in situ cemented backfill performance / by D. R. Tesarik, J. D. Vickery, and J. B. Seymour.

p. cm. — (Report of investigations; 9360)

Includes bibliographical references (p. 26).

Supt. of Docs. no.: I 28.23:9360.

1. Ground control (Mining)—Washington (State)—Wenatchee Region. 2. Mine filling—Washington (State)—Wenatchee Region. I. Seymour, J. B. II. Vickery, J. D. III. Title. IV. Series: Report of investigations (United States. Bureau of Mines); 9360.

TN23.U43 [TN288] 622 s—dc20 [622'.28] 90-23307 CIP

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## UNIT OF MEASURE ABBREVIATIONS USED IN THIS REPORT

| | | | |
|---|---|---|---|
| deg | degree | $\mu\epsilon$ | microstrain |
| ft | foot | oz | ounce |
| $ft^2$ | square foot | oz/st | ounce per short ton |
| h | hour | pct | percent |
| in | inch | psi | pound per square inch |
| $in^3$ | cubic inch | st | short ton |
| $lb/ft^3$ | pound per cubic foot | $yd^3$ | cubic yard |
| MPa | mega pascal | yr | year |

# EVALUATION OF IN SITU CEMENTED BACKFILL PERFORMANCE

By D. R. Tesarik,[1] J. D. Vickery,[2] and J. B. Seymour[3]

## ABSTRACT

As part of its research program to investigate ways of improving resource recovery and reducing subsidence, researchers from the U.S. Bureau of Mines placed instruments in the B-North ore body of the Cannon Mine, Wenatchee, WA, to monitor cemented backfill and rock deformation during mining. The vibrating-wire gauges proved to be reliable and versatile, and approximately half of the instruments are providing data after 2 years of use.

A two-dimensional, finite-element model was used to analyze the Cannon Mine's multilevel bench cut-and-fill mining method and predict rock and backfill displacements. The model accurately predicted rock displacements, but the predicted and measured displacements in cemented backfill had a correlation coefficient near zero, indicating that the model should only be used to predict rock displacements and not backfill displacements. A finite-difference model was also used to evaluate the stability of a cemented backfill pillar. Results can be used to conservatively predict backfill stresses, but on-site observation of pillar failures coupled with in situ measurements are needed to make more accurate predictions.

An ongoing evaluation of the mining system has indicated that filling the primary stopes tight to the back with cemented backfill allowed these pillars to carry overburden loads soon after the cemented backfill was placed.

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## INTRODUCTION

The Ca[illegible]ine is located in central Washington in the eastern foothills of the Cascade Mountains and lies just outside the city limits of Wenatchee, WA, an agricultural community with a population of about 45,000 (fig. 1). Traditionally known for its fruit orchards, Wenatchee has recently become a major gold-producing region. Because of the mine's proximity to the city, special provisions were made to minimize the environmental impacts associated with mining. Subsidence was a particularly important consideration since the upper boundary of the B-North ore body is only 200 ft beneath the stables and arena of a local riding club. To minimize surface subsidence and yet enable a high percentage of the ore body to be recovered, the mining staff decided to use a cut-and-fill mining method and cemented backfill.

The U.S. Bureau of Mines' ongoing purpose in this study is to monitor a backfilling operation to determine how the entire mine structure, especially the backfill, responds to total extraction of the ore body. Although cemented backfill is an important structural component in mine design (*5*, *8*, *10*, *13*, *16*),[4] in situ evaluation is needed to verify initial design values. Data collected from instruments, along with a validated computer model of the mine, will help in the design of future backfill pillars.


## ACKNOWLEDGMENTS

The authors wish to thank the following Cannon Mine personnel for providing access to the B-North ore body, participating in instrument placement plans, providing mine maps and schedules, and assisting in instrument installation: Mark Anderson, general manager; Jim Knowlson, chief mining engineer; Clark Gilbert, senior mine engineer; Mark Mudlin, mine engineer; John Baz-Dresch, mine engineer; Mike Bernard, task trainer; and Chris Jacobsen, surveyor. The authors also wish to thank Carl Brechtel, associate, J. F. T. Agapito and Associates, Inc., Grand Junction, CO, for his views regarding instrument placement and design; Tony Simmonds, geotechnical engineer, Geokon, Inc., Lebanon, NH, for customizing some of the instruments; and William G. Pariseau, professor of mining engineering, University of Utah, Salt Lake City, UT, for providing the finite-element code and assisting in the interpretation of results.


## BACKGROUND

### HISTORY

As early as 1885, gold was discovered in the Wenatchee district in a series of silicified sandstone outcrops known as reefs. However, because the gold occurs as a very fine-grained material, the ore proved difficult to process, and as a result, most of the early ventures were unsuccessful. A long period of intermittent mining followed, until the region's first significant mining operation began production in 1949. From 1949 until 1967, the Lovitt Mining Co., Wenatchee, WA, produced over 410,500 oz of gold and 625,800 oz of silver from the D-Reef deposit (*2*).

After years of sporadic exploration in the area of the B-Reef (fig. 2), two Canadian companies, Asamera Minerals, Calgary, AT, and Breakwater Resources, Vancouver, BC, conducted a diamond drilling exploration project that confirmed the extent of the B-Reef and B-West deposits. In 1983, the high-grade B-North and B-Neath deposits were discovered, and in 1984, construction of the Cannon Mine facilities began with Asamera as the managing partner. The first ore from the Cannon Mine was processed through the Asamera mill in July of 1985. With the identification of further ore reserves southeast of the B-Reef ore zone, the Cannon Mine is now regarded as one of the most significant recent gold discoveries in North America and is one of the largest underground gold mines in the United States (*1*).

### GEOLOGY

As shown in figure 2, the B-Reef complex of the Cannon Mine contains several distinct ore bodies that together form an elongate ore zone approximately 1,800 ft long. As the managing partner in the Cannon Mine operation, Asamera began mining one of the larger, higher grade ore bodies, the B-North. Containing 4,000,000 st of ore at an average grade of 0.257 oz/st of gold and 0.40 oz/st of silver, this tabular ore body has a maximum width of 500 ft, a maximum length of 800 ft, an average thickness of 130 ft, and lies 200 to 400 ft beneath the surface.

[4]Italic numbers in parentheses refer to items in the list of references at the end of this report.


Figure 1.—Location map of Cannon Mine.

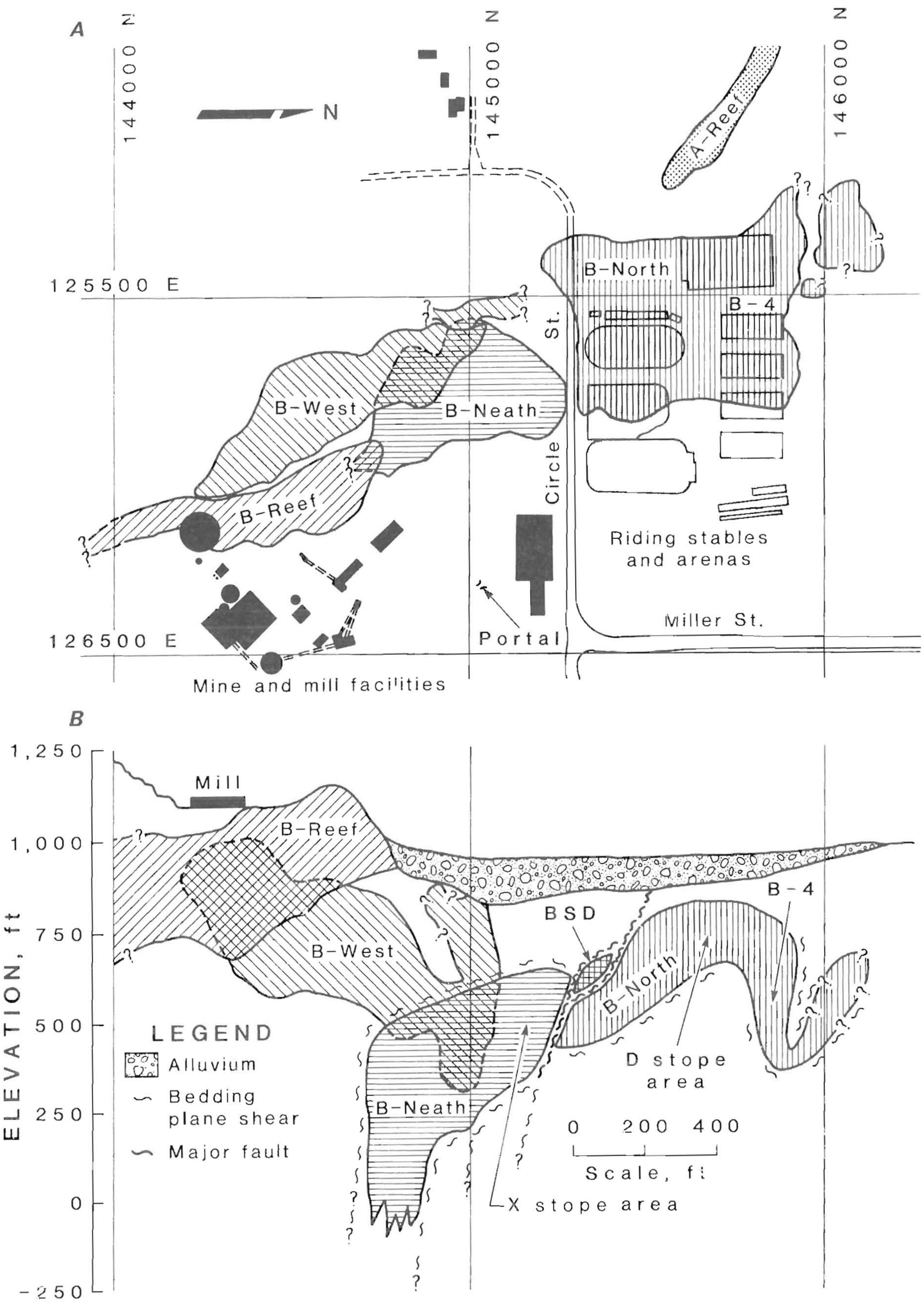


**Figure 2.—B-Reef ore zone.** *A*, **Plan view;** *B*, **cross section looking southwest (courtesy of Cannon Mine).**

The B-North ore body lies within a repetitive sequence of interbedded feldspathic sandstones, siltstones, and claystones that have been folded, faulted, and intruded by a hornblende-andesite dike to the west and a biotite-rhyodacite porphyry stock to the east. A zone of extensively sheared sedimentary beds separates the ore body from the adjacent rhyodacite intrusion, while deformed claystone or mudstone beds delineate the ore body's upper and lower limits. Ground conditions within the ore body are generally good, except near these sheared or deformed sediments. The rhyodacite intrusion served as a heat source and focusing mechanism for hydrothermal solutions that deposited gold and silver (primarily free gold, electrum, pyrargyrite, acanthite, and auriferous pyrite) throughout the fractured sandstone sediments.

Gold is disseminated within the silicified portions of the sandstone host rock in localized stocks, hydrothermal breccias, and widely spaced veins of quartz, adularia, and calcite. Although the gold is predominantly very fine grained, visible coarse-grained gold can occasionally be found in high-grade veinlets. Silver accompanies the gold at a fairly constant 2:1 ratio (Ag:Au) throughout the ore body. A detailed explanation of the geology of the B-North ore zone is presented by Ott, Groody, Follis, and Siems (*14*).

## MINE FACILITIES

The mine's portal, decline, shaft, and main access drifts were developed in the competent rhyodacite porphyry footwall to the immediate east of the ore body. A concrete-lined shaft, 18 ft in diameter and 620 ft deep, is used to hoist crushed ore out of the mine and to serve as an exhaust path for the ventilation system. A 15- by 15-ft decline with a 15-pct slope provides access to production levels; with an auxiliary portal, this decline serves as the primary ventilation intake. To help reduce surface noise, the main ventilation fan is located underground. Horizontal crosscuts lead from the main ramp to the ore body. Blasted ore is loaded onto 26-st diesel trucks using load-haul-dump units (LHD's) and hauled to an internal ore pass system that feeds an underground primary crusher. After the jaw crusher reduces the ore to a minus 5-in size, a conveyor transports the crushed ore to a storage bin and loading pocket at the shaft, where it is hoisted to the surface in two 7-st skips by a drum hoist and processed through a flotation mill. Currently operating at about a 90-pct recovery rate, the mill processes 1,500 st of ore per day, producing approximately 70 st of concentrates having an average grade of about 3.5 oz/st of gold and 7 oz/st of silver.

The mill tailings slurry is thickened to a solids concentration of 50 pct by weight and then pumped 6,000 ft up nearby Dry Gulch to a tailings impoundment area where the tailings slurry is cycloned. The cyclone underflow is deposited on the upstream face of an earthen dam that was constructed to contain the mill tailings. This 300-ft-high embankment is the largest earthen structure in the State of Washington (*17*). Designed for zero discharge to ensure that no effluents would be released from the property, the tailings impoundment system was constructed to withstand maximum conceivable flood conditions (24 in of rainfall within 24 h, a 10,000-yr weather event) at a cost of approximately $18 million (*4*).

## MINING METHOD

### PRIMARY MINING

Stopes are arranged across the ore body in parallel 24-ft-wide panels and then mined and filled in an alternating sequence as in slot-and-pillar mining at the Keretti Mine in southeast Finland (*10*). Primary stopes are excavated and backfilled before secondary mining begins; therefore, no two adjacent stopes are mined at the same time. Stopes are excavated from access drifts in 50-ft vertical intervals by driving an upper and a lower sill cut 15 ft high by 24 ft wide. After these sublevel headings are driven the length of the stope, a drop raise and a slot are excavated at the end of the stope connecting the two levels. The resulting stope block is then benched toward the access drift on the upper level while blasted ore is removed on the lower level (fig. 3). LHD's either load the blasted ore onto 26-st diesel trucks or tram the ore directly to an ore pass.

### BACKFILLING

After the entire stope block is benched, cemented fill is dumped into the open stope from an upper heading using 26-st teledumper trucks equipped with a unique telescoping bed that allows backfill to be end dumped in the limited height (15 ft) of the upper sill cut. The fill pile is leveled with 2-yd$^3$ loaders to establish a working platform from which the trucks continue to dump backfill until the excavated stope is completely filled. This sublevel then serves as the mucking level for the next vertical stope interval, and the mining sequence is repeated. When the top of the ore block is reached, cemented fill is pushed tight to the back of the top sill cut using a 4-ft² plate mounted on a beam bolted in the bucket of an LHD (fig. 4). Placing the cemented fill tight to the back of the top sill cut minimizes subsidence by enabling the backfill pillar to provide roof support as soon as secondary mining commences.

During the mine's initial stages of production, fill barriers were erected to prevent the fill from slumping into the access drift on the lower level of the open stope during backfilling. As shown in figure 3, access to the muck haulage drift was blocked by constructing timber barricades or suspending 2-in chain-link fencing across the opening with cable slings anchored by Split-Set[5] rock bolts. Instead of barricading the fill, it is now allowed to slump into the access drift, which is then cleared 6 to 8 h later with a mucking machine. If a stope block is mined through an access drift, a steel arch tunneling form is assembled before backfilling to maintain access through the fill. After the fill has cured, the tunnel forms are removed.

Consisting of approximately 55 pct coarse aggregate (minus 2-in river gravel purchased from a local commercial source), 40 pct alluvial sand (available on-site), 5 to 6 pct cement, and a water-reducing agent, cemented fill for Asamera's overhand sublevel bench stopes is mixed underground with an automatic batching system operated by a programmable controller. As shown in figure 5, the

[5]Reference to specific products does not imply endorsement by the U.S. Bureau of Mines.

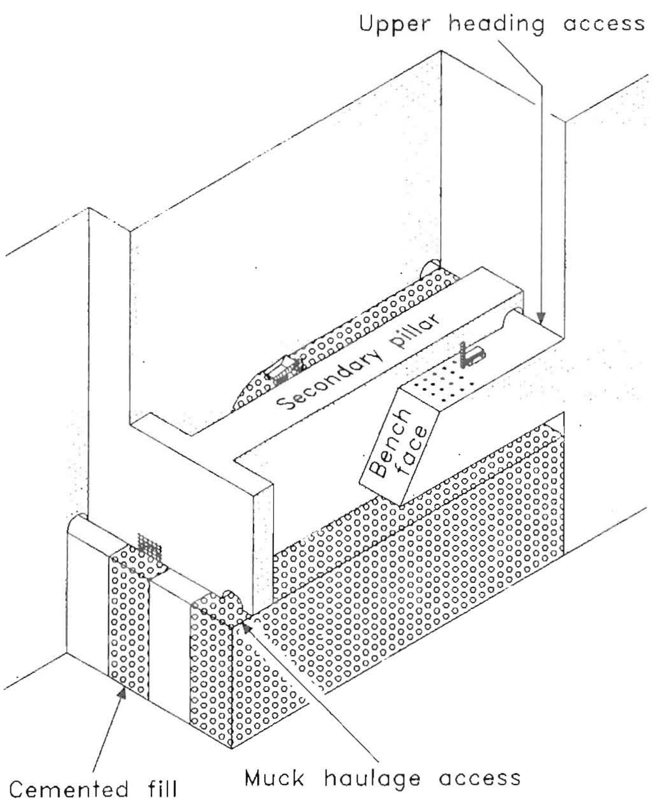


Figure 3.—Mining method schematic.

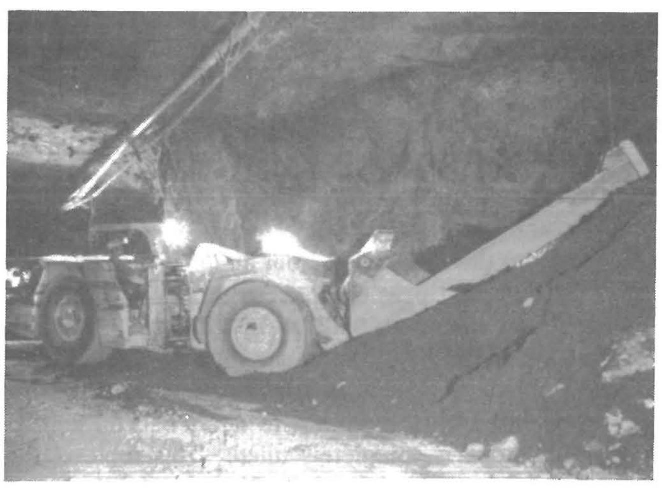

Figure 4.—Load-haul-dump unit equipped with backfill ram (courtesy of J. J. Baz-Dresch).

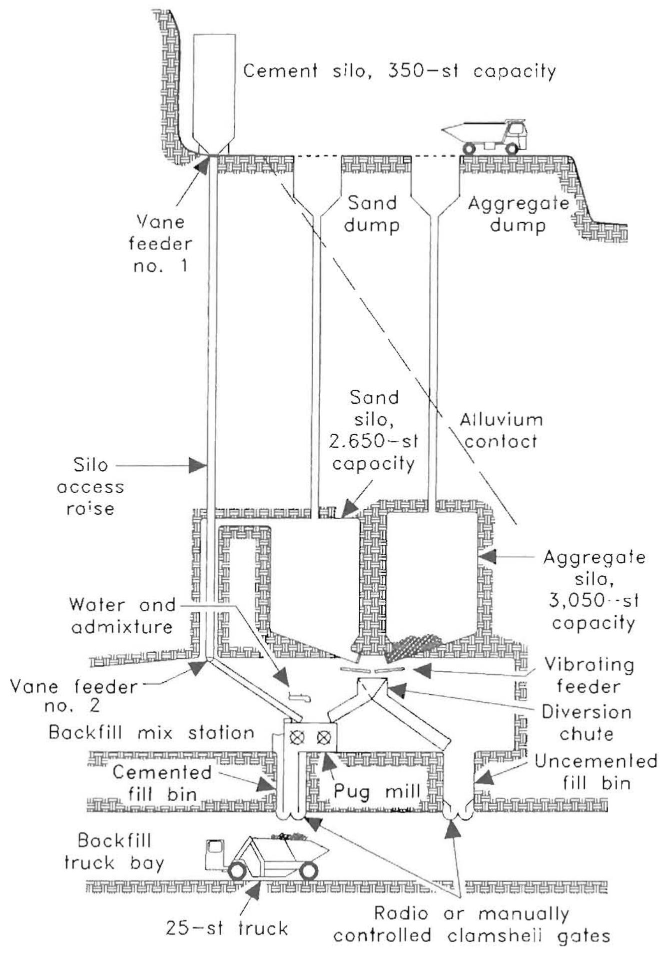


Figure 5.—Backfill plant schematic (after 2).

proper amount of each component is blended in a double-screw pug mill and discharged into a loading bin from which the backfill trucks are filled. The resulting backfill mix resembles concrete, except that it is much drier and, consequently, has a very low slump.

A 6- by 12-in cylinder of cemented backfill is cast during each shift. Uniaxial compressive strengths are approximately 1,200 psi, the design strength of the backfill. As the backfill is dumped into the open stope from the upper level, the aggregate tends to separate from the mix and collect at the toe of the stope. Although this creates a weak zone within the fill pillar, only very localized stability problems have been observed. To minimize segregation of the aggregate, the size of the coarse fraction has been reduced to a minus 2 in.

## SECONDARY MINING

After mining and backfilling the primary stopes in 50-ft vertical lifts, secondary pillars are extracted using a similar procedure, except that blasted ore is removed from between the cemented fill pillars using remote-controlled equipment. Depending on ground conditions and mining plans, the secondary stopes are backfilled with either cemented fill or waste rock. Completed stopes range in height from 30 to 130 ft, depending on their location in the ore zone.

# STUDY AREA

The North 5650 transverse cross section was chosen for the location of the instruments because the mining width-to-depth ratio (approximately 1:3.8) is greater here than at any other place in the B-North ore body. This high ratio reduces the likelihood of stresses arching to the abutments and thereby decreasing the loads carried by the pillars. Higher loads produce larger displacements and are desirable when installing instruments to ensure that displacement values are of a large enough magnitude to be within the accuracy range of the instrument's transducer.

This area of the B-North ore body also has fewer development entries for the volume being mined, it is probably less affected by adjacent mining, and it has fewer shear structures, all of which complicate numerical models. With a pillar length of up to 200 ft, displacements in a direction orthogonal to the cross section can be assumed to be zero, and pillars can be adequately modeled in plane strain. A transverse cross section is shown in figure 6, and minable blocks in the ore body are shown in figure 7 (*3*).

Because primary mining and primary stope backfilling were nearly finished at the east side of the North 5650 cross section while the instrument plan was being developed, the west and central sections of this cross section were chosen for study. A general plan was developed and modified as necessary to accommodate changes in the production schedule.

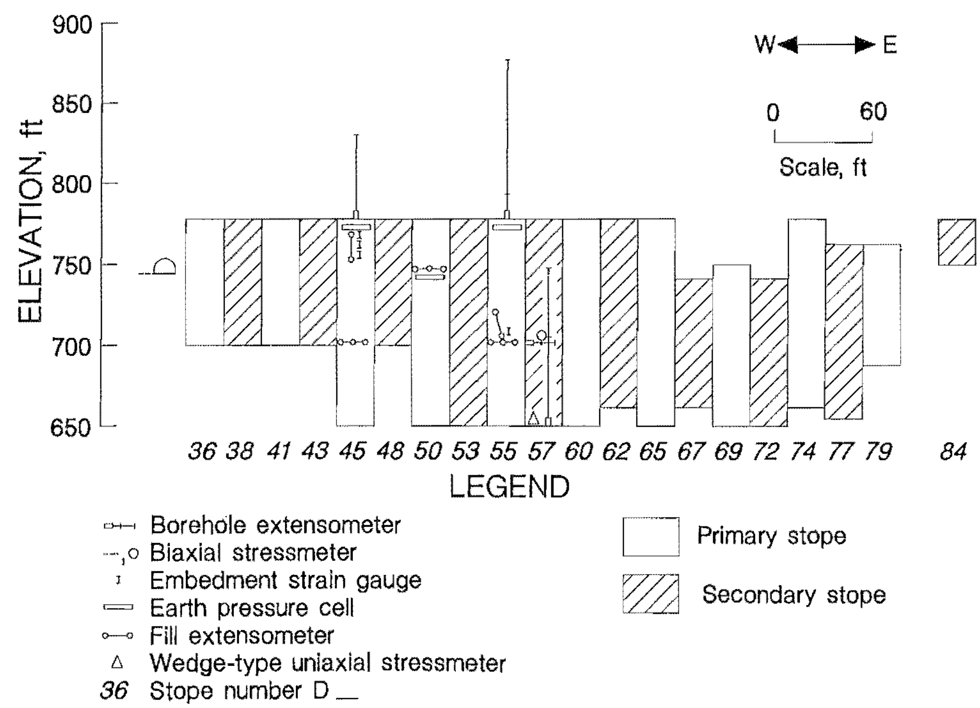


**Figure 6.—Transverse cross section of B-North ore body at North 5650.**

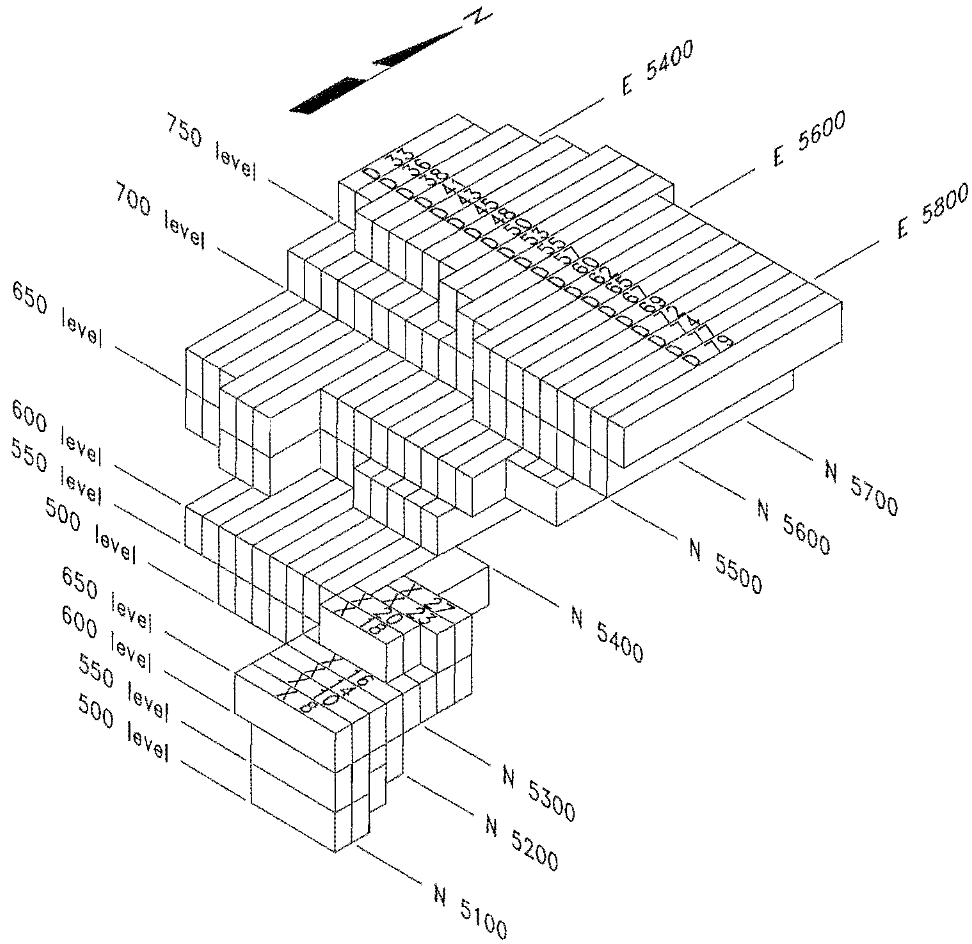


**Figure 7.—Minable blocks in B-North ore body (after 3).**

## INSTRUMENTS

Because safety and structural stability are major concerns, the mine staff installed geotechnical instruments where unstable conditions might exist. For the Bureau's study, embedment strain gauges, earth pressure cells, and two types of extensometers were placed to monitor the response of the cemented backfill, while borehole extensometers and biaxial stress meters were installed in rock pillars and abutments to measure displacements and stress changes. Instrument location, orientation, and anchor depth are shown in table 1.

Digital, vibrating-wire, remote-reading instruments were chosen because cable splices and long transmitting distances would not affect readings. In other types of instruments, changes in wire length caused by cable splicing substantially affect the electrical resistance of analog transducers and must be accounted for during data reduction. The principle behind all vibrating-wire gauges is that the natural frequency of a tensioned wire is proportional to the square root of the wire's tension. Consequently, displacements between two points can be indirectly determined by measuring the frequency of the wire before and after displacement. A coil-and-magnet assembly surrounding the tensioned wire generates a full spectra of frequencies that span the natural frequency of the wire in the range of expected tensions. When the generated and natural frequencies are the same, an alternating current (ac) voltage with this frequency is induced in the coil. This frequency is in turn transmitted through a cable to the readout box, which accurately measures the period for a given number of cycles. The readout box also can perform the mathematical functions to linearize the readings so that the strain of the wire is directly proportional to its frequency.

**Table 1.—Instrument location**

| Instrument | Location: Stope | Location: Level | Orientation, deg from horizontal | Anchor depth, ft |
|---|---|---|---|---|
| Biaxial stressmeter ....... | West rib | 740 | 0 | 9.3 |
| Do. ................ | D57 | 700 | 0 | 55.0 |
| Wedge-type stressmeter ... | D57 | 650 | 0 | 50.0 |
| Do. ................ | D57 | 650 | 0 | 40.0 |
| Embedment strain gauge .. | D45 | 755 | 90 | NAp |
| Do. ................ | D55 | 700 | 90 | Nap |
| Borehole extensometer .... | D45 | 780 | 90 | 20.2, 49.7 |
| Do. ................ | D55 | 780 | 90 | 14.6, 34.6, 64.6 |
| Do. ................ | D57 | 700 | 0 | 10.0, 20.0 |
| Do. ................ | D57 | 650 | 90 | 54.0, 99.6 |
| Backfill extensometer ..... | D45 | 755 | 90 | 17.4 |
| Do. ................ | D45 | 700 | 0 | 16.6 |
| Do. ................ | D50 | 750 | 0 | 16.6 |
| Do. ................ | D55 | 700 | 0 | 16.6 |
| Do. ................ | D55 | 700 | 56 | 27.8 |
| Earth pressure cell ....... | D45 | 780 | 0 | NAp |
| Do. ................ | D50 | 740 | 0 | NAp |
| Do. ................ | D55 | 780 | 0 | NAp |

NAp Not applicable.

To measure large displacements, the wire is attached to a spring with a constant force-displacement coefficient. Figure 8 shows the Geokon vibrating-wire transducers used in the rock and backfill extensometers. Transducers were either 2 or 4 in, depending upon estimates of expected displacements. To measure fluid pressure, the wire is attached to a diaphragm having a constant stress-displacement coefficient. Each instrument has a gauge factor that has been determined by calibration in a laboratory. Engineering units of strain, displacement, or stress are obtained by multiplying the linearized reading by the gauge factor. Relative measurements are obtained by subtracting the initial reading taken at the time of installation from a current reading and then multiplying by the gauge factor.

## ROCK EXTENSOMETERS

To install the rock extensometers, B-size diamond or 2.5-in percussion boreholes were drilled by Cannon Mine personnel in the rock several feet deeper than the instrument length. The collar of the hole was enlarged to facilitate the transducer installation in the collar anchor and to protect the head assembly from physical damage. The extensometer rods and anchors were assembled in the stope or drift and inserted in the borehole. Special setting rods were used to position the anchors at the correct depth and orientation. The anchors were then actuated by inflating the anchor bladder with hydraulic oil. The transducers were coupled to the rod end and positioned according to the amount of tensional or compressional displacement expected (fig. 9).

Pillar D57 was chosen as the secondary rock pillar in which to place rock extensometers because it would be one of the last secondary pillars to be mined. This pillar is located near the center of the cross section and would most likely take the largest load as stress was redistributed

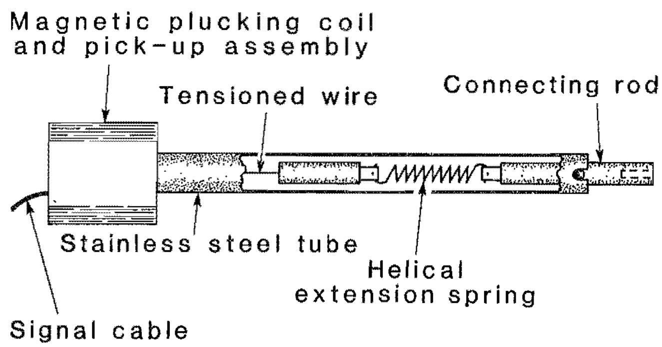


**Figure 8.—Vibrating-wire displacement transducer.**

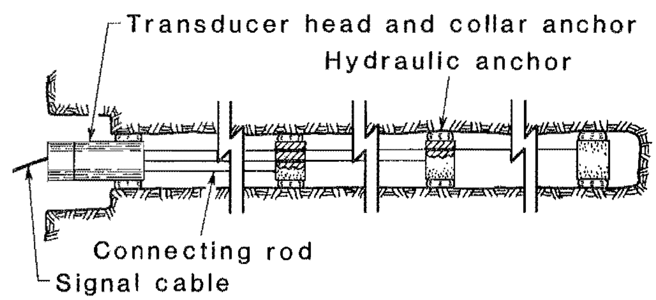


**Figure 9.—Multiple-position borehole extensometer.**

when other secondary pillars were mined. A comparison of strains from a horizontal and a vertical extensometer was intended to give an indication of the ratio of horizontal to vertical strain in the pillar and thus an approximation of Poisson's ratio for the host rock.

Vertical borehole extensometers were also installed in the back above primary backfill pillars. A Boros point anchor was used at a depth of 64.55 ft in pillar D55 to secure the extensometer in a mudstone seam. This type of anchor uses hydraulically activated arms to grip the sidewalls of a borehole. Data from these extensometers and earth pressure cells placed in the backfill near the back were to be used to indicate when the backfill was loaded by overburden.

## ROCK STRESSMETERS

Because stress or change in stress in rock cannot be directly measured by any type of available instrument, it must be determined indirectly, generally by applying elastic theory to measured rock strains to obtain stresses. This approach has been commonly applied using overcoring to determine in situ stress. At the Cannon Mine, vibrating-wire biaxial stressmeters were installed because minor and major principal stress changes and orientation can be determined for the plane perpendicular to the axis of the borehole with one instrument. This is possible because the instrument has two sets of three vibrating-wire gauges oriented at 60° angles from each other. The gauges measure radial deformation of a borehole and transfer the measurements directly to a stressmeter through a thin grout annulus (fig. 10).

To install the biaxial stressmeters, a B-size, diamond-drilled borehole was slightly declined from horizontal just past the desired depth. A special high-strength, non-shrinking grout was then pumped into the hole until the hole was completely filled. A special installation rod equipped with a mercury position-sensing switch and an audible signal device was used to push the stressmeter into the grout and orient one of the vibrating-wire gauges vertically. Before removing the installation rods, the stressmeter was secured by pulling a cable-activated pin that released a snap-ring anchor. Laboratory tests determined that the average elastic modulus of the grout was 1,700,000 psi.

One biaxial borehole stressmeter was installed at the 700 level of pillar D57, 55 ft into the pillar from an access drift, so that the stress readings would be in the same plane as readings from all other instruments. Mine personnel also installed two wedge-type uniaxial borehole stressmeters at North 5550 on the 650 level to monitor vertical stress. Another biaxial borehole stressmeter was installed in the west abutment to measure the load distributed to the abutment because of pillar removal. Orientation of this instrument was in a plane perpendicular to the cross section shown in figure 6.

## BACKFILL EXTENSOMETERS

Two types of backfill extensometers, horizontal and vertical, were constructed in the open stopes while the stopes were being filled. The horizontal extensometers were made by connecting the ends of two borehole jointmeters so they would almost span the width of the stope. As shown in figure 11, these extensometers consisted of an inner connecting rod and an outer polyvinyl chloride (PVC) jacket with a slip joint that allowed movement between end plates.

In the field, a steel cable was attached to each rib across the stope and the entire assembly was hung in place. The PVC jacket of each jointmeter was wrapped with brattice cloth to prevent concrete from adhering and to ensure that movement of the backfill would not be restricted by the extensometer. Because of the flexibility of the jointmeter, the instrument had to be handled carefully to prevent damaging the displacement transducer during

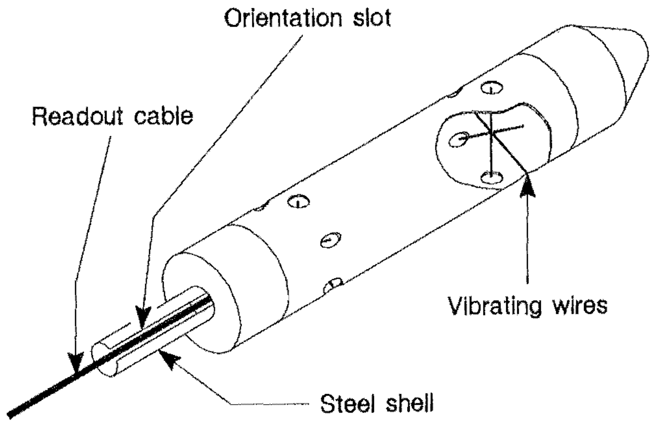


Figure 10.—Vibrating-wire biaxial stressmeter.

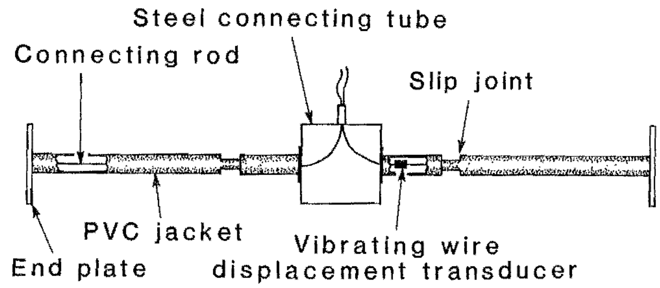


Figure 11.—Horizontal backfill extensometer.

installation. As backfill was dumped from the bench above, the fill slope surged forward unevenly. Consequently, the backfill distorted this flexible instrument into a slight arc, which may have damaged the transducer and caused erroneous readings. In early installations, all the backfill instrument readout cables were threaded through 3/4-in steel pipe to prevent the cables from being cut or damaged. Because the aggregate was uncrushed rounded rock, the added protection was found to be unnecessary, and later installations were made by directly burying the cables in the backfill.

The vertical extensometers were fabricated at the Spokane Research Center. They consisted of a steel base plate and a pipe encased in loosely coupled PVC pipe, which prevented backfill from adhering to the inner steel pipe (fig. 12). Sections of steel and plastic pipe were added as the stope was filled until the extensometer reached the desired length (fig. 13). The head assembly, consisting of a displacement transducer, slip joint, and embedment anchor, was then connected to the top of the vertical pipe section. Relative displacements between the base plate and the anchor plate were measured by the displacement transducer to determine strain.

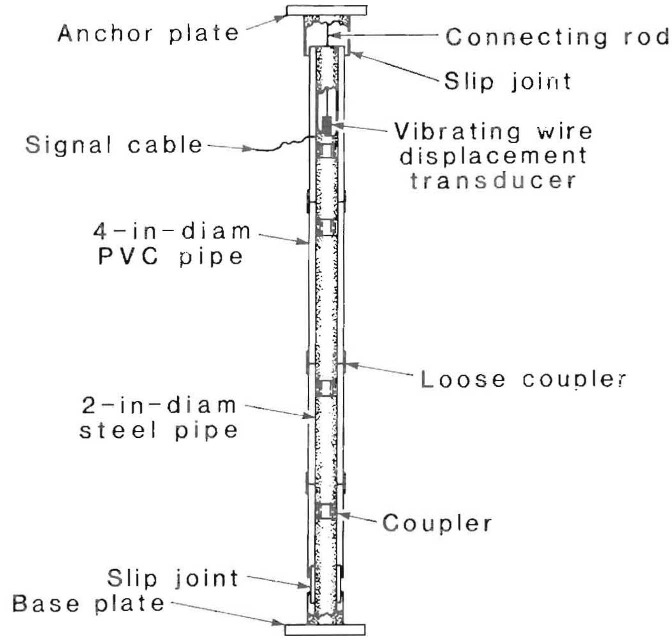


**Figure 12.—Vertical backfill extensometer.**

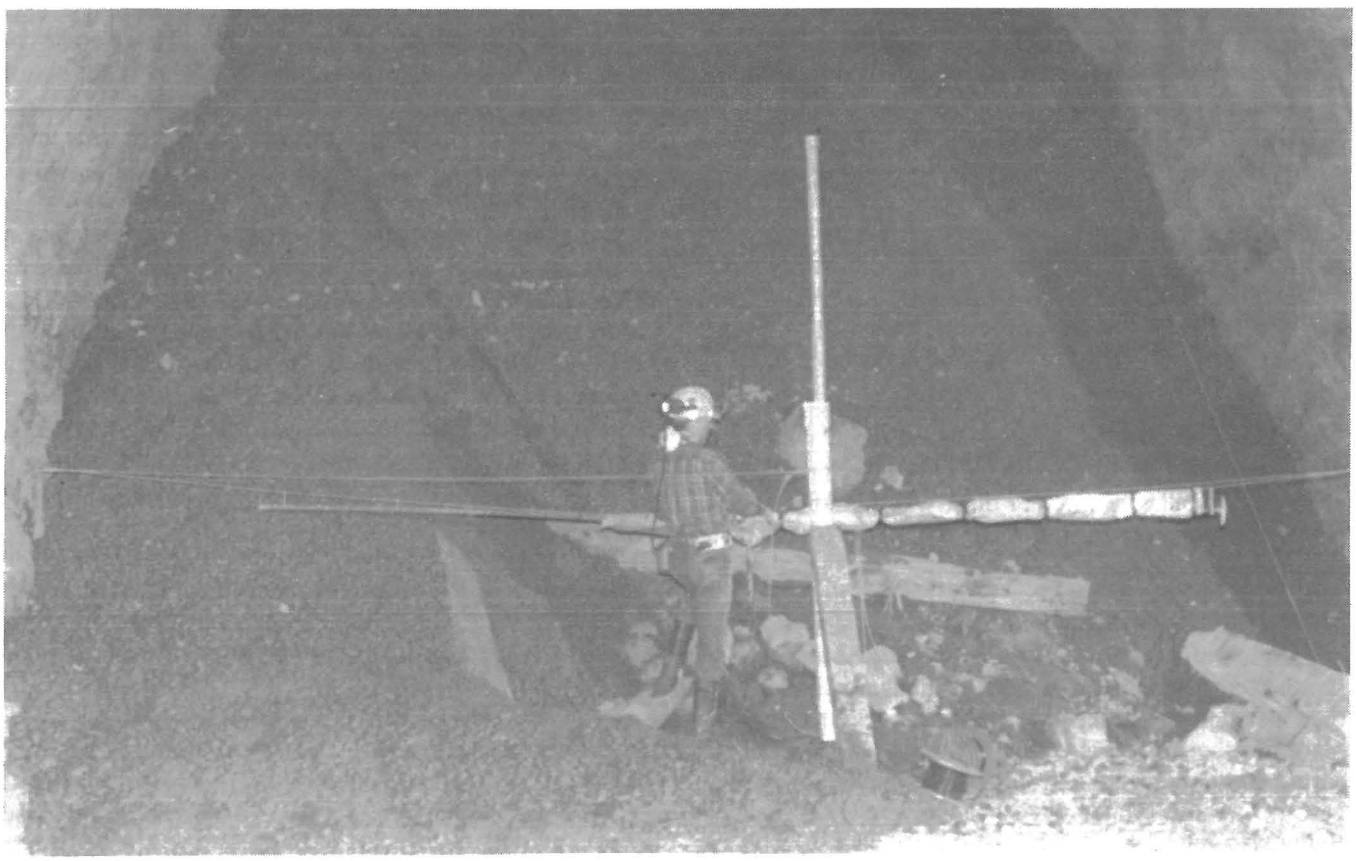

**Figure 13.—Partially assembled horizontal and vertical backfill extensometers at toe of cemented backfill.**

Installation of the vertical extensometers began by attaching 10 ft of steel and plastic pipe to the base plate and then burying it with about 5 ft of backfill. Within a few hours, the fill had set sufficiently to allow the additional sections of pipe to be added and filling to be resumed. If filling progressed too fast, the fill would build up and then the slope would fail, causing the pipe to bend. Steel cables strung from the ribs provided support to the pipe. It was found that if the cables were left attached and backfill covered them, the weight of the fill would pull on the cables. In stope D55, one of the support cables broke, and the tension of the remaining cable bent the pipe 34° from its vertical orientation. Another vertical extensometer was partially constructed at the 700 level in stope D45 but was not completed because the weight of the sloping backfill caused the instrument to fail at a pipe joint. Therefore, installation was best accomplished by allowing the fill to achieve initial set after each 5- to 10-ft lift and by never allowing the support cables to be buried.

Cemented backfill pillars D45, D50, and D55 were selected for instrumentation because they were located near the center of the North 5650 cross section and would be mined to a height of 130 ft, the maximum vertical extent of the ore body.

## EMBEDMENT STRAIN GAUGES

Vibrating-wire embedment strain gauges were installed in the cemented backfill to measure vertical strain over the 6-in length of the gauge. These instruments are capable of measuring up to 1,500 $\mu\epsilon$ in either tension or compression, yielding a maximum strain of 3,000 $\mu\epsilon$. The gauge wire is attached to two end plates and is surrounded by a stainless steel tube and a magnetic plucking coil and pick-up assembly (fig. 14). Because the gauge wire is strained by the same amount as the medium in which it is embedded, the measurement capability of this instrument is limited to the maximum elastic strain of the gauge wire.

The gauges were placed at different elevations near the longer vertical backfill extensometer in D45 (fig. 6) to determine whether strain was uniform throughout the height of the pillar or whether there was a gradient. Only one gauge was placed near the long extensometer in pillar D55, near the base of this instrument.

Because of the volume of backfill, the height of the stopes, and the large size of the aggregate, accurate orientation of the small gauges could only be achieved by first casting them in 6- by 12-in cardboard cylinders and then positioning the cylinders as the backfill was being placed in the stope. The plus 1/2-in aggregate was hand sorted from the cemented backfill before the gauge was cast. Once the cylinder had been covered with backfill, filling was generally stopped to ensure that the cylinders would remain vertical.

## EARTH PRESSURE CELLS

Three 9-in-diam earth pressure cells were cast in the cemented backfill to measure relative changes in vertical stress and to indicate when the backfill began to carry overburden loads (fig. 15). The cells in D45 and D55 were cast directly in the backfill while the other was cast in a 1-yd$^3$ wooden form in the stope. Once the backfill had set, the form was removed and the cube was buried.

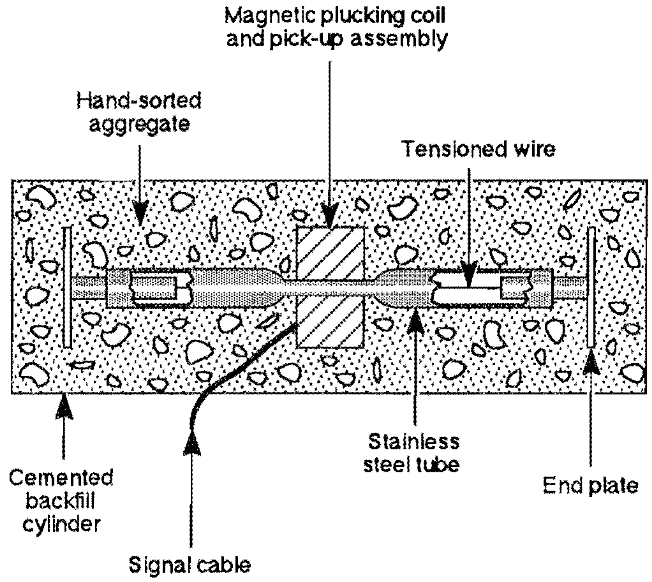


**Figure 14.—Vibrating-wire embedment strain gauge cast in backfill cylinder.**

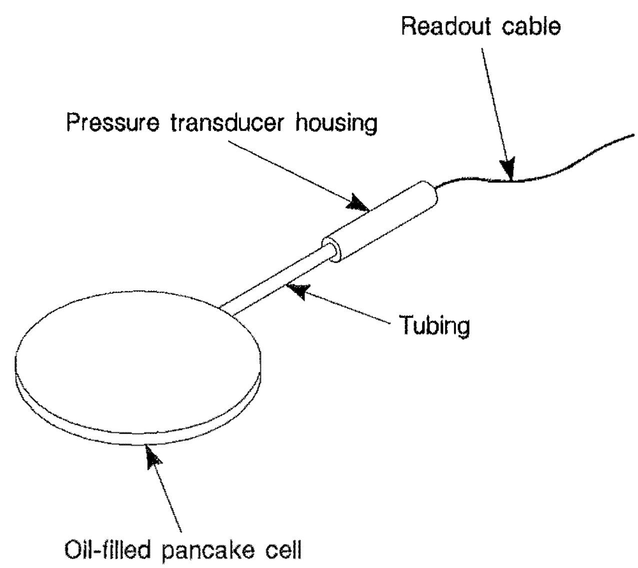


**Figure 15.—Earth pressure cell.**

Backfill stress was determined by correlating the pressure of the hydraulic oil within the flatjack cell to an enclosed vibrating-wire pressure transducer that had been calibrated before assembly of the cell. Backfill surrounding the cell was screened to minus 1/4 in to minimize the chance that large pieces of aggregate would point load the flatjack.

## INSTRUMENT PERFORMANCE AND RESULTS

If active mining was going on one or two stopes away from the instruments, the instruments were read daily by Cannon Mine personnel. Otherwise, readings were taken weekly. Approximate excavation and backfilling dates for headings and benches are shown in figure 16. The data were entered in computer spreadsheets, reduced to engineering units, and plotted. The plots of the reduced data are included in figures 17 to 28. Tensile stress and displacement caused by tension are positive. Major principal stresses are algebraically the largest. Conversely, minor principal stresses are algebraically the smallest, but they are the largest compressive stresses.

The vibrating-wire transducers proved to be reliable and versatile, and approximately 50 pct of the instruments still functioned after 2 yr of use. Cable splices were required during the installation of several instruments, but these splices did not cause any degradation in instrument reading capabilities or a need to adjust data reduction formulas. Causes of instrument failure included cut or inaccessible readout cables, transducers exceeding their planned range, or mining at the instrument location. For some instruments, no specific reason for failure could be identified. Clustering instruments prevented complete loss of data when one instrument failed and provided a data check when they all functioned.

By installing instruments in both the rock pillars and the cemented fill pillars, relationships between instrument data sets were identified as mining proceeded. For example, the vertical backfill extensometer in stope D55 was compressed when the adjacent heading was mined in secondary rock pillar D57, indicating that the backfill was taking additional load (fig. 17). At the same time, the earth pressure cell in cemented fill stope D50 recorded an increase in stress of approximately 150 psi (fig. 18), while the vertical extensometer in the rock pillar extended, indicating a release in stress (fig. 19). The two wedge-type uniaxial stressmeters also recorded decreases in stress (fig. 20).

This stress redistribution did not occur at the same time in all parts of the cross section. After the heading of pillar D57 was excavated, deformation continued in cemented backfill pillar D55 for 7 months (fig. 17). However, the abutment did not show the effects of excavation until 2 months later (fig. 21). There was no immediate response to mining recorded by the vertical fill extensometer in stope D45, but the instrument indicated that the backfill continued to take load (fig. 22).

The horizontal backfill extensometers in pillar 50 provided little information for a global evaluation of the mining system because one of the transducers was damaged during installation and the other transducer failed several days later, as did one of the horizontal extensometers in pillar D55. In these latter two cases, failure could have been caused by the weight of the backfill bending the steel extension rod. Plots of these instruments are not included here.

The horizontal extensometers in pillar D45 recorded an average initial compressive strain of approximately 410 $\mu\epsilon$, some of which could have been caused by backfill shrinkage[6] (fig. 23*A*). Displacements changed from compressive to tensile when adjacent pillar D48 was benched. The horizontal extensometer on the western side of stope D55 also recorded compressive displacement (fig. 23*B*), but the time of these occurrences could not be related either to initial backfill shrinkage or a specific mining event.

Coupling stress readings from the earth pressure cells at the top of the cemented backfill stopes with readings from the vertical extensometers placed in the back above the stope did not indicate when the backfill began to take load, which was the purpose of installing these instruments. Readings from the pressure cell in D45 did not show a significant increase in stress (fig. 24*A*), and the earth pressure cell in D55 stopped working 5 months after it was installed (fig. 24*B*). However, vertical backfill extensometers in the lower levels and an earth pressure cell in D50 indicated that the backfill was taking load.

The three-point borehole extensometer in the back of stope D55 failed before the heading in D57 was mined. Anchored in weak, unsilicified material, the 34.6- and 64.6-ft-long anchors may have slipped down the borehole, as evidenced by the fact that the transducers recorded less strain than the 15.1-ft anchor in silicified sandstone (fig. 25).

An in situ modulus of 115,140 psi was estimated for the cemented backfill by dividing the stress indicated by the

[6]Tests by the U.S. Bureau of Reclamation have indicated that shrinkage during drying is approximately 200 $\mu\epsilon$ after 1 month of curing (*11*).

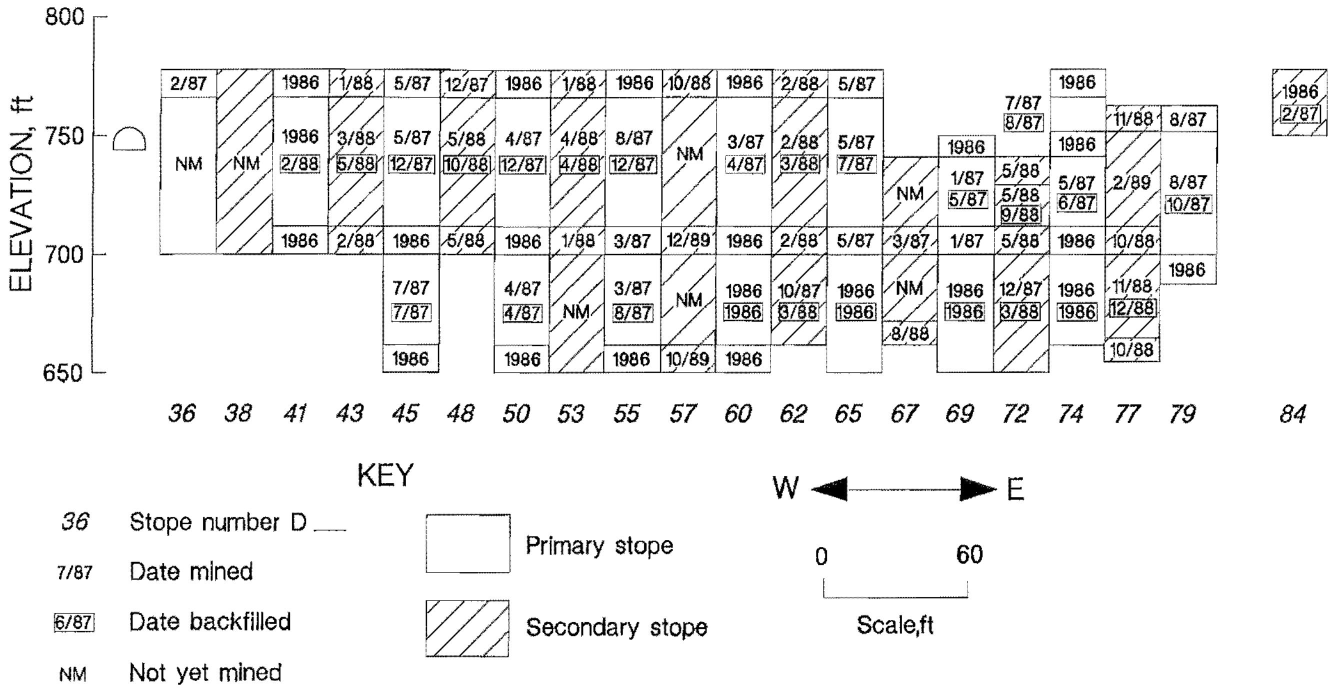


**Figure 16.—Excavation and backfill dates at North 5650.**

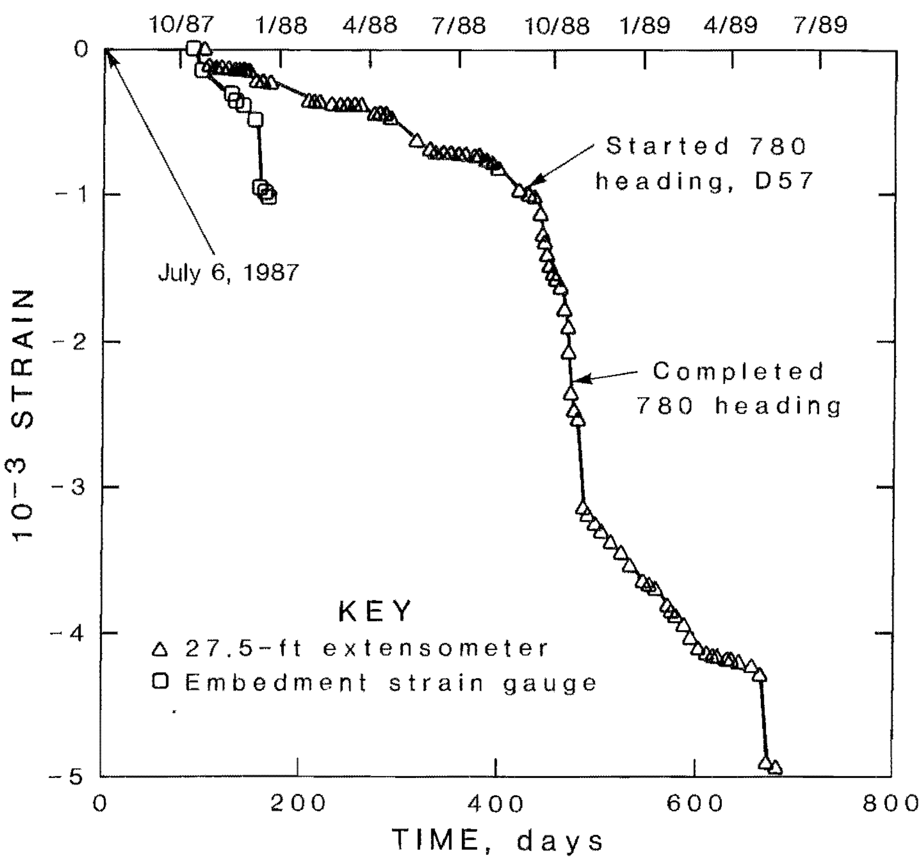


**Figure 17.—Plot of embedment strain gauge and 27.5-ft vertical backfill extensometer measurements in stope D55, 700 level.**

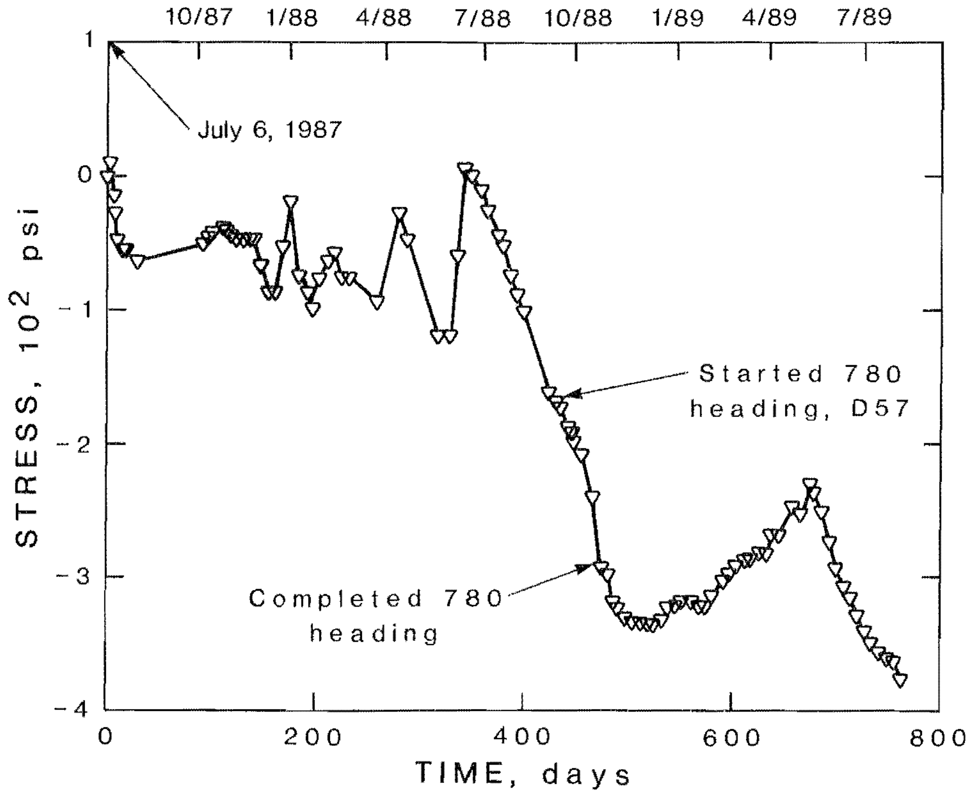


Figure 18.—Plot of earth pressure cell measurements in stope D50, 740 level.

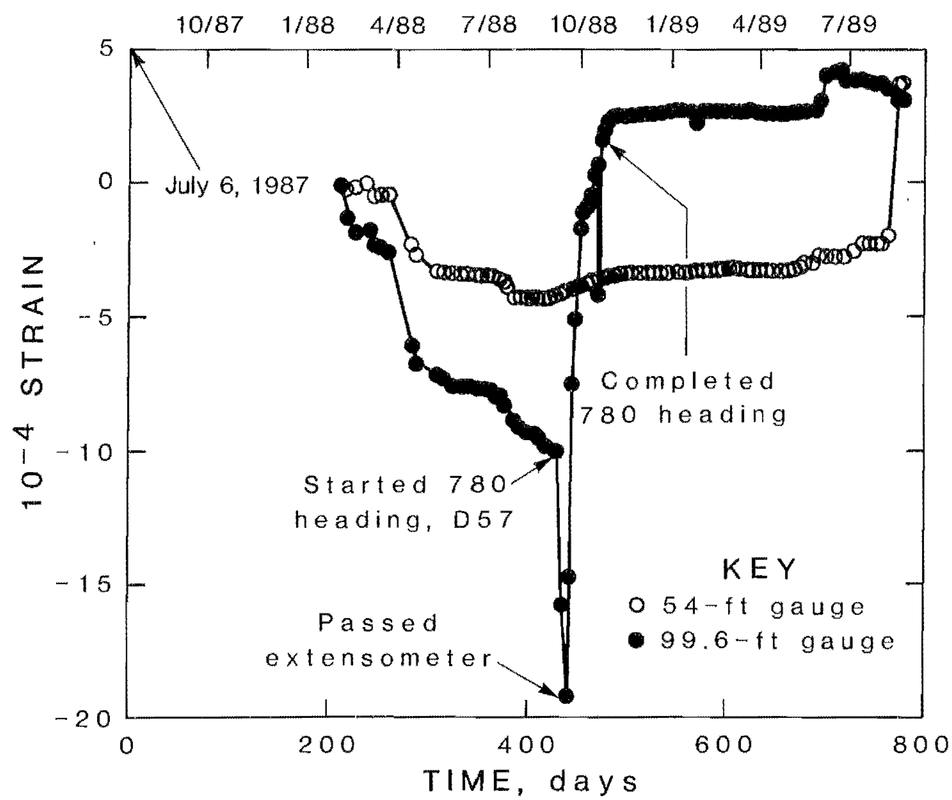


Figure 19.—Plot of vertical borehole extensometer measurements in pillar D57, 650 level.

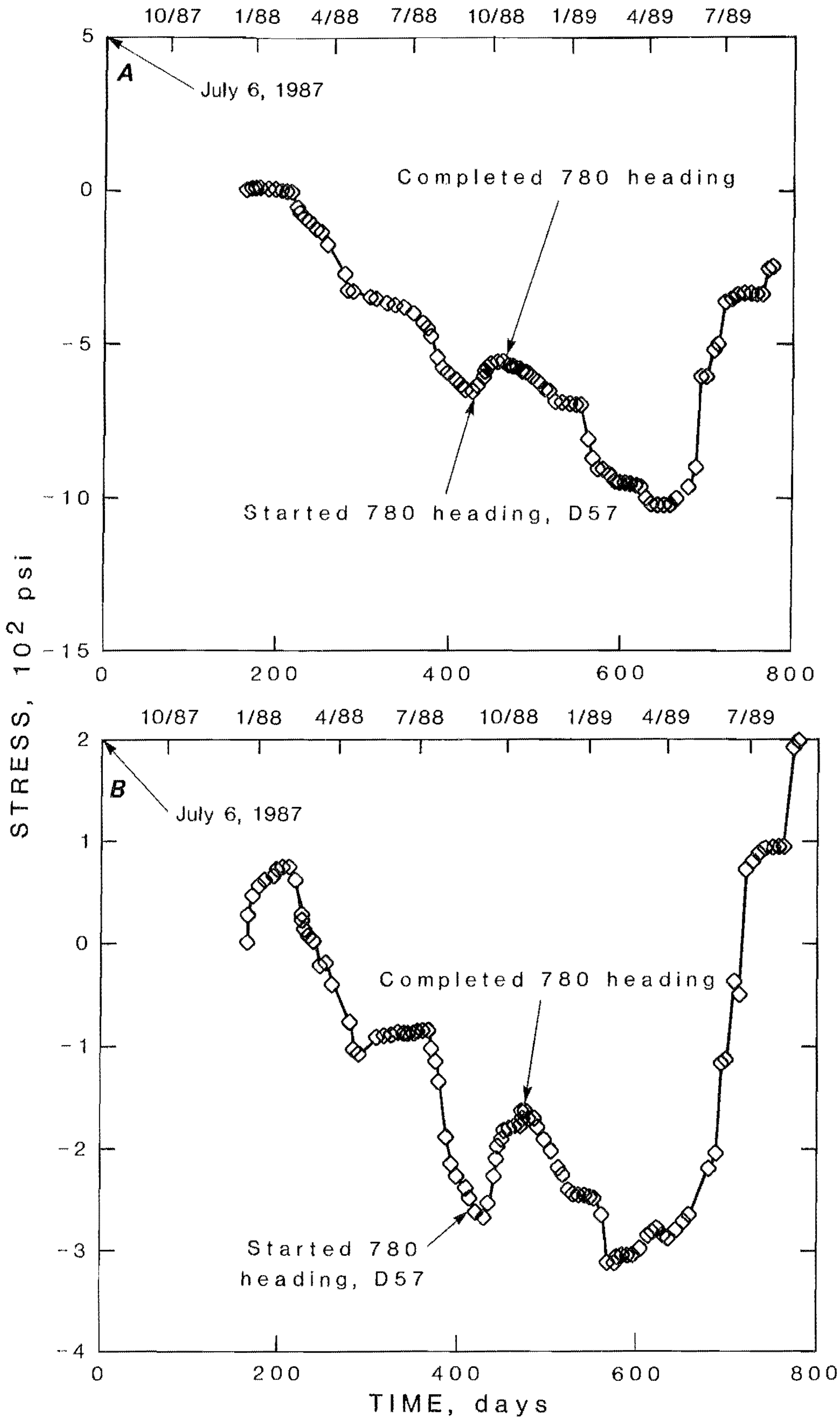


**Figure 20.—Plot of uniaxial stressmeter measurements in pillar D57, 650 level.** ***A*****, No. 1;** ***B*****, no. 2.**

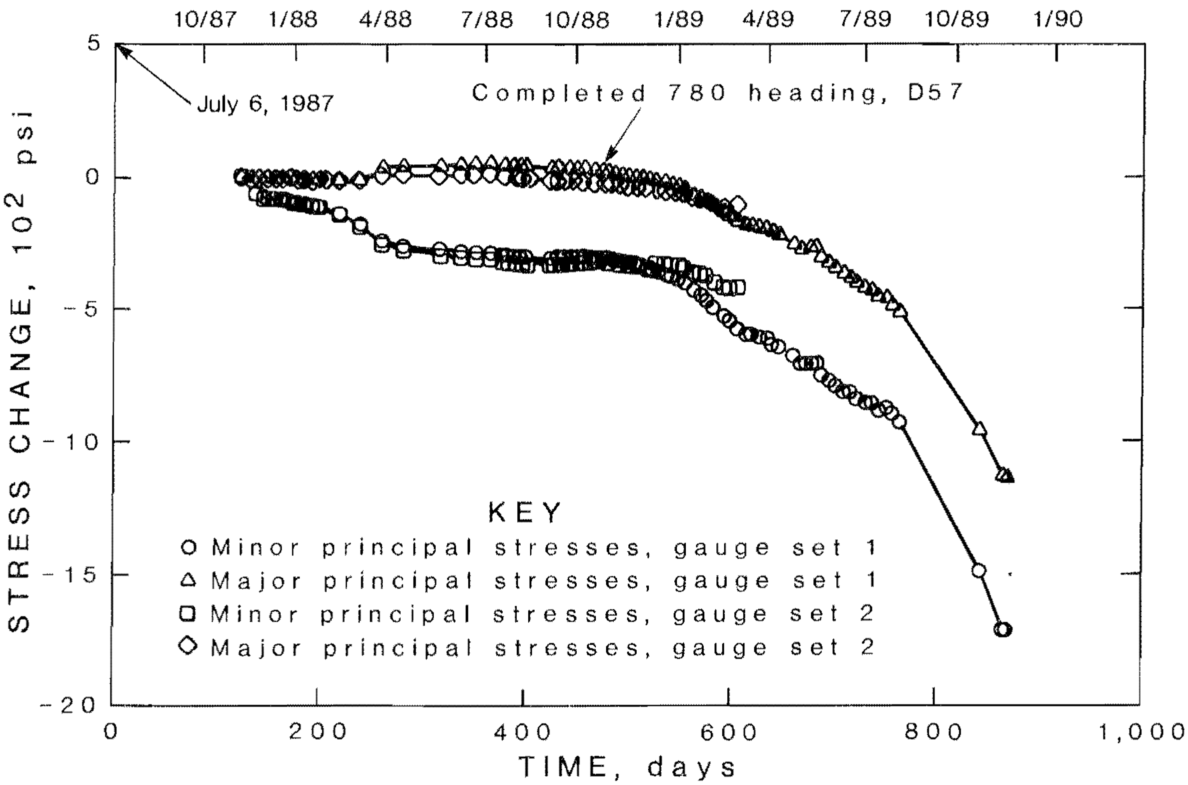


**Figure 21.—Plot of biaxial stressmeter measurements of principal stresses in west abutment.**

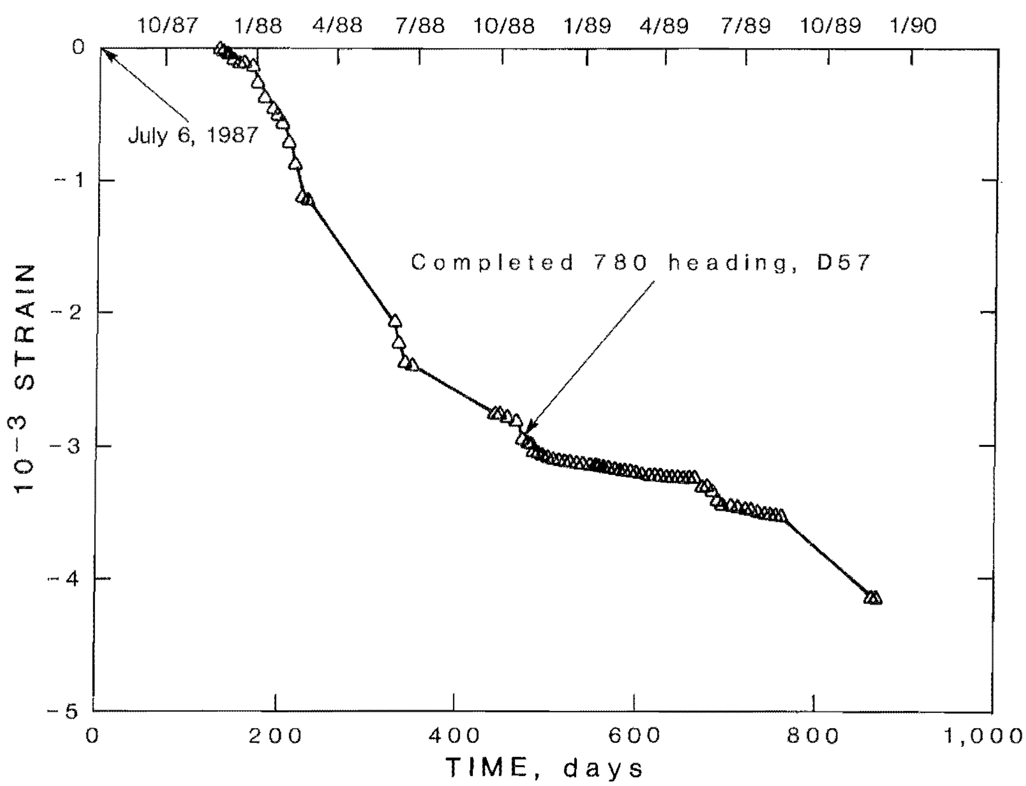


**Figure 22.—Plot of 17.4-ft vertical backfill extensometer measurements in stope D45, 755 level.**

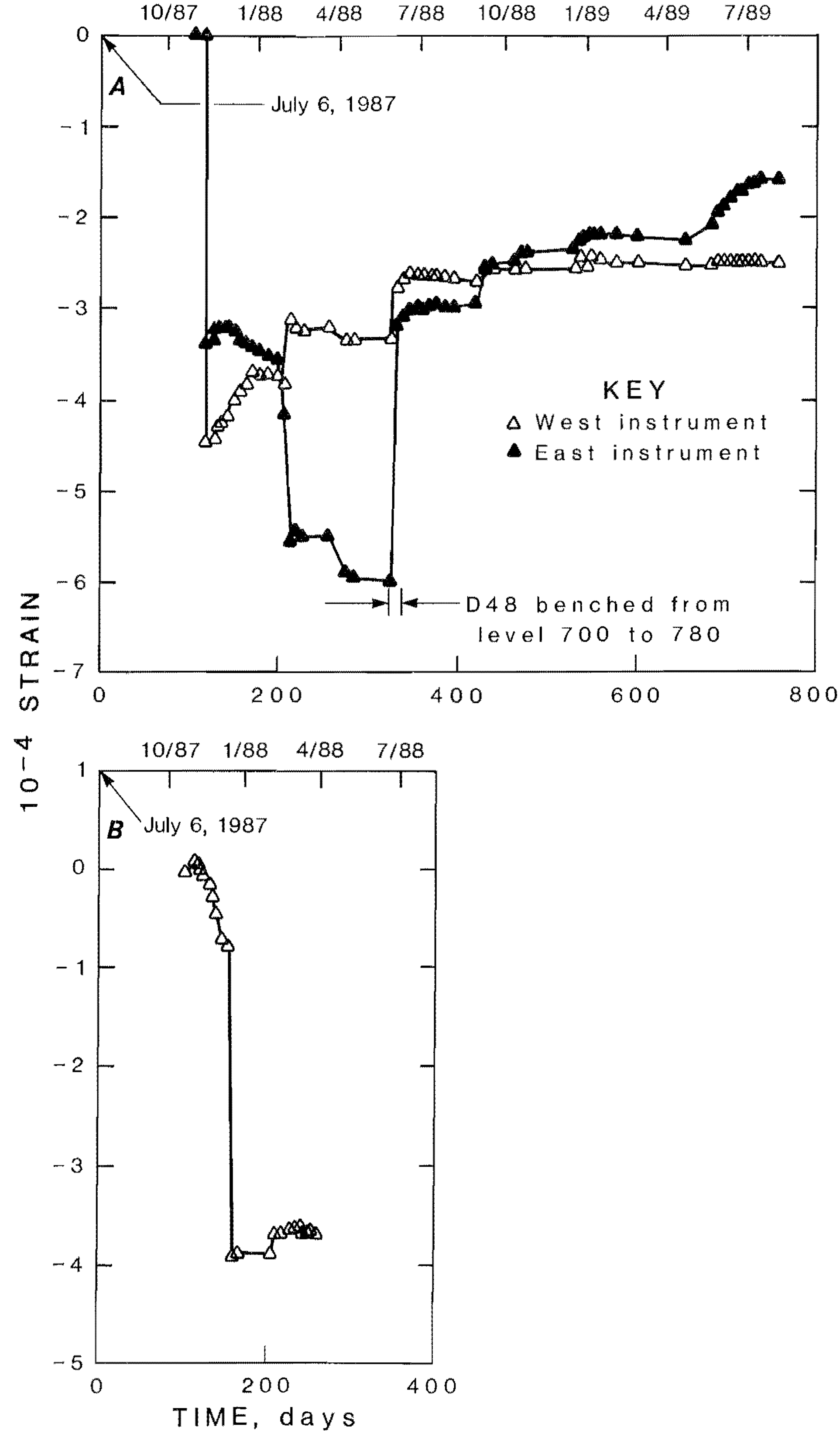


**Figure 23.—Plot of horizontal backfill extensometer measurements. *A*, Stope D45, 700 level; *B*, west side of stope D55, 700 level.**

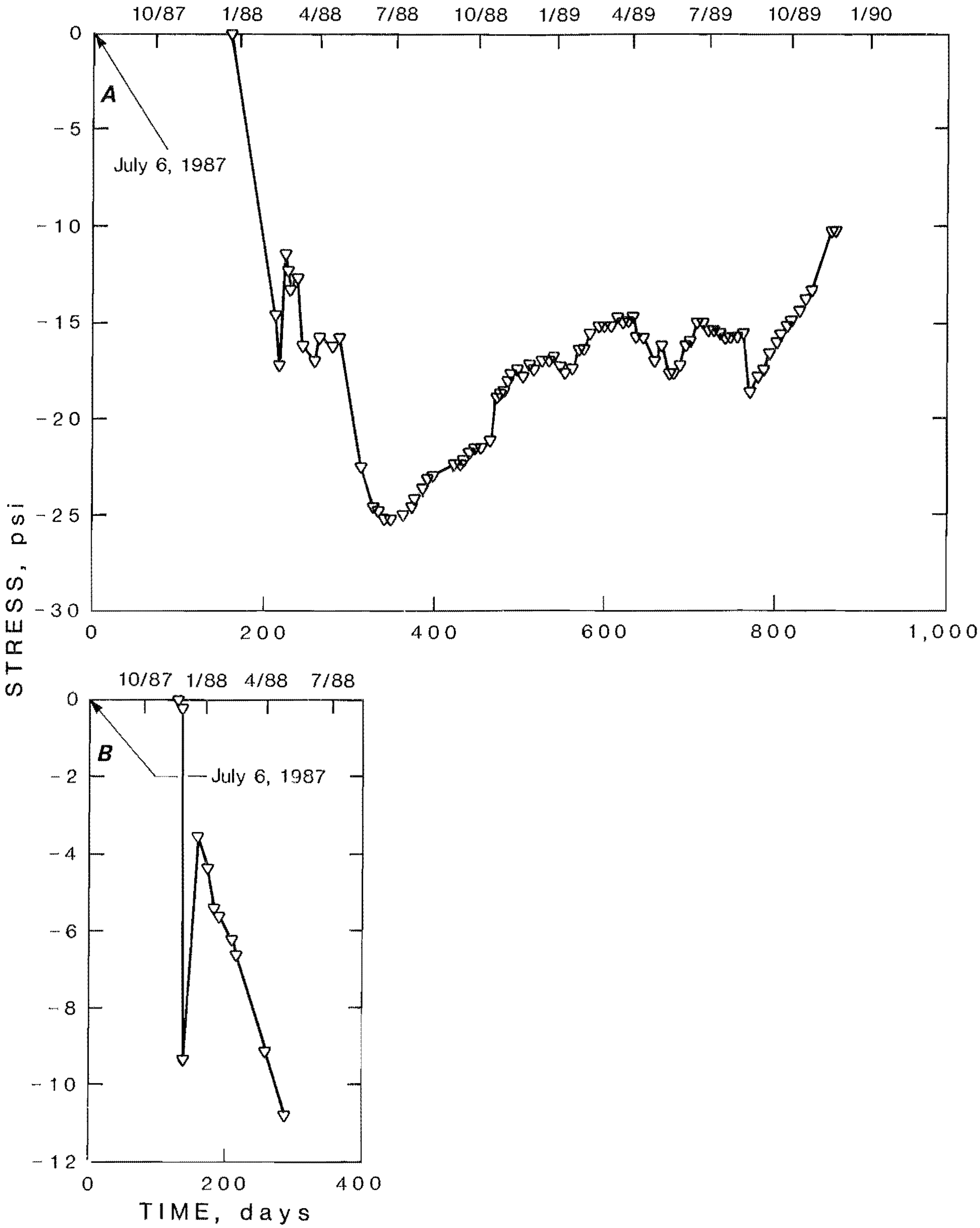


**Figure 24.—Plot of earth pressure cell measurements.** ***A*****, Stope D45, 780 level;** ***B*****, stope D55, 780 level.**

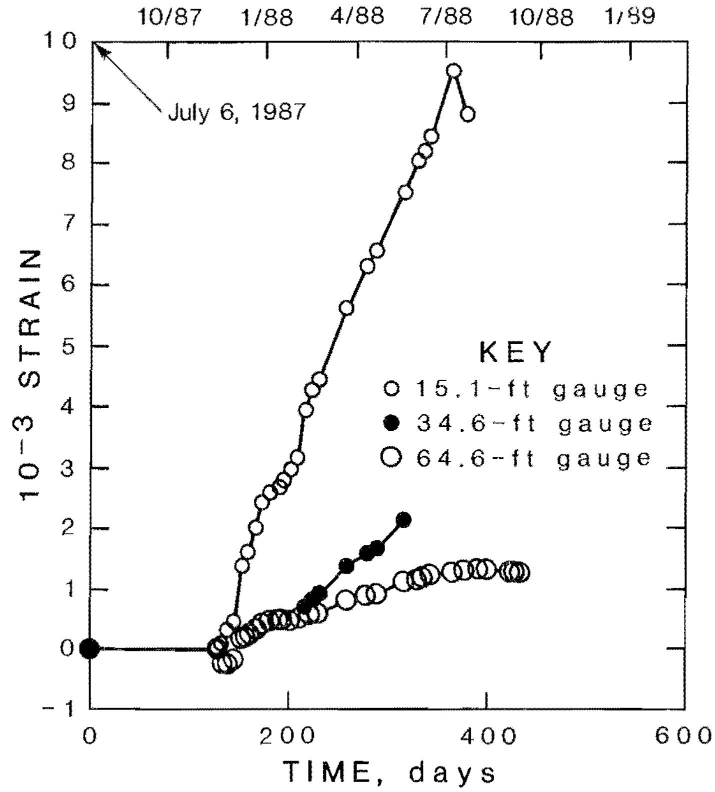


**Figure 25.—Plot of vertical borehole extensometer measurements in stope D55, 780 level.**

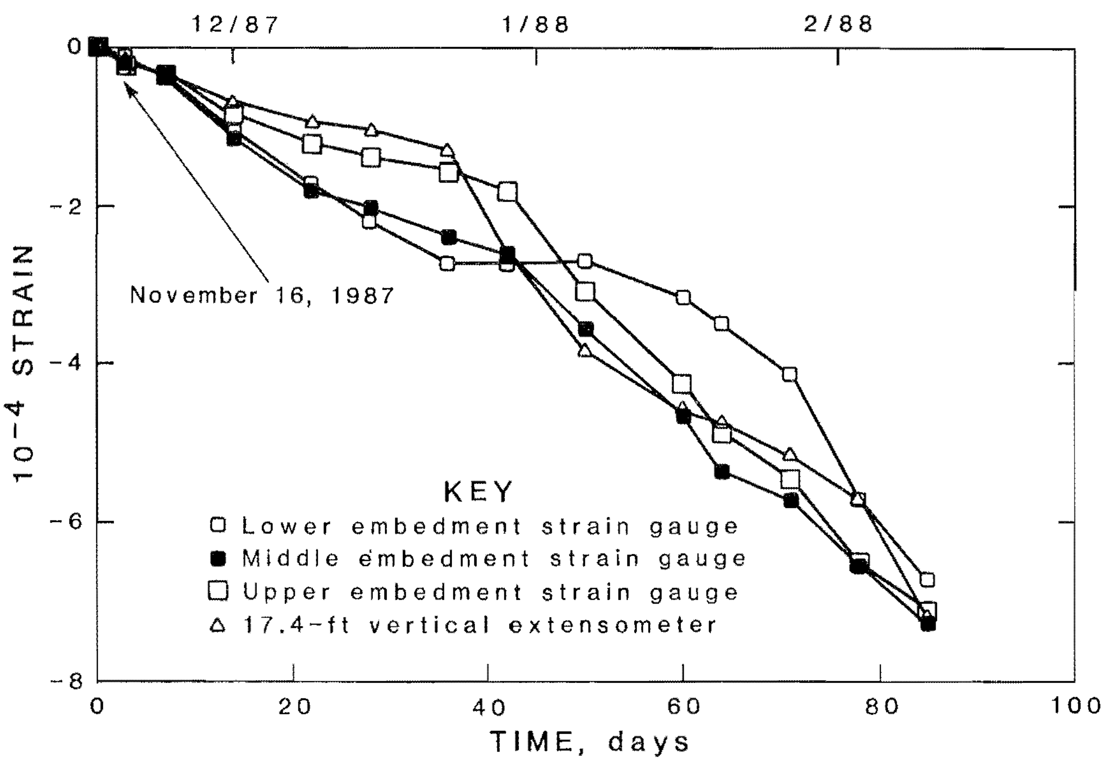


**Figure 26.—Plot of embedment strain gauge and 17.4-ft vertical extensometer measurements in stope D45, 755 level.**

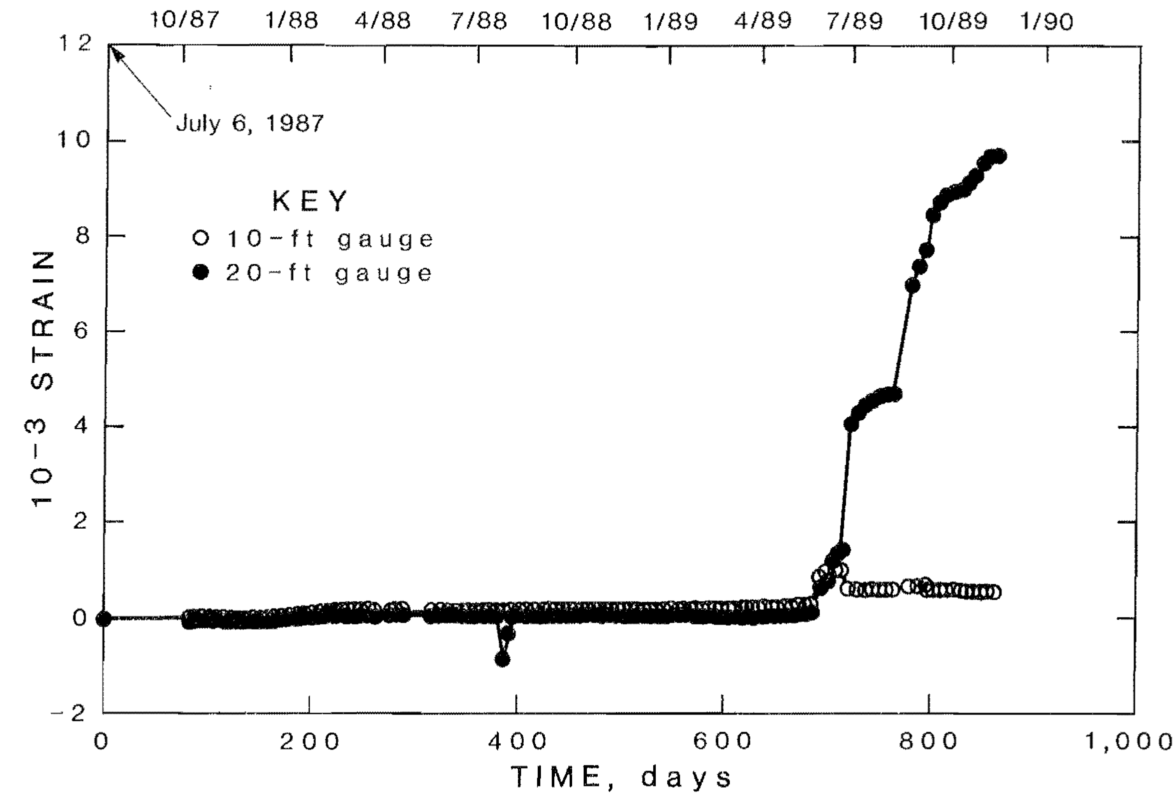


**Figure 27.—Plot of horizontal borehole extensometer measurements in pillar D57, 700 level.**

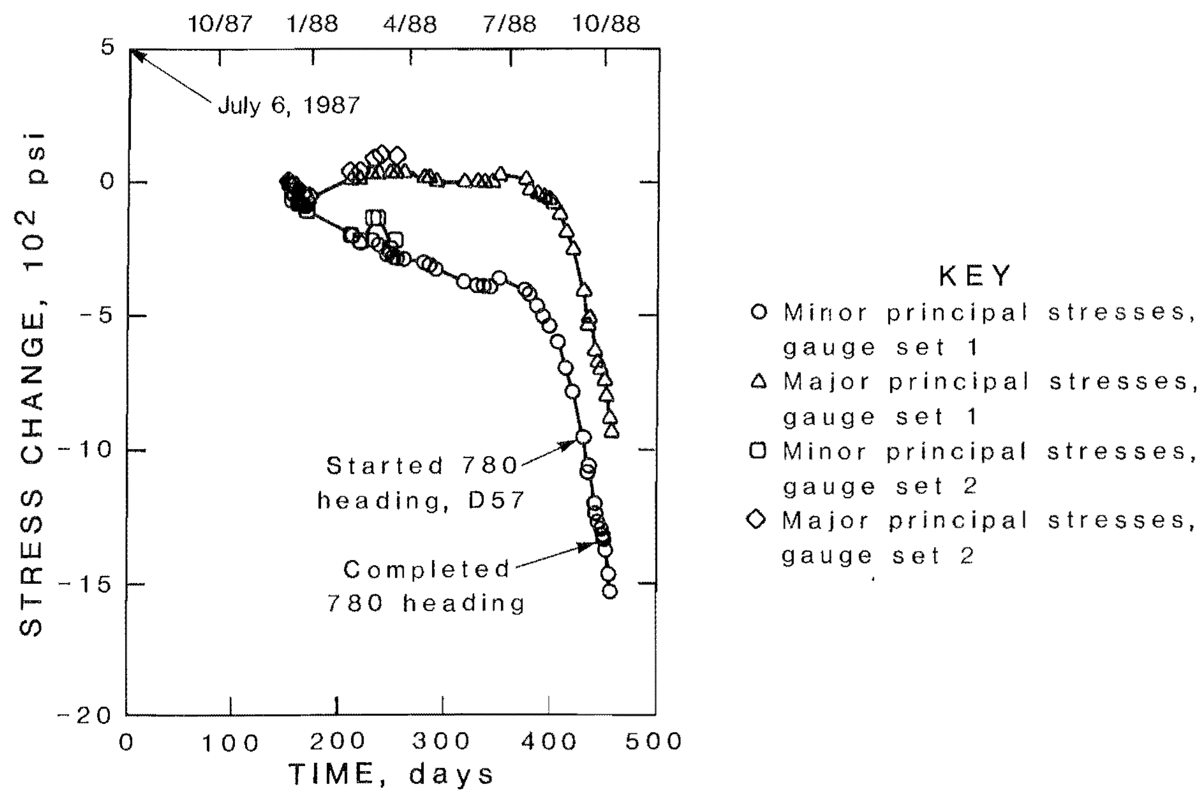


**Figure 28.—Plot of biaxial stressmeter measurements of principal stresses in pillar D57, 700 level.**

earth pressure cell in D50 by the strain measured by the vertical backfill extensometer in pillar D55. This estimate included data collected up to November 22, 1989, after the D57 heading had been excavated, when the stress measured by the earth pressure cell in D50 stabilized. Data collected after this time were not included because the pressure cell indicated an unexplainable unloading of the backfill. The total stress was 381 psi, which included 335 psi as measured by the earth pressure cell at the 700 level and an additional 46 psi for the 50 ft of cemented backfill between the 700 and the 750 levels. Strain was -0.003309.

By using the last strain reading (-0.005) available from the large vertical fill extensometer before its cable became inaccessible (fig. 17), an estimate of stress at the 650 level of stope D55 was made. Using an in situ modulus of 115,140 psi, the stress was 576 psi at the base of the instrument. An additional 50 ft of backfill at 132 lb/ft$^3$ gave a measurement at the 650 level of 622 psi. Visual inspection of the cemented pillar did not reveal any apparent degradation.

At the end of the first 3 months following installation, readings from the embedment strain gauges and the vertical fill extensometer in stope D45 gave the same strain rate. Figure 26 is a plot of strain changes for all four instruments from November 16, 1987, to February 9, 1988. The upper embedment strain gauge exceeded its capacity at 700 $\mu\epsilon$ and the lower instruments about 1,100 $\mu\epsilon$ (not shown in fig. 26). This similarity in readings suggests that embedment strain gauges could be used in place of a longer vertical fill extensometer, thus saving installation time and costs.

There is evidence that pillar D57 failed or spalled after the 780 heading was driven. As shown in figure 27, the horizontal strain recorded by the 20-ft anchor of the extensometer on the 700 level increased rapidly from May to December 1989. The final value of 0.0097 is equivalent to 2.3 in. It is hypothesized that the pillar split in half, because the 10-ft anchor did not record a significant change in strain. Numerous vertical tension cracks had been observed when the backfill extensometer was installed in adjacent pillar D55. One of these cracks could have propagated to produce the abrupt change in strain rate. Visual observations during the driving of the 700 heading revealed several fractures and blocky ground conditions. Stressmeters installed in the pillar at the 650 level at North 5550 showed evidence of pillar failure when readings of stress decreased rapidly at the beginning of May 1989 (fig. 20). Although both sets of gauges in the biaxial stressmeter in pillar D57 failed (although not simultaneously) before the heading was completed, the instrument recorded changes in loading rate starting in mid-July of 1988 (fig. 28). The angle of the minor principal (largest compressive) stress, as recorded both by this instrument and the biaxial stressmeter in the west abutment, averaged approximately 77° counterclockwise from the vertical. A near-vertical direction would be expected because the overburden load would be transferred to the pillars and abutments as mining progressed. The reason for this discrepancy is not known.

## NUMERIC ANALYSIS

### FINITE-ELEMENT ANALYSIS

The two-dimensional, finite-element program UTAH2 (*15*) was used to model the North 5650 transverse cross section in plane strain with elastic, perfectly plastic material properties. The yield criterion is Drucker-Prager, where strength depends on the three principal stresses, and the associate flow rules are applied to determine strains in yielded elements. The finite-element mesh had 4,443 elements and 4,286 nodes. The sides of the mesh were two ore body diameters away from the east and west ribs of the ore body to eliminate the influence of boundary conditions. Each stope was five square elements wide, each element representing an area of rock approximately 5 by 5 ft. The top of the mesh represented the ground surface and was coarser than the mesh in the stopes because large stress gradients were not expected away from the ore body.

Nineteen cuts and fifteen filling steps were required to model the mining sequence from the time the first heading was driven to the time the 780 heading of pillar D57 was excavated. At this point, most of ore body had been mined and backfilled.

Average values for material properties used in the computer model are listed in table 2. Values for rock and cemented backfill properties were obtained from uniaxial compression tests reported by J. F. T. Agapito and Associates, Inc.[7] Uncemented fill and overburden modulus values were obtained from one-dimensional compression tests on mine backfills (*12*), and the unit weight for overburden was based on textbook values for soil (*18*).

[7]Unpublished report submitted to Asamera Minerals (U.S.), Inc., October, 1985.

**Table 2.—Average values for material properties**

| Material | Elastic modulus, psi | Uniaxial compressive strength, psi | Tensile strength, psi | Poisson's ratio | Density, lb/ft³ |
|---|---|---|---|---|---|
| Unsilicified sandstone ..... | 1,700,000 | 3,690 | 370 | 0.25 | 152 |
| Silicified sandstone ..... | 2,500,000 | 6,610 | 660 | .27 | 155 |
| Cemented fill .... | 550,000 | 1,200 | 300 | .30 | 132 |
| Uncemented fill .. | 23,000 | 600 | 150 | .30 | 150 |
| Overburden ..... | 2,800 | 100 | 5 | .30 | 130 |

Figure 29 shows measured versus predicted relative displacement readings from extensometers placed in rock and backfill. The linear correlation coefficients are 0.81 and 0.095, respectively. One possible reason for the lack of correlation in the backfill data might be that the horizontal fill extensometers were deformed by the flow of backfill when they were installed, which could have caused some damage to the rod-transducer assembly.

Another possibility is that the interface between the rock and the backfill was not properly represented without frictional interface elements. In the model, elastic strains in the rock pillars caused failures in adjacent backfill elements that probably did not occur in the mine. A third possibility is that the model mining sequence may not represent the true mining sequence. In actual mining, the headings and benches were extracted in a succession of small excavations, and these were not taken into account by the two-dimensional, plane strain code. A three-dimensional model might improve results; however, a material model incorporating creep would be necessary to account for the apparent time-dependent stress redistribution.

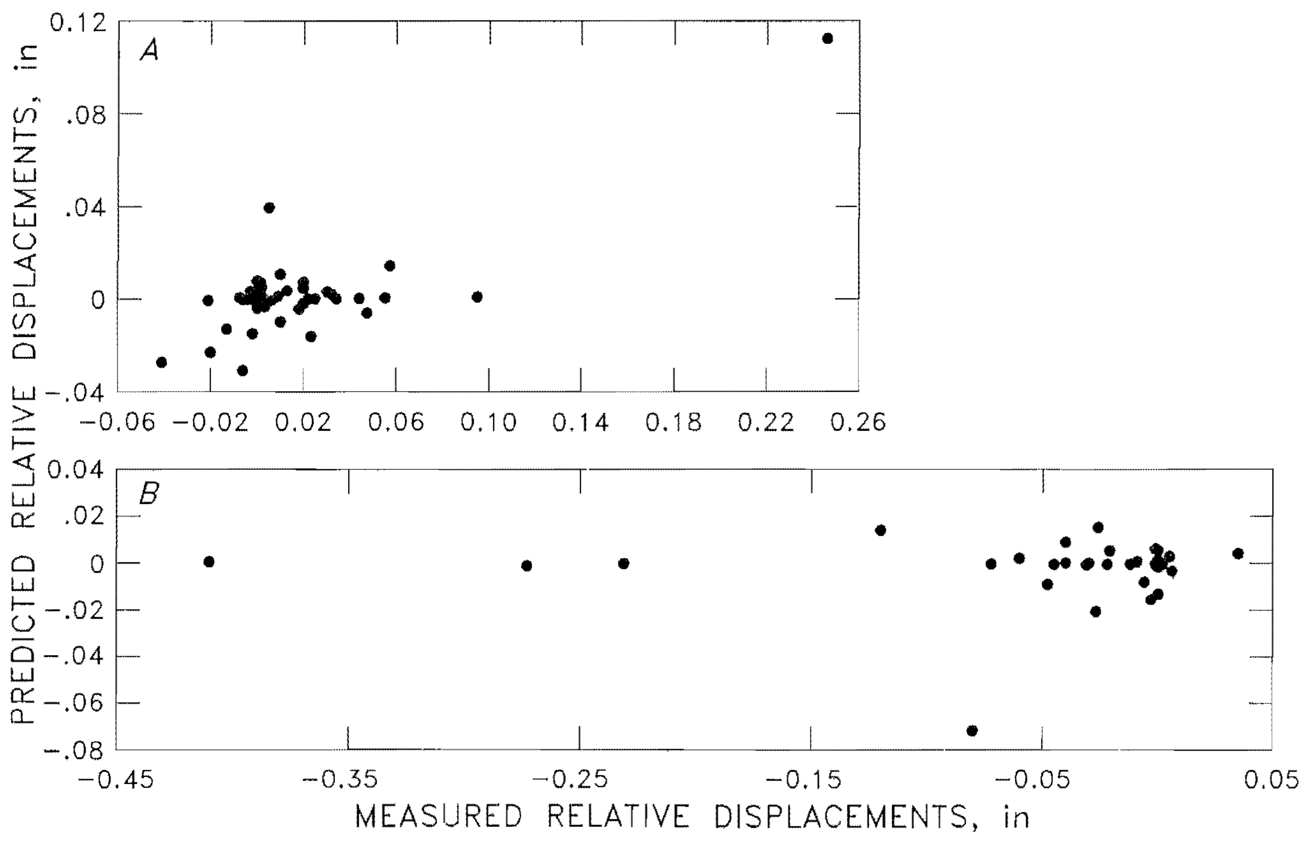


**Figure 29.—Measured versus predicted relative displacements for extensometers. *A*, Rock; *B*, backfill.**

## FINITE-ELEMENT ANALYSIS WITH REDUCED MATERIAL VALUES

Additional finite-element runs were made with reduced values for material properties (table 3) to account for rock mass discontinuities and weaker cemented backfill caused by aggregate separation during placement. A lower cemented backfill modulus of 115,140 psi was calculated from readings of the backfill instruments, and a uniaxial compressive strength of 600 psi was calculated from average values obtained from large-scale laboratory tests. The average modulus of elasticity for the test cylinders was 326,500 psi, a value substantially larger than the modulus calculated from backfill instrument readings. An equation (*9*) containing characteristic compressive strength was used to estimate tensile strength of the cemented backfill. The characteristic compressive strength is defined as that strength below which 5 pct of all possible strength measurements for the specified mix may be expected to fall, which in this case was 600 psi. This value was chosen because a reliable distribution of large-scale strength values was not available. The equation is shown below.

$$f_{ctm} = 0.3 f_{ck}^{2/3}, \tag{1}$$

where $f_{ctm}$ = mean tensile strength, MPa,

and $f_{ck}$ = the characteristic compressive strength, MPa.

Reduced compressive strength of the silicified sandstone was calculated using equation 2. This equation is based on an empirical strength formula and is the same one used by J. F. T. Agapito and Associates, Inc., for design purposes (*6*).

$$\sigma_C = \sigma_L \times (V_L/V_F)^{\alpha} \times (S_F/S_L)^{\beta}, \tag{2}$$

where $\sigma_C$ = field uniaxial compressive strength, psi,

$\sigma_L$ = laboratory strength, psi,

$V_L$ = volume of the laboratory sample, $in^3$,

$V_F$ = volume of the pillar, $in^3$,

$\alpha$ = volume reduction factor (0.10),

$S_F$ = width-to-height ratio of the pillar,

$S_L$ = width to height ratio of the laboratory sample,

and $\beta$ = shape effect factor (0.80).

Laboratory samples were 2.4 in. in diameter and 5 in long. A pillar 24 ft square and 65 ft high was used to calculate volume. The result was an estimated field uniaxial compressive strength of 1,210 psi.

Reducing sandstone strengths resulted in the formation of localized plastic zones in the back and floor of excavated stopes (fig. 30). Some cemented backfill elements adjacent to rock elements became plastic in tension when the rock was excavated, but these failures are believed to be erroneous because of the coupled interface. These elements are not shown.

**Table 3.—Reduced values for material properties**

| Material | Elastic modulus, psi | Uniaxial compressive strength, psi | Tensile strength, psi | Poisson's ratio | Density, $lb/ft^3$ |
|---|---|---|---|---|---|
| Unsilicified sandstone ..... | 680,000 | 695 | 68 | 0.25 | 152 |
| Silicified sandstone ..... | 1,000,000 | 1,210 | 150 | .27 | 155 |
| Cemented fill .... | 115,140 | 600 | 110 | .30 | 132 |
| Uncemented fill .. | 23,000 | 600 | 110 | .30 | 150 |
| Overburden ..... | 2,820 | 100 | 5 | .30 | 130 |

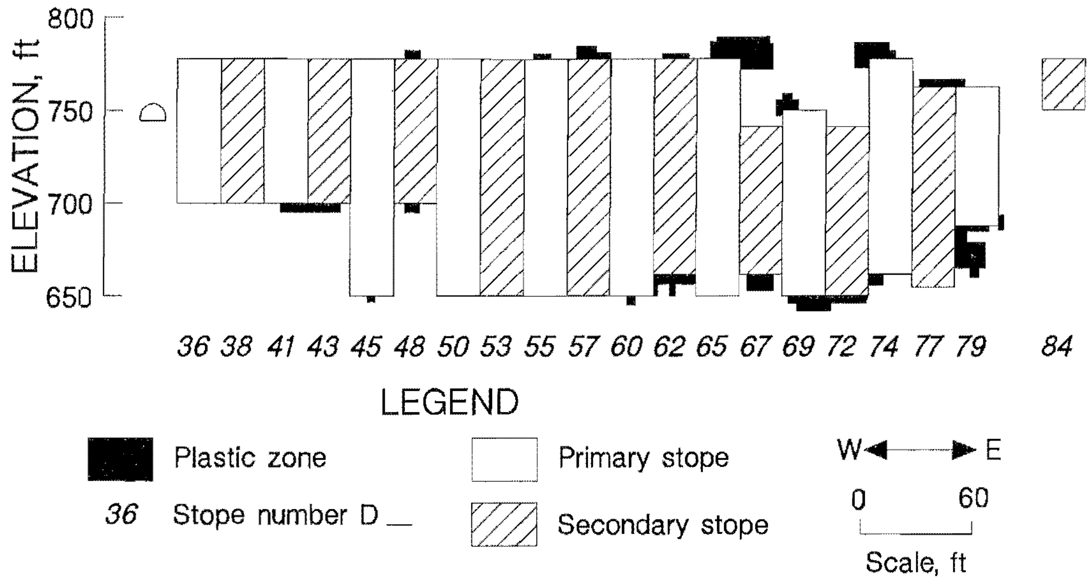


**Figure 30.—Plastic zones in rock as predicted by finite-element model with reduced material properties.**

## FINITE-DIFFERENCE ANALYSIS OF PRIMARY BACKFILL PILLAR D55

Cemented backfill pillar D55 was analyzed separately with the finite-difference program Fast Lagrangian Analysis of Continua (FLAC) (7) and the Mohr-Coulomb failure criterion. Strain values imposed on the modeled pillar were obtained from vertical backfill extensometer measurements in the pillar. The bottom and sides of the modeled pillar had rollered boundaries. A total vertical displacement of -6.697 in was applied to the top boundary and represented a strain value of -0.004293 over a pillar height of 130 ft. This value was recorded by the vertical backfill extensometer in pillar D55 on May 3, 1989, before the last rapid change in strain rate occurred (fig. 17). This change could represent pillar failure, although, as noted previously, there was no visual evidence for pillar degradation.

Mining the 650, 700, and 780 headings and the 700 bench was achieved in the model by removing the respective rollers on one side of the pillar. Small plastic zones 8 ft wide by 4 ft high developed in the pillar near the excavations when the 650 and 700 headings were driven, but the remainder of the pillar was in an elastic state. Mining the 700 bench, however, caused most of the elements from the 650 to 700 levels to approach or enter the plastic state. The loss of confinement resulted in tensile stresses in the horizontal direction, producing a Mohr's circle radius that approached or exceeded the failure envelope. Because pillar D55 did not show signs of failure when observed underground after D57 was mined, the estimated field strength of 600 psi for the cemented backfill could be low. However, results from the model using this value could be used as a conservative estimate of fill behavior.

## CONCLUSIONS

The use of vibrating-wire instruments proved to be a reliable method of recording mining-induced stress redistributions from overburden to the secondary rock pillars and from these pillars to the cemented backfill in the primary stopes. Installing instruments in the stope back, ore body abutment, secondary rock pillars, and primary cemented backfill pillars verified pillar loading, unloading, and failure. Along with an immediate response to mining characterized by changes in stress and strain, a time-dependent stress increase was recorded by instruments in the cemented backfill and in the ore body abutment.

A possible improvement in instrument installation procedures might be to cast earth pressure cells in forms and allow the backfill in the form to cure for a short time before backfilling operations were resumed and covered the cell. Modifications to the horizontal backfill extensometer, such as replacing the PVC casing with steel pipe, would make the instrument more rigid and help prevent damage when it was buried in backfill.

Installing instruments in clusters allowed checks on instrument readings and provided data at given locations even when some of the instruments stopped working. By using stress readings from earth pressure cells and strain readings from backfill extensometers, an estimate of in situ backfill elastic modulus could be made. Also, by installing instruments in adjacent rock and cemented fill pillars, stress redistribution from the rock pillars to the fill pillars could be monitored.

The correlation coefficient for displacements measured in the rock versus displacements predicted by the finite-element code was 0.81, indicating that the numeric model could be used as a predictive tool. Although there was no linear relationship between measured versus predicted displacements for the instruments in the backfill, loading on cemented backfill pillars could be estimated from rock displacements at the top and bottom of the fill pillars.

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