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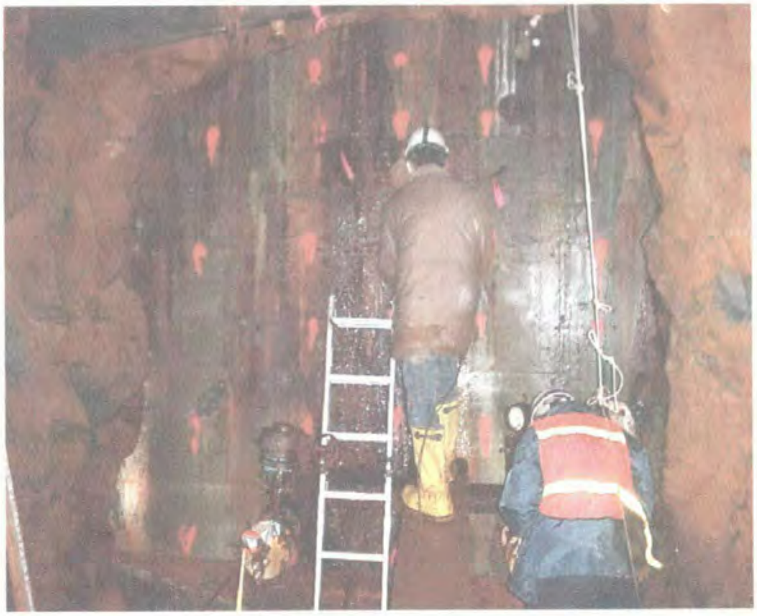
Walker Mine
Plumas County, CA

Seal Testing and Evaluation Report



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SUBMITTED TO
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Central Valley Region
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Project Number 003870

Walker Mine

Plumas County, CA

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Signature Page

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1. Background and History

1.1 Background

The Walker Mine is an inactive underground copper mine located approximately 23 miles northwest of Portola, in Plumas County, California (Figure 1). Access to the site is by County Road 112 (Figure 2). North of Lake Davis the road is graveled and is not plowed during the winter. The mine site is located at altitudes ranging from 6,000 to 7,000 feet (mean sea level datum) and is generally inaccessible to motor vehicles between November and May.

Mining operations took place between 1916 and 1941. Underground workings were developed between about El. 5,400 and El. 7,000. The "700 Level Main Access Adit," which is the subject of this project, is located at about El. 6,200 and is the lowest point at which the underground workings reach the surface. This adit, driven in the early 1920's from the mill site at Dolly Creek, reportedly intersected the South Orebody at a distance of about 3,000 feet from the portal. It then followed the mineral vein through the Central, North, 712 and Piute Orebodies (Figure 3). At approximately 10,000 feet from the portal, the Piute shaft was raised in the vein from the 700 Level to the ground surface in 1927/1928 (SRK, 1985).

The mine was closed down in 1941 by a subsidiary of Anaconda Copper Company. After operations ceased, acidic and metal-laden drainage water (Acid Mine Drainage or AMD) issuing from the adit portal began to affect the downgradient streams (Dolly Creek and Little Grizzly Creek). The discharge from the mine was reported to have totally eliminated aquatic life in Dolly Creek, downstream from its confluence with the mine drainage water, and in Little Grizzly Creek downstream from its confluence with Dolly Creek for a distance of approximately ten miles downstream of the Walker Mine (SRK, 1985).

In November of 1987, the California Regional Water Quality Control Board (RWQCB) constructed a concrete plug, or seal, in the adit with the purpose of stopping AMD discharges. The seal was located at a distance of about 2,700 feet from the portal based on an evaluation of the rock conditions along the adit performed by SRK in 1985 (SRK, 1985). The seal is seated in granodiorite. Its cross section is about 9 feet wide by 12 feet high and its length is 15 feet. Two 4-inch-diameter stainless steel pipes are embedded in the seal to allow draining of the impounded water. The pipes are controlled by rotary control valves that apparently, as of the date of the commencement of these studies, had not been operated since they were installed in 1987. A sampling port with a pressure transducer is mounted on one of the drain pipes, upstream of the control valve. The transducer is connected to a data logger that is monitored by the RWQCB. A view of the seal shortly after its construction in 1987 is shown as Photo No. 1 (the photos can be found after the figures).

Installation of the seal has been a success. Discharge of AMD from the adit has reportedly ceased. Surface water monitoring by the RWQCB has not detected any springs or seepage areas into the valleys of the Dolly or Little Grizzly Creeks that could be identified as groundwater recharged from the Walker Mine workings. The seal impounds the AMD, which now partly floods the mine workings. As shown on Figure 4, the water level behind the seal varies seasonally, peaking after the spring snowmelt. At its peak, the hydraulic head on the seal has been recorded at over 200 feet, and the reservoir created by the seal holds back more than 90 acre-feet of AMD.

The RWQCB operations and maintenance plan for the Walker Mine site requires the RWQCB to perform integrity testing of the mine seal every 10 years. Testing of the mine seal is a critical and necessary project because if the seal were to fail a large volume of AMD would be released, impacting aquatic life in downstream creeks.

1.2 Available Data

Available information from previous studies on the Walker Mine seal includes the following:

(1) Steffen Robertson and Kirsten (SRK), "Walker Mine Project, Draft Final Feasibility and Design Report," September 1985. This is the design report for the seal. The stated design life for the seal is in excess of 100 years. The report contains interpretive information on the site geology and hydrogeology. It also presents the rock mechanics evaluations performed to assess the shear strength along the rock/concrete interface. An allowable shear strength of about 190 psi was estimated along the interface. The design was prepared assuming a maximum hydraulic head of 570 feet and applying a factor of safety of 2.5 to the hydraulic head. The maximum hydraulic head is controlled by existing adits above the Main Access Adit. The Piute Shaft landing tunnel has a portal at an elevation approximately 390 feet higher than the 700 Level Adit (Figure 3). However, for the design it was assumed that the Piute Shaft also could be sealed (to date it has not been sealed). The next point of egress would be the Old Sawmill adit, located at an elevation about 570 feet higher than the 700 Level Adit.

(2) SRK, "Walker Mine, Final Construction As-Built Report," March 1989: This report includes: a letter report describing the key construction activities; Attachment 1 entitled "Quality Assurance Data Summaries" which contains test data related to concrete materials and concrete; Attachment 2 entitled "Specifications"; Attachment 3 entitled "Correspondence," which includes additional data on the concrete, information on instrumentation, and miscellaneous correspondence; and an unnumbered attachment dated September 1989 containing the SRK "Construction Activities Reports." According to this report the concrete mixture used for the seal had a design 28-day compressive strength of 3,000 psi. Each cubic yard of concrete contained approximately 1,476 pounds of fine aggregate, 1,546 pounds of 3/4-inch to No. 4 coarse aggregate (limestone), 450 pounds of Type II low alkali portland cement, and 150 pounds of lassenite, a natural pozzolan. Silica fume, also pozzolanic

in nature, was added at a rate of 49.5 pounds per cubic yard to improve the workability and pumpability of the concrete. A superplasticizer was used to obtain a slump of about 7 inches at the pump while maintaining a water to cementitious material ratio of about 0.45. The slump at the end of the pump line generally ranged from 4 ½ to 6 inches. The concrete was placed in a single shift on November 13, 1987. The total volume placed was approximately 66.2 cubic yards. The temperature of the mix ranged from 51 to 56 degrees Fahrenheit. The ambient temperature in the mine during placement was 47 degrees Fahrenheit. Concrete test cylinders made from one sample taken during the placement averaged a 28-day compressive strength of 5,500 psi.

(3) Regional Water Quality Control Board, "Walker Mine Site Safety Plan," January 20, 1998. This is the RWQCB safety plan for their site management operations, and was used for the site visit described in this report.

(4) SRK drawing entitled "As-Built Walker Mine Plug Plan, Profile and Details," dated March 1989. This is the construction drawing for the seal. A copy is included in Appendix A.

(5) Westec drawing entitled "Walker Mine Tunnel Rehabilitation, 700 Level Adit Mapping Data" dated March 1995. It presents geologic mapping data along the adit alignment.

(6) Video entitled "Walker Mine Seal Project" dated November 13, 1987. It shows the site of the seal just prior to concrete placement. It shows views of the bulkheads upstream and downstream of the seal and the interior of the seal, including the drain pipes, grout pipes, tremie pipe and rock condition.

(7) Water quality data:

- Typical water quality parameters for portal water and unimpacted nearby streams before construction of the seal were reported in 1985 by SRK and are shown in the first four columns of Table 1 below. A seasonal variation was reported by SRK both in flow amounts and copper concentration, with the latter being highest during periods of greatest flow. This was attributed to the spring flushing of acid generated in the mine all winter, resulting in spring flows that have a lower pH and a higher copper content than flows later in the year. Reported copper concentrations for AMD ranged from about 10 to 50 mg/l (SRK, 1985).

- The RWQCB monitors the chemistry of the water exiting the mine portal on a semiannual basis. Median values for selected parameters measured during 1996-2000 (after seal construction) are listed in the fifth column of Table 1. A comparison of the third and fifth columns of Table 1 shows the improvement in the quality of portal water resulting from seal construction, e.g., the pH increased from 4.1 to 7.4, the sulfate concentration decreased from 146 to 1 mg/kg, the alkalinity increased from 0 to 56 mg/kg, the copper concentration decreased from 29 to 0.1 mg/kg, and the zinc concentration decreased from 0.9 to 0.02 mg/kg.

- The quality of the water impounded by the seal is illustrated by data from two samples taken by the RWQCB in June 2000: One of them (labeled "seep") was taken from seepage water emerging from the rock/concrete interface along the crown of the seal. The other (labeled "pool") was taken from a pool of water at the downstream toe of the seal. Selected parameters from these tests are shown in the sixth and seventh columns of Table 1.

- Data on the discharge rate of portal water has been obtained by the RWQCB. Fifty-one measurements taken from 1957 to the date of construction of the seal in 1987 give an average discharge flow rate of about 200 gpm and a peak of about 1,100 gpm. Twenty-four measurements taken after the construction of the seal give an average discharge flow rate of 6 gpm and a peak discharge of about 18 gpm.

Table 1
Water Quality Parameters

Parameter	Unit	Prior to Construction of Mine Seal		After Construction of Mine Seal		
		Portal[1]	Streams[1]	Portal[2]	Seep[3]	Pool[3]
pH	units	4.1	7.6	7.4	4.0	3.7
Ca	mg/l	24.5	5.8	11.5	29	33
Na	mg/l	2.7	2.8	4.6	2.6	3.1
K	mg/l	1.6	0.7	NA	2.4	2.8
Mg	mg/l	6.4	2.2	NA	6.8	6.8
SO_4	mg/l	146	5	1.1	200	200
HCO_3	mg/l	0	23	56	ND	ND
NO_3	mg/l	4.5	0.7	NA	ND	ND
NH_3	mg/l	0.01	0.01	NA	NA	NA
Cl	mg/l	1	ND	0.66	ND	ND
Cu	mg/l	29	0.03	0.13	14	12
Zn	mg/l	0.93	0.01	0.02	0.76	0.76
Fe	mg/l	1.0	0.15	0.09	3.3	1.2

Notes:
1. Typical water quality parameters reported by SRK before the seal was constructed (SRK, 1985; Table 2).
2. Median values from water samples collected at the portal and tested approximately semiannually from 1996 through 2000 for the RWQCB.
3. Tests performed by CLS Labs, Rancho Cordova, California for the RWQCB. Samples were collected just downstream of the seal. Test results are dated July 14, 2000.

ND = Not Detected
NA = Not Analyzed

2. Purpose and Scope

2.1 Purpose

The purpose of the Walker Mine seal testing and evaluation program was to assess the integrity of the seal. Specifically, the following characteristics of the mine seal were assessed as requested by the RWQCB:

a. Location, depth, and extent of major cracks,
b. Seepage locations and volumes,
c. Physical condition of concrete on the submerged side of the seal,
d. Condition (corrosion) of the two pipes and valve assemblies installed in the seal,
e. Condition of the support rock at the seal area,
f. Condition of the rock/concrete interface at selected locations,
g. Condition of the seal with regard to its ability to withstand the design hydraulic head,
h. Maximum (optimum) head on seal to minimize seal degradation.

2.2 Scope

The scope of the seal testing and evaluation work is summarized as follows:

1. Perform a site visit to inspect the visible features of the seal and conduct an initial phase of nondestructive tests. Prepare a Site Visit Report that presents the observations, test data, and initial evaluations (issued on April 13, 2001).

2. Based on the results of the site visit and initial tests, plan additional testing and evaluation activities to characterize the seal. Prepare a Sampling and Analysis Work Plan that summarizes the proposed integrity evaluation program (issued on April 13, 2001).

3. Upon approval of the work plan by the RWQCB, conduct the additional testing and evaluation activities (conducted between May and December 2001).

4. Prepare a Seal Testing and Evaluation Report summarizing the data and findings from the testing activities, data interpretation, conclusions and recommendations.

This document, entitled "Seal Testing and Evaluation Report", presents a comprehensive summary of data and findings from the site visit and testing activities, as well as data interpretation, conclusions, and recommendations. The findings and conclusions of the site visit are presented in Section 3. The investigations performed for additional characterization of the seal are discussed in Section 4. Section 5 presents additional analyses and evaluations pertaining to the seal design, construction and long-term performance. Conclusions, recommendations, limitations, a disclosure statement, and references are presented in Sections 6 through 10, respectively. Data and additional information are included in appendices.

3. Initial Site Visit

3.1 General

The mine seal was observed in detail during the initial site visit on October 31 through November 2, 2000. The weather was sunny on October 31, partly cloudy on November 1, and again sunny on November 2. Temperatures were below freezing at night and in the upper 30s to low 40s during the day. There was about two to three inches of snow on the ground at the mine site. Vehicular access to the site required 4-wheel drive due to mud and snow on the road in the vicinity of the site.

Participants and their affiliation included the following:

- Patrick Morris, Regional Water Quality Control Board project manager
- Alberto Pujol, P.E., Mike Knarr and Nick Kollerer, GEI Consultants, project director, structural engineer and staff engineer, respectively
- Malcolm Lim, Ethan Dodge and Jerry Harrano, Construction Technology Laboratories (CTL), concrete nondestructive testing specialists
- Jose Cercone and Cynthia Fox, Washington Group International (Washington), engineering geologist and civil/process engineer, respectively
- Gary Mass, independent concrete consultant
- Lara Pucik, Walker & Associates (WAI), geochemistry specialist
- Patrick Morrison, SDV-ACCI, graphics and project support

The general sequence of activities occurred as follows:

- Preparatory activities occurred on October 31st. A ladder, hand pump, hoses and various supplies were brought to the vicinity of the seal. The seepage pool next to the seal was pumped out to enable access to the valves and observation of the concrete face.

- The main visit took place on November 1st. Pucik took AMD samples and performed field water quality determinations. Kollerer pumped out water that had accumulated in the seepage pool overnight. The CTL group performed nondestructive tests (impact echo and impulse response) on the concrete of the mine seal. Knarr and Mass inspected the condition of the concrete face. Fox measured and inspected the piping and valves. Cercone observed rock conditions immediately downstream of the seal. The visit was concluded shortly before 5 pm.

- A smaller group including Mass, Fox, Cercone, Pujol and Kollerer returned to the mine site on November 2nd to wrap up the site visit and demobilize. Kollerer again dewatered the seepage pool next to the seal. Fox, Cercone and Mass concluded their observations. A chain with padlock was placed around the handwheel of each control valve actuator to prevent unauthorized actuation of the valves. The site inspection equipment was retrieved from the seal area and the area cleaned up. The ladder, pump, hoses and wheelbarrow were stored near the entrance to the adit, by the battery packs. The visit was concluded shortly after 1 pm.

Throughout the site visit, air quality in the adit was monitored using a gas monitor (MG140 Four Gas Monitor manufactured by Industrial Scientific Corporation) which continuously monitored oxygen, hydrogen sulfide, carbon monoxide and combustible gas content. Whenever personnel were in the adit, at least one person remained outside, at the portal, to monitor the generator and for safety purposes.

3.2 Seepage Conditions Observed During Initial Site Visit

From the portal, the adit begins with a 150-foot-long cut-and-cover section supported by a corrugated metal pipe lining. This section, constructed recently, appears in good condition and dry. The next 1,100 feet of the adit is heavily timbered and very wet. The surrounding ground is decomposed or highly weathered granodiorite. The adit drains groundwater from the hillside above the portal. Water drips from the roof at many locations, and pools are present on the ground to a depth of up to 6 inches. The water overflows into a ditch that runs along the left side of the adit (looking toward the seal), under the ventilation duct. On the average, the adit grade is roughly one percent sloping down toward the portal.

Beyond the first 1,300 feet or so from the portal, the adit runs through generally fresh or slightly weathered granodiorite. The opening is unsupported and essentially dry. During the site visit we did not observe water in this section of the adit until we reached a distance of about 200 feet from the seal. The rock surface is partly covered with a thin coating of reddish mud, reportedly deposited during a collapse (date unknown) in the timbered section that caused the adit to be flooded. The floor of the adit is covered with soil which supports ties and a narrow gauge rail track.

At the seal, the hydraulic pressure of the impounded water measured on the day of the visit was 61 psi. Thus, a hydraulic head of approximately 140 feet was acting on the upstream face of the seal. As a result of the hydraulic gradient through the seal and adjacent rock mass, water seeps through the concrete-rock interface and existing joints in the rock mass. The leakage primarily daylights to the downstream face of the seal along the interface between the concrete and the roof of the adit, and appears in the form of drips running down the face of the seal. Large iron hydroxide deposits of a dark reddish color have formed over the seal face. Most of the leakage daylights at the left-center portion of the crown and along both side walls. We did not see any evidence of perceptible seepage flow occurring through the concrete itself. Wet reddish mud covers the floor of the adit at the toe of the seal. We tried to push it aside with a shovel and did not see any perceptible evidence (in the form of bubbling or eddies in the mud) of seepage occurring along the rock-concrete contact at the floor of the seal. However, the mud and water impeded a good view of the contact and small amounts of seepage would escape detection.

There appears to be minimal leakage through the rock around the seal. We observed a very slow drip occurring from existing rock joints that daylight at the roof of the adit about 25 feet downstream of the seal.

The seepage accumulates in a pool on the floor of the adit at the downstream toe of the seal. The pool is confined by the seal face, the adit sidewalls, and the ballast soil covering the adit floor away from the seal. Approximate pool dimensions are 10 feet by 6 feet and the depth of water is up to about 20 inches. Water also accumulates in the ditch on the left side of the adit for a distance of about 200 feet from the seal. The water in the ditch appears stagnant, and it seeps into the ground and disappears at the same rate that it discharges from the seal area. The grade of the adit within a few hundred feet downstream of the seal is essentially flat.

During the first day of our visit (October 31), we used a hand pump and 125 feet of 3/4-inch garden hose to pump water out of the pool next to the seal. We also bailed water using 5-gallon buckets and discharged it at the end point of the hose. We estimate that we removed about 1,000 gallons from the pool using these methods. The discharged water simply pooled within the irregularities of the adit floor at the discharge point of the hose and in the ditch along the left wall. We blocked the ditch next to the seal to prevent the water from flowing back into the pool. We stopped pumping at 3:00 pm. Before we left for the evening we set up a temporary staff gauge to measure the water level in the pool. When we left at about 4:00 pm, the water level was at about 3 inches above the bottom. The following morning at 10:45 am the water level had risen to about 8-¼ inches, and we pumped about 200 gallons out to return the water level to about 3 inches. This volume of water over a period of 19 hrs 45 minutes gives a seepage rate of 0.17 gpm.

When we left on November 1 at 4:35 pm, the water level was 2-1/8 inches. When we arrived at 9:55 am on November 2, the water level was at 7-1/8 inches and we estimate that we pumped about 150 gallons to draw down the water level to about 2 -1/8 inches. This volume of water over a period of about 17 hrs 30 minutes gives a seepage rate of 0.14 gpm. This rate is slightly lower than that observed the previous day possibly because during the previous day, water may have been seeping back into the pool from the soil deposits that cover the floor of the adit adjacent to the pool. Based on these observations and considering the approximate nature of the method used, we believe that 0.15 gpm is a reasonable estimate of the seepage rate occurring in the immediate vicinity of the plug at a hydraulic head of 140 feet.

3.3 Rock Condition Observed During Initial Site Visit

Rock exposed along the 700 level adit consists mainly of coarse-grained, light gray granodiorite containing chiefly quartz, plagioclase feldspar phenocrysts, and dark colored mafic minerals. Structures observed along the adit include joints, cleavage, and sheared zones.

Beyond the initial timber-supported zone, the rock condition is generally good, and there is no installed support. We observed no indications of rock distress, stress-induced relief, post-excavation overbreaks or cave-ins. The overbreak is generally in the roof and is believed to have occurred during the original excavation (SRK, 1985).

Generally the rock is slightly to moderately jointed, slightly weathered to fresh, and dense. The predominant joint orientations observed, two sets along the adit drive, are N 50 W and E-W. Both sets dip between 45 to 55 degrees to the south. Typically the joints are spaced 1 to 3 feet apart, and are rough, tight and clean or with a thin filling of clay (weathered felsic mineral).

3.4 Concrete Condition Observed During Initial Site Visit

The exposed face of the concrete seal was closely examined by Gary Mass and Mike Knarr. A sketch of the face is presented as Figure 5. The purpose of the examination was to determine the condition of the concrete that was accessible, as an indication of the condition of the concrete throughout the seal. This section presents the observations that were made.

3.4.1 General Appearance

The entire face of the seal was moist. Except for those areas covered by deposits of iron hydroxide and calcium carbonate from seepage water, the original formed surface of the seal was plainly visible and in good condition (See Photo No. 2). Surface deposits appear to be associated with those areas of seepage and seepage flow including the active seep in the left-center portion of the crown and along both sidewalls adjacent to the concrete/rock contact. Efflorescence, whitish deposits of calcium carbonate, was isolated to two locations. These locations were the lower left sidewall and upper right side of the seal. A green-colored copper sulfate staining was also observed on the right side of the seal and on the rock adjacent to the seal.

The concrete/rock contact appeared to be good (See Photos No. 6, 7 and 8). No discontinuities were observed. Concrete against rock was well consolidated and free of visible voids, honeycomb, or other defects.

No surface erosion was observed. The concrete appeared dense and well consolidated. Several horizontal lines were observed above mid-height on the face. The lines are composed of a series of small surface voids, generally less than 1 mm diameter. These lines were not continuous across the face, but were generally limited to several feet in length. There was no evidence of seepage, past or present, along any of these lines.

No cracking of any nature was observed on the face of seal. The only damage appeared to be a 1-inch-deep by 8-inch-long gouge in the concrete face on the lower left side of the seal. The cause of this gouge is unknown but it appeared to be produced by a pointed object.

A horizontal water line on the concrete face, as evidenced by a solid iron staining, was observed approximately 18 inches above the floor of the adit. This water line was generated by the presence of the ponded seepage against, and immediately downstream of the seal. Since the ponded water had been removed for access and inspection, a mud line of soft iron sediment could also be seen along the floor. This mud line followed the contour of the rock along the floor and appeared to be about 3 to 4 inches in thickness.

Several items penetrate the full thickness of the seal including twelve (12) 5/8-inch-diameter threaded form ties and the two (2) 4-inch-diameter stainless steel drain pipes. No seepage, or evidence of prior seepage, was observed around any of these penetrations. The embedded pipe that was used for pumping concrete into the form and the embedded grout pipes do not penetrate the full thickness of the concrete seal and no evidence of seepage was observed around either of these features.

3.4.2 Surface Examination

Since portions of the surface of the concrete had been exposed to acidic water, either from seepage flowing down the face or from standing water in the pond, a close examination was made to determine hardness and quality of the concrete surface. The pointed end of a geology hammer was used to check surface hardness.

The first area examined was in the center of the calcium carbonate deposit on the lower left side of the seal (See Photo No. 2). This area was wet from active seepage flow. It was found that the thickness of the deposit ranged from 1 to 2 mm and that this deposit was reasonably well bonded to the concrete. The underlying concrete surface was soft and could be easily removed by scraping with the hammer point. Depth of the softness was estimated at 1 to 1.5 mm. Once this soft paste and mortar were removed the concrete immediately underneath was hard and sound.

The second area examined was above and below the water line that had been created by the ponded seepage. Below the water line the concrete surface paste and mortar could easily be removed by scraping to a depth of 1 to 2 mm before hard, sound concrete was reached. Above the water line the depth of soft paste and mortar was approximately 1 to 1.5 mm, or slightly less than below the water line (See Photos No. 3 and 4).

The third area examined was immediately below the protruding steel pipe that had been used for pumping concrete into the seal. The surface in this area was moist but there did not appear to be any evidence of seepage flow. The surface paste was found to be hard and sound and could not be removed by scraping (See Photo No. 5).

As mentioned earlier, iron staining and iron deposits have developed on the exposed face of the seal. The heaviest accumulation of iron deposits is down the face on the right-of-center side of the seal (See photo No. 2). This deposit is composed of fairly soft material that is up to one inch in thickness with a consistency of mush. The iron staining and deposits do not affect the durability of concrete in any manner.

3.5 Nondestructive Testing of Concrete Performed During Initial Site Visit

The concrete seal was tested by Construction Technology Laboratories, Inc. (CTL) using the nondestructive impulse response, impedance log and impact echo testing techniques. Testing was performed on November 1, 2000. A detailed report by CTL describing the test methods and results is included in Appendix B.1.

3.5.1 Impulse Response Testing

For the Impulse Response (IR) method of testing, a receiving sensor (geophone) was coupled to the face of the seal using grease. A sledgehammer impacted the seal face at selected test points, and the time history of the force measured by the hammer and receiving sensor after impact was recorded for analysis. Two IR testing approaches were adopted:

- A matrix of test points at 2-ft vertical and horizontal spacing was established, and each test point was impacted with the geophone located approximately 6 inches from the point of impact. This test methodology gave information on the concrete condition to a depth of approximately 3 feet into the seal from its face.

  The IR test in this mode produces two principal parameters: element mobility and dynamic stiffness. The measured values of mobility and stiffness remain constant for a given unit thickness and concrete quality. If the concrete thickness, quality or density reduces, the mobility increases and the stiffness decreases. A separation within the concrete such as a cold joint or delamination will also result in significant changes in these two parameters.

  A 2-ft x 2-ft test grid was laid out over the total seal face which gave a grid with 5 test points in the horizontal direction and 6 test points in the vertical direction. The measured values of dynamic stiffness and average mobility are plotted in contour in Figures D.1 and D.2 of Appendix B.1.

- The geophone was positioned at the center of the seal face, and testing was performed by striking the face at points around the periphery of the seal. The IR test results obtained in this mode were analyzed to measure the distance from the face to the back of the seal and the equivalent dynamic shear modulus at the concrete-rock interface at different points around the seal. The test data was interpreted using simulation methods such as the Impedance Log. Test results are presented in Appendix B.1.

3.5.2 Impact Echo Testing

Like the Impulse Response Test, the Impact Echo (IE) Test uses stress wave to detect flaws within concrete structures. However, the frequency range used is considerably higher in the IE test, since much shorter wave lengths are required to detect small anomalies.

Limited IE testing was performed due to the soft nature of the surface of the seal which acted as a damper to the impactor. Several different size impactors were used to generate the energy necessary to penetrate the entire length of seal. Despite using three different size impactors, the energy generated was insufficient to penetrate the entire length of seal.

3.6 Arrangement and Condition of Drain Pipes Observed During Initial Site Visit

3.6.1 General

The drain piping consists of two independent 4-inch diameter stainless steel pipes that extend through the concrete seal (See Photos No. 9 through 14). The SRK drawing entitled "As-Built Walker Mine Plug Plan, Profile and Details" (attached in Appendix A) indicates that the pipes extend a minimum of 6 inches upstream of the seal face and the inlets are protected with screens. The drawing also shows that rubber rings were placed around each pipe at 4-foot intervals to serve as waterstops. The screens and waterstops are visible in the videotape of the original seal construction (RWQCB, 1987).

Downstream of the seal, each projecting pipe has a closed 4-inch shutoff valve with a handwheel actuator and terminates in a blind flange tapped for two 3/4-inch-diameter pipe stubs, each with a closed ball valve. Upstream of the 4-inch shutoff valve, the drain pipe on the right side (looking toward the seal) has a 3/4-inch pipe branch used for a pressure transmitter connection that includes an isolating ball valve, a pressure gauge, a sampling port, and a pressure transmitter sensor. The left drain pipe has only a 3/4-inch pipe stub-up with a ball valve and threaded end plug. Sketches showing the approximate dimensions and materials of the installed arrangement of the drain pipe and valve assemblies are included as Figures 6 (plan) and 7 (elevation). These figures depict the conditions observed during the site visit and do not include the pipe and valve modifications made after the site visit. The modifications are discussed in Section 4, and the approximate modified (and current) pipe and valve arrangements are depicted in Figures 8 and 9.

3.6.2 Valves and Instruments

4-inch Shutoff Valves: The two 4-inch valves are K-Max rotary control valves (RCV) manufactured by DeZurik. DeZurik sold this line of valves in 1994 to Leslie Controls, which continues to produce RCV K-Max valves. Leslie Controls has a California representative, Birmingham Controls, that can supply information and field service for the K-Max valves. Product information for the 4-inch K-Max valves currently being manufactured by Leslie Controls is included in Appendix C. This literature depicts valves that are similar but not exactly equal to those installed in 1987.

The installed valves' nameplate data are as follows:

Code: GKM
352363
P/N: 2029397 (west valve); 2029399 (east valve)

PO NO: 12676 Guide: STELL
Body: 4.0 STL SS Plug: LIN
Trim Size: 4.0 Trim: S3
DEZURIK

According to the nameplate data, the installed valves are 4-inch, wafer style (flangeless), rotary half-ball plug valves with stainless steel bodies, plugs, and trim. The original assembly number indicates that the valves were ordered without actuators. According to Leslie Controls, each valve was originally equipped with the following items: a high strength 17-4 PH stainless steel shaft, Stellite alloy bearings, teflon packing, "full capacity" sized seat ring, and a special trim hardening feature composed of a Stellite alloy coating on the plug and seat ring. The valves are installed stem up with the plug-end oriented upstream. The plug can operate through a 90-degree rotation and, unlike most valves, the shutoff valve stem is rotated clockwise to open. The valves are precision flow control valves designed for highly erosive service and have a minimum shutoff rating of 285 psi (ANSI Class IV). The DeZurik service representative provided the information that the service life of the teflon packing is at least 25 years and that the packing should be in good condition in the mild acid (pH 4) environment. The valves are installed with black, 1/16-inch thick, raised-face flange gaskets that are in good condition. The exteriors of the valves are not corroded and are in good condition.

It was our understanding that the two 4-inch valves had been in the closed position, under upstream pressure, and as of November 1, 2000, had not been operated since they were installed in 1987.

Valve Actuators: Each 4-inch shutoff valve is equipped with a rotary, manual, handwheel actuator. The actuators are totally enclosed and have 6-inch diameter handwheels. The actuator enclosures are painted cast iron. The actuators appeared to be in operable condition, although the enclosures and handwheel stems are encrusted with metal oxide deposits and show external corrosion due to constant exposure to dripping acidic water. Unlike most valves, the handwheel is rotated clockwise to open the valve.

Manufacturer information for these actuators is not available. Manufacturer information for similar, but not equal, actuators is included in Appendix C.

Auxiliary Valves: The 3/4-inch ball valves are lever-operated and have threaded ends. The valves are marked "1000 wog CF-8M", which indicates that they are suitable for water, oil, or gas service at pressures up to 1000 psi, and are made of Type 316 stainless steel. All the ball valves are normally closed except for the isolation valve on the pressure transmitter piping. The ball valves showed no corrosion and appeared to be in good condition. There is one other 1/4-inch valve at the sample port on the pressure transmitter piping. The valve appears to be a globe valve and has a round plastic handwheel operator. The valve body material is slightly bronze in color and is marked "796F MUR". The valve was operated during the site visit to take a water sample from the pipeline. It is in good condition, has no corrosion, and is providing leak-tight closure.

Pressure Gauge and Pressure Transmitter: The pressure gauge was a 5-inch-diameter, glass-faced gauge with a range from 1 to 300 psi manufactured by Royal. The pressure transmitter has markings identifying it as a Model PTX 520 industrial pressure transmitter with a range from 1 to 300 psi manufactured by Druck, Inc (see Appendix C for product data). The transmitter has a 2-wire current output that provides a continuous 4-20mA signal to a Telog data logger located near the entrance of the mine. The direct current power for the transmitter and the data logger is provided by a multiple battery pack near the entrance of the mine. The wiring connecting the transmitter to the data logger is strung unprotected along the floor of the mine adit, making it vulnerable to being tread on and to rock falls.

3.6.3 Piping General Arrangement

4-inch Piping: The 4-inch piping is stainless steel of unknown type and is identified as Schedule 40 pipe on the original construction drawings. The spool flanges are welded, slip-on, raised face flanges and are marked 150 lb., Type 316 stainless steel with ANSI Type B16 bolt hole drilling. The tapped blind flange on the end of each drain pipe also is a raised face flange, and is marked 150 lb., Type 304L stainless steel with ANSI Type B16 bolt hole drilling. The 4-inch piping and flanges appeared in good condition and, although coated with metal oxide deposits, showed no corrosion.

The flanges take eight 5/8-inch diameter bolts. There are eight machine bolts with nuts securing the blind flanges to the adjacent spool pieces. At least one of the eight machine bolts and nuts appears to be carbon steel. Each 4-inch wafer (flangeless) shutoff valve is held in place by six tie bolts and two threaded rods that restrain the valve between the flanges of the adjacent pipe spools. The tie bolts and threaded rods were examined visually and with a magnet. At least one of the six tie bolts that restrain each valve between spool flanges appears to be carbon steel, not stainless steel. In addition, the threaded rods (wall anchors), which are embedded into the concrete of the plug wall, are also highly magnetic and appear to be carbon steel.

The stainless steel bolts and nuts appeared to be in good condition and showed no corrosion. The carbon steel tie bolts and nuts appeared to be in fair condition and showed noticeable corrosion, especially where the carbon steel is in contact with stainless steel. The threaded rod wall anchors appeared to be in good to fair condition. These rods are covered with oxide deposits, but showed only mild surface corrosion.

Auxiliary Piping: Most of the 3/4-inch and smaller piping is stainless steel. However, three segments of the 3/4-inch piping appear to be carbon steel pipe. Pipe metals were inspected visually, examined for markings, and checked with a magnet. The stainless steel piping was silver in color and not magnetic; the carbon steel piping was dull gray in color and strongly magnetic. One carbon steel pipe segment was the first segment on the piping branch for the pressure transmitter, just upstream of the isolating ball valve. (Note: This segment was replaced after the initial site visit. The pipe repairs are described in Section 4.) The other two carbon steel pipes are connected to the blind flanges, one on each flange. (See Figure 7.)

The stainless steel piping was found to be in good condition and showed no exterior corrosion. The carbon steel segment on the pressure transmitter piping appeared to be galvanized and was in fair condition with some visible corrosion at the threaded connections at each end. (As discussed in Section 4, the threaded connections in contact with [inside of] the stainless steel pieces were found to be severely corroded when the pipe broke during a subsequent site visit on May 31, 2001. Emergency remedial repairs, as discussed in Section 4, were required to replace this piece with a stainless steel piece.)

The entire surfaces of the carbon steel pipes downstream of the blind flanges are covered with a thin layer of corrosion. These segments have not been replaced with stainless steel, but they are not as critical since they are downstream of normally-closed valves.

Generic product information for piping and fittings, as well as data on various types of stainless steel, is included in Appendix C for reference.

3.6.4 Variances of Installed Drain Pipes from As-Built Drawing

During the inspection, it was observed that the installed drain pipes and valves do not entirely match the as-built drawing in Appendix A. The main differences between the as-built drawing and the actual installed arrangement are summarized below:

1. The two projecting piping assemblies are not strapped down and are not supported from below by the concrete block pipe supports that are shown on the as-built drawing. In actuality, the piping is cantilevered from the face of the seal.

2. The 4-inch shutoff valves do not have the flanges shown on the as-built drawing. In actuality, each valve is a wafer-type valve and is held in place by tie bolts that extend across the body of the valve between the flanges of the adjacent spool pieces.

3. The pipe branch for the pressure gauge and pressure transmitter is more complex than shown on the drawing. The installed pressure transmitter piping includes an isolating ball valve and a valved sampling port, along with the pressure gauge and pressure transmitter.

In addition, while the original design intent as we understand it was to use stainless steel for all wetted piping components, flange bolts, and wall anchors, the inspection revealed that the following piping components and fasteners appear to be carbon steel rather than stainless steel:

- The 3/4-inch pipe segment on east drain pipe pressure transmitter line. (The segment was replaced with a stainless steel pipe segment after the initial site visit.)

- One 3/4-inch pipe downstream of the blind flange for both the east and west drain pipes.

- The tie bolt at the 10 o'clock position on each 4-inch valve (looking downstream).

- The threaded rod wall anchors and the FxF threaded turnbuckle-type connectors that connect to the tie bolts at the 5 and 7 o'clock positions on each 4-inch valve.
- Miscellaneous tie bolts and nuts restraining the blind flanges. (The bolts and nuts restraining the blind flanges were replaced with stainless steel bolts and nuts after the initial site visit.)

3.7 AMD Sampling During Initial Site Visit

Acid Mine Drainage (AMD) was collected on November 1, 2000 from the following four areas:

- Pond: The standing pond in front of the seal was sampled, after about 20 hours of recharge and prior to any other activity in the mine, in order to provide a sample with as little disturbance as possible.
- Seep: Water seeping from the seal was sampled by collecting a drip stream emanating from the top of the seal.
- Drain pipe: Water in long-term contact with the east drain pipe was collected by sampling the first volume of water to flow out of the pipe upon opening of the sampling valve.
- Upstream: Water from the main body of AMD behind the seal was sampled by collecting flow from the drain pipe after approximately 25 gallons of water had been allowed to drain from the pipe. This represents 2 pipe volumes and means that water flowing through the pipe at the time of collection was sourced from AMD that had not been in long-term contact with the pipe prior to sampling.

Two unpreserved, one liter, samples from each area were collected. One sample from each area was placed into a cooler with ice for transport under standard chain of custody protocols to Sequoia Analytical in Sacramento, CA. Laboratory analysis of these samples is described in Section 4.8.

The other sample split was analyzed in the field for:

- pH
- Oxidation Reduction Potential (ORP)
- Dissolved Oxygen (DO)
- Iron (Fe)

These constituents were all analyzed via portable electrode except for iron, which was measured using a Hach DR2010 portable spectrophotometer and the Hach Ferrozine colorimetric method for iron.

Results of the November 1, 2000 field analyses are presented in Table 2 below:

Table 2
Field Determined Water Quality Parameters
(November 1, 2000)

Sample ID	Time	pH [Units]	ORP [mV]	DO [mg/L]	Fe [mg/L]
Seep	11:15	4.17	350	6.7	11.6
Pond	10:45	4.80	312	7.4	6.3
Drain pipe	11:20	4.53	286	1.7	40.3
Upstream	11:40	4.57	279	1.6	39.9

3.8 Discussion and Conclusions of Initial Site Visit

3.8.1 Seepage

The leakage observed just downstream of the seal is considered low and within the range of what can be expected for a well-constructed tunnel plug. As noted in Section 3.2, the seepage rate was estimated during the site visit to be 0.15 gpm. SRK estimated an average hydraulic conductivity of 10^{-5} cm/sec or less for the rock penetrated by the mine (SRK, 1985, p. 18). The measured seepage rate is consistent with this hydraulic conductivity, as demonstrated in this paragraph. For simplicity, the estimated seepage flow of 0.15 gpm can be assumed to be carried through an annular envelope of rock around the seal. Selection of the thickness of the ring of rock is somewhat subjective. Based on the observation that seepage is concentrated along the interface and within a short distance (a few feet) of the interface, we chose a thickness of 5 feet, although a slightly different thickness would yield similar results. An average hydraulic conductivity of 7 x 10^{-6} cm/sec can then be back-calculated for this ring of rock by applying Darcy's Law to the measured seepage flow, the assumed flow cross section, and an average hydraulic gradient of approximately 7. Thus, it can be concluded that the observed seepage conditions and flow rate are consistent with (1) an essentially impervious concrete seal and (2) bedrock hydrogeologic conditions as described in the original design documents.

More important than absolute flow amounts is the trend of flows versus time. In this respect we have only the following data to compare:

- MetaCon, Inc. letter dated June 6, 1988 (contained in SRK, March 1989): Describes a pressure of 11 psi (25 feet of hydraulic head) and a dry face of the seal except for a stained area approximately one-third of the way up on the west wall.

- SRK letter dated September 30, 1988 (contained in SRK, March 1989): Describes unquantified seepage under a hydraulic head of approximately 32 feet of water. Seepage was seen at the concrete/rock interface along much of the left side of the seal face and over a short section of the upper right side of the seal face.

- SRK letter dated December 20, 1989: Describes a small quantity of seepage, on the order of tenths of a gallon per minute, under a hydraulic head of 97 feet. Most of the leaking is reported to be confined to the interface between the seal and the roof, with no apparent leakage through the rock around the seal.

A comparison of these qualitative observations with the measured seepage rate of 0.15 gpm at 140 feet of head suggests that leakage is not increasing over time and may have decreased somewhat. However, it has not completely sealed itself over the 14 years since the seal was constructed.

We believe that periodic measurements of seepage flow (versus hydraulic head) are very important for long-term performance monitoring of the seal.

3.8.2 Rock Condition

With the exception of poor ground conditions in the first 1,300 feet of the adit, the adit appears to be sound and stable. The overbreak is generally in the roof and is believed to have occurred during the original excavation (SRK, 1985).

3.8.3 Concrete Seal

Based on the findings of the nondestructive tests and our visual observations, we conclude that, in general, the concrete forming the seal is in good condition.

The CTL report on nondestructive testing of the concrete (Appendix B.1) concludes that the overall condition of the concrete in the seal is good, except for a zone of poor concrete consolidation immediately surrounding and below the location of the original concrete tremie (pumping) pipe in the upper right quadrant of the seal (see Figure 5 for the location of the tremie pipe). This zone, with approximate dimensions of 4 feet high by 2 feet wide (see Figure D.2 in Appendix B.1) was probably caused by "blind spots" developing during the concrete placement, i.e. poor consolidation under and around the elbow of the original concrete tremie pipe. This zone is likely relatively shallow in thickness.

Typical values for concrete stress wave velocity in integral foundation piers with good concrete quality vary between 12,500 and 14,000 feet per second, with average values around 13,125 feet per second. The thickness of the seal calculated from the mobility plots presented in Appendix B.1 assuming a stress wave velocity in the concrete of 13,125 feet per second varies between 14.5 feet and 15.5 feet (versus a design length of 15.0 feet). Thus, the measured thickness of sound concrete agrees closely with the original design thickness. Conversely, for an assumed seal thickness of 15 feet, the average calculated concrete compression wave velocities are between 12,690 and 13,560 feet per second, indicating good quality concrete in the body of the seal with no discontinuities.

The shear wave velocity at the rock/concrete interface for different points around the plug perimeter was measured, ranging from 1,500 to 2,500 feet per second (450 to 750 m/s). As explained in the CTL report, these are very high values, indicating a good contact between the rock and the concrete. The lower values within this range are concentrated around the bottom center and the top of the plug, with the higher values along the sides of the plug over the lower two thirds. This is expected as a result of the concrete placement technique employed.

Based on surface examination of the concrete seal described in Section 3.4.2, it can be concluded that the concrete surface exposed to the acid mine water has been affected to some extent and has resulted in a softening of the surface paste and mortar. At present, the depth of this softening is small: generally less than 1.5 mm except below the pond water line where the depth may reach approximately 2 mm at the location we checked. Furthermore, softening has not resulted in loss of the surface paste by erosion. Concrete below this thin zone of softening is sound and hard, and most likely indicative of the concrete mass within the seal. We believe that concrete that is not exposed to direct contact with the acid water can be expected to be sound and hard. Additionally, we have no reason to believe that the physical condition of the concrete on the upstream face will differ significantly from that observed on the submerged portion of the downstream face because the acidity of the water upstream of the plug is similar to, or slightly lower than, that of the water in contact with the downstream face.

A small amount of seepage is occurring along the concrete/rock contact at the roof and sidewalls of the adit. This suggests that there are small, hairline openings between the rock and the concrete along the top and sides of the seal. The concrete along these hairline openings where seepage is occurring has probably been affected in a similar manner as that observed on the face, i.e., by softening of the surface paste. Over a long period of time, one could anticipate that the softened paste could be gradually eroded away, making the interface opening between the rock and the concrete gradually larger and allowing more water to flow through the interface, resulting in gradually increasing seepage flows. At this time, no increase in seepage rate has been observed that would indicate that the softening of the paste and mortar has been detrimental or has affected the integrity of the seal. Monitoring of the long-term seepage rate through the seal would provide advance warning for gradual deterioration of concrete along seepage paths.

Even if seepage were to gradually increase in the future, the increase in flows would not be a threat to the structural integrity of the seal until the water began to flow with significant velocity through the opening, causing mechanical damage to the rock or concrete at the interface. If this were to occur, it would take a significant loss of material to jeopardize the integrity of the seal, since the seal is wedged-in between the irregular surfaces of the adit rock. Monitoring of seepage rates should provide ample advance warning and would allow the RWQCB to undertake high-pressure grouting of the seepage areas to rehabilitate the seal.

Efflorescence is generally associated with the leaching of $Ca(OH)_2$ from the cement due to movement of water through, or adjacent to the concrete. When seepage water exits the concrete and reacts with air, $CaCO_3$ forms and is deposited on the concrete surface. The existing deposits of $CaCO_3$ observed on the face of the seal as described in Section 3.4 appeared to be very minor in nature and substantially less than the iron staining and iron deposits, particularly the iron deposits on the left-center portion of the seal.

Several horizontal lines were observed above mid-height on the face that might suggest the possibility of cold joints in the concrete during placement of the seal. However, close examination of these lines showed no evidence of poor consolidation or actual existence of cold joints. These lines appeared to be the result of the occasional flow of concrete against the form as it was filled by pumping.

3.8.4 Piping and Valves

Piping: The exterior of the exposed piping is generally in good condition. Although it is covered with iron oxide deposits, the exterior of the stainless steel components is not corroding. During this initial site visit, the exterior of the carbon steel components exhibited a variable amount of corrosion. As revealed by the pipe failure during the site visit of May 31, 2001, described in Section 4, the carbon steel 3/4-inch pipe segment in the pressure transmitter line was highly corroded and required emergency replacement. While corrosion of the remaining carbon steel components is not a cause for immediate concern at this time, attention will need to be given to the components over the long term, particularly for corrosion occurring at the contacts between dissimilar metals (carbon steel and stainless steel). Eventually, when there is an opportunity, all carbon steel components should be replaced with stainless steel components.

Valves: The valves appeared to be in good condition. However, as of the date of the initial site visit, the 4-inch shutoff valves in the seal drain lines, as well as most of the 3/4-inch ball valves, had not been operated in 13 years. When valves are pressurized and not exercised for such a long time there is a possibility that the moving parts may become bonded to the internal components of the valve. The actuators could also have become frozen. The valves were not leaking. (Later operational testing of the valves described in Section 4 revealed that the valves and actuators are operational.)

Pipe Supports: It was mentioned earlier that pipe supports for the 4-inch drain lines were contemplated in the original design but were not constructed. While pipe supports are not strictly required because of the short cantilevered length of the pipes, it would be advisable to install them to reduce the possibility of inadvertent overstressing of the pipes by human activity. Temporary supports were installed during the emergency pipe repair described in Section 4. Installation of permanent pipe supports is advisable.

3.8.5 AMD Chemistry

Samples of AMD were obtained from upstream and downstream of the seal for laboratory testing and geochemistry evaluations as described in Section 3.7. Based on the limited data collected in the field, the following observations were made:

- The samples are all moderately acidic with pH ranging from 4.2 to 4.8. The pH observed in these waters suggests that some of the host or wall rock contains some buffering capacity and therefore is able to neutralize some of the pyrite oxidation-generated acidity.

- The samples all contain some dissolved oxygen, which is expected in an actively oxidizing orebody. Oxygen is required for sulfide to oxidize to sulfate. The pond and seep samples are both in higher oxygen containing environments, since they are directly exposed to air.

- The dissolved oxygen content supports the Oxidation Reduction Potential (ORP) measurements. Most oxygen is observed in the seep and pond samples directly exposed to air, while the AMD behind the seal is much lower in dissolved oxygen. Behind the seal, diffusion of oxygen into the AMD and concurrent consumption by sulfide oxidation limits the dissolved oxygen content.

- Dissolved Fe in the seep is low (11.6 mg/L) compared to the AMD behind the seal (upstream sample) as is the pH of the sample. This is due to exposure of the leaking AMD in front of the seal to oxygen, which assists in conversion of Fe (II) to Fe (III). Once formed the Fe (III) rapidly hydrolyzes and precipitates as Fe(OH)3 solid. This reaction series liberates 3 moles of proton for each mole of Fe converting to Fe(OH)3, thereby lowering the pH. This explains the lower pH of seep AMD (4.2) compared to upstream AMD (4.6).

- Since the water behind the seal remains mildly acidic, with a pH slightly over 4, it is concluded that acid generating reactions continue in the mine workings. This is to be expected since a significant portion of the mine workings (between approximate elevations 6,400 and 7,000) is not flooded.

4. Additional Field and Laboratory Investigations

4.1 Purpose and Scope

Additional testing activities were planned and performed based on the observations and findings from the initial site visit, the initial phase of nondestructive tests, and the AMD sampling which are described in Section 3. This section describes the scope, procedures and results of the additional seal testing.

The scope of additional field and laboratory testing activities is summarized as follows:

(1) Drilling, Sampling and Grouting: Seven holes were cored through the downstream portion of the seal and adjacent rock to recover concrete and rock samples and to allow nondestructive testing of the seal and rock-concrete interface from the cored holes. The holes were grouted after the testing was complete.

(2) Nondestructive Testing of Concrete Seal (Second Phase): The condition of the concrete was evaluated by performing cross-hole sonic logging of the concrete mass. In addition, Schmidt hammer tests were performed on the face of the seal and adjacent rock to obtain a rough estimate of unconfined compressive strength and to develop a baseline, thus documenting the current conditions for use in future monitoring events.

(3) Nondestructive Testing of Exposed Mechanical Components: The ultrasonic thickness measurement technique was used to measure the wall thickness of solid steel in pipes and valves, therefore providing an indication of internal corrosion.

(4) Laboratory Testing of Concrete and Rock Samples: Core samples of concrete were tested to assess the pulse wave velocity and compressive strength of the concrete. Petrographic examinations of selected concrete cores were made to ascertain evidence of acid attack. Lastly, the compressive strength and pulse velocity of the support rock was also measured.

(5) Mapping of Rock Characteristics: The rock joints exposed immediately downstream of the seal were characterized in detail to estimate the rock mass rating (RMR) and to establish a baseline that can be used for future monitoring events.

(6) Operational Testing of 4-inch Shutoff Valves: The 4-inch shutoff valves were operated to verify that they still open and close. This activity included procuring and installing 4-inch backup valves downstream of the existing shutoff valves.

(7) Repair of Auxiliary Piping: Due to the pipe failure described in Section 4.7, the section of carbon steel pipe upstream of the pressure gauge was replaced with a stainless steel pipe section.

(8) Geochemical Evaluation of AMD Chemistry: The AMD samples taken from upstream and downstream of the seal were analyzed for major metals to assist in the identification of indications of concrete dissolution and steel corrosion.

These investigations are described in more detail below.

4.2 Coring, Sampling and Grouting

4.2.1 Coring and Sampling

Coring, sampling and grouting were performed by Jensen Drilling Company of Eugene, Oregon, from August 21 to September 16, 2001. Jose Cercone, an engineering geologist with Washington Group International, supervised the field work and logged concrete and rock samples collected during drilling. The scope of work and technical specifications for the coring and grouting subcontract are included in Appendix D. Core logs are presented in Appendix E.

Seven HQ (3.78-inch-diameter) holes were cored through the downstream portion of the seal and adjacent rock to recover concrete and rock samples and to allow nondestructive testing (NDT) of the seal and the rock-concrete interface. The core holes were used for the sonic logging described in Section 4.3 below. The locations of the core holes are shown on Figure 10. Six of the core holes, BH-1 through BH-6, were located along the perimeter of the seal and were angled away from the axis of the seal, with the objective of intersecting the rock-concrete interface within the middle third of the seal's length. This was in order to obtain maximum information on concrete and rock conditions while at the same time minimizing the risk of intercepting a pressurized seam. However, due to the extreme roughness of the rock walls, only four of the six holes intersected the rock-concrete interface. The seventh hole, BH-7, was drilled through the center of the seal and subparallel to its axis. The core hole lengths ranged from 7 to 10 feet. The total core hole length drilled was 61.5 feet.

Coring was accomplished using a CP55 air-driven core drill, a Bean air-driven water pump and a 750 Ingersoll Rand air compressor. A five-foot long Longyear HQ3 core barrel (3.62 inches outside diameter) was used. To obtain core hole alignment, a 12-inch-long, 4-inch-diameter Schedule 40 steel pipe was welded to a steel plate at the selected angle of the core hole. The plate was then bolted and sealed with epoxy to the surface of the concrete seal (See Photos No. 15 and 16).

Drilling was performed through the pipe into the face of the seal. The pipe was equipped with a valve and pressure gage to allow the control of water loss through the core hole should a pressurized joint or crack be intercepted (See Photo No. 17).

Water was intercepted in only one drill hole, BH-4, although water build-up was not enough to register on the pressure gauge. It was estimated that the seepage rate was 0.01 to 0.02 gpm.

4.2.2 Pressure Testing

Water pressure packer tests were conducted in the holes that intersected rock, i.e., BH-1, BH-2, BH-4, and BH-6, using a single packer. The holes were flushed with clean water before performing the tests. A Moyno pump was used to pump the high-pressure water. Flows were recorded using an Ashcroft flowmeter. The pressure tests typically tested the concrete closest to the rock-concrete interface, the interface itself, and the adjacent rock. Hole BH-7 was also pressure-tested because the core exhibited a diagonal crack and it was not obvious whether the crack was a mechanical break or a preexisting crack. The results of the packer tests are presented in Table 3 below.

Table 3
Water Pressure Testing of Boreholes

Borehole	Zone Tested* (ft)	Maximum Pressure (psi)	Maximum Take (cfm)	Estimated Permeability (cm/sec)
BH-1	5' to 10'	100	0.0	0
BH-2	3' to 8'	100	0.125	5×10^{-6}
BH-4	5' to 7'	100	0.12	2×10^{-5}
BH-6	5' to 9'	100	0.01	5×10^{-6}
BH-7	3' to 8'	100	0.0	0

(*) Measured from downstream face of seal

4.2.3 Grouting

Upon completion of drilling and nondestructive testing, the core holes that intersected the rock-concrete interface were pressure-grouted. A JDC 150 Grout Plant equipped with a high-speed colloidal mixer and a Moyno 3L-6 grout pump were used. Both were mobilized to the vicinity of the seal by transporting them over the rail tracks using a specially manufactured steel-wheeled cart. A thin grout using microfine cement with 3 to 1 water-cement ratio was injected to attempt to seal hairline cracks along the rock-concrete interface. A maximum grouting pressure of 150 psi was selected based on (1) the need to use a pressure greater than the water pressure behind the seal (41 psi at the time of grouting) and (2) the desire not to significantly exceed the grouting pressure originally used in the construction of the seal (reported to be 125 to 200 psi in SRK, September 1989). The pressures used and the grout take required to seal the core holes were measured and recorded and are summarized in Table 4 below.

Table 4
Pressure Grout Data

Borehole	Zone of Grouting (ft)	Maximum Pressure (psi)	Total Take After Initial Filling (cf)
BH-1	5' to 10'	150	0
BH-2	3' to 8'	150	0.08
BH-4	5' to 7'	150	0.09
BH-6	5' to 9'	150	0.08

Once grouting refusal was achieved, a thick, zero-bleed, non-shrink grout was used to backfill all holes. The collar pipes and plates were then removed, and the holes drypacked with portland cement grout (See Photo No. 18). The materials used to grout the holes are listed in Table 5.

Table 5
Materials Used to Grout Boreholes

Material	Manufacturer	Product	Total No. of Bags Used*
Microfine Cement	De Neef	Microfine Cement MC-500	3
Non-Metallic, Non-Shrink, Hydraulic Cement	Dayton Superior	Grout 1107	8
Portland Cement	Ash Grove	Type I-II	2

(*) Includes waste material filling hoses, partial bags, etc.

4.2.4 Miscellaneous Items

A technical consultation was received from Cal-OSHA during the coring and sampling program. The consultation indicated the need to upgrade the tunnel ventilation system during coring and grouting activities, with the objective of achieving a minimum air flow velocity of

60 feet per minute. The existing fan and duct system were measured to provide about 600 cubic feet per minute, or an air flow velocity of only about 10 feet per minute. The existing duct, an 18-inch-diameter fiberglass pipe manufactured by Schauenburg Flexadux Corporation, leaked badly at over one third of its joints. In addition, it was deemed that the existing fan, a 3 HP Jet Air propeller fan model AAA13 (serial # 1987491) might be underpowered to provide the required air flow even if the condition of the duct was upgraded. Therefore, a 20 HP propeller fan was rented by Jensen Drilling Company from American Mine Service Inc., 11808 West Highway 93, Boulder, CO 80303. The fan, model number 5624BP2 manufactured by Hartzell Propeller Fan Co., Piqua, Ohio, barely achieved the required air flow.

To further improve air flow, an inquiry was made with Schauenburg about sealing the duct joints. The suggested methods were (1) using industrial shrink wrap applied around the joints as a temporary fix, or (2) using an expandable foam sealer as a long-term repair. Due to time constraints, the shrink wrap method was used to seal all leaky and accessible joints. This resulted in a significant improvement in air flow through the adit.

To transport compressed air and clean water from the portal to the seal face, Jensen Drilling Company installed about 2,700 feet of 2-inch diameter galvanized steel pressure pipe labeled Hyundai A-53A-E, 2"x154x21'. This pipe, as well as the steel-wheeled cart, were purchased from Jensen at the end of the work and left inside the adit for potential use in the future.

4.3 Nondestructive Testing

4.3.1 Cross-hole Sonic Testing

Cross-hole sonic logging between the seven core holes was carried out by CTL on September 10 and 11, 2001. The testing was performed following the coring of the seal but before grouting. This method is described in detail in the American Concrete Institute Report ACI 228.2R-98. The method uses either core holes or pre-placed tubes in the concrete element to be tested. A transmitter probe placed at the bottom of one hole emits an ultrasonic pulse that is detected by a receiver probe at the bottom of the second hole. A recording unit measures the time taken for the ultrasonic pulse to pass through the concrete between the probes. The probes are sealed units, and the holes are filled with water to provide coupling between the probes and the concrete. The probe cables are withdrawn over an instrumented wheel that measures the cable length and thus probe depth. Continuous pulse measurements are made during withdrawal, at increments ranging from 10 mm to 50 mm (0.4 inches to 2 inches), providing a series of measurements that can be printed out to provide a profile of the material between the holes.

The ultrasonic pulse velocity (UPV) is a function of the density and dynamic elastic modulus of the concrete. If the signal path is known and the transit time is recorded, the apparent UPV can be calculated to provide a guide to the quality of the concrete. A reduction in modulus or density will result in a lower UPV. If the path length is not known, but the tubes are reasonably parallel, the continuous measurement profile will clearly show any sudden changes in transit time caused by a low pulse velocity due to low modulus or poor quality material, such as contaminated concrete or inclusions. Voids have a similar effect by forcing the pulse to detour around them, thus increasing the path length and the transit time. By varying the geometric arrangement of the probes, the method can resolve the vertical and horizontal extent of such defects, and locate relatively fine cracks or discontinuities. The major advantage is that the method has no depth limitation, unlike surface reflection methods.

For the test to be successfully performed, the core holes must be full of water. Holes 1, 5, 6 and 7 were inclined slightly downward making it possible to retain water in these holes during testing. Holes 2, 3 and 4 in the upper part of the plug were inclined upward in order to intercept the concrete/rock interface. Therefore, it was necessary to devise a valve system that would hold water in the hole while at the same time allowing the CSL probe and cable to be placed and operated in the hole. The system developed at the plug face to achieve the seal was successful, and valid CSL test profiles were obtained for all holes.

The CTL report presented in Appendix B.2 discusses the results of this testing. Perimeter and diagonal sonic profiles were obtained. The results are presented as plots of equivalent pulse velocity of the concrete for each sonic log profile in Appendix B.2.

4.3.2 Schmidt Hammer Tests

Nondestructive Schmidt hammer tests were performed on the downstream face of the concrete seal and on the rock surface immediately downstream of the seal to estimate the unconfined compressive strength of the rock and concrete surfaces and to compare these readings with the values obtained from laboratory testing of core samples. The tests were performed on the face of the seal and the adit's rock surface within 20 feet of the seal by GEI on September 10, 2001. Results and locations of the Schmidt hammer tests are presented in Table 1 of Appendix B.3 for the left side of the adit, Table 2 for the roof of the adit, Table 3 for the right side of the adit, and Table 4 for the face of the seal. An N Type Schmidt hammer was used. A rebound, or R value, was obtained for each test. The R value was corrected for the angle of the test relative to horizontal, based on the correlation presented in the operation manual of the N Type Schmidt hammer.

For the index values for rock, the R value from the N Type hammer (R_N value) was converted to an R value for the L Type hammer (R_L value) using the conversion expression from the manufacturer, as presented in Appendix B.3. Using the correlation developed by Deere and Miller (1966), the unconfined compressive strength was estimated from an average R_L value, using a rock unit weight of 165 pounds per cubic foot. The expression for this correlation is presented in Appendix B.3. The unit weight of rock was based on laboratory measurements from two rock core samples obtained at the seal in September 2001 (Appendix B.2, Table 2).

4.3.3 Nondestructive Testing of Exposed Mechanical Components

The internal condition of the exposed mechanical components (piping and valves) was assessed by thickness testing using ultrasonic testing equipment. Testing was performed on September 11, 2001, by Kleinfelder, Inc., at 63 points spaced along the drain pipes and valves. Ultrasonic waves were applied to the outside surface of the metal element. The waves travel through the element and are reflected back from the inside surface. The method measures the time of travel of the ultrasonic waves. The equipment was calibrated by inputting wave velocities for the applicable metal (stainless steel or carbon steel). A direct readout of thickness was obtained from each test. The results of this testing are presented in Appendix B.4 and summarized in Table 6 below.

4.4 Laboratory Testing of Concrete and Rock Samples

The concrete and rock cores were visually examined, and samples were selected by Alberto Pujol of GEI, Gary Mass, and Dr. Allen Davis of CTL for laboratory testing by CTL. Samples for testing were selected from holes BH-3, BH-6 and BH-7 for the following reasons:

- BH-3 was selected because it was the hole drilled through the top of the seal. The upper portion of the seal is an area of potentially weaker concrete and also is the area where most seepage is occurring.

- BH-6, the hole drilled through the bottom of the seal and into the floor of the adit, revealed a localized area of weak, possibly segregated, concrete. Samples of this and adjacent material were selected for testing.

- BH-7, the hole drilled through the center of the adit, was selected as a potential indicator of average concrete characteristics.

The following tests were performed:

- Unconfined compressive strength of five concrete (ASTM C42) and two rock cores (D2938) to confirm strength values assumed in the design. The locations and depths of the samples tested are shown on the drill logs in Appendix E. The results of the testing are summarized in Table 2 of Appendix B.2. The full reports are also included in Appendix B.2.

- Petrographic examination of three concrete samples according to the methodology presented in ASTM C856 to assess the condition of the concrete and evaluate possible deterioration from AMD attack. The full petrographic report is presented in Appendix B.2.

Table 6
Ultrasonic Thickness Measurements (Inches)

Item	Left (West) Drain Pipe			Right (East) Drain Pipe			Nominal Wall Thickness of Schedule 40 Pipe
	Minimum	Maximum	Average	Minimum	Maximum	Average	
4-inch Drain Pipe	0.224	0.240	0.231	0.213	0.226	0.221	0.237
4-inch DeZurik Valve Body	0.370	0.403	0.379	0.398	0.408	0.403	N.A.
4-inch Pipe Spool	0.227	0.229	0.228	0.229	0.238	0.232	0.237
4-inch Apollo Valve	0.227	0.236	0.230	N.T.	N.T.	N.T.	N.A.
3/4-inch Pipe Stubs Off Blind Flange	0.106	0.116	0.111	0.105	0.119	0.110	0.113
½-inch Pipe in Pressure Transmitter	N.A.	N.A.	N.A.	0.104	0.100	0.107	0.109
3/4-inch Pipe in Pressure Transmitter Branch (New)	N.A.	N.A.	N.A.	0.145	0.151	0.148	0.154*

Notes:
N.A.- Not Applicable, N.T.-Not Tested
* Schedule 80S

- Pulse velocity through five concrete core and two rock core samples according to the methodology described in ASTM C597 to correlate compressional wave velocities with the values assumed for NDT evaluations. The results of the testing are presented in Appendix B.2. Figure 2 of Appendix B.2 shows the relationship between core compressive strength and core pulse velocity derived from the five concrete samples.

4.5 Mapping of Rock Characteristics

Observations and measurements of the rock condition and quality next to the seal were made by GEI on September 10, 2001 to develop a Rock Mass Rating (RMR). The RMR was prepared in accordance with Table 1 of ASTM D 5878 (included in Appendix E.3). Six classification parameters are used to develop the RMR. They include the strength of the rock, the rock quality designation (RQD), the spacing of the joints, the condition of the joints (aperture, roughness, weathering), the groundwater inflow, and an adjustment for the joint orientation. A summary of the RMR for the rock in the 20-foot-long reach of adit immediately downstream of the seal is presented in Table 7. Details of RQD and joint spacing data used for classification parameters of the RMR are presented in Tables 2 and 3 of Appendix E.3, respectively.

4.6 Operational Testing of 4-inch Shutoff Valves

4.6.1 Background

As discussed in Sections 3.6 and 3.8.4, during the site visit of October 31, 2000, the two 4-inch shutoff valves appeared to be in good condition, but it was noted that the valves likely had not been operated for 13 years since they were installed in 1987. When valves are pressurized and not exercised for such a long time there is a possibility that the moving parts become bonded to the internal components of the valve. The actuators may also have become frozen.

It was recommended that appropriate measures be taken to verify that the valves remain operational and, thereafter, operate all the valves in the system on a regular maintenance schedule to ensure that they continue to operate smoothly and do not become frozen. However, when the valves are opened, the seat rings or valve stems could be damaged. If this occurs, it may not be possible to close the valve completely, or when closed, the valve may leak. Since the valves cannot be removed from the pressurized drain lines, a damaged valve may need to be abandoned in place until it can be removed and serviced in the future, perhaps when the mine is drained. Therefore, it was recommended that the existing piping be modified by installing a new 4-inch backup valve downstream of each existing 4-inch shutoff valve and the blind flanges reinstalled. Then the new backup valves could be closed and the operation of the existing 4-inch valves could be tested. The new backup valves and downstream 3/4-inch ball valves could then be opened to verify that there is no leakage past the existing 4-inch valves after the testing. If an existing 4-inch valve were damaged during

Table 7
Summary of Rock Mass Rating
for 20-Foot-Long Reach Downstream of Seal

Location of Measurement In Tunnel[1]		Classification Parameters[2]						Rock Mass Rating[8] (RMR)
		No. 1	No. 2	No. 3	No. 4	No. 5	No. 6	
		Strength of Intact Rock Material[3] (MPa)	Drill Core Quality, RQD[4] (%)	Spacing of Joints[5] (mm)	Condition of Joints[6]	Ground Water[7]	Rating Adjustment for Joint Orientation	
Right Wall	Measurement	132	64	660	slightly rough surfaces, separation < 1 mm, unweathered walls	damp	unfavorable[9]	
	Rating	12	13	15	25	10	-10	**65**
Roof	Measurement	132	82	864	slightly rough surfaces, separation < 1 mm, unweathered walls	damp	unfavorable[9]	
	Rating	12	17	15	25	10	-10	**69**
Left Wall	Measurement	132	64	772	slightly rough surfaces, separation < 1 mm, unweathered walls	damp	unfavorable[9]	
	Rating	12	13	15	25	10	-10	**65**

Notes:

1) Right and left sides of tunnel are relative to the direction of tunnel drive, facing the seal. All measurements are within 20 feet of the seal.
2) Parameters are from RMR Table 1, Geomechanics Classification Of Jointed Rock Mass, in ASTM D 5878 – 95, Standard Guide for Using Rock-Mass Classification Systems for Engineering Purposes.
3) Based on the average strength from two uniaxial compression tests performed by CTL, 16,240 and 22,020 psi, or 112 to 152 MPa, average 132 MPa. The unconfined compressive strength estimated from Schmidt Hammer tests confirm this value, see Appendix E.3.
4) Rock Quality Designation (RQD) is calculated as total of intervals greater than 4 inches between joint planes over measurement distance (see Appendix E.3, Table 2 for data).
5) Calculated as the average spacing of the joints within each joint set (see Appendix E.3, Table 3 for data).
6) Joint surfaces are slightly rough, unweathered, and have a separation <1 mm, but are continuous.
7) Inflow is minimal but rock surfaces are damp within 75 feet of concrete seal.
8) RMR range from 61 to 80 is considered "good rock" in ASTM D 5878 - 95.
9) Unfavorable rating is due to orientation of joints relative to tunnel axis, per RMR Table 2 in ASTM D 5878 - 95, drive against dip with dips from 20 to 45 degrees. A few joints strike parallel to tunnel axis mostly with near vertical dips, but range from 45-90 degrees

the test, it could be abandoned in place and the new 4-inch backup valve could be used as the drain shutoff valve.

4.6.2 Valve Testing and Maintenance

The operational testing of the valves was conducted on June 13, 2001 by Roy Parcell and Phillip Parcell of Pacific Mechanical Corp., piping/mechanical construction subcontractor to GEI. The work was performed under the supervision of Alberto Pujol of GEI and Cynthia Fox of Washington Group International. As a precaution, 4-inch backup valves were installed downstream of the existing shutoff valves prior to the operating test. The modified arrangement of the drain pipe assemblies, after installation of the backup valves, is shown on Figures 8 and 9. Catalog information on the new valves is presented in Appendix C.3. Details of the procedure are described below.

First, the 3/4-inch valves downstream of each blind flange were opened. Some water flowed out, but the flow stopped after a few seconds. This indicated that (1) the 3/4-inch valves had been sealing tightly and were not blocked; (2) once the fluid was released, there was no fluid pressure acting behind the blind flanges; and (3) the 4-inch control valves were sealing nearly leak-tight since flow out of the 3/4-inch valves stopped once the space between the 4-inch control valves and the blind flanges was drained, although it appears that over the years the 4-inch control valves have allowed passage of some fluid, which was contained in that space.

The 4-inch blind flanges were removed from the downstream end of the two drain pipes to allow the installation of the two new backup valves. With the blind flanges removed, it was noted that the existing control valves were installed with the plug–ends oriented upstream. Consequently, the inside chambers and stems of the control valves were visible and appeared to be in good condition with only slight amounts of dark deposits inside (See Photo No. 19). A new Apollo ANSI Class 150, flanged, 4-inch, stainless steel, lever operated, ball valve was added as a backup valve downstream of each control valve (Photo No. 20). The backup valves were installed using 1/8-inch thick, full-face, natural rubber gaskets and stainless steel bolting. The blind flanges were then reinstalled with the same type of gaskets and bolting.

The backup valves are equipped with actuator mounting plates that have standard ISO bolt holes and dimensions. This would allow these valves to be fitted with remotely controlled actuators if desired in the future.

To test the operation of the control valves, the downstream backup valves and 3/4-inch valves were closed. The left and then the right control valve was opened and closed four times using the handwheel actuators. Both the valves and the actuators operated smoothly, without hesitation, and shut tightly. Each pipeline was purged once for a few seconds by opening the downstream valves to clear debris from upstream of the valve seats. Fluid release was limited by the 3/4-inch valves. The control valves, backup valves, and 3/4-inch valves were all closed at the end of the valve testing.

Two adjustable stainless steel pipe supports were cut to length and installed between the drain pipes and the rock floor of the adit. Because of the irregular rock surface, blocks of wood were used to provide a flat surface under the supports. These supports are considered a temporary feature. The new backup valves were fitted with padlocks to lock the valves in the closed position. Lastly, the control valve actuators were covered with plastic to help minimize corrosion and the build-up of mineral deposits from seeping water (Photo No. 21).

4.7 Repair of Auxiliary Pipe

4.7.1 Background

During the site visit of November 1, 2000, it was observed that a pressurized segment of the auxiliary 3/4-inch piping upstream of the pressure transmitter (and upstream of any valves) appeared to be carbon steel rather than stainless steel. It was also noted that the pipe appeared to be only in fair condition with corrosion at the threaded connections at each end. On May 31, 2001, during a prebid site visit for the drilling and grouting subcontract, a break occurred in the carbon steel segment of the 3/4-inch pipe at the threaded connection into a 1-inch by 3/4-inch reducing bushing which in turn threaded into a 1-inch threadolet welded to the 4-inch drain pipe (see Figure 7 and Photo No. 27). Observation of the fracture indicated that the carbon steel pipe at this location was badly corroded and paper-thin. A short duration release of a limited quantity of AMD occurred. Emergency, temporary plugging of the pipe was immediately accomplished with a wooden plug to stop the flow.

4.7.2 Piping Repair

The pipe repairs were accomplished on June 14, 2001 by Roy Parcell and Phillip Parcell of Pacific Mechanical Corp., piping/mechanical construction subcontractor to GEI. The work was performed under the supervision of Alberto Pujol of GEI and Cynthia Fox of Washington Group International. The repair involved first removing the large wooden brace holding the temporary wooden plug in place (Photos No. 22 and 23). Then the broken stub was removed by unscrewing and removing the 1-inch by 3/4-inch reducing bushing from the main pipe, allowing fluid to escape the pipe through the 1-inch opening. An assembly of stainless steel connecting piping was quickly screwed into the main pipe. The assembly included a new 1-inch by 3/4-inch bushing, a new 3/4-inch threaded nipple, and the old 3/4-inch ball valve (Photos No. 24 and 25). The connection work took about one minute. The ball valve was then closed, stopping the discharge. The rest of the pressure gauge assembly was reinstalled and the pressure transducer sensor was reconnected. Teflon tape and pipe dope were used for all screwed pipe connections. The piping was checked for leaks. The old pressure gauge had cracked and was replaced with a new 4.5-inch-diameter, stainless-steel, oil-filled gauge with a range from 1 to 300 psig manufactured by Ashcroft. The pressure gauge registered a fluid pressure of 45 psi (Photo No. 26). Data on the pressure gauge is included in Appendix C.5. Finally, the date and time on the pressure transducer data logger were reset by Rob Busby and Steve Rosenbaum of the RWQCB using a portable computer.

4.8 AMD Geochemistry

As described in Section 3.7, Acid Mine Drainage (AMD) was collected on November 1, 2000 from the following four areas:

- Pond: The standing pond in front (downstream) of the seal.
- Seep: Water seeping from the contact between the seal and the roof of the adit.
- Drain pipe: Water in long-term contact with the east (right) drain pipe.
- Upstream: Water from the main body of AMD behind the seal.

One unpreserved, one-liter sample from each area was placed into a cooler with ice for transport under standard chain of custody protocols to Sequoia Analytical in Sacramento, CA for analysis of:

- pH
- Total Dissolved Solids (TDS)
- Acidity
- Alkalinity
- Calcium, Copper, Iron, Aluminum, Potassium, Magnesium, Manganese, Sodium, Lead, Silicon, Chromium, Nickel, Molybdenum, Titanium, and Zinc.

These constituents were analyzed via the standard USEPA methods. Results from Sequoia Analytical Laboratory are included in Appendix F.

4.9 Evaluation of Data from Additional Investigations

4.9.1 Observations from Drilling and Grouting

Both drilling conditions and examination of the concrete samples indicate that the concrete in the seal is generally dense, sound, and hard (see core logs and photographs in Appendix E). The coarse aggregate consists of angular, light colored, limestone particles. The matrix of cement paste and siliceous and calcareous fine aggregate is generally dense and hard. No indication of weathering or reaction rims was observed. All breaks in core samples were deemed to be mechanical breaks. No large voids or poorly consolidated concrete were found.

The core from hole BH-7 presented a diagonal fracture at a depth of 3 feet. The evidence on whether this is a mechanical break or a pre-existing fracture is inconclusive. Close examination revealed that the fracture passes through aggregate particles, suggesting a mechanical break, perhaps along a plane of weakness. However, the examination also revealed small amounts of carbonate material on the fracture surface. If these are secondary mineral deposits, they would suggest that the crack was pre-existing, perhaps a cold joint formed during concrete placement (or from shrinkage). In any event, water pressure testing of hole BH-7 yielded zero water loss through this zone. Furthermore, the cross-hole sonic logging profiles between this hole and adjacent holes did not indicate any defects, suggesting that this occurrence was either a mechanical break or a very localized feature.

In general, no segregation was noted, except for an area of soft, weak, concrete that was intercepted at the bottom of the seal by hole BH-6. The concrete in this area lacked coarse aggregate, probably from segregation during placement. However, the cross-hole sonic logging profiles between this hole and adjacent holes did not reveal an area of weakness, indicating that the area of poor concrete intercepted by hole BH-6 is likely to be very localized.

Based on examination of the cores retrieved, the concrete in the vicinity of the contact with the adjacent rock is of the same quality as that of the concrete in the center of the seal, with the exception of BH-6 noted above. No weathering, reaction rims, or significant indications of acid attack were observed.

While four holes penetrated the rock-concrete interface, we obtained only one sample of concrete that was adhered to rock (from hole BH-1). Elsewhere, breaks occurred between the concrete and the rock. This is not surprising given that the angle of the holes to the rock wall was generally only a few degrees; the small angle and the difference in hardness between the concrete and rock may have caused the drill bit to try to deviate from its alignment, possibly vibrating and causing mechanical breakage of the cores.

The rock around the seal consists of granitic rock and is hard, generally moderately fractured, fresh, and with generally clean joints. No evidence of acid attack of the rock was observed.

Only one of the holes extending into rock yielded some water, at an estimated rate of 0.01 to 0.02 gallons per minute. Water pressure tests of the rock and rock-concrete interface indicated a hydraulic conductivity in the range of 10^{-5} cm/sec, which is consistent with the hydrogeologic model of the rock mass developed for design of the seal. Pressure grouting of the rock and rock-concrete interface took only minimum quantities of a thin microfine-cement-based grout, again confirming the tightness of the seal.

4.9.2 Cross-Hole Sonic Logging Results

Twelve cross-hole sonic logging profiles were obtained by CTL and are presented in Appendix B.2. Measurements of equivalent pulse velocities gave between 12,000 and 12,500 ft/sec. All test profiles showed continuous concrete between holes, with no breaks in arrival signals at any point. Where the core holes penetrated the rock, if poor concrete consolidation or open fractures were to exist at the rock-concrete interface, a break in the signal arrival time would be present in the test traces. This was not seen in any of the profiles. The tests showed sound and integral concrete in the areas of the seal that were explored.

The first phase of nondestructive concrete testing performed by CTL had indicated a surficial area of potentially poor concrete consolidation in the northeast quadrant of the seal, under the original tremie pipe. The cross-hole sonic logging profile between holes BH-3 and BH-4 explored this area. No significant reductions in equivalent pulse velocities were noted, suggesting that the concrete in this area is reasonably sound.

4.9.3 Lab Testing Results

Five compression strength tests were performed on concrete core samples, yielding compressive strengths ranging from 5,780 to 6,750 psi and averaging 6,150 psi. In December 1987, the average compressive strength of the concrete in the seal 28 days after placement was determined to be 5,550 psi based on tests of two cast cylinders (SRK, March 1989). The compressive strengths measured for this study compare well with the original strength, indicating that a limited increase in compressive strength has occurred with age. No adverse effects on the compressive strength from acid attack are apparent in the tested materials.

The unit weight of the concrete cores tested for compressive strength was measured. The unit weight values range from 137 to 146 pounds per cubic foot (pcf) and average 140 pcf. These values are indicative of reasonably dense, well-consolidated concrete. The laboratory ultrasonic pulse velocity of these cores was also measured, ranging from 13,211 to 14,889 feet per second (fps) and averaging 13,884 fps. These values are in the range expected for well-consolidated concrete of this type.

Petrographic examinations were made of three core samples obtained from holes BH-3, BH-6, and BH-7. The sample from hole BH-3 was taken from close to the roof of the adit, an area of the seal where weaker concrete may be expected to occur. The sample from hole BH-7 was from the center of the seal and included the diagonal crack discussed above. The sample from hole BH-6 was taken from the weak, segregated concrete also discussed above; selection of this sample was made with the express purpose of testing the worst concrete revealed by the core holes. The petrographic reports are included in Appendix B.2. In summary, the core samples from BH-3 and BH-7 were found to be similar in composition and quality. The general quality of the concrete was found to be good, with no major abnormalities observed. By contrast, the quality of the core sample from BH-6 was poor, exhibiting soft paste and no coarse aggregate, possibly due to segregation. Please refer to the Petrographic Services Report in Appendix B.2 for a detailed explanation of findings and photographs of the samples examined.

Similar tests were performed on two cores of granitic rock. The measured compressive strengths were 16,240 and 22,020 psi. The measured ultrasonic pulse velocities were 18,251 and 19,118 fps. Both cores had a unit weight of 166 pcf. These results are typical of sound, unweathered granitic rock. Petrographic examinations of the rock were not deemed necessary because the rock appeared fresh and sound to the naked eye.

4.9.4 Schmidt Hammer Test Results

The Schmidt hammer provided good index measurements of the unconfined compressive strength of the rock and joint surfaces along the walls and roof of the adit near the seal. The average index value for the adit's left wall, right wall, and roof provided a correlation to the unconfined compressive strength of the rock, which ranged from 17,000 psi to 22,000 psi for the rock, and 20,000 to 21,000 psi for the joint surfaces. These strengths are within the range measured in laboratory tests on rock core samples taken near the rock-concrete contact of the seal, which were from 16,240 to 22,020 psi.

The Schmidt hammer provided index strength measurements of the concrete surface of the seal. The average index value, 2,600 psi, provided a correlation to the unconfined compressive strength for the concrete at the surface of the seal. Strength measurements from laboratory tests on concrete core samples of the seal were more than twice those estimated using the Schmidt hammer. It is likely that the thin layer of deteriorated, soft, cement paste observed on the face of the seal had the effect of significantly reducing the Schmidt hammer index values. The main value of these data is to establish a baseline that can be used in future monitoring events to assess trends.

4.9.5 Rock Conditions Downstream of Seal

The observations and measurements of the rock condition and quality document the current conditions in the 20-foot-long reach of the adit immediately downstream of the seal. Selected data (on joint strength and condition and groundwater inflow) can be adopted as a baseline for use in future monitoring events.

The Rock Mass Rating (RMR) for the rock near the seal indicates "good" quality rock. The RMR range was 65 to 69, as listed in Table 7. This agrees with the observations made in the adit during site visits in November 2000 and September 2001. The rock appeared sound and the joints were tight. There was little evidence of rockfall in this section of the adit, indicating that the rock has been stable since the mine closed six decades ago.

GEI obtained a lower RMR rating (RMR=65 to 69) for the reach of adit immediately downstream of the seal than presented in SRK's 1985 report for the seal location (RMR=81). This is mostly due to the reduction in rating caused by unfavorable joint orientations, RMR parameter number six, which SRK apparently did not evaluate. It is also possible that rock conditions at the location of the seal were marginally better than those immediately downstream.

4.9.6 Valves and Pipes

All valves in the drain pipe system were exercised and were found to be functional. The actuators of the 4-inch DeZurik control valves were not frozen and operated smoothly. The valves gave tight shutoff upon closing. During the pipe repair activities, the interior of the 1/2-inch stainless steel pipe immediately under the pressure gauge was examined and observed to have no noticeable corrosion (Photo No. 27). This pipe is normally filled with water under pressure. Likewise, the interior of the DeZurik valves was observed from the downstream side (the nonpressure side, see Photo No. 19). While this side of the valve was filled with water prior to the inspection, there was no discernible corrosion of the visible interior stainless steel components of the valves.

The results of the ultrasonic thickness testing of pipes and valves are given in Appendix B.4 and summarized in Table 6 in Section 4.3.3. The table also lists the nominal wall thickness for Schedule 40 pipe. The data confirms that the installed piping is Schedule 40, except for the new piece connecting to the pressure gauge, which is Schedule 80. As can be seen from the table, variances between the nominal wall thickness and the ultrasonic thickness measurements are relatively small and judged not significant. All measurements are within the permitted manufacturing tolerance, which is +/- 12.5% of nominal thickness. The main value of this data is to establish a baseline that can be used in future monitoring events to assess trends.

4.9.7 Water Chemistry

Selected results of the laboratory analysis of samples collected from the seep and upstream samples are presented below in Table 8 (see Appendix F for WAI's report on the geochemistry of the AMD including complete water chemistry data).

Table 8
Selected Results of Laboratory Chemical Analyses of Water Samples Collected November 1, 2000

Analyte	Upstream Sample (mg/L)	Seep Sample (mg/L)	Difference (mg/L) (Increase or Decrease)
Acidity	92	59	-33
Alkalinity	0	0	0
Aluminum	2.2	4.0	+1.8
Calcium	29.3	24.7	-4.6
Copper	1.8	9.1	+7.3
Iron	36.1	7.8	-28.3
Potassium	2.1	2.0	-0.1
Magnesium	7.3	5.8	-1.5
Manganese	5	3.2	-1.8
Zinc	0.6	0.7	+0.1
Sodium	2.3	2.2	-0.1
Silicon	13	12.9	-0.1
pH	4.6	3.8	-0.8

In general the major chemical and minor chemical constituents of the water upstream of the seal and the water seeping from the rock-seal contact are nearly identical. There are only a few noteworthy differences. The important points are noted below:

- The pH of the upstream sample is higher than that of the downstream (seep) sample, while the Fe in the upstream sample is much higher than in the seep sample. As noted before, the Fe (II) oxidizes to Fe (III) and then precipitates as the hydroxide, with the generation of additional acidity and the corresponding decrease in pH. Field observations on the condition of the concrete seal face confirm that heavy iron staining has occurred on the concrete face and drain pipes, consistent with the precipitation of Fe (III) upon oxygenation.

- Silicon in the two samples is identical, suggesting that the silicate portion of the concrete seal matrix is intact. The observed silica concentrations are likely from wall or host rock dissolution upstream of the seal. If the silicate portion of the seal were dissolving, the seep water would likely show a gain in silica content. This is not observed.

- Zinc and copper both increase in the seep water compared to the upstream sample. The cause of this increase is unknown but might be due to the decrease in pH.

- Magnesium, calcium, sodium and potassium are all major components of concrete and possibly the mine wall rock. Concrete dissolution would likely increase these components as AMD passes through the seal if significant dissolution occurred. The observed data show that there is very little difference in chemistry at the two locations. The slight changes noted in the table are well within typical analytical variance and cannot be used to conclusively identify an increase or decrease in concentration.

- Aluminum tends to increase in the seep water compared to the upstream water suggesting that a gain in Al occurs as AMD seeps from the seal. Aluminum is a significant component of both concrete and wall rock. However, aluminum is usually more resistant than cations such as Mg, Na, K and Ca to acid attack due to its role as a molecular structural element. Therefore, increases in Al should be accompanied by increases in these other cations as well. This is not observed. The increase in Al in the seep may be due only to the decrease in pH which solubilizes Al in the rock through which the seep runs.

In summary, based on the chemistry discussion above, there is no apparent difference in most chemical constituents occurring in the AMD behind the seal or in the seep flowing along the contact between the seal and the wall rock. The differences in chemistry between the locations are limited mainly to pH and Fe. In other cases where small differences in chemistry occur, they tend to be within typical analytical variance (<25%) suggesting that most differences may not be significant. Repeated sampling of the different locations would be necessary to determine if the differences are consistent and real.

It should also be noted that since the pH of the seep and pond samples is lower than that observed in the AMD upstream of the seal, acid attack may be slightly more aggressive on the downstream face of the seal.

5. Additional Analyses and Evaluations

5.1 Review of Geotechnical and Structural Design

5.1.1 Description of the Seal Design

The seal in the adit of the Walker Mine was designed to prevent direct discharge of acid and heavy metal-laden waters from the mine. SRK evaluated the seal design in their report, "Walker Mine Project Draft Final Feasibility and Design Report," dated September 1985. In the report, SRK determined that a 15-foot-long concrete seal in the adit would be adequate to resist the design water pressures expected in the mine. The seal would resist the water pressures by transferring the load from the concrete to the rock mass next to the seal through the rock-concrete interface.

The maximum water pressure head expected on the back of the seal is 570 feet, which is the elevation difference between the Main Access Drive Adit and the location where water could exit at the Old Sawmill Adit. SRK used a factor of safety of 2.5 for this water pressure head, giving a design water pressure head of 1,400 feet. For design, this pressure was applied to the back surface of the concrete seal.

SRK reported that the rock at the seal was a coarse grained granodiorite, and estimated a uniaxial compressive strength of 22,000 to 29,000 psi, which is considered strong rock. The weathering of the granodiorite was fresh. SRK found that the quality of the rock mass at the seal was "good" based on the Rock Mass Quality (Q) system, and "very good" based on the Rock Mass Rating (RMR) system. These classification systems evaluate the strength of the rock mass and are described in the 1985 SRK report.

SRK assumed a concrete strength for the seal of 2,000 psi, and determined that the concrete's resistance to punching shear was adequate. SRK also checked the shear strength of the seal for a potential failure surface along the rock-concrete interface. SRK's design evaluated the shear strength along this failure surface based on its surface area and the estimated shear strength of the rock mass along the rock-concrete interface. SRK estimated the shear strength of the rock mass based on Hoek's principal stress relationship for the type and quality of the rock mass observed at the seal, as quantified by the RMR and Q ratings. SRK assumed that the normal stress acting across the rock-concrete interface would be negligible. For design of the seal, SRK estimated a shear strength of 182 psi for the rock mass adjacent to the rock-concrete interface.

Water seepage around the seal is controlled by the condition of the rock mass surrounding the seal and the water pressure gradient along the length of the seal. A maximum allowable water pressure gradient of 40 psi per foot and a design water pressure at the back of the seal of 1,400 feet of head were used for design to establish a minimum seal length of 15 feet. This condition controlled the design length of the seal.

5.1.2 Construction Information

SRK described the seal construction and as-built condition in a letter report dated March 7, 1989. SRK reported that the seal location was properly prepared in accordance with the specifications before concrete was placed to allow a good bond between the concrete and the rock. Soil debris was excavated from the tunnel invert and loose rock pieces were scaled to expose a sound rock surface in the tunnel. Water was used to clean the rock surface of mud and dirt. Joints and fractures exposed in the rock surface were tight and not considered by SRK as a conduit for unacceptable seepage around the seal. The joints form irregular rock surfaces with a stepped pattern of asperities about 8 inches deep and spaced several feet apart. SRK considered the asperities to provide a good interlocking surface for the rock-concrete contact.

A video was recorded by the RWQCB documenting the condition of the rock during preparation of the seal area and before concrete placement. For the most part, the video confirms the observations in SRK's 1989 report. The surface of the tunnel walls, invert, and roof appear to have sound rock with tight joints. When the video was recorded, it appears that the rock in the lower walls and invert had been cleaned. Some reddish mud appeared to remain coating portions of the rock in the upper walls and roof. Cleaning operations were still in progress, and these areas may have been cleaned but not recorded.

Mine water was controlled at the seal to prevent accumulation in the tunnel invert. However, at the start of the concrete pour an unexpected surge of water entered the seal area and accumulated in the invert of the tunnel. During the concrete pour, the concrete reportedly displaced the water and the contractor did not consider the water harmful to the performance of the seal, even though the specifications required the concrete to be placed in the dry.

5.1.3 Data and Results from the Testing and Evaluation Activities

Rock

As indicated earlier, Schmidt hammer index values provided a correlation to the unconfined compressive strength of the rock at the surface and indicated a range of 17,000 psi to 22,000 psi for the rock mass, and 20,000 to 21,000 psi for the joint surfaces. Laboratory test results on rock core samples yielded compressive strengths of 16,240 and 22,020 psi.

GEI reviewed the Rock Mass Rating (RMR) for the rock immediately downstream of the seal. The RMR indicates "good" quality rock with values of 65 to 69.

Based on these observations and measurements, GEI estimated the shear strength of the rock mass around the seal using Hoek's principal stress relationship for the type and quality of the rock mass observed immediately downstream of the seal. GEI estimated a shear strength of 150 psi for the rock mass adjacent to the rock-concrete interface based on good quality granodiorite with an RMR of 65 and an approximate average rock uniaxial compressive strength of 20,000 psi and maintaining SRK's assumption that the normal stress acting across the rock-concrete interface is negligible.

Concrete

Field tests on the seal indicate that, to the depth tested, the concrete is dense, continuous, and in good contact with the surrounding rock. Strength measurements from laboratory tests yield values of 5,600 to 6,200 psi. The Schmidt hammer measured an index strength of 2,600 psi for the concrete surface of the seal. SRK's stability analyses of the seal used a concrete strength of 2,000 psi; thus, the concrete is stronger than assumed in the design.

Rock-concrete interface

Rock conditions exposed in the walls and roof of the tunnel next to the seal are similar to the rock conditions along the rock-concrete interface observed in the rock core obtained during our investigation and documented by others in the as-built report and video recording. The information on rock quality next to the seal can therefore be used to estimate the rock quality at the rock-concrete interface of the seal.

5.1.4 Discussion

The condition of the 15-foot-long concrete seal and surrounding rock mass is adequate to resist the design water pressure head of 570 feet, with a factor of safety of 2.5 on the water pressure based on our interpretation of the original seal design, seal construction records, field and laboratory data collected for this study, and our estimates of the material properties of the seal and surrounding rock. The factor of safety used for the original design, i.e., 2.5 times the maximum water pressure, is still reasonable. In our review of the seal design, we evaluated stability for (1) shear through the concrete and (2) shear through the adjacent rock mass. Shear along the rock-concrete contact is not considered to be physically possible given the large asperities in the rock surface. Shear failure along the rock-concrete interface would require dilation of the interface to allow the concrete to slip along the large rock asperities. A large dilation cannot physically occur in the adit because the rock-concrete interface of the seal is confined on all sides.

5.2 Review of Concrete Mixture Data Versus Published Data on Acid Attack of Concrete from the Iron Mountain Mine

5.2.1 General

The concrete mixture and placement data from the Walker Mine Seal - Final Construction As-Built Report, by SRK dated March 1989, was analyzed and compared with the results of the Iron Mountain Mine (IMM) studies involving field performance of concrete in an AMD environment. The purpose of this comparison was to develop an order-of-magnitude estimate of the rate of acid attack and depth of affected concrete at present, and an order-of-magnitude prediction of expected depth of attack at the end of the 100-year design life of the Walker

Mine seal. The analysis was performed by Gary Mass, an independent concrete consultant under subcontract to GEI. The complete study is presented in Appendix G.1 and is summarized herein. A summary of the field studies for IMM is presented in the September 2000 article entitled "Resistance of Concrete to Acidic Water" (Connell et al., 2000).

5.2.2 Characterization of Concrete in the Seal

The concrete mixture design for the Walker Mine seal was prepared and presented by Engineering Testing Associates, Inc. of Sparks, Nevada in their report dated October 14, 1987 (contained in SRK, 1989). The mixture and sources of materials are presented in Table 9 below.

On November 13, 1987 the concrete seal was placed. Details of the placement of the seal are presented in Appendix G.1. A total of ten 8-cubic-yard batches (80 cubic yards) were prepared using an onsite batching plant. Only 6 cubic yards of the ninth batch and none of the tenth batch were discharged into the concrete pump. The estimated volume of concrete actually placed in the roughly 9 foot wide by 12 foot high by 15 foot long seal was 66.2 cubic yards.

Water content of each batch was varied to maintain as consistent a slump of concrete entering the pump as possible. Except for the partly-used ninth batch, the water content of concrete was less than the mixture design water content (292 pcy). Water content and water-to-cementitious materials ratio for the batches are presented in Appendix G.1. The average water-to-cementitious materials ratio was 0.41.

During the placement, concrete samples were taken at the pump and at the discharge end of the pump line. The average slump at the pump was 6.6 inches and average slump at the seal was 4.4 inches. Ambient temperature at the seal was a stable 47° F and average concrete temperature at placement was 54° F. At approximately mid-point of the third batch, a set of cylinders was cast at the seal. The cylinders were later tested with the results shown on Table 10.

Table 9
Concrete Mixture Design Data
Walker Mine Seal

Material	Source	Pounds per Cubic Yard
Portland Cement, ASTM C 150, Type II Low Alkali	Calaveras	450
Pozzolan, ASTM C 618, Class N	Lassenite	150
Silica Fume, FORCE 10,000	W.R. Grace	49.5
Water		292
Fine Aggregate, ASTM C 33	Teichert Aggregates	1476 (S.S.D.)
Coarse Aggregate, ASTM C 33, Size No. 67 (limestone)	Sierra Aggregates	1546 (S.S.D.)
High-range Water-reducer, ASTM C 494, Type G, Daracem 100	W.R. Grace	(108 oz.)
Total Weight		3963.5
Design compressive strength, 28 days		3000 psi
Design slump, after addition of Daracem 100		7 inches
Water-to-cementitious materials ratio		0.45, by wt.

Table 10
Compressive Strength of Concrete Sample

Concrete Age	Compressive Strength (psi)
3 day	1390
7 day	2620
28 day	5550 (average of two specimens)

5.2.3 Exposure Conditions

During the concrete seal design phase and more recently during these investigations, AMD water samples were tested. It is well recognized that acid waters, when in direct contact with concrete, attack portland cement by dissolving and removing part of the cement constituents, leaving behind a soft, mushy mass. The rate of attack has been reported to depend, in part, on the acidity of the water's pH (Lea, 1971). A summary of the pH test results on the various water samples is presented in Table 1, Section 1, and in Appendix G.1.

It can be assumed that AMD water began to pond against the upstream face of the seal while concrete placement was completed. Recent water quality tests would suggest that the pH of this water was about 4.6. The actual time for the AMD reservoir to reach full-height of the seal is not known. Contact grouting of the uppermost crown of the seal was performed on November 23, 1987, ten days after concrete placement.

Since installation of the seal some seepage has developed. The most significant seepage appears to be along the concrete/rock contact at the top of the seal. However, staining indicates some seepage along both sidewalls. Water quality samples of the uppermost seepage show an average pH value of about 4.0. In addition to the seepage, a permanent, approximately 18-inch-deep, pond has formed in the low area at the downstream toe of the seal. The pH of the pond water appears to vary from 3.7 to 4.8. Based on the above, pH values for analysis of the seal were taken as 4.6 for waters against the upstream face of the seal and 4.0 for seepage.

5.2.4 Iron Mountain Mine Studies

In late December of 1998, as part of the mine reclamation studies for the Iron Mountain Mine in California, a series of concrete mixtures were batched and mixed at a local concrete plant in Redding, California. The objective of this work was to investigate the performance of various types of concrete when exposed to AMD water for an extended period of time.

A total of seven different mixtures were prepared in 2-cubic-yard batches. For each mixture, the concrete was tested for fresh concrete properties and twenty 6-inch-diameter by 12-inch-high cylindrical concrete specimens were cast. Twelve of these specimens for each of the seven mixtures (total of 84 specimens) were cured under standard ASTM C 39 conditions and were tested for compressive strength at ages of 7, 28, 56, 90, 180, and 302 days; two specimens were tested at each age. The remaining eight specimens for each of the seven mixtures (total of 56 specimens) were cured, surface-dried, and weighed. These specimens were then taken to the Iron Mountain Mine area and immersed in acidic waters. Four specimens of each mixture were immersed in AMD water with a pH of approximately 2.4 and the other four specimens of each mixture were immersed in a small stream of surface drainage with a pH of approximately 3.1. Two of the specimens from each mixture were removed from each immersion site at 2-month intervals, brushed, surface-dried and weighed. The remaining two specimens of each mixture at each immersion site were left in the acidic water for a period of 6 months and 8 months, respectively. At the end of each period, the remaining specimens were brushed, surface-dried, and weighed. Immersion testing was started on March 1, 1999 and was completed on October 22, 1999.

Of the seven different mixtures tested, the mixture most resembling the concrete mixture used in the Walker Mine seal was the one identified as SCRR-3. Based on the immersion test results and the relationship of volume-to-surface area and weight-loss-to-initial-weight of the cylindrical specimens, the rate of AMD attack was calculated as an average depth of surface loss. The results of this calculation for mixture SCRR-3 are summarized in Table 11 below. Material properties for this mixture and detailed test results are presented in Appendix G.1.

Table 11
AMD Immersion Test Results at Iron Mountain Mine

Immersion Testing, Months	~pH 2.4 Avg. Surface Loss, in.	~pH 3.1 Avg. Surface Loss, in.
Brushed Samples (2-month intervals):		
6	0.0471	0.0157
Unbrushed Samples:		
6	0.0417	0.0063
8	0.0498	0.0075

5.2.5 Discussion

It is apparent from the immersion testing results that unbrushed concrete, when exposed to AMD waters, forms a layer of reaction by-products that retards the progress of acid attack, likely by acting as a diffusion barrier. This condition was visually observed on the immersion test specimens. Since there is no abrasive action in the movement of AMD water through the Walker Mine seal, the unbrushed condition can be considered more representative of the rate of attack. Accordingly, the 8-month values for rate of attack for unbrushed specimens can be used. A conversion of this value to rate of attack per year is plotted on Figure 11. Using a semi-logarithmic scale and assuming a straight-line relationship, the curve can be extrapolated to pH values of 4.0 and 4.6 to provide a rough estimate of the annual rate of attack of concrete in the Walker Mine seal. The corresponding average annual rate of attack is 0.001 in/yr for a pH of 4.0 and 0.0002 in/yr for a pH of 4.6.

A comparison can be made between the IMM results, as presented herein, and the visual examination of the Walker Mine seal. From Figure 11 an average value of 0.001 in/yr, for a pH of 4.0, can be taken as the rate of attack of the seepage water on the downstream face of the seal. This equates to approximately 0.33 mm of surface softening in 13 years of service.

Visual observations during inspection of the seal in November of 2000 indicated that the maximum depth of surface softening was on the order of 1 to 2 mm. The difference in these values may be attributed to one or more of the following:

- Exposure of the Walker Mine concrete seal to AMD water at a very early age when attack is much more severe
- Gradual increase in pH values since installation of the seal, wherein attack may have been more severe in early years
- Seasonal variation of pH
- Slightly higher water-cementitious materials ratio of the Walker Mine concrete
- Inaccuracy in the visual estimate of surface softening during inspection
- Inaccuracy in the predicted rate of surface softening

As seen from the IMM studies, the pH of the AMD water has a significant effect on the rate of attack on concrete. The higher pH value of the Walker Mine AMD water suggests that the rate of attack on the concrete seal will be rather slow. This is confirmed by current performance as indicated by examination of the portion of the seal's downstream face that is in contact with AMD. At a design life of 100 years, the estimated maximum softening of concrete exposed to AMD waters is on the order of 0.1 inches based on IMM testing. If a value of 2 mm in 13 years is extrapolated, this softening could reach a depth of 15.4 mm, or slightly more than ½ inch in 100 years. If either of these conditions occurred along a continuous seepage path such as a crack along the rock-concrete interface, a marked increase in seepage would occur before the structural integrity of the seal were threatened. The occurrence of seepage and the seepage rate are thus key indicators of concrete condition and performance.

5.3 Potential Presence of Transverse Thermal Cracks

A simplified thermal study was made by Gary Mass, an independent concrete consultant under subcontract to GEI. The study was made using ACI 207.2R-95, Effect of Restraint, Volume Change, and Reinforcement on Cracking of Mass Concrete, to analytically investigate the potential for thermal transverse cracking in the concrete seal during curing. Parameters used in this study were taken from the final construction report prepared by SRK. Details of the methodology used are presented in Appendix G, including the calculations performed (Appendix G.2) and the above-mentioned ACI publication (Appendix G.3). The results of this study indicate a potential that transverse cracks may be present within the concrete. However, the existence of such cracks is not supported by the findings of the nondestructive testing and coring programs described earlier in this report. On the contrary, the test results indicate that the concrete is of good quality and without significant discontinuities. It is believed that, if they exist, these cracks are most likely well distributed and do not alter the structural integrity of the seal.

6. Conclusions

The conclusions are structured in accordance with the specific characteristics of the mine seal that were to be assessed during this study, as outlined in Section 2.1.

6.1 Location, Depth, and Extent of Major Cracks

Based on the findings from our visual observations, nondestructive tests and intrusive explorations, we conclude that, in general, the concrete forming the seal is sound, hard, and in good condition. No major cracks, large voids, or large zones of poorly consolidated concrete were identified. Three defects were detected by our explorations:

- Hole BH-6 encountered a 2-foot-long zone of soft, weak, concrete at the bottom of the seal. The concrete in this area lacked coarse aggregate, probably from segregation during placement. However, the cross-hole sonic logging profiles between this hole and adjacent holes did not reveal an area of weakness, indicating that the area of poor concrete intercepted by hole BH-6 is likely to be localized.

- The first phase of nondestructive (impulse-response) testing identified a zone of poor concrete consolidation immediately surrounding and below the location of the original concrete tremie pipe in the upper right quadrant of the seal. This zone, with approximate dimensions of 4 feet high by 2 feet wide was probably caused by "blind spots" developing during the concrete placement, i.e. poor consolidation under and around the elbow of the original concrete tremie pipe. The presence of this zone was not intercepted by core holes nor detected by subsequent nondestructive tests (cross-hole sonic logging), indicating that it is shallow in thickness.

- A possible crack or weakened plane was intercepted by hole BH-7 at a depth of 3 feet. As discussed in detail in Section 4.9.1, the evidence as to whether the separation was a mechanical break caused by coring or a pre-existing fracture or weakness, such as a cold joint formed during concrete placement, is inconclusive. In any event, water pressure testing of hole BH-7 yielded zero water loss through this zone. The cross-hole sonic logging profiles between this hole and adjacent holes did not indicate any defects, suggesting that this occurrence was either a mechanical break or a very localized feature.

6.2 Seepage Locations and Volumes

Water impounded behind the seal induces a hydraulic gradient through the seal and adjacent rock mass. As a result of this hydraulic gradient, water seeps through the concrete-rock interface and existing joints in the rock mass. The leakage primarily daylights to the downstream face of the seal along the interface between the concrete and the roof of the adit, and appears in the form of drips running down the face of the seal. Most of the leakage daylights at the left-center portion of the crown and along both side walls. We did not see any evidence of perceptible seepage flow occurring through the concrete itself. Likewise, we did not see any perceptible evidence (in the form of bubbling or eddies in the mud) of seepage occurring along the rock-concrete contact at the floor of the seal. However, the mud and water impeded a good view of the contact and small amounts of seepage would escape detection.

The leakage observed just downstream of the seal is considered low and within the range of what can be expected for a well-constructed tunnel plug. The seepage rate was estimated during our site visit on November 1, 2000 to be approximately 0.15 gpm for a hydraulic head behind the seal of 140 feet. The measured seepage rate is consistent with an average hydraulic conductivity of 10^{-5} cm/sec or less for the rock penetrated by the adit. We would expect the seepage rate to be directly proportional to the hydraulic head.

More important than absolute flow amounts is the trend of flows versus time. A review of data and qualitative observations suggests that, so far, the hydraulic conductivity of the seal/rock system has not been increasing over time and may have decreased somewhat. However, the flow paths have not completely sealed themselves over the 14 years since the seal was constructed.

6.3 Physical Condition of Concrete on the Submerged Side of the Seal

Based on surface examination of the submerged portion of the downstream face of the seal, it can be concluded that the concrete surface exposed to the acid mine water has been affected to some extent and has resulted in a softening of the surface paste and mortar. At present, the depth of this softening is only approximately 2 mm at the locations tested. Furthermore, softening has not resulted in loss of the surface paste by erosion. Concrete deeper than this thin surficial zone of softening is sound and hard, and is indicative of the concrete mass within the seal as generally confirmed by the exploratory core holes. The concrete that is not exposed to direct contact with the acid water can be expected to be sound and hard.

We have no reason to believe that the physical condition of the concrete on the upstream face of the seal differs significantly from that observed on the submerged portion of the downstream face. Since the pH of the seepage water ponded against the downstream face of the seal is somewhat lower than that observed in the AMD upstream of the seal, acid attack may be slightly more aggressive on the downstream face of the seal. The results of nondestructive testing indicate that the thickness of sound concrete in the seal is approximately equal to the 15-foot design thickness of the seal, suggesting that the thickness of soft, deteriorated concrete is small. A review of a study of acid attack on concrete performed at another Northern California mine confirms that the observed rate of attack, i.e., 2 mm in 13 years, is reasonable for acidic water with a pH of about 4.

6.4 Condition (Corrosion) of the Two Pipe-and-Valve Assemblies

Although covered with iron oxide deposits, the exterior of the stainless steel pipe and valve components is not corroding. The interior of the stainless steel components that could be observed was also in good condition. Ultrasonic thickness testing of the pressurized piping components confirmed wall thicknesses approximately equal to the design thicknesses.

The exterior of the carbon steel components exhibited a variable amount of corrosion. As revealed by the pipe failure during the site visit of May 31, 2001, described in Section 4, the carbon steel 3/4-inch pipe segment in the pressure transmitter line was highly corroded and required emergency replacement. While corrosion of the remaining carbon steel components (assorted bolts, tie-rods and nonpressurized 3/4-inch pipe) is not a cause for immediate concern at this time, attention will need to be given to these components over the long term, particularly for corrosion occurring at the contacts between dissimilar metals (carbon steel and stainless steel).

All valves in the drain pipe system appeared to be in good condition. All valves were exercised and were found to be functional. The actuators of the 4-inch DeZurik control valves were not frozen and operated smoothly. The valves gave tight shutoff upon closing. The actuator enclosures are painted cast iron. The enclosures and handwheel stems are encrusted with metal oxide deposits and show external corrosion due to constant exposure to dripping acidic water. Unlike most valves, the handwheel is rotated clockwise to open the valve.

6.5 Condition of the Support Rock at the Seal Area

The rock around the seal consists of granitic rock and is hard, generally moderately fractured, fresh, and with generally clean and tight joints. No evidence of acid attack of the rock was observed. Water pressure tests of the rock and rock-concrete interface indicated a hydraulic conductivity in the range of 10^{-5} cm/sec, which is consistent with the hydrogeologic model of the rock mass developed for design of the seal.

6.6 Condition of the Rock-Concrete Interface

While four holes penetrated the rock-concrete interface, only one sample of concrete adhered to rock was obtained. Elsewhere, breaks occurred between the concrete and the rock. This is not surprising given that the angle of the holes to the adit wall was generally only a few degrees; the small angle and the difference in hardness between the concrete and rock may have caused the drill bit to deviate from its alignment, possibly vibrating and causing mechanical breakage of the cores.

It is also possible that the concrete and the rock are not well bonded throughout. The video from the day of the seal construction appears to show portions of the adit wall that were not clean enough to result in a good bond between concrete and rock. It is not known whether these areas were cleaned after the video was taken. However, the results of both nondestructive testing programs do indicate that the concrete is in intimate contact with the rock. In the first phase of testing, consistently high values of shear wave velocity along the interface were calculated (see Table 1 of Appendix B.1), indicating a good contact between the concrete and the rock. In the second phase of testing, all cross-hole sonic logging profiles showed continuous material between test holes, with no breaks in arrival signals at any point. If poor concrete consolidation or open fractures were to exist at the rock-concrete-interface, a break in the signal arrival time would be present in the test traces. This was not seen in any of the profiles.

In our opinion, complete adhesion of the concrete to the rock is not required for stability of the seal. Shear along the rock-concrete contact is not considered to be physically possible given the large asperities in the rock surface. Shear failure along the rock-concrete interface would require dilation of the interface to allow the concrete to slip along the large rock asperities. A large dilation cannot physically occur in the adit because the rock-concrete interface of the seal is confined on all sides.

No evidence of acid attack was observed in the rock-concrete interface where it was traversed by the core holes. However, a small amount of seepage is occurring along the concrete/rock contact at the roof and sidewalls of the adit. This suggests that there are water-bearing, hairline openings between the rock and the concrete along the top and sides of the seal. The concrete along these hairline openings where seepage is occurring has probably been affected in a similar manner as that observed on the face, i.e., by softening of the surface paste. Over a long period of time, it is possible that the softened paste may gradually erode away, making the interface opening between the rock and the concrete gradually larger and allowing more water to flow through the interface, resulting in gradually increasing seepage flows. At this time, no increase in seepage rate has been observed that would indicate that the softening of the paste and mortar has been detrimental or has affected the integrity of the seal. Concrete sometimes has a tendency to plug itself through the leaching and redeposition of calcium carbonate. Monitoring of long-term trends in the seepage rate through the seal would provide advance warning for gradual deterioration of concrete along seepage paths.

Even if seepage were to gradually increase in the future, the increase in flows would not be a threat to the structural integrity of the seal until the water began to flow with significant velocity through the opening, causing mechanical damage to the rock or concrete at the interface. If this were to occur, it would take a significant loss of material to jeopardize the integrity of the seal, since the seal is wedged-in between the irregular surfaces of the adit's rock walls. Monitoring of seepage rates should provide ample advance warning and would allow the RWQCB to undertake high-pressure grouting of the seepage areas to rehabilitate the seal.

6.7 Condition of the Seal with Regard to Its Ability to Withstand the Design Hydraulic Head

In our opinion, the condition of the 15-foot-long concrete seal and surrounding rock mass is adequate to resist the design water pressure head of 570 feet based on our interpretation of the original seal design, seal construction records, field and laboratory data collected for this study, and our estimates of the material properties of the seal and surrounding rock.

6.8 Maximum (Optimum) Head on Seal to Minimize Seal Degradation

We believe that the rate of degradation of the concrete in the seal is rather slow (less than 1 mm per year). Within the normal operating range of hydraulic heads experienced by the seal to date, we do not believe that there is a threshold value of hydraulic head that will minimize acid attack. Obviously, lower hydraulic heads generate lower gradients, less seepage, and less mechanical stress; thus, lower heads are desirable over higher heads all other things being equal. However, as indicated above, it is our opinion that the seal is currently safe to operate up to the full design head of 570 feet.

7. Recommendations

(1) Seepage Monitoring

We believe that monitoring of seepage locations and seepage flow rate (versus hydraulic head) is very important for long-term performance monitoring of the seal. Monitoring of long-term seepage rates, adjusted for hydraulic head, should provide advance warning of gradual deterioration of the concrete along seepage paths, and should allow the RWQCB to undertake high-pressure grouting of the seepage areas to rehabilitate the seal if it becomes necessary. In addition to seepage flow measurements, the seepage locations at the seal face should be monitored and recorded yearly, and apparent changes should be evaluated.

It is recommended that a convenient means of measuring seepage rates be installed to facilitate this monitoring task. A feasible way of doing this would be to construct a concrete weir wall just downstream of the seal, as shown conceptually on Figure 12. The weir wall would be located to avoid interference with the existing valves, including access to the actuator handwheels, tie bolts, and port at the bottom of the shutoff valves. The top of the wall would be just below the bottom of the drain pipes and valves. The flow metering element could potentially be a perforated stainless steel plate bolted to and sealed against a rectangular opening in the concrete wall. The perforations in the steel plate would have to be arranged to facilitate the accurate measurement of flows. It appears that 1/8-inch-diameter perforations arranged at two-inch vertical spacing would accomplish this objective. The lowest perforation would have to be just above the static water level in the adit downstream of the weir wall (the "static water level" shown on Figure 12). The contact between weir wall and foundation would have to be very well sealed to avoid seepage losses.

An approximate hydraulic head versus flow rate curve is shown on Figure 12. However, since this is not a standard weir configuration, it is recommended that this arrangement be calibrated under controlled conditions. The curve should then be revised to agree with the calibration test results.

The flow would be measured by measuring the water level within the pool upstream of the weir wall and converting to flow rate by the use of the calibrated hydraulic head versus flow rate curve. The water level measurement could be done with a high-sensitivity pressure transducer. A Druck PTX 520 with a range of 0 to 5 psi should be sufficient, since the accuracy of the instrument is 0.15% of full scale, or about 0.2 inches. The pressure transducer would be connected to a data logger powered by a battery pack which could be located near the entrance to the mine, in an arrangement similar to the existing system. The existing data logger only has one channel, so either the existing data logger would need to be replaced with a multi-channel unit, or a separate unit installed. Seepage flow data could then be downloaded and analyzed twice per year by the RWQCB during their semiannual visits to the mine portal. Confirmatory manual readings of flow rate could be made during the annual visits to the seal by measuring the time that the weir flow takes to fill a container of known volume.

The orifices in the weir plate should be periodically cleaned to keep them open and free-flowing. It is quite possible that the orifices will tend to plug or to develop a build-up of iron deposits or slime. Such build-up would change the head-versus-flow relationship and at worst it may make the flow measurements using the perforated plate impractical. If this occurs, two fallback options are available:

- Use the weir structure to obtain a manual flow measurement during each annual visit. This option would provide one reading per year.

- Measure the seepage flow by installing a pumping system in the pool behind the weir structure. For this option, the perforations in the steel plate would need to be sealed (or the plate replaced with a solid plate). A small diaphragm metering pump, powered by a direct current motor and a battery pack, would be installed to pump the water from behind the weir. The pump would be turned on and off by the transducer measuring the water level behind the weir, or by a level switch. The pump would discharge to the downstream side of the weir. The data logger would record pump run time, which is directly related to flow for a positive displacement pump; thus, a flowmeter would not be needed. This option could provide daily to weekly flow readings.

(2) Replacement of Carbon Steel Components

When there is an opportunity, all remaining carbon steel components should be replaced with stainless steel components, since carbon steel components will continue to corrode, particularly at the contacts with stainless steel pieces. The known remaining carbon steel components are addressed below. All existing nuts and bolts should be reexamined carefully, both visually and with a magnet, to locate any additional carbon steel components.

- Tie bolt at the 10 o'clock position on each 4-inch shutoff valve (looking downstream): These carbon steel tie bolts should be replaced with stainless steel tie bolts before corrosion reduces the strength of the bolts.

- Threaded tie rod wall anchors and FxF threaded turnbuckle-type connectors that connect to the tie bolts at the 5 and 7 o'clock positions on each 4-inch shutoff valve (see Figure 9): The sections of carbon steel tie rod that pass through the flanges of the valves should be replaced with stainless steel tie bolts, while at the same time retaining the embedded tie rods. This can be accomplished by cutting each tie rod at a location upstream of the valve and flanges. The downstream tie rod section should then be replaced using stainless steel tie bolts that run through both flanges and are restrained by stainless steel nuts, and that extend upstream to connect to the cut end of the embedded tie rod with the turnbuckle. The modified configuration is shown on Section A-A of Figure 12.

- One 3/4-inch pipe downstream of the blind flange for both the east and west drain pipes: These are not as critical because under normal conditions they are not pressurized.

- Carbon steel padlocks: These were used to prevent tampering with the 4-inch backup valves downstream of the shutoff valves. These padlocks should be replaced with ¼-inch-diameter, long hasp, corrosion resistant padlocks.

Proper precautions should be taken when replacing carbon steel fasteners under liquid pressure. Only one tie rod should be cut or unfastened at any time, and temporary restraining across the flanges on each side of the shutoff valve may have to be provided depending on the fluid pressure at the time the repair is made.

(3) Installation of Pipe Supports

Pipe supports for the 4-inch drain lines were contemplated in the original design but were not constructed. While pipe supports are not strictly required because of the short cantilevered length of the pipes, it would be advisable to install them to reduce the possibility of inadvertent overstressing of the pipes by human activity. Temporary pipe supports were installed during the emergency pipe repair described in Section 4. However, the temporary supports could not be fixed to the uneven rock floor of the adit and, consequently, they can come off easily. Installation of permanent pipe supports is advisable. The existing temporary supports could be used for the permanent installation, since they are corrosion resistant, adjustable, and removable. However, for a permanent installation, each support should have a concrete footing. Ideally, the permanent pipe supports could be constructed at the same time that the concrete weir wall is constructed. Figure 12 shows one idea that uses the existing pipe support retained at the bottom with a pin made from a 1-inch-diameter stainless steel rod embedded into the concrete footing of the weir wall. Other configurations are possible.

(4) Regular Operation of Valves

It is recommended that the RWQCB operate all the valves in the system on a regular maintenance schedule to ensure that they continue to operate smoothly and do not become frozen. To test the operation of the 4-inch control valves, we suggest the following sequence:

- Open the downstream 3/4-inch valve; document the flow conditions.

- Open the backup 4-inch valve; document the flow conditions.

- Clear debris from upstream of the 4-inch control valve seat by opening this valve for a few seconds. To open this valve the handwheel actuator needs to be rotated clockwise. Fluid release will be limited by the 3/4-inch valve. However, make sure that no person is standing in front of the flow path during this procedure. Close the control valve. Check that it is shut tightly.

- Close the downstream backup valve. Leave the 3/4-inch valve open.
- Open fully and then close the 4-inch control valve several times. Check that the actuator operates smoothly, without hesitation.
- With the control valve closed, open fully and then close the 4-inch backup valve several times. Verify that the closed control valve is shut tightly.
- Close all valves to end the test, and reinstall safety padlocks.

We recommend that this procedure be performed at least once each year.

(5) Protection and Monitoring of Valve Actuators

It is recommended that the RWQCB monitor the condition of the manual rotary actuators mounted on the 4-inch shutoff valves. The existing actuators are fabricated with painted cast iron housings and, although they are still in good condition, they are more susceptible to corrosion than the stainless steel drain piping and valves. Consequently, the existing actuators should be protected from the constant dripping of acidic mine seepage and monitored for excessive corrosion.

It is suggested that the actuators be cleaned of deposits using a non-abrasive method that will not damage the paint, such as a plastic scrub brush or pad. The actuators should then be dried and covered with a plastic tent. The "tent" can simply be a heavy-duty PVC garbage bag, although the plastic needs to be restrained using elastic cord or waterproof tape to prevent puddling water from tearing or dragging the plastic material off and exposing the actuator. An off-the-shelf item, such as a plastic garbage can, could be modified to make a serviceable tent, so long as it was waterproof, acid resistant, and light weight.

The rotary-type actuators may be difficult to replace for several reasons: 1) the shutoff valves are control valves and rotate in the opposite direction of most valves; 2) each shutoff valve has a special tapered spline at the top of the valve stem rather than the typical 2-inch nut; and 3) the shutoff valves were fabricated before the bolt spacing and dimensions of actuator mounting plates were standardized to use ISO bolting. Eventually, if the actuators need to be serviced, they could be unbolted from the valves, removed from the mine, and refurbished by a valve specialist or replaced with new actuators. More detailed considerations on the servicing of the actuators are included in Appendix C.2.

(6) Subsequent 10-Year Inspections

We understand that the June 1997 Operations and Maintenance Procedures for the Walker Mine requires the RWQCB to perform integrity testing of the mine seal every 10 years. We recommend that the next 10-year testing and evaluation program include at least the following:

- Visual inspection of the condition of the seal.

- Measurement of seepage flow from the seal and review of seepage rates, locations, and historical data to determine seepage trends.

- Evaluation of the rock quality next to the seal, including Rock Mass Rating (RMR) and in-situ index strength tests (using the Schmidt Hammer). Estimation of the RMR requires the estimation of the Rock Quality Designation (RQD) among other factors. To estimate the RQD for an excavated rock face without coring, a line is drawn along the rock face. Those distances between fractures in the rock along the line that are greater than four inches, are summed and divided by the total length of the line to obtain RQD values.

- Evaluation of the concrete quality of the seal, including in-situ index strength (using the Schmidt Hammer) and nondestructive testing (using the Impulse Response test method).

- Visual inspection of the condition of valves, actuators and drain pipes and operation of all valves. Evaluate the need for servicing the actuators.

- Ultrasonic thickness testing of pipes and valves.

- Chemical testing of the water.

- Recommendations for the subsequent 10-year inspection.

The data presented in this Seal Testing and Evaluation Report can be used to document the baseline conditions (as of year 2001). Future data can be compared to the baseline conditions to detect and evaluate trends.

(7) Other

If the RWQCB decides to construct a seal in the Piute adit, we recommend that, at that time, a detailed inspection of the seal in the main adit be performed prior to construction of the Piute seal to confirm that the existing seal and drain piping remain safe to withstand the design hydraulic head of 570 feet.

8. Limitations

This report presents observations made and conclusions drawn from (1) visual inspections of the Walker Mine Seal; (2) a review of documents made available by the RWQCB relating to the design and construction of the seal; and (3) the results of limited nondestructive as well as intrusive testing of the seal components. The purpose of the inspection, testing and review has been to assess the integrity and safety of the seal for continuing operation in the interest of environmental protection and public safety.

In the context intended above, the term "safety" is interpreted to be restricted specifically to major structural and control features of the seal in regard to its adequacy against possible catastrophic failure. No consideration is given or intended to the safety or integrity of the adit downstream of the seal. Likewise, no consideration is given or intended to the safety of individuals who could be exposed to personal mishaps by entering the mine or utilizing the seal's features.

GEI Consultants, Inc., and its employees and agents who performed the inspections, tests, and investigations, reviewed available information, and prepared this report, desire that it be clearly understood that the conclusions regarding the condition and safety of the seal and related facilities are not guaranteed, but do represent our best judgment. Inevitably, such judgment must be recognized to be affected to an uncertain degree by the practical limitations which affect all evaluations of constructed water-retention structures, relative principally to approximate knowledge of the internal condition and properties of the existing structures and their foundations, and the uncertainties that are known to exist in estimating factors of safety. We endeavored to perform our professional services for this project in accordance with generally accepted engineering practices; no other warranty, expressed or implied, is made.

The conclusions and recommendations in this report are based in part upon the data obtained from limited subsurface testing and surface observations. Actual subsurface conditions may escape detection by the testing techniques used and may therefore be different from those described. The nature and extent of such variations may become evident when further tests and evaluations are performed. If variations from the anticipated conditions are encountered, it may be necessary to revise the recommendations in this report.

9. Disclosure Statement

This report was prepared through Agreement 0-058-150-0 between the State Water Resources Control Board and GEI Consultants, Inc. This disclosure statement is provided pursuant to Exhibit D, Item 6 of said agreement.

This report is the last of a series of written reports prepared by GEI Consultants under the agreement. Additional reports prepared by GEI include a Site Visit Report and Sampling Analysis Work Plan, a Health and Safety Plan Addendum, Monthly Status Reports, and Subcontract Documents for various activities.

The maximum amount of this agreement is $350,000. The main subcontracts issued by GEI relating to the preparation of documents and written reports for this project are listed below:

Subcontractor	Subcontract Amount
Washington Group International, Inc.	$ 57,100
Construction Technology Laboratories, Inc.	46,000
Walker and Associates, Inc.	15,000
Gary Mass	10,000
SDV-ACCI	10,500
Jensen Drilling Company, Inc.	88,200
Pacific Mechanical Corporation, Inc.	11,500

10. References

American Concrete Institute, "Nondestructive Test Methods for Evaluation of Concrete in Structures," ACI 228.2R-98.

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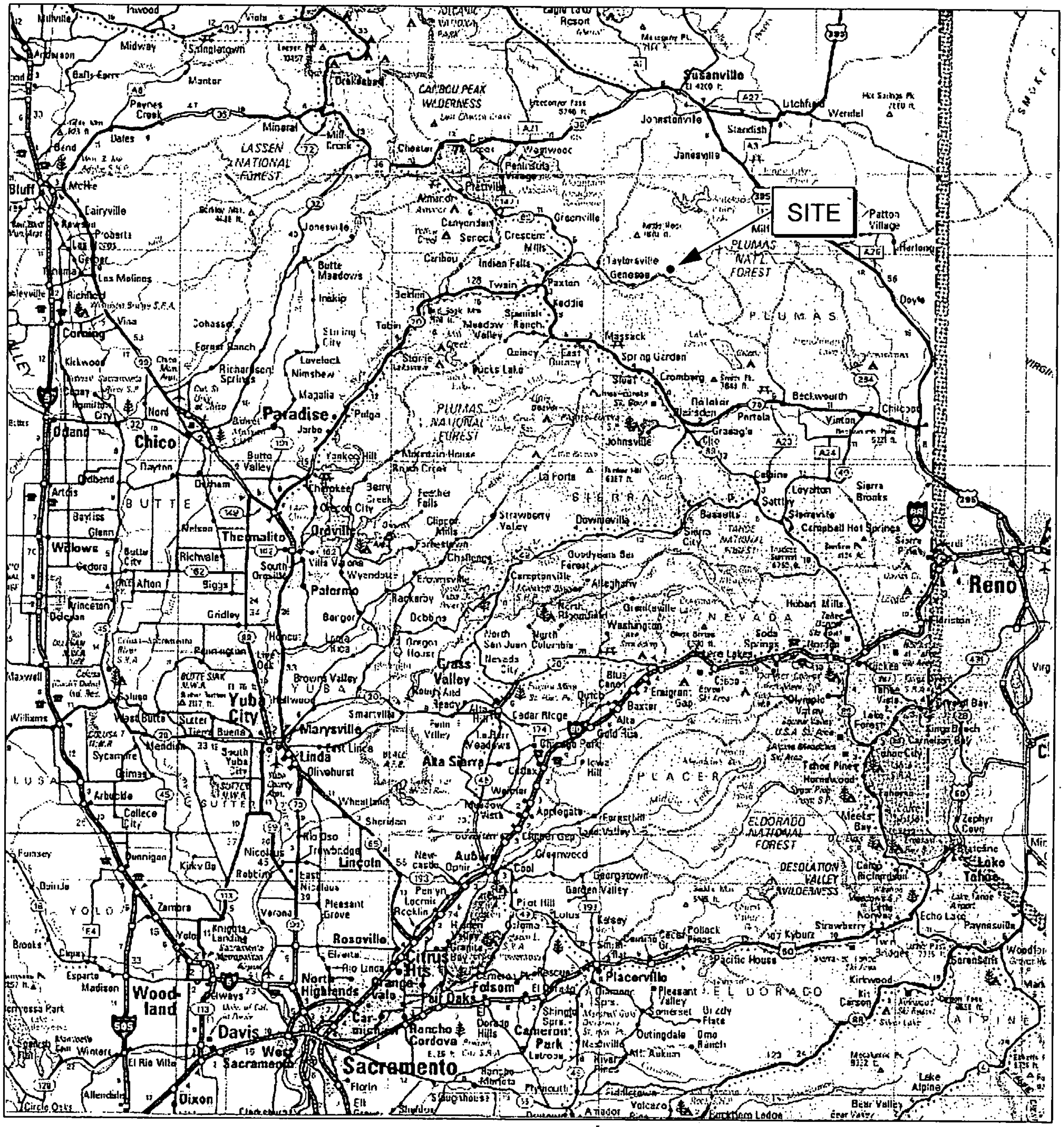
Westec, Drawing No. 1, Rev. 1, "Walker Mine Tunnel Rehabilitation, 700 Level Adit Mapping Data," March, 1995.

FIGURES



0 10 20 40 60

SCALE, MILES

LOCATION OF SITE ABOVE

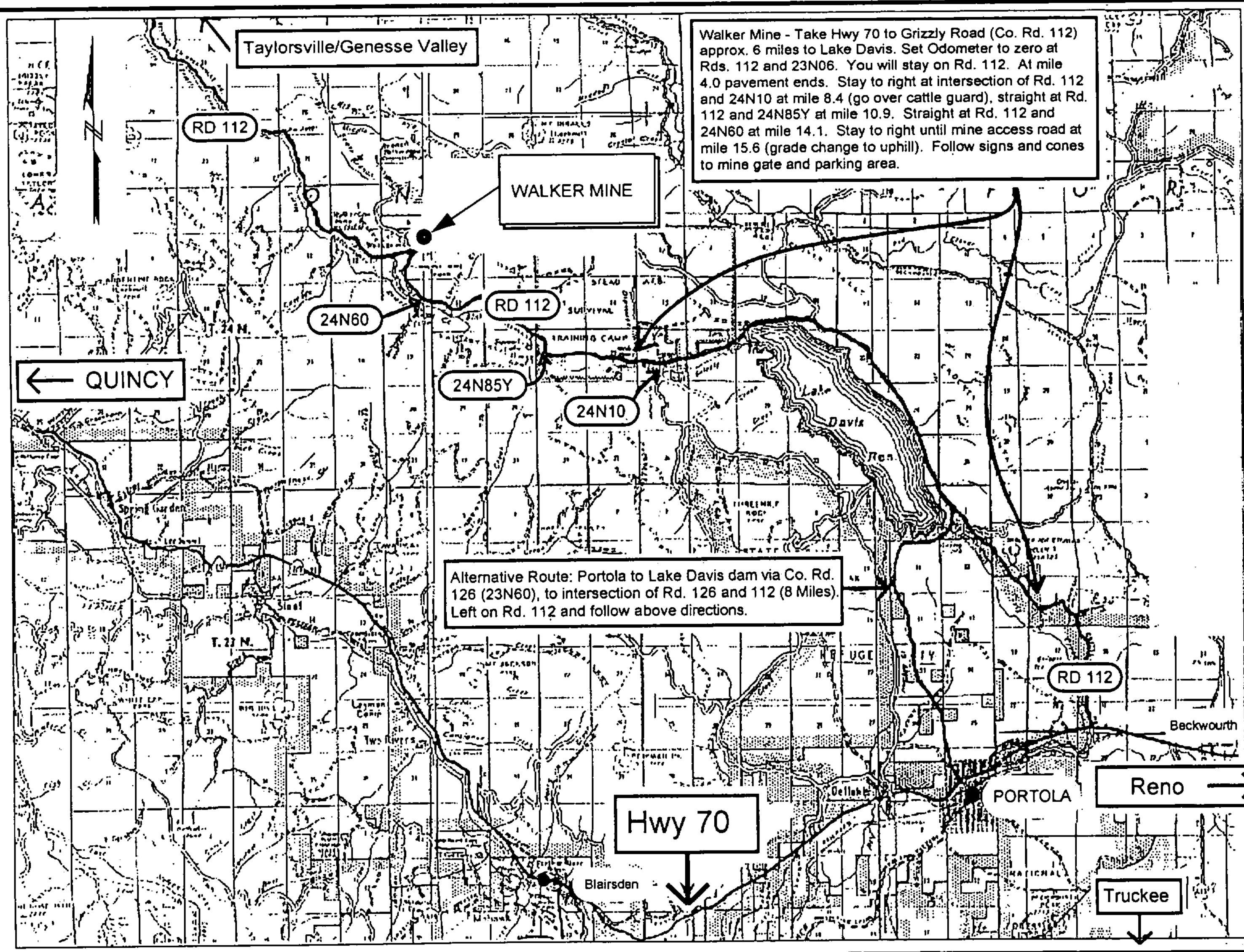
NOTE

Map is taken from California road map dated 1998.

Regional Water Quality Control Board Central Valley Region	Walker Mine Seal Testing & Evaluation Portola, CA	PROJECT LOCATION MAP
GEI Consultants, Inc.	Project 00387	Dec. 2001 / Figure 1

Fig-1.dwg 12-08-00 PYM

Fig-2.dwg 12-08-01 PYM



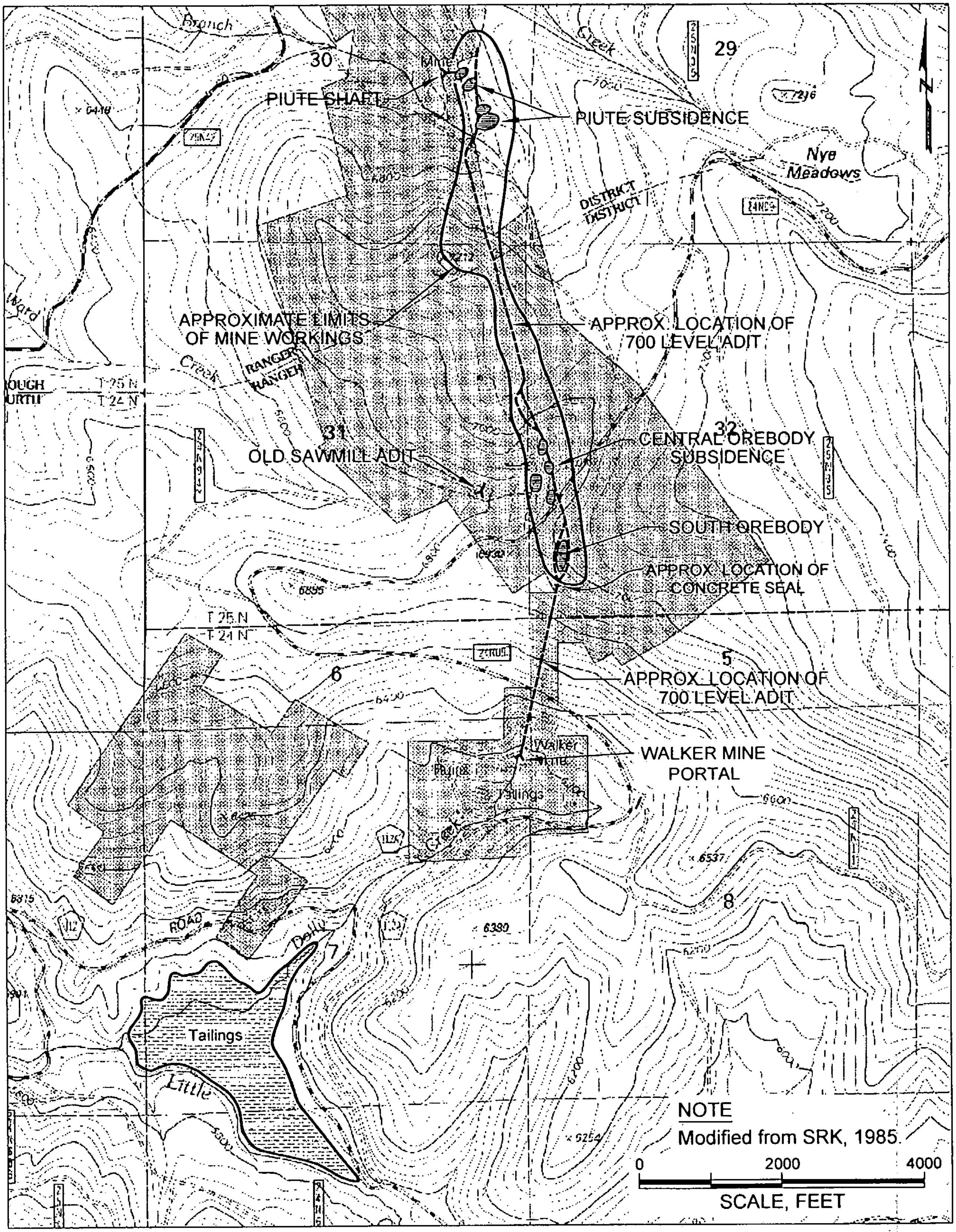
0 2.5 5 10

(Approx.) SCALE, MILES

NOTE

Map is provided by RWQCB.

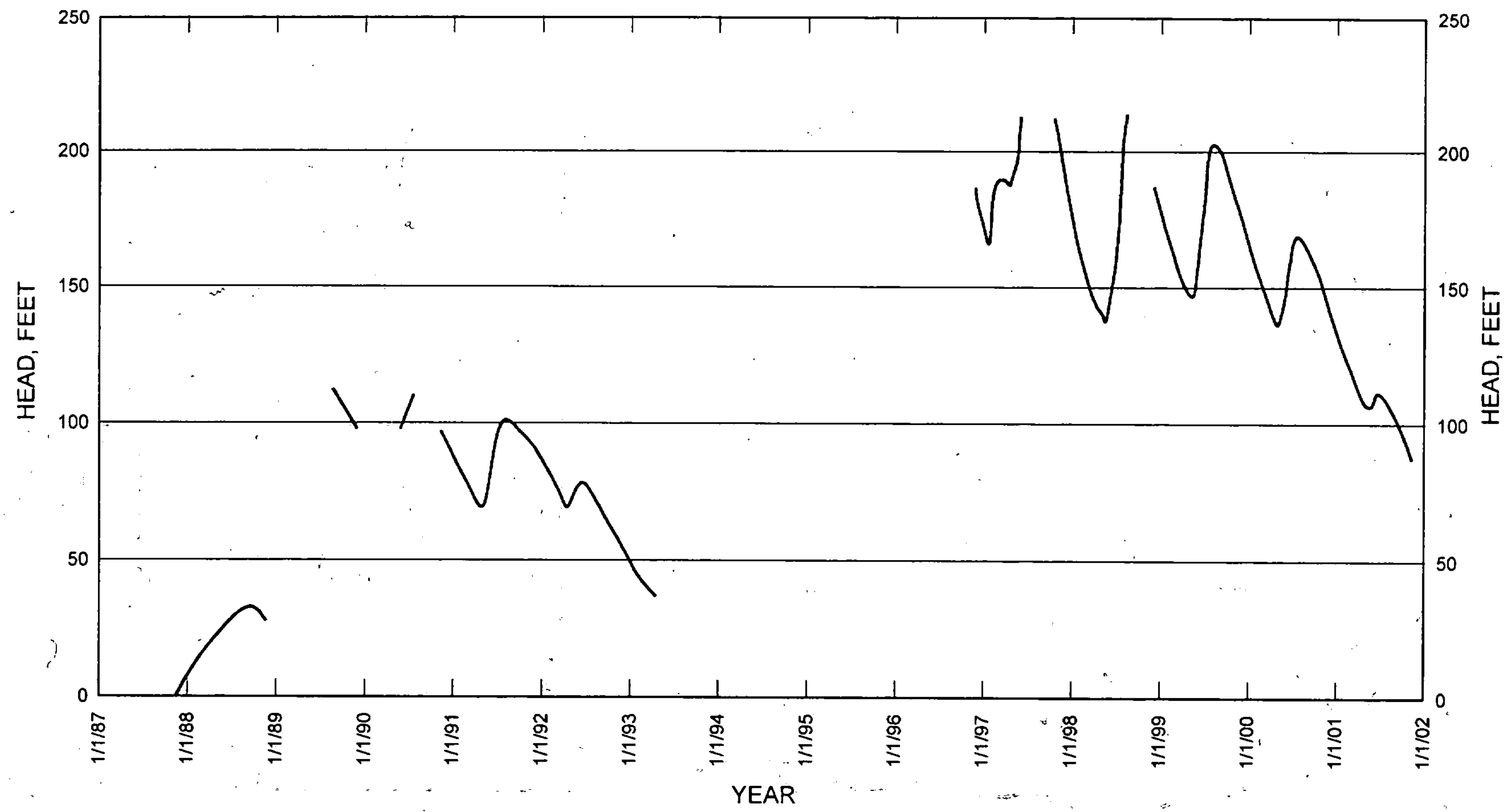
Regional Water Quality Control Board Central Valley Region	Walker Mine Seal Testing & Evaluation Portola, CA	DIRECTIONS TO SITE
GEI Consultants, Inc.	Project 00387	Dec. 2001 — Figure 2



Regional Water Quality Control Board Central Valley Region	Walker Mine Seal Testing & Evaluation Portola, CA	SITE PLAN AND TOPOGRAPHY	
GEI Consultants, Inc.	Project 00387	Dec. 2001	Figure 3

Fig-3.dwg 12-07-01 PYM

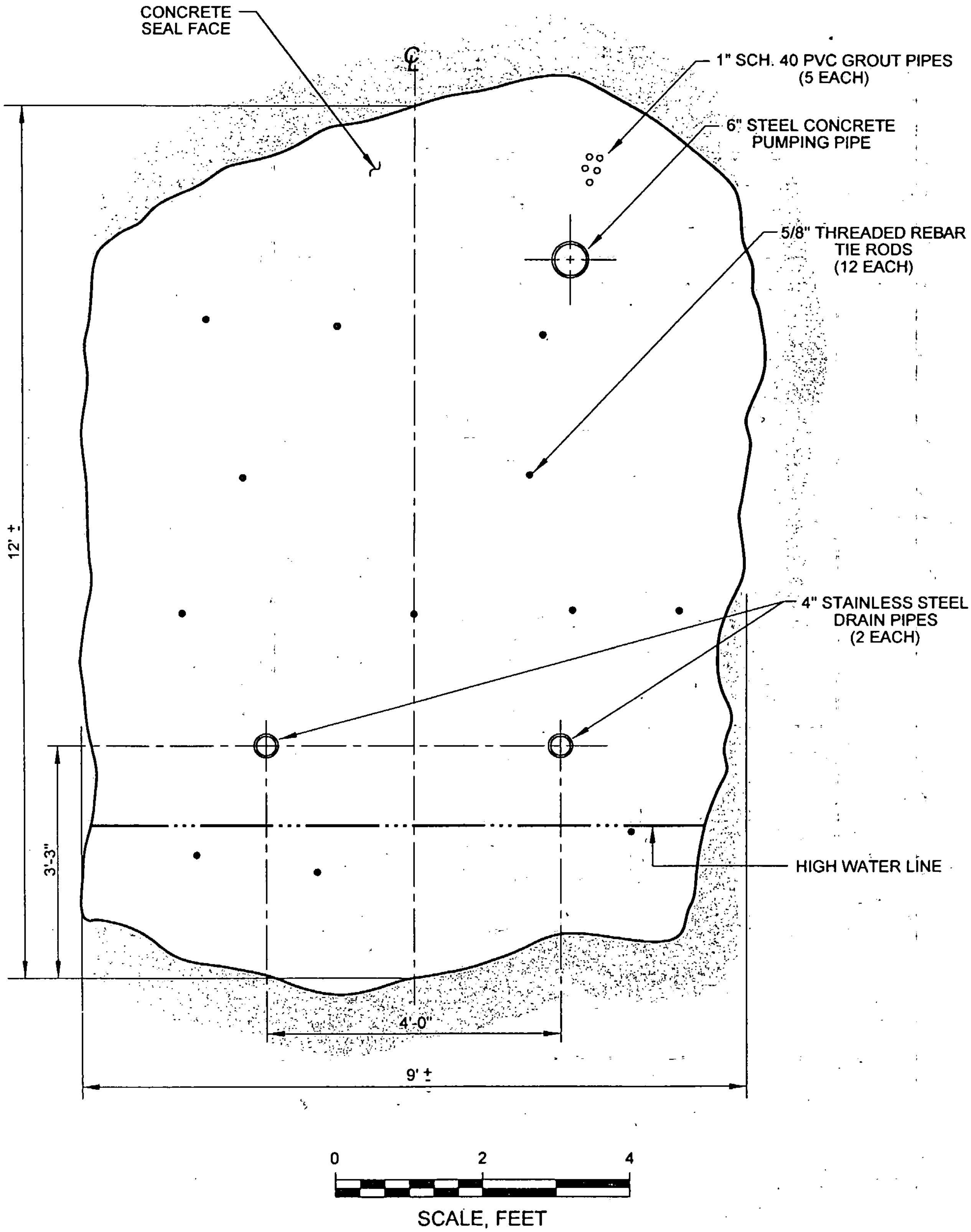
GRAPH 12-08-01 PYM



NOTE

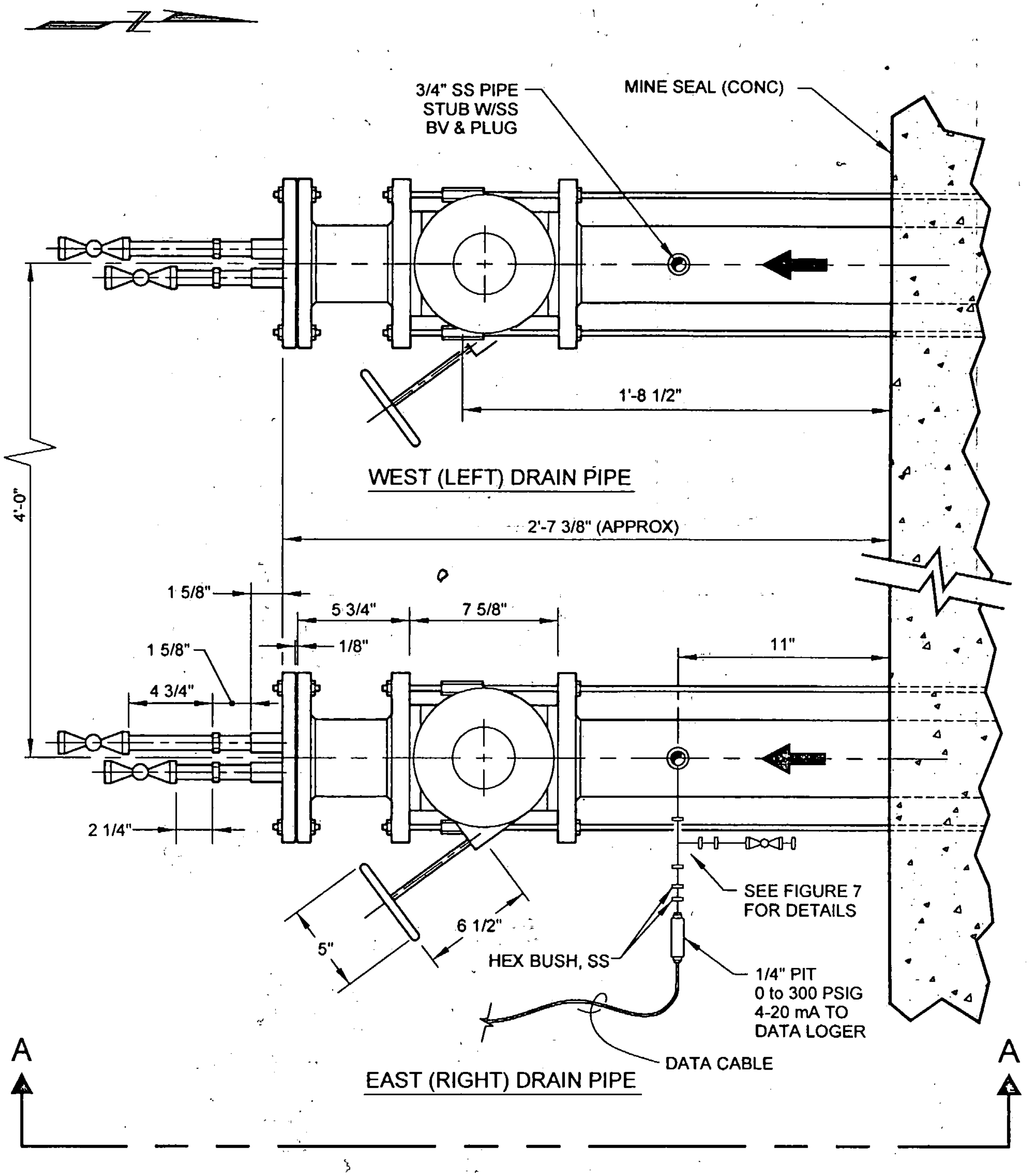
DATA SOURCE: RWQCB, 2001.

Regional Water Quality Control Board Central Valley Region	Walker Mine Seal Testing & Evaluation Portola, California	WATER PRESSURE HEAD BEHIND CONCRETE SEAL
GEI Consultants, Inc.	Project 00387	Dec. 2001 / Figure 4



Regional Water Quality Control Board Central Valley Region	Walker Mine Seal Testing & Evaluation Portola, CA	SKETCH OF SEAL FACE AS OBSERVED ON 11/1/00	
GEI Consultants, Inc.	Project 00387	Dec. 2001	Figure 5

Fig-5.dwg 12-08-01 PYM

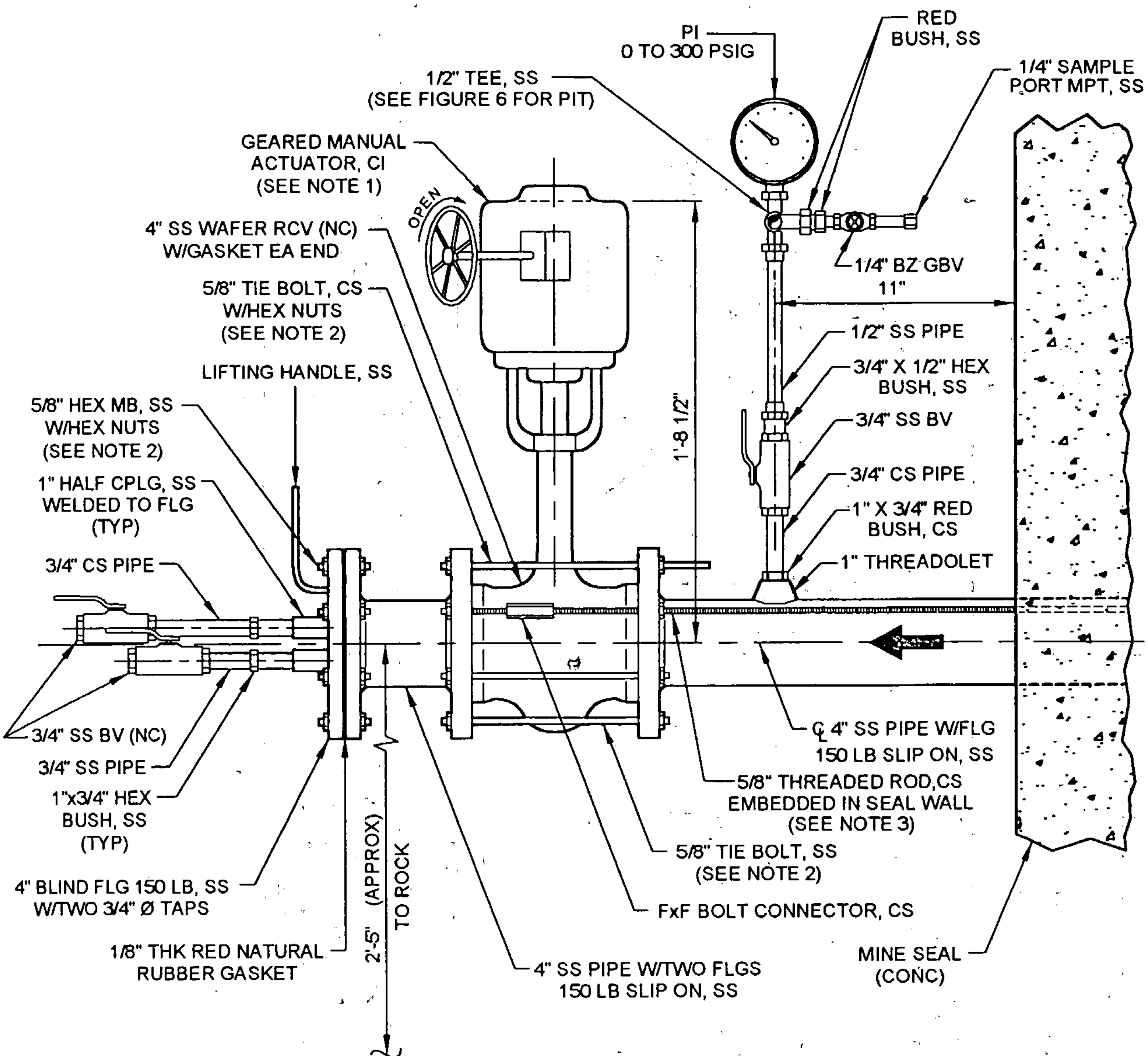


NOTES

1. SEE FIGURE 7 FOR LEGEND OF ABBREVIATIONS.

Regional Water Quality Control Board, Central Valley Region Washington Group International, Inc. GEI Consultants, Inc.	Walker Mine Seal Testing & Evaluation Portola, CA	4-INCH DRAIN PIPES PLAN (BEFORE MODIFICATIONS)
	Project 00387	Dec. 2001 — Figure 6

Fig-6.dwg 12-08-01 PYM



LEGEND

BZ	BRONZE	BV	BALL VALVE
CI	CAST IRON	GBV	GLOBE VALVE
CS	CARBON STEEL	RCV	ROTARY CONTROL VALVE
SS	STAINLESS STEEL	PI	PRESSURE INDICATOR
BUSH	BUSHING	NC	NORMALLY CLOSED
FLG	FLANGE	NO	NORMALLY OPEN
CONC	CONCRETE	PIT	PRESSURE INDICATOR TRANSMITTER
F	FEMALE	MB	MACHINE BOLT
M	MALE	EA	EACH
THK	THICK	MPT	MALE PIPE THREADS
		RED	REDUCING

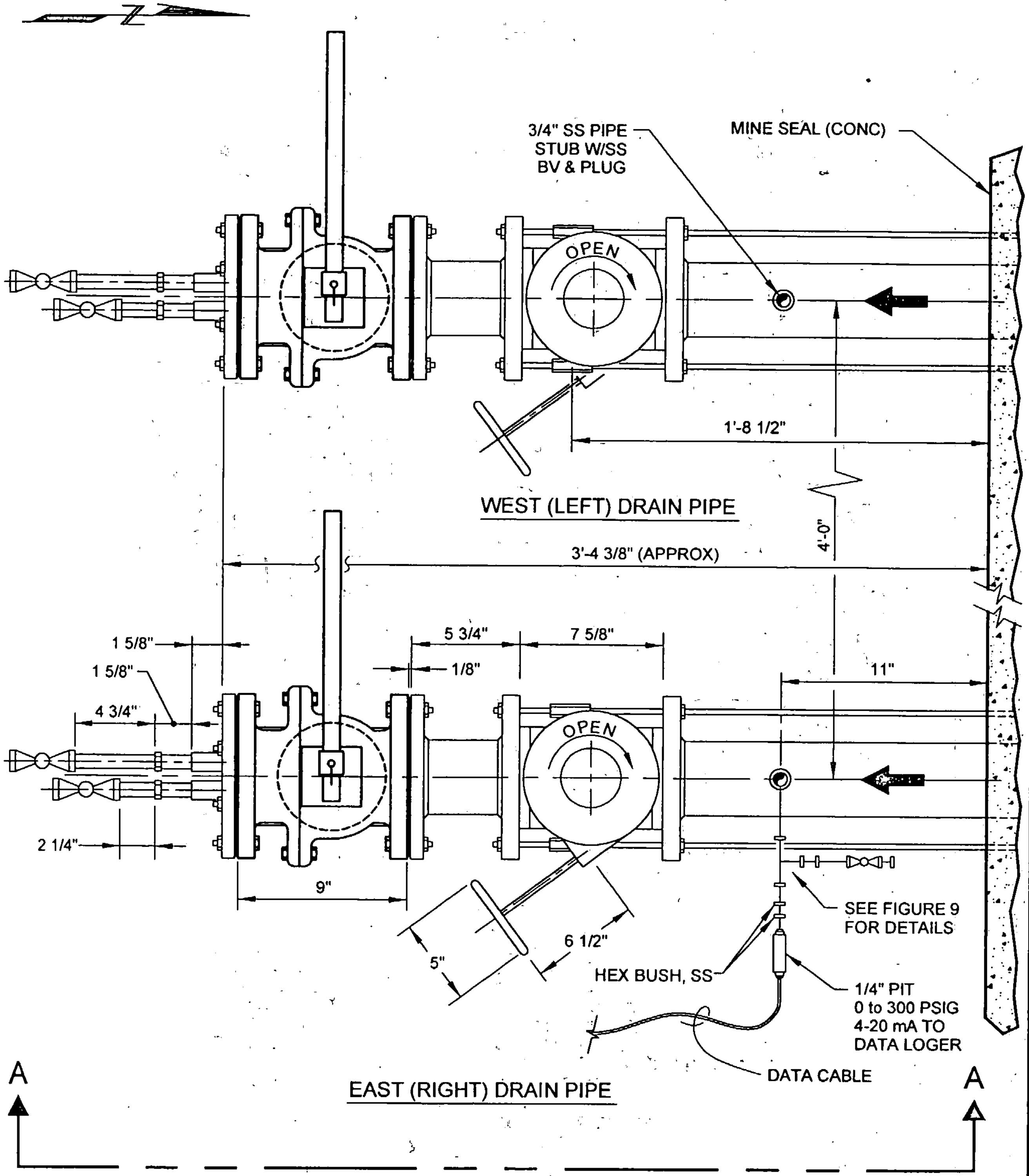
NOTES

1. ACTUATOR HANDWHEEL LOCKED IN CLOSED POSITION WITH CHAIN AND PADLOCK.
2. SOME FLANGE FASTENERS ARE CARBON STEEL.
3. EMBEDDED ROD POSITION VARIES. RIGHT SIDE ROD ON EAST PIPE IS ABOVE 4-INCH PIPE CENTERLINE. ALL OTHERS ARE BELOW PIPE CENTERLINE.

0 6 12
1 1/2" = 12"

Regional Water Quality Control Board, Central Valley Region Washington Group International, Inc.	Walker Mine Seal Testing & Evaluation Portola, CA	EAST (RIGHT) DRAIN PIPE ELEVATION A-A (BEFORE MODIFICATIONS)	
GEI Consultants, Inc.	Project 00387	Dec. 2001	Figure 7

Fig-7.dwg 12-08-01 PYM



NOTES

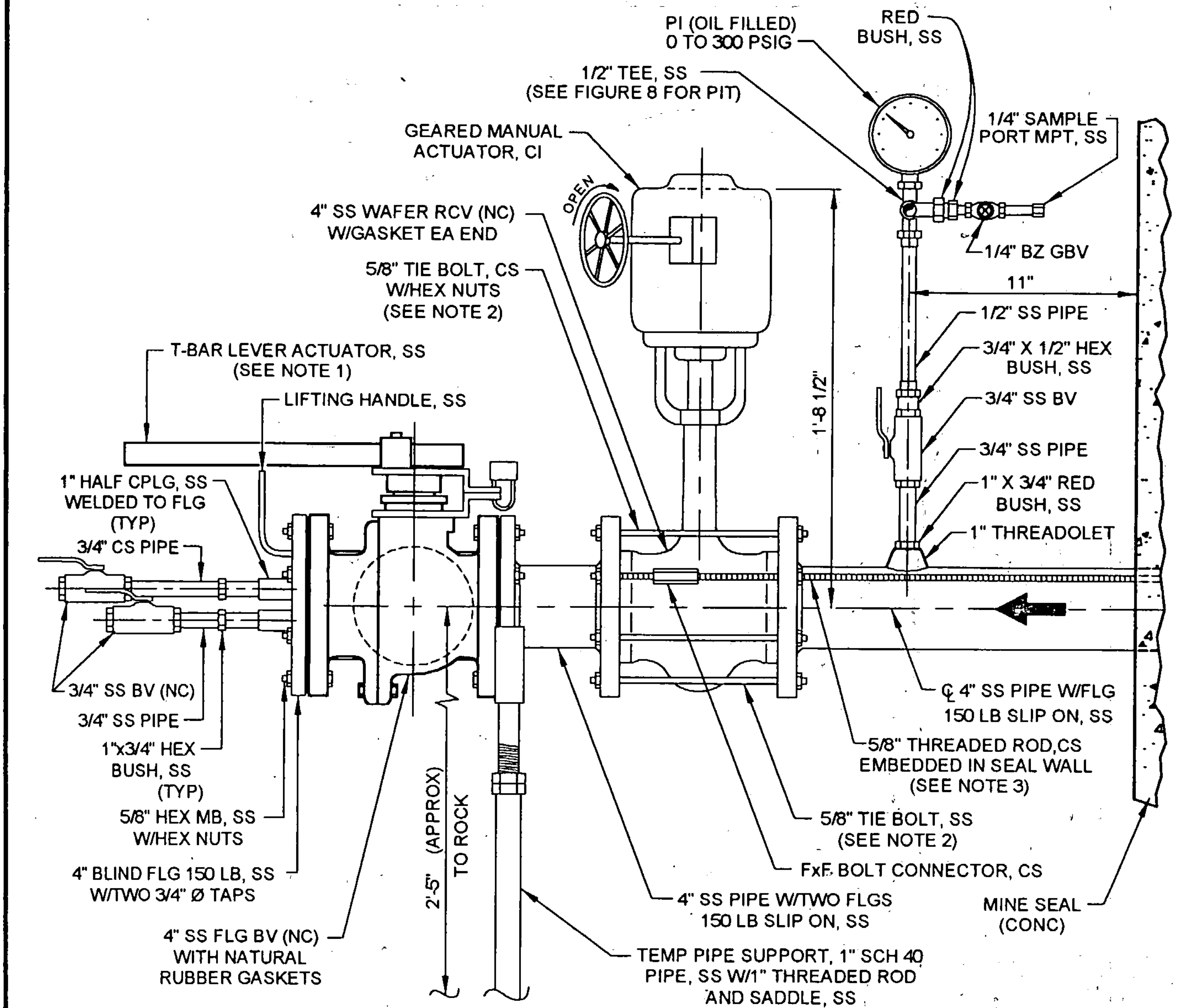
1. SEE FIGURE 9 FOR LEGEND OF ABBREVIATIONS.

0 6 12

1 1/2" = 12"

Regional Water Quality Control Board, Central Valley Region Washington Group International, Inc.	Walker Mine Seal Testing & Evaluation Portola, CA	4-INCH DRAIN PIPES PLAN (AFTER MODIFICATIONS)	
GEI Consultants, Inc.	Project 00387	Dec. 2001	Figure 8

Fig-8.dwg 12-08-01 PYM



LEGEND

BZ	BRONZE	BV	BALL VALVE
CI	CAST IRON	GBV	GLOBE VALVE
CS	CARBON STEEL	RCV	ROTARY CONTROL VALVE
GS	GALVANIZED STEEL	PI	PRESSURE INDICATOR
SS	STAINLESS STEEL	NC	NORMALLY CLOSED
BUSH	BUSHING	NO	NORMALLY OPEN
FLG	FLANGE	PIT	PRESSURE INDICATOR TRANSMITTER
CONC	CONCRETE	MB	MACHINE BOLT
F	FEMALE	EA	EACH
M	MALE	MPT	MALE PIPE THREADS
THK	THICK	RED	REDUCING

NOTES

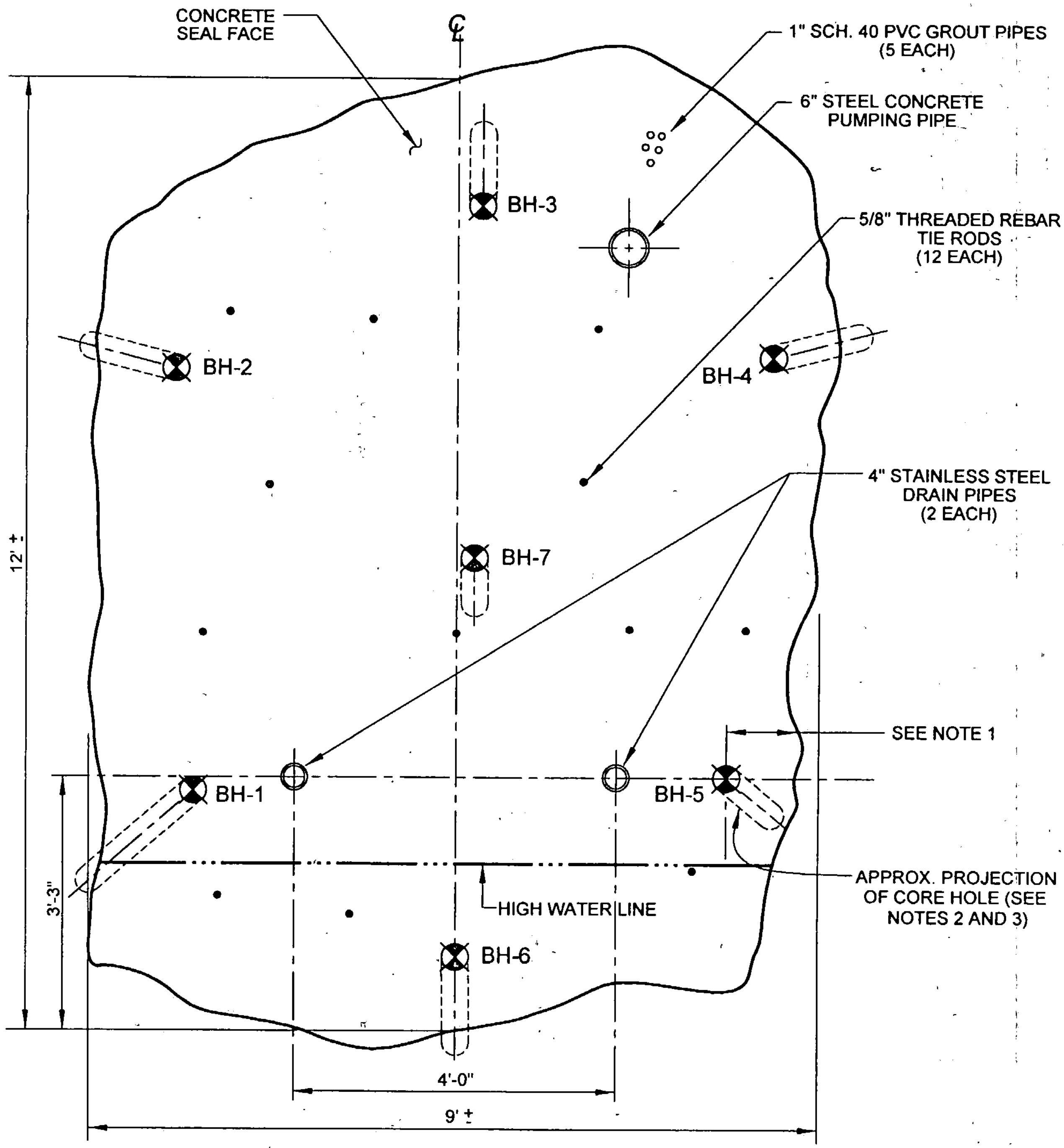
1. ACTUATOR LOCKED IN CLOSED POSITION WITH PADLOCK.
2. SOME FLANGE FASTENERS ARE CARBON STEEL.
3. EMBEDDED ROD POSITION VARIES. RIGHT SIDE ROD ON EAST PIPE IS ABOVE 4-INCH PIPE CENTERLINE. ALL OTHERS ARE BELOW PIPE CENTERLINE.

0 6 12

1 1/2" = 12"

Regional Water Quality Control Board, Central Valley Region Washington Group International, Inc.	Walker Mine Seal Testing & Evaluation Portola, CA	EAST (RIGHT) DRAIN PIPE ELEVATION A-A (AFTER MODIFICATIONS)	
GEI Consultants, Inc.	Project 00387	Dec. 2001	Figure 9

Fig-9.dwg 12-08-01 PYM

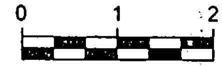


NOTES

1. Core hole entry point is about 10 to 19 inches from the rock face. Hole diameter is 3.8 inches.
2. Hole orientation projected on a vertical plane passing through the entry point:
   - Holes 1, 5, 6 and 7 are oriented at angles of 5, 7, 17 and 7 degrees downward, respectively.
   - Holes 2, 3 and 4 are oriented at angles of 6, 7 and 5 degrees upward, respectively.
3. Hole orientation projected on a horizontal plane passing through the entry point:
   - Holes 1 and 2 are oriented at angles of 8 and 6 degrees toward the left (toward the rock sidewall), respectively.
   - Holes 4 and 5 are oriented at an angle of 5 degrees toward the right (toward the rock sidewall).
   - Holes 3, 6 and 7 are parallel to the axis of the seal.

LEGEND

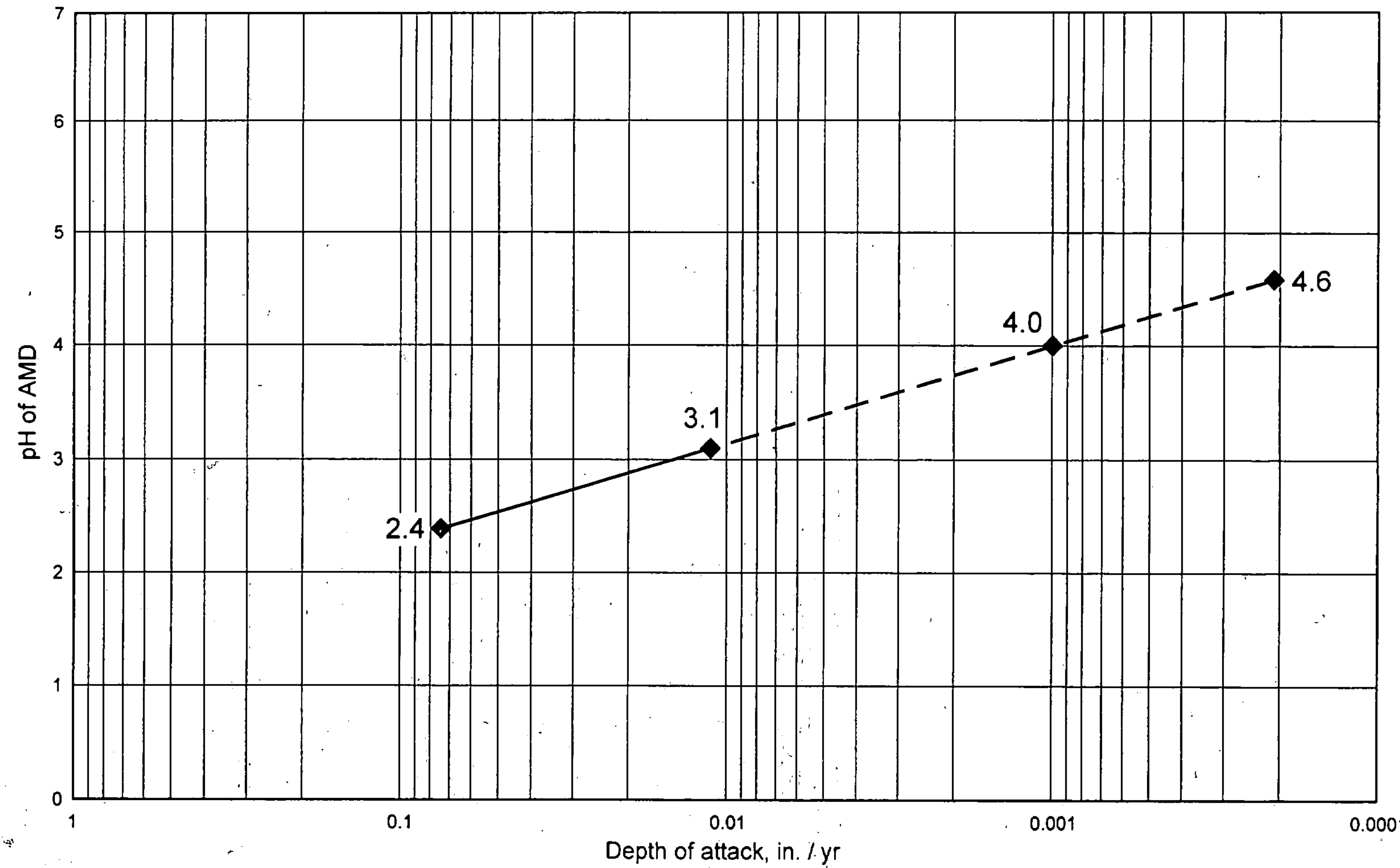
BH-6 ⊗ APPROX. CORE HOLE LOCATION AND NUMBER



Regional Water Quality Control Board Central Valley Region	Walker Mine Seal Testing & Evaluation Portola, CA	APPROXIMATE BOREHOLE LOCATIONS AND ORIENTATIONS
GEI Consultants, Inc.	Project 00387	Dec. 2001 — Figure 10

Fig-10.dwg 12-07-01 PYM

Fig-11 12-07-01 PYM

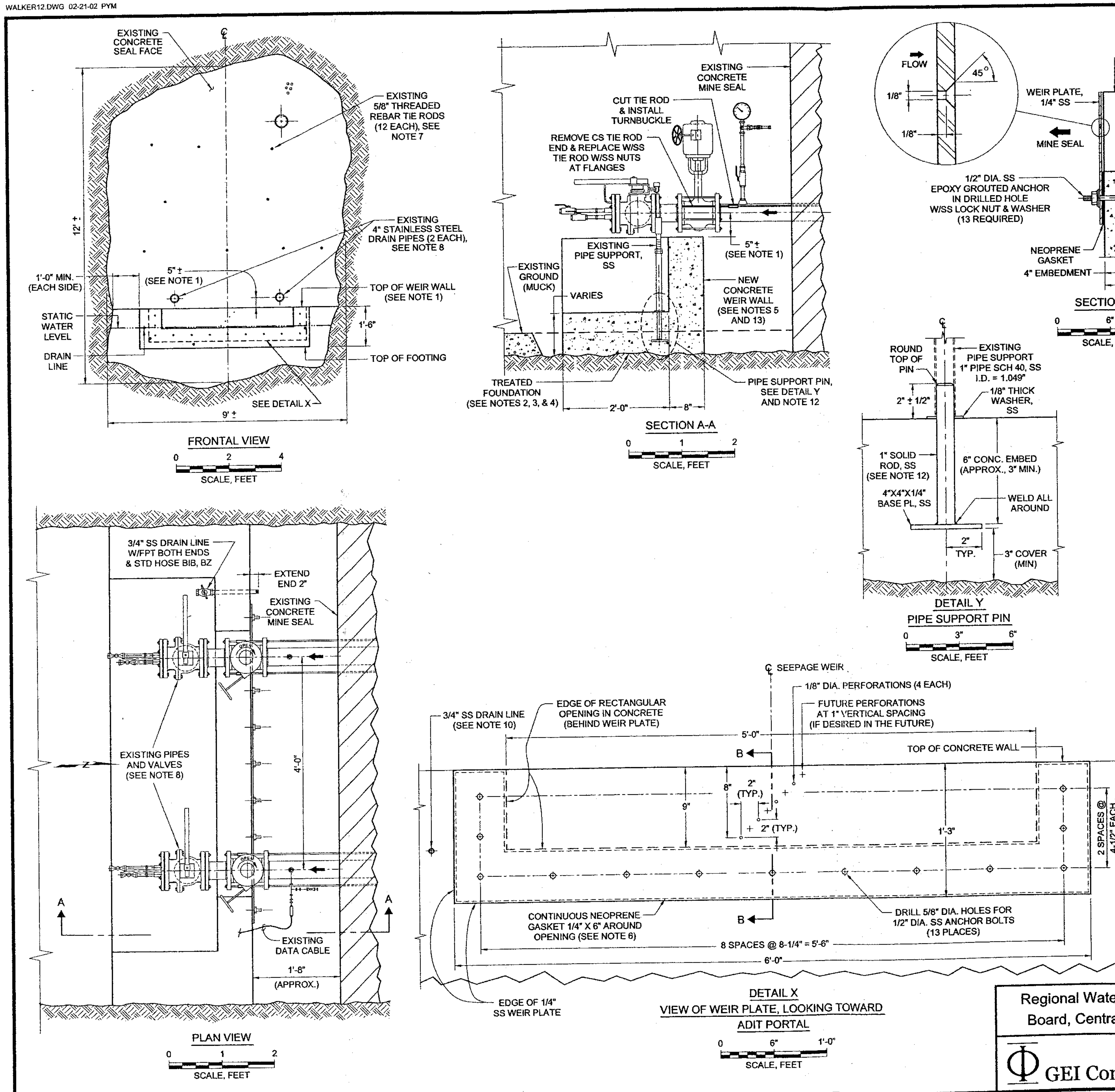


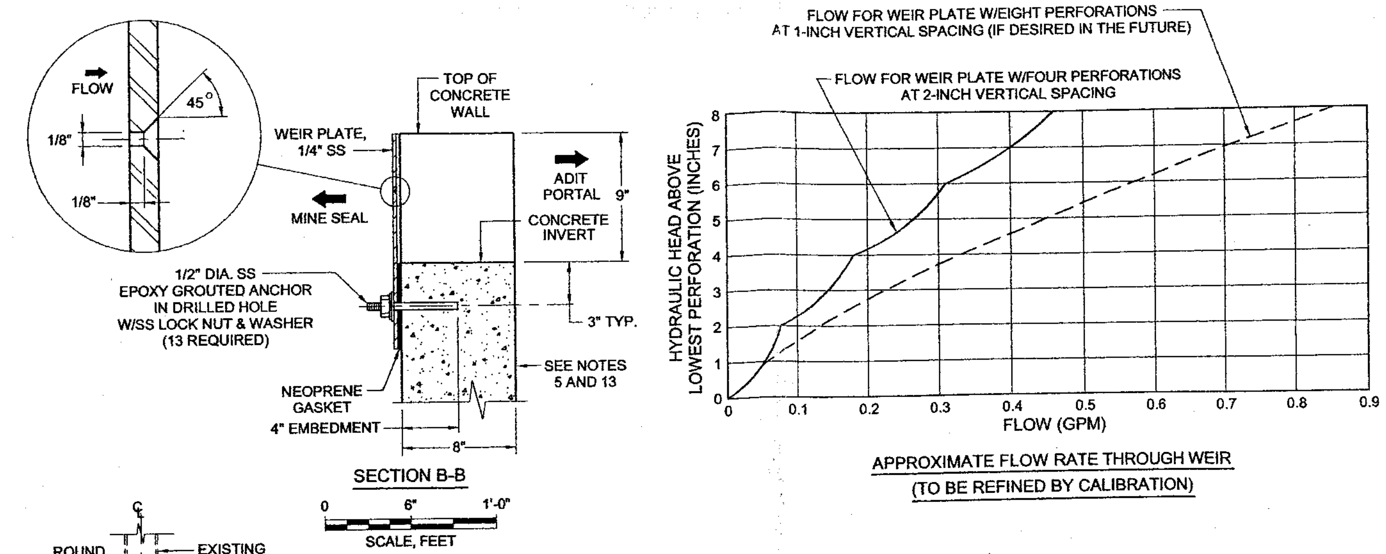
NOTE

Data points at pH 2.4 and 3.1 are from IMM studies. Points at pH 4.0 and 4.6 are based on a straight line interpolation. See cautions and discussion in the text of the report.

Regional Water Quality Control Board Central Valley Region	Walker Mine Seal Testing & Evaluation Portola, California	ESTIMATED RATE OF AMD ATTACK ON CONCRETE MINE SEAL BASED ON IMM STUDIES	
GEI Consultants, Inc.	Project 00387	Dec. 2001	Figure 11

WALKER12.DWG 02-21-02 PYM





NOTES

1. **BEFORE DISTURBING SITE, ACCURATELY MEASURE THE STATIC WATER LEVEL IN THE EXISTING SEEPAGE POOL AT THE TOE OF THE MINE SEAL. SET THE TOP OF CONCRETE WALL TO BE 8-3/4 INCHES ABOVE THE STATIC WATER LEVEL, SO THAT THE LOWEST PERFORATION WILL BE 3/4 INCHES ABOVE THE STATIC WATER LEVEL.**
2. DEWATER WEIR SITE THOROUGHLY. MAINTAIN SITE DEWATERED THROUGHOUT CONSTRUCTION.
3. REMOVE ALL MUCK, DEBRIS AND WATER FROM TUNNEL FLOOR AND WALLS UNDER WALL FOOTING AND WITHIN 6 INCHES AT EITHER SIDE. PRESSURE WASH TUNNEL FLOOR AND WALLS UNDER FOOTING TO ENSURE A CLEAN FOUNDATION OF UNWEATHERED ROCK.
4. BEFORE PLACING CONCRETE, TREAT THE CRACKS IN THE ROCK UNDER THE WEIR FOOTING AND WALLS BY PRESSURE WASHING THEM WITH WATER AND FILLING THEM WITH EPOXY GROUT OR OTHER APPROVED ACID-RESISTANT GROUT MATERIAL. APPLY GROUT IN ACCORDANCE WITH MANUFACTURER'S DIRECTIONS.
5. COAT ALL EXPOSED CONCRETE SURFACES OF THE WEIR WALL WITH THREE COATS OF EPOXY RESIN.
6. NEOPRENE GASKET SHALL BE OF HIGH-GRADE RUBBER MATERIAL. DRILL 5/8-INCH HOLES TO MATCH WEIR PLATE.
7. CUT THREADED BARS FLUSH WITH CONCRETE SURFACE.
8. THROUGHOUT THE WORK, PROTECT AND RETAIN THE EXISTING PIPES AND VALVES.
9. PREPARE THE CONCRETE USING PACKAGED, DRY, COMBINED MATERIALS FOR CONCRETE MEETING ASTM C 387. CONCRETE SHALL BE NORMAL WEIGHT, NORMAL STRENGTH CONCRETE. WATER SHALL BE CLEAN, NON-ACIDIC WATER. IN MIXING THE CONCRETE, ADD THE AMOUNT OF WATER RECOMMENDED BY THE MANUFACTURER IN THE PACKAGE, BUT NOT MORE.
10. PLACE 3/4" SS DRAIN LINE AND VALVE AT THE STATIC WATER LEVEL.
11. AFTER WEIR CONCRETE HAS BEEN CAST AND HAS GAINED SUFFICIENT STRENGTH, PERFORM A LEAK TEST BY FILLING THE UPSTREAM SIDE WITH WATER. MAINTAIN THE DOWNSTREAM SIDE DEWATERED THROUGHOUT THE TEST. OBSERVE DOWNSTREAM SIDE FOR LEAKS FOR A MINIMUM OF 8 HOURS. SEAL ALL LEAKS FROM THE UPSTREAM SIDE USING THE APPROVED GROUT MATERIAL, AND REPEAT TEST UNTIL NO MORE LEAKS ARE OBSERVED.
12. CENTER THE PIPE SUPPORT PIN ON PIPE CENTERLINE BELOW THE DOWNSTREAM FLANGE OF THE SPOOL PIECE BETWEEN THE VALVES. SET THREADED HEIGHT ADJUSTMENT OF PIPE SUPPORT TO MIDPOINT AND TRIM LENGTH OF SUPPORT TO MATCH TOP OF FINISHED CONCRETE. PRIOR TO CONCRETE PLACEMENT, VERIFY THAT PIN HEIGHT WILL ALLOW SUFFICIENT CLEARANCE FOR PIPE SUPPORT INSTALLATION AND REMOVAL.
13. CONCRETE REINFORCING STEEL IS NOT SHOWN ON THIS FIGURE.

Regional Water Quality Control Board, Central Valley Region	Walker Mine Seal Testing & Evaluation Portola, CA	CONCEPTUAL DESIGN OF SEEPAGE WEIR
GEI Consultants, Inc.	Project 00387	Feb. 2002 — Figure 12

PHOTOGRAPHS

Photo No. 1 Seal after construction (December 1987).

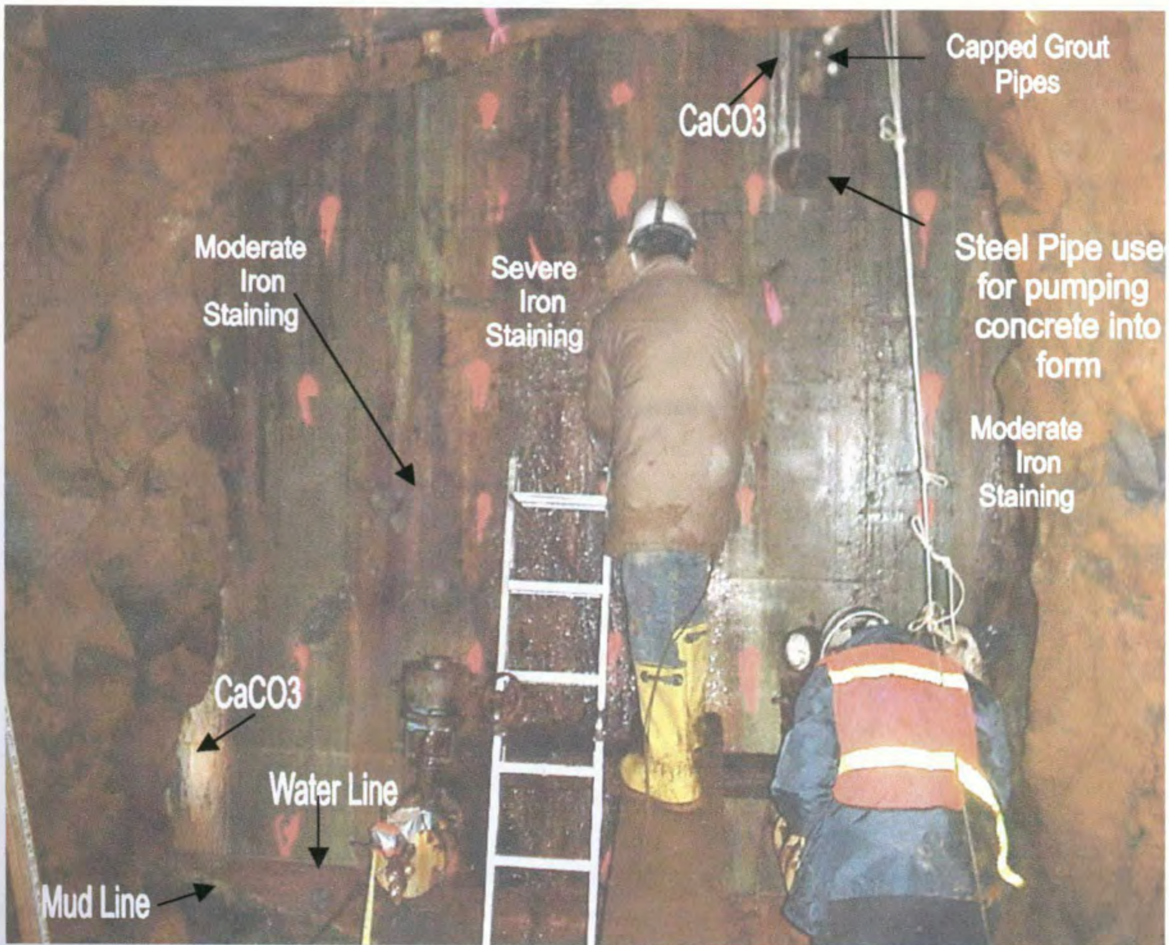


Photo No. 2 Walker Mine Concrete Seal (November 1, 2000). Red paint marks are impact points for nondestructive testing.

Photo No. 3 Bottom left quadrant of the seal (November 2, 2000).

Photo No. 1 Seal after construction (December 1987).


Photo No. 6 Upper part of seal and adit roof (November 2, 2000).


Photo No. 7 Closeup of rock-concrete interface at adit roof (November 2, 2000).

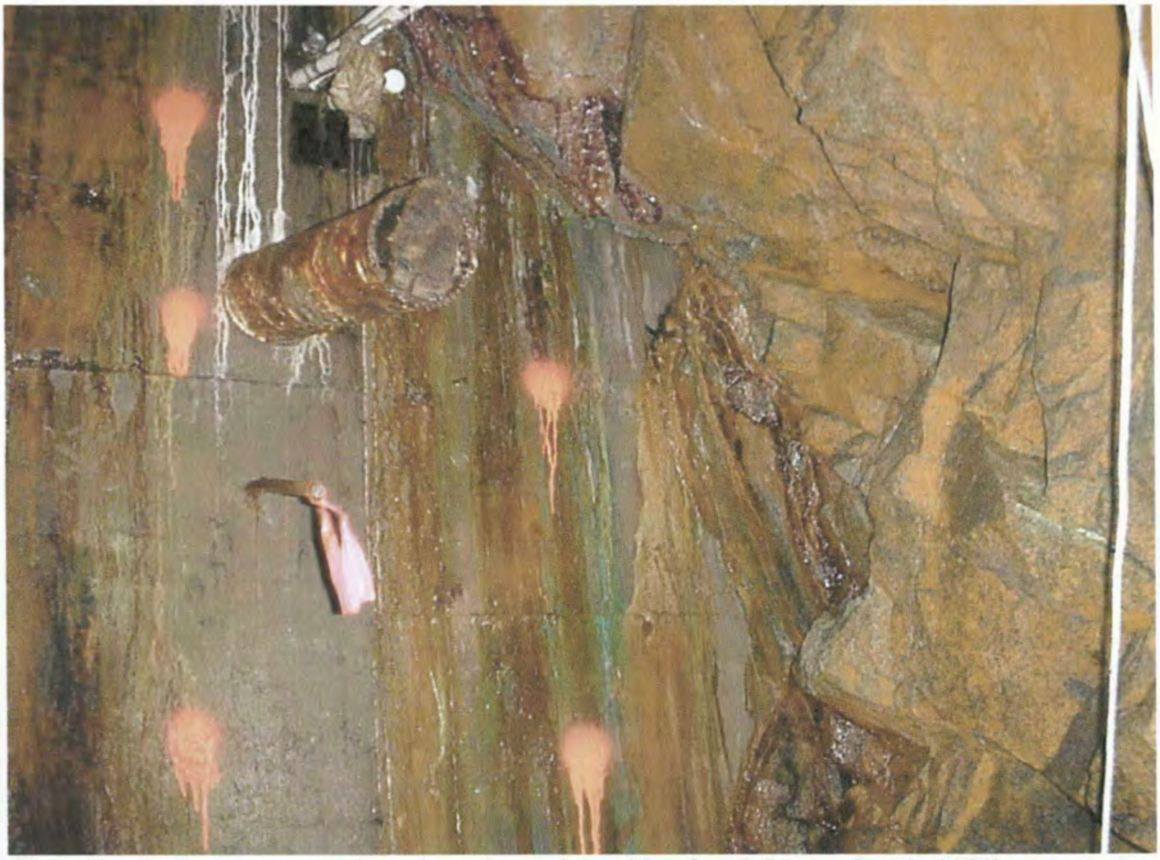


Photo No. 8 Rock-concrete interface along right side of seal (November 2, 2000).

Photo No. 9 Seal face and chained-up valves on November 2, 2000. Seepage pool next to seal is practically dewatered.

Photo No. 10 Left drain pipe and valve with measuring tape attached (November 2, 2000).

Photo No. 11 Left drain pipe on November 2, 2000.

Photo No. 12 Right drain pipe and valve (November 2, 2000).

Photo No. 13 Right drain pipe on November 2, 2000.

Photo No. 14 Right drain pipe from top showing pressure gauge, sampling valve, pressure transducer and control valve actuator (November 2, 2000).

Photo No. 15 Coring of hole BH-7 on August 29, 2001. Protective timber frames were constructed and placed over the drain pipes while the upper holes (BH-2, -3, -4 and –7) were drilled.

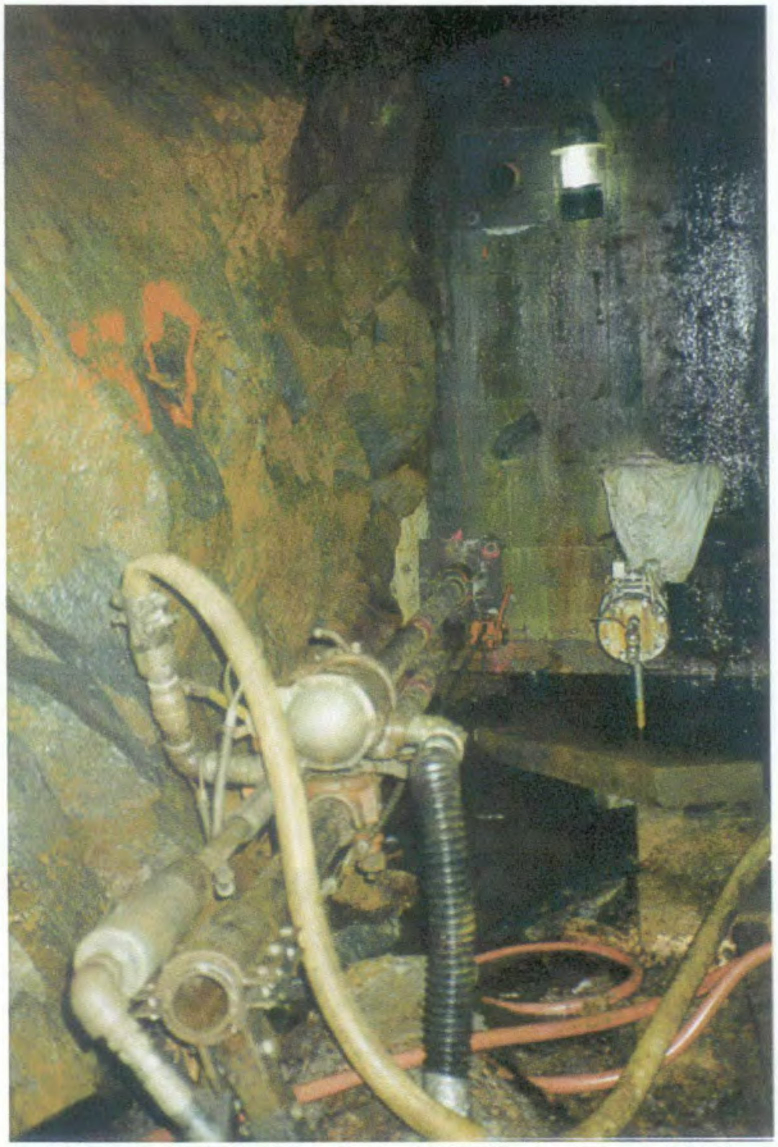


Photo No. 16 Coring of hole BH-1 (September 9, 2001).

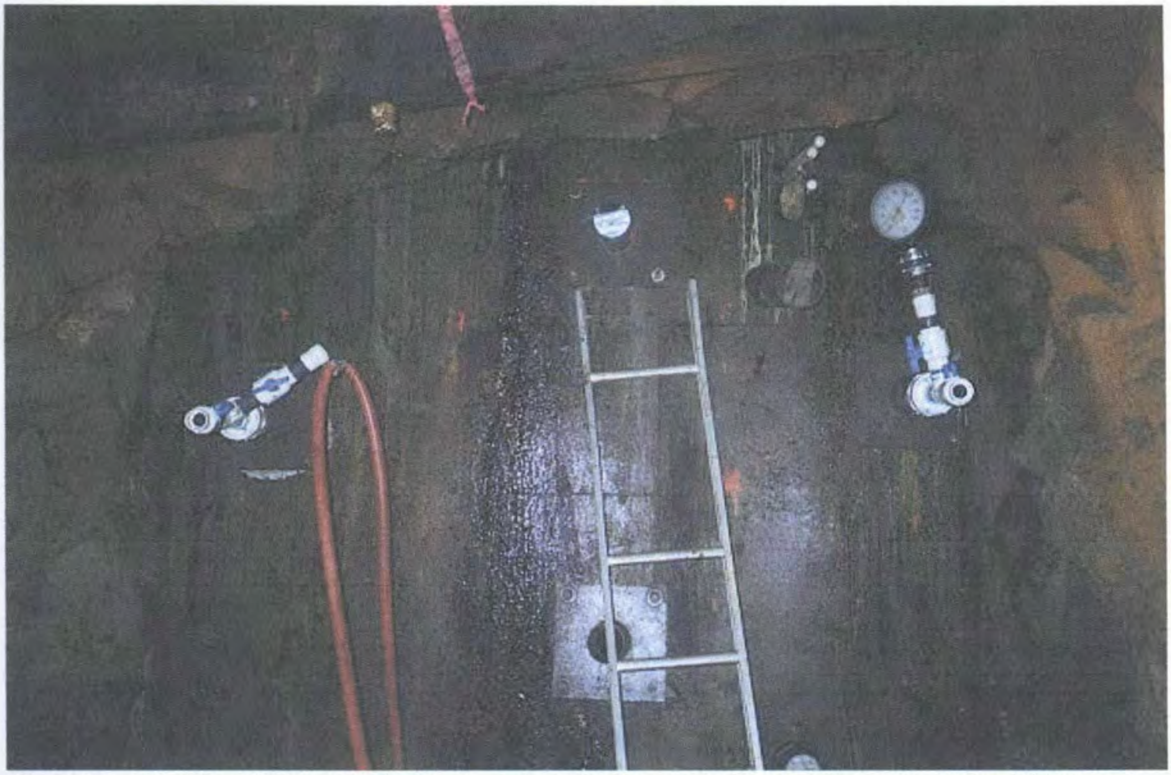


Photo No. 17 Core holes BH-2, -3, -4, and –7 in the upper half of the seal. Holes BH-2 and BH-4 are equipped with shutoff valves (September 10, 2001).

Photo No. 18 Seal face after grouting of holes and removal of collar pipes and plates. The holes were backfilled and drypacked (September 16, 2001).

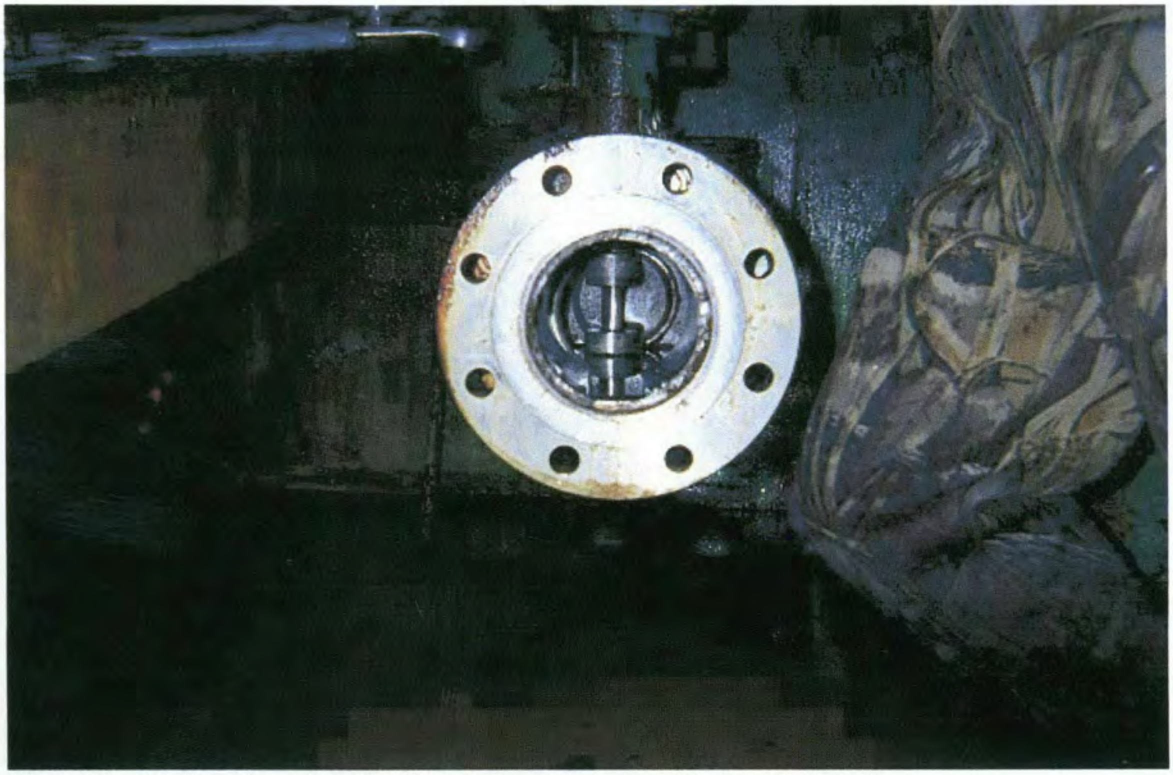


Photo No. 19 Interior of DeZurik 4-inch control valve (June 13, 2001).

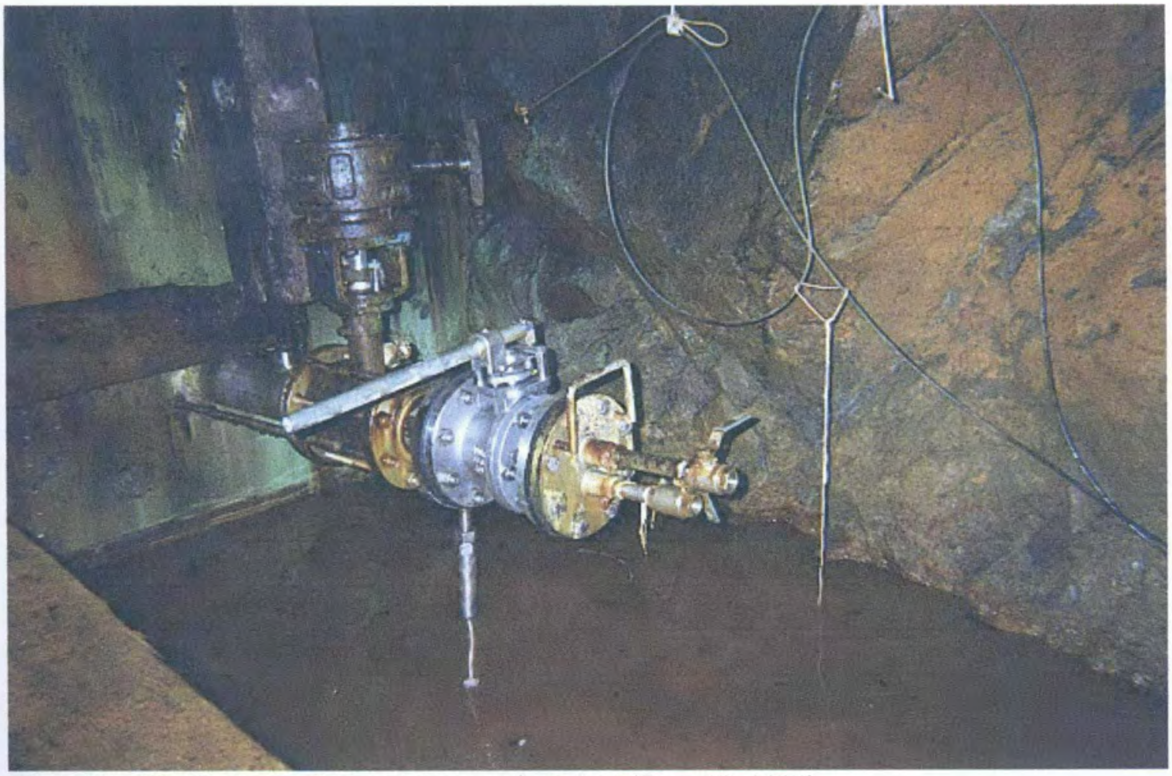


Photo No. 20 Backup valve and support installed (June 13, 2001).

Photo No. 21 Both backup valves and pipe supports installed. Control valve actuators covered with plastic bags (June 14, 2001).

Photo No. 22 Overview of mine seal showing east 4-inch pipe with control valve. Wood post behind valve with wedge at top braces temporary plug inserted in ¾ inch fitting in 4-inch pipe (June 13, 2001).

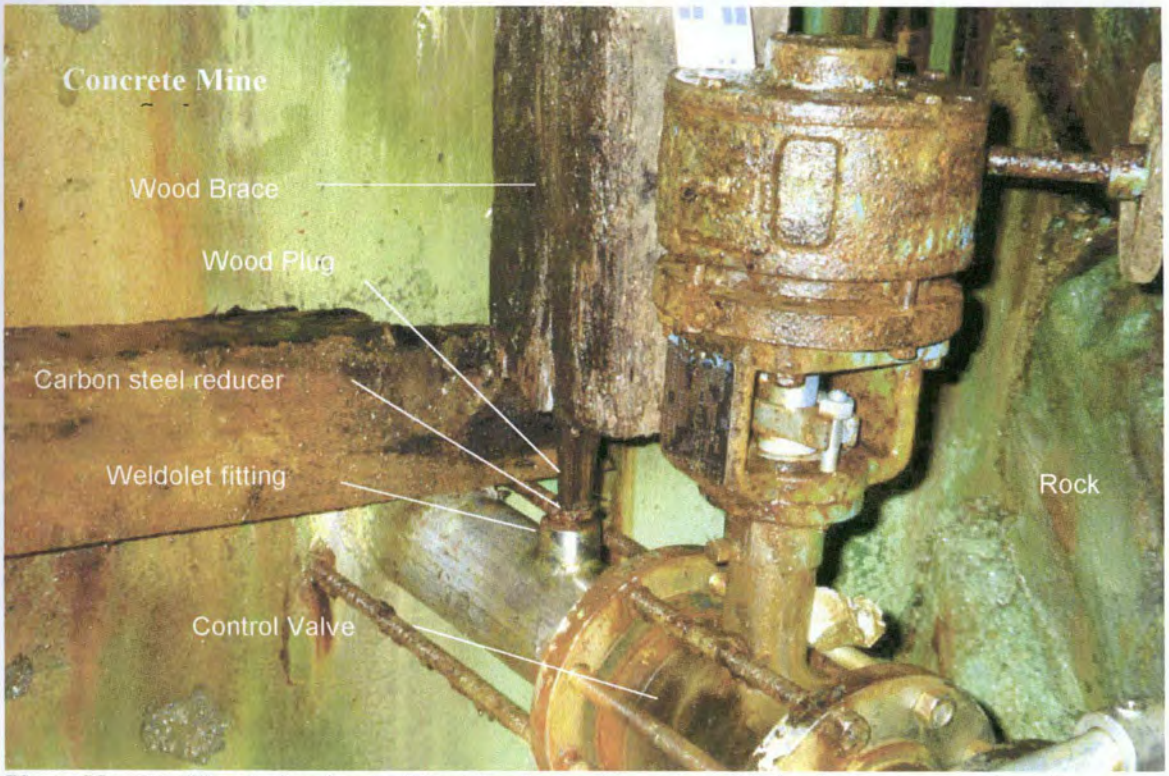


Photo No. 23 Wood plug in steel bushing braced by post. Fitting is between seal and control valve (June 14, 2001).

Photo No. 24 New stainless steel bushing and pipe in threadolet fitting (June 14, 2001).

Photo No. 25 Repair completed (June 14, 2001).

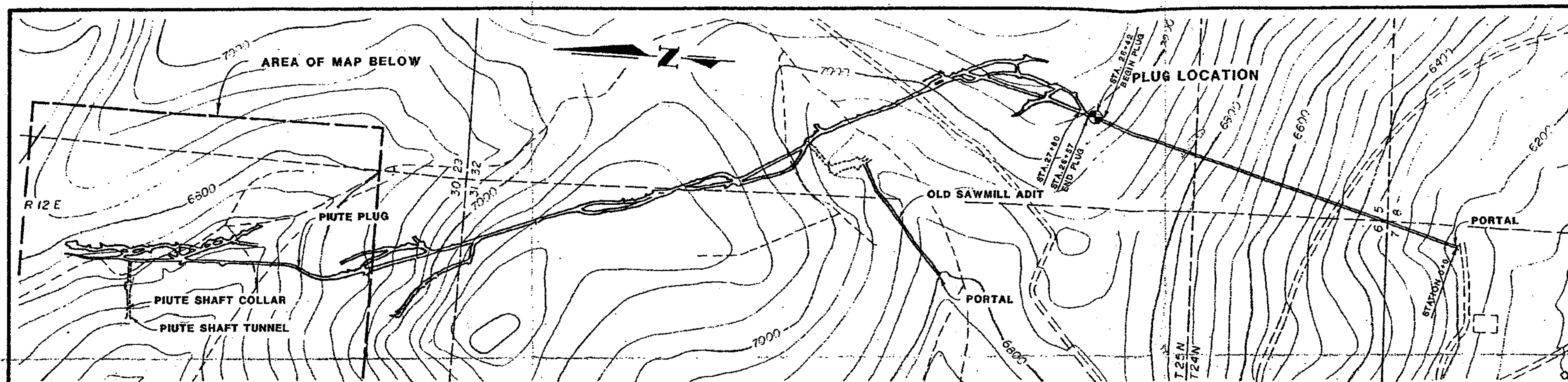
Photo No. 26 New gauge installed and pressure transducer attached. Gauge shows 45 psi (June 14, 2001).

Photo No. 27 Broken ¾-inch carbon steel nipple, stainless steel ¾-inch ball valve, stainless steel ½-inch pipe, and cracked pressure gauge (June 1, 2001).

Appendix A

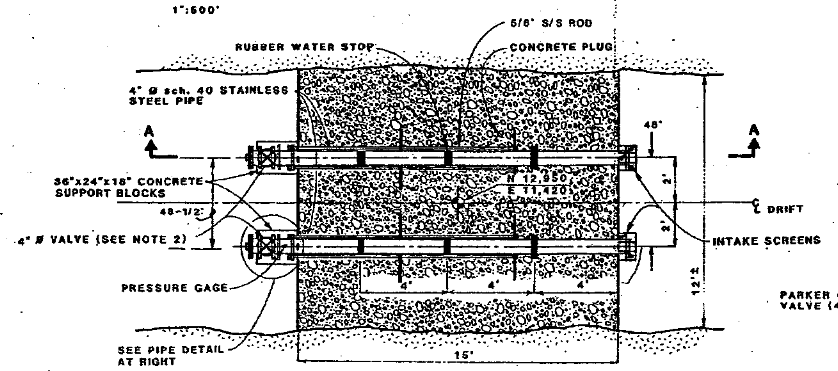
Appendix A

SRK's As-Built Drawing for Walker Mine Seal



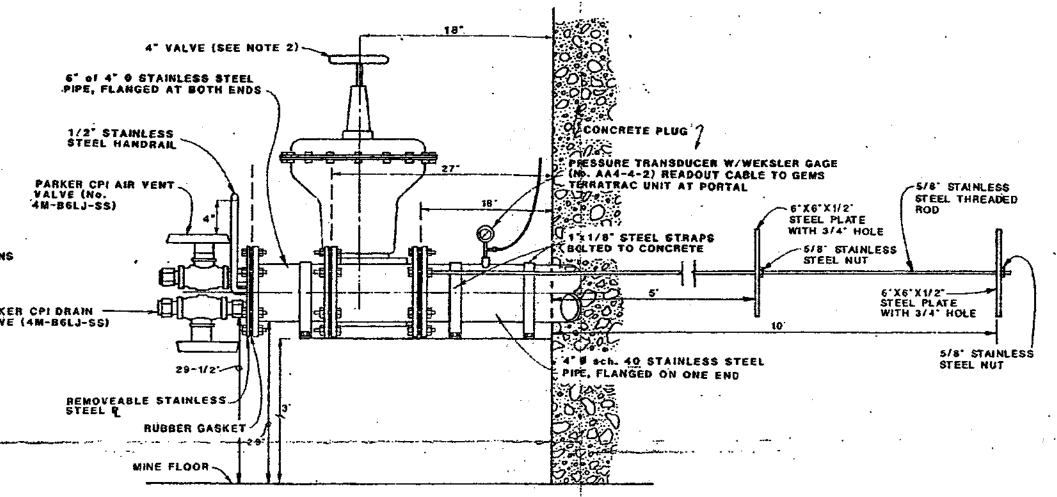
OVERALL MINE PLAN AND PLUG LOCATION

1":600'



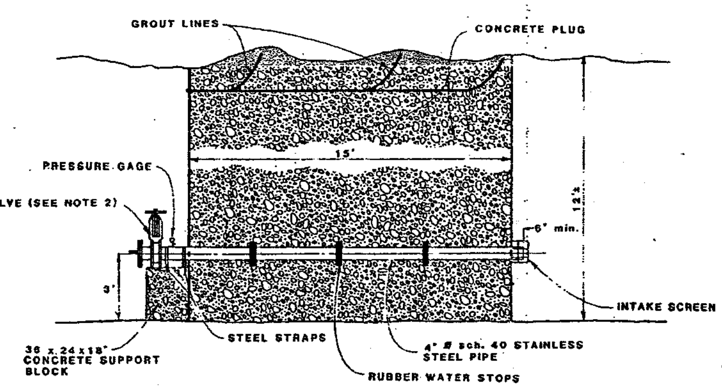
PLAN OF MINE PLUG

1":4'



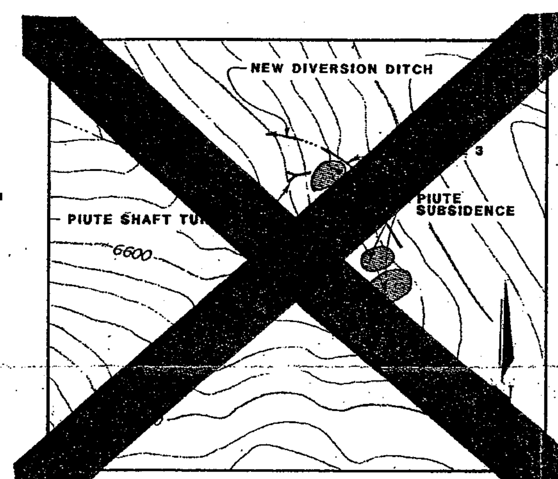
PIPE OUTLET SYSTEM DETAIL

1-1/2":1'



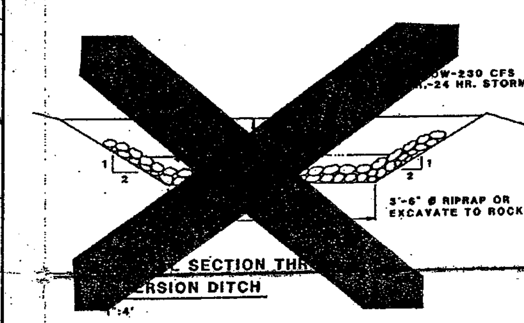
SECTION A-A SECTION THROUGH MINE PLUG

1":4'



DIVERSION DITCH LOCATION PLAN

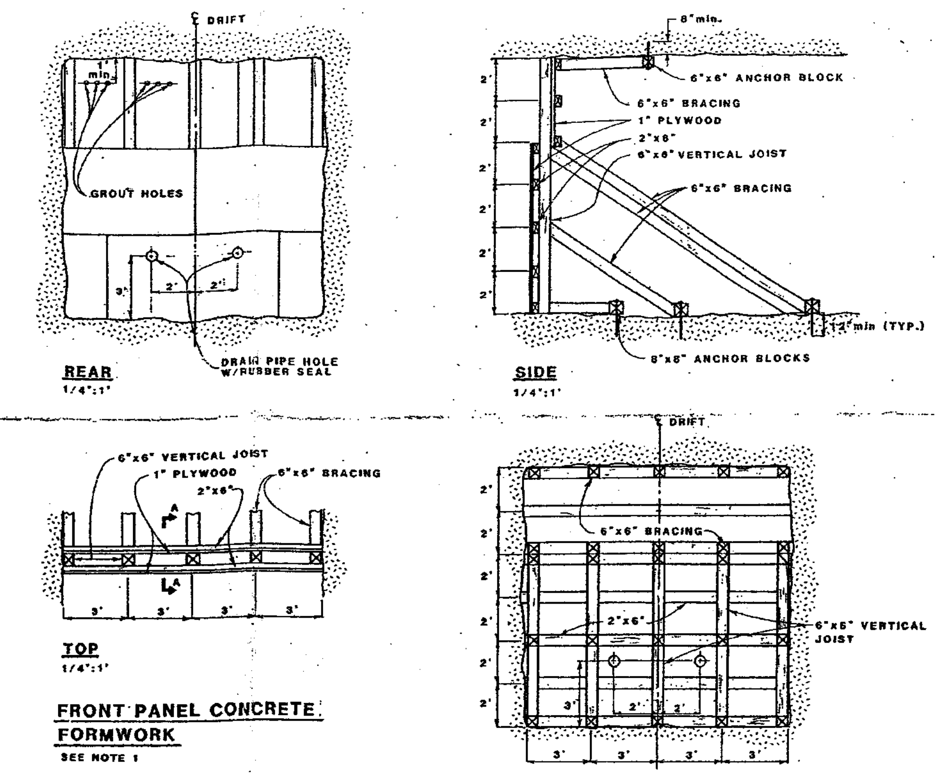
1":500'



SECTION TH...
...RSION DITCH

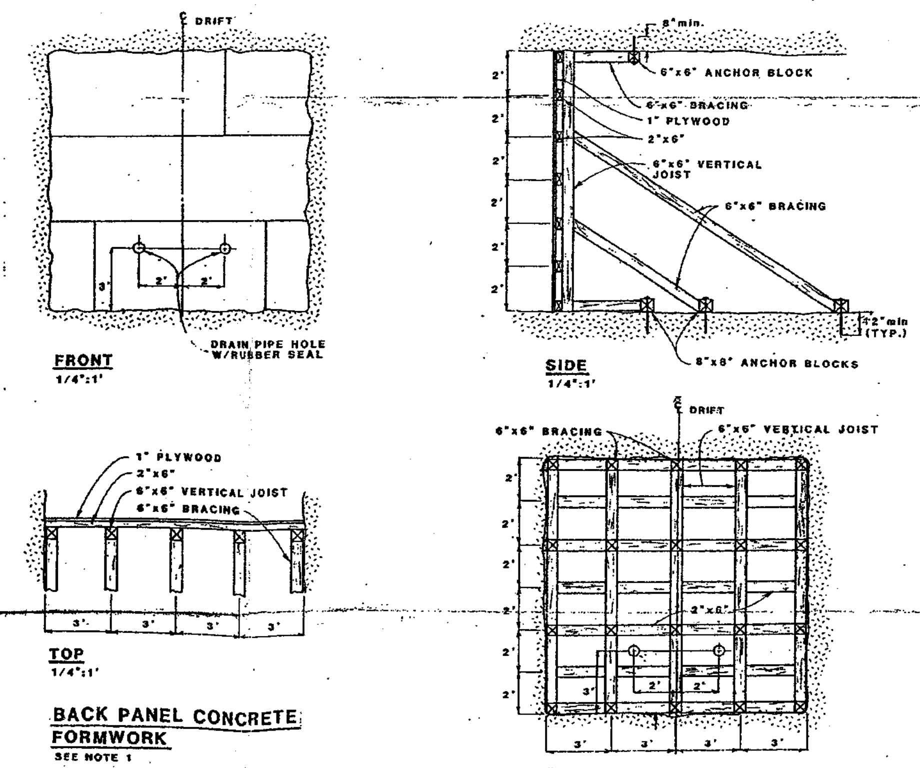
NOTES:

1. Formwork shown in this drawing is for reference only. Contractor shall provide final plans prior to construction.
2. The 4" Ø valve will be a DeZurik K-Max valve with a master gear or approved equivalent. (DeZurik part no. GKM,6,W2,S2,TC,S2).
3. Actual ditch alignment to be field located by the Engineer.



FRONT PANEL CONCRETE FORMWORK

SEE NOTE 1



BACK PANEL CONCRETE FORMWORK

SEE NOTE 1

STATE APPROVAL

ENGINEER: IAN P.G. HUTCHISON, PE No. 039319

PREPARED FOR: CALIFORNIA REGIONAL WATER QUALITY CONTROL BOARD CENTRAL VALLEY REGION

PREPARED BY: STEFFEN ROBERTSON & KIRSTEN Consulting Engineers

REVISIONS No.	DESCRIPTION	BY	CKD	APPROV	DATE
A	ISSUED TO CRWQCB FOR REVIEW	DAP		DAP	11/85
B	AS BUILT REVISIONS	DAP		IH	2/89

REFERENCES: CONTRACT No. 4-051-150-0 REPORT AND ADDENDUM 1, BY SRK, NOV. 1985

DESIGN	AM/CS/DP		
DRAWN	SLH		
CHECKED	DP	3/89	
APPROV	DAP	3/89	
APPROV			
APPROV			

TITLE CONTRACT No. 4-051-150-0

AS-BUILT WALKER MINE PLUG PLAN, PROFILE AND DETAILS

SCALE AS SHOWN

DRAWING No. 06901-01

REVISION B

Appendix B

Appendix B

Nondestructive Testing of Mine Seal

B.1 "Evaluation of Concrete Mine Seal Using Nondestructive Testing, Walker Mine, Portola, Plumas County, California", CTL, November 28, 2000.

B.2 "Evaluation of Concrete Mine Seal Using Nondestructive Testing, Walker Mine, Portola, Plumas County, California, 2nd Visit", CTL, October 31, 2001.

B.3 Schmidt Hammer Test Results

B.4 "Ultrasonic Thickness Evaluation, Walker Mine Seal", Kleinfelder, Inc., October 17, 2001.

Appendix B.1

"Evaluation of Concrete Mine Seal Using Nondestructive Testing, Walker Mine, Portola, Plumas County, California"
CTL, November 28, 2000.

CTL
Construction Technology Laboratories, Inc.

www.ctlgroup.com
5420 Old Orchard Road
Skokie, Illinois 60077
800.522.2CTL (2285)

November 28, 2000

Mr. Alberto Pujol
GEI Consultants, Inc.
2201 Broadway Suite 321
Oakland, CA 94612

**RE: EVALUATION OF CONCRETE MINE SEAL USING NONDESTRUCTIVE TESTING
WALKER MINE, PORTOLA, PLUMAS COUNTY, CALIFORNIA
CALIFORNIA REGIONAL WATER QUALITY CONTROL BOARD
CENTRAL VALLEY REGION RESD-PSB-AD 2000-11
CTL PROJECT NO. 320098**

Dear Mr. Pujol:

GEI Consultants, Inc. instructed Construction Technology Laboratories, Inc. (CTL) to carry out an evaluation of a mass concrete mine seal (plug), located in the Walker Mine, Portola, Plumas County, California. The first phase of this evaluation comprised nondestructive testing from the downstream face of the plug.

Testing was performed on site on November 2, 2000. The nondestructive Impulse Response, Impedance Log and Impact Echo testing techniques were used to evaluate the piers. All test methods are fully described in Appendix A, B and C to this report.

CONCRETE SEAL PLUG DESCRIPTION

The mass concrete plug fills the original rock tunnel, and is approximately 9 ft wide by 12 ft high at the visible face, and approximately 15 ft deep. The rock surrounding the plug appears to be hard and compact at the plug face. Water seepage is visible at the top face, and it is not clear whether this seepage is coming from the rock or from the concrete/rock interface, or both.

Plate 1 in the Appendix D shows the plug. Two drain outlet valves are visible on the plug face: at approximately 3 ft from the left and right edges respectively and approximately 3 ft from the base of the plug. In addition, a sealed pipe is apparent in the top right corner of Plate 1, which is thought to be the entry point of the original concrete tremie. The sheen on the concrete face is from the water seepage. Several 1 in. diameter grout pipes were also noted at the top right corner of the plug.

TEST PROGRAM

The plug was tested using the Impulse Response (IR) method and Impact Echo as described in Appendices A through C to this report. For the IR method, a receiving sensor (geophone) was coupled to the face of the plug using grease. A sledgehammer impacted the plug face at selected test points, and the time history of the force measured by the hammer and of the receiving sensor after impact was recorded for future analysis. All data was stored on a field portable computer for analysis in the office.

Two IR testing approaches were adopted:

- A matrix of test points at 2-ft vertical and horizontal spacing was established, and each test point was impacted with the geophone located at approximately 6 inches from the point of impact. This test methodology gives information on the concrete condition to a depth of approximately 3 ft into the plug from its face, and is fully described in Appendix A. This program is entitled *IR – Concrete Quality*.
- The geophone was positioned at the center of the plug face, and each test was performed by striking the face at points around the periphery of the plug. A digital gain of either 10 or 20 was applied to the velocity response from the geophone to amplify the signal strength, in view of the damping effect of the seated plug. The test data can be interpreted using simulation methods originally developed for testing the length and integrity of drilled shafts and caissons such as the Impedance Log, and the test method is fully described in Appendix B. This program is entitled *IR – Plug/Rock Interface*.
- Limited Impact Echo testing was performed due to the soft nature of the surface of the plug. The soft nature of the surface of the plug acted as a damper to the impactor used. Several different size impactors was used to generate the energy necessary to penetrate the entire length of the plug. Despite using three different size impactors, the energy generated was insufficient to penetrate the entire length of the plug.

DATA ANALYSIS

1. IR – Concrete Quality

The IR test in this mode (see Appendix A) produces two principal parameters: element mobility and dynamic stiffness. The measured values of mobility and stiffness remain constant for a given unit thickness and concrete quality (modulus, density), and for similar support conditions. If the concrete thickness decreases, the mobility increases and the stiffness decreases. If the concrete quality reduces, the mobility increases and the stiffness decreases. A separation within the structure such as a cold joint or delamination will result in significant changes in these two parameters. Also, for poor density concrete in the outer 3 ft of the unit being tested, the measured stiffness and mobility reflect the concrete consolidation conditions; in the event of poor consolidation, the stiffness will decrease and the average mobility will increase.

The 2-ft x 2-ft test grid laid out over the total plug face gave a grid with 5 test points from left to right in the horizontal direction (Columns A to E) and 6 test points from bottom to top (Rows 1 to 6). The measured values of dynamic stiffness and average mobility are plotted in contour form in Figures D.1 and D.2 respectively.

2. IR – Plug/Rock Interface

The Impulse Response results obtained in this mode were analyzed to measure:

a) the distance from the face to the back of the plug,
b) the equivalent dynamic shear modulus at the concrete rock interface at different points around the plug.

Typical values for concrete stress wave velocities in integral foundation piers with good concrete quality vary between 12,500 and 14,000 ft/s, with average values around 13,125 ft/s.
The depth of the plug measured with the mobility plots assuming a stress wave velocity in the concrete of 13,125 ft/s varied between 14.5 ft and 15.5 ft (See Figure D.3).

It was also possible to measure the characteristic impedance (Impedance Log and Sonic Echo) of the plug from some of the IR test results as described in Appendix B. This value of characteristic impedance was then combined with the Sonic Echo test results, to produce an Impedance Log for the plug. The Impedance Log method is also described in Appendix B. A simulated mobility plot from the Impedance Log is compared with an actual test result in Appendix D, Figure D.4. The matching value for the shear wave velocity, ß at the concrete/rock interface is obtained from the parameters used in the simulation model (Davis & Dunn, 1974).

DISCUSSION OF TEST RESULTS

1. IR – Concrete Quality

Figures D.1 and D.2 show that the values of stiffness and average mobility are relatively consistent over most of the plug face, apart from a zone around column D, from test rows 3 to 6. It is expected that the average mobility will decrease from the center of the plug to the edge, and that the inverse will happen for the measured stiffness. This is the case for three of the four plug face quadrants (SE, SW and NW), whereas the fourth quadrant shows considerably higher values for average mobility, and correspondingly much lower values of stiffness.

It is of interest to note that these poorer results come from a zone immediately surrounding and below the location of the original concrete tremie. The very high mobility values indicate that poor concrete consolidation is present in a zone approximately 4 ft high by 2 ft wide at the locations shown in the northeast quadrant, as shown in Figure D.2.

2. IR – Plug/Rock Interface

The depth of the plug measured with the mobility plots assuming a stress wave velocity in the concrete of 13,125 ft/s (4,000 m/s) varied between 14.5 ft and 15.5 ft. If the projected plug depth of 15 ft is substituted, then the concrete stress wave velocity is higher, between 12,690 and 13,560 ft/s (3,870 to 4,130 m/s), indicating that the concrete in the plug is of good quality.

Values for the shear wave velocity at the rock/concrete interface for different points around the plug perimeter are presented in Table 1 below.

Table 1

Test Location (Column-Row)	Shear Wave Velocity, ß (m/s)
A-2	750
A-3	750
A-5	500
B-2	750
C-1	450
C-6	450
D-2	500
D-5	600
D-6	450
E-1	750
E-2	750
E-3	750
E-5	450

As a comparison, measured values for ß for rock socketed caissons are usually in the range of 300 to 400 m/s for high concrete/rock bond. The very high values measured here indicate a very good bond between the concrete and the rock.

Lower values for ß (between 450 and 500 m/s) are concentrated around the bottom center and the top of the plug, with high values along the sides of the plug over the lower two thirds. This is to be expected, as a result of the concrete placement technique employed.

DISCUSSION

The overall condition of the concrete in the plug is good, except for areas with high mobility and low stiffness values, indicating zones of poor concrete consolidation. This is present in a zone approximately 4 ft high by 2 ft wide around column D in the northeast quadrant. This was probably caused by "blind spots" developing during concrete placement. We recommend that this zone be inspected by cross hole sonic logging and coring to a limited depth during the second phase of the investigation. The test program for the second phase will be provided at a later date.

Very high values of shear wave velocities for the concrete/rock interface were calculated, indicating a very good bond between the rock and concrete plug. The bond at the interface is greater at the lower two thirds of the plug edges. For a plug depth of 15 ft, the average calculated concrete compression wave velocities are between 12,690 and 13,560 ft/s (3,870 and 4,130 m/s), indicating good quality concrete in the body of the plug with no discontinuities. It is our recommendation that this limited nondestructive survey of the plug/rock interface in this first phase be complemented by cross hole sonic logging in core holes intersecting the interface. It should be noted that the wave speed velocities noted on above is based on industries standards. The wet environment present, and the expected higher concrete compressive strength will affect the wave speed velocity. CTL recommends that the cores be tested to obtain actual wave speed velocity.

We sincerely appreciate the opportunity to work with GEI Consultants, Inc. on this project. If you have any questions, please call us at 1-800-522-2CTL.

Respectfully:

CONSTRUCTION TECHNOLOGY LABORATORIES, INC.

Malcolm K. Lim, P.E.
Project Manager

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Attachments

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APPENDIX A

STRESS WAVE TESTS FOR CONCRETE

1. THE IMPULSE RESPONSE TEST

The Impulse Response (IR) test method is a nondestructive, stress wave test, used extensively in the evaluation of machined metallic components in the aircraft industry. Its application to concrete structures in Civil Engineering is less well known, and the method has received far less publicity than the recently developed Impact-Echo (I-E) test (Sansalone & Streett, 1997). Both methods are described in the American Concrete Institute Report ACI 228.2R-98, *"Nondestructive Test Methods for Evaluation of Concrete in Structures"*.

The IR method (also referred to in earlier literature as the Transient Dynamic Response or Sonic Mobility method) is a direct descendant of the Forced Vibration method for evaluating the integrity of concrete drilled shafts, developed in France in the 1960's (Davis & Dunn, 1974). The basic theory of dynamic mobility developed at that time has not changed; however, its range of applications to different structural elements has increased to incorporate the following problems:

- voiding beneath concrete highway, spillway and floor slabs (Davis & Hertlein, 1987),
- delamination of concrete around steel reinforcement in slabs, walls and large structures such as dams, chimney stacks and silos (Davis & Hertlein, 1995),
- low density concrete (honeycombing) and cracking in concrete elements (Davis & Hertlein, 1995; Davis *et al,* 1997),
- the depth of ASR attack in drilled shafts used as pylon foundations (Davis & Kennedy, 1998),
- debonding of asphalt and concrete overlays to concrete substrates (Davis *et al*, 1996),
- the degree of stress transfer through load transfer systems across joints in concrete slabs (Davis & Hertlein, 1987).

<u>IR Testing Equipment</u>

The method uses a low strain impact to send a stress wave through the tested element. The impactor is usually a 1-kg sledgehammer with a built-in load cell in the hammerhead. The maximum compressive stress at the impact point in concrete is directly related to the elastic properties of the hammer tip. Typical stress levels range from 5 MPa for hard rubber tips to more than 50 MPa for aluminum tips. The response to the input stress is normally measured using a velocity transducer (geophone). This receiver is preferred because of its stability at low frequencies and its robust performance in practice. Both the hammer and the geophone are linked to a portable field computer for data acquisition and storage.

<u>Method Description</u>

When testing plate-like structures, the Impact-Echo method uses the reflected stress wave from the base of the concrete element or from some anomaly within that element (requiring a frequency range normally between 10 and 50 kHz). The IR test uses a compressive stress impact approximately 100 times that of the I-E test. This greater stress input means that the plate responds to the IR hammer impact in a bending mode over a very much lower frequency range (0-1 kHz for plate structures), as opposed to the reflective mode of the I-E test.

Both the time records for the hammer force and the geophone velocity response are processed in the field computer using the Fast Fourier Transform (FFT) algorithm. The resulting velocity spectrum is divided by the force spectrum to obtain a transfer function, referred to as the *Mobility* of the element under test. The test graph of Mobility plotted against frequency over the 0-1kHz range contains information on the condition and the integrity of the concrete in the tested elements, obtained from the following measured parameters:

- *Dynamic Stiffness:* The slope of the portion of the Mobility plot below 0.1 kHz defines the compliance or flexibility of the area around the test point for a normalized force input. The inverse of the compliance is the dynamic stiffness of the structural element at the test point. This can be expressed as:

  Stiffness *f [concrete quality, element thickness, element support condition]*

- *Mobility and Damping:* The element's response to the stress wave imposed will be damped by the element's intrinsic rigidity (body damping). The mean mobility value over the 0.1-1 kHz range is directly related to the density and the thickness of a plate element, for example. A reduction in plate thickness corresponds to an increase in mean mobility. As an example, when total debonding of an upper layer is present, the mean mobility reflects the thickness of the upper, debonded layer (in other words, the slab becomes more mobile). Also, any cracking or honeycombing in the concrete will reduce the damping and hence the stability of the mobility plots over the tested frequency range.

- *Peak/Mean Mobility Ratio:* When debonding or delamination is present within a structural element, or when there is loss of support beneath a concrete slab on grade, the response behavior of the uppermost layer controls the IR result. In addition to the increase in mean mobility between 0.1 and 1 kHz, the dynamic stiffness decreases greatly. The peak mobility below 0.1 kHz becomes appreciably higher than the mean mobility from 0.1-1 kHz. The ratio of this peak to mean mobility is an indicator of the presence and degree of either debonding within the element or voiding/loss of support beneath a slab on grade.

References

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2. Davis A.G. and B.H. Hertlein 1987, "Nondestructive testing of concrete pavement slabs and floors with the transient dynamic response method," *Proc. Int. Conf. Structural Faults and Repair,* London, July 1987, Vol. 2, pp. 429-433.

3. Davis, A.G. and B.H. Hertlein, 1995, "Nondestructive testing of concrete chimneys and other structures," *Conf. Nondestructive Evaluation of Aging Structures and Dams*, Proc. SPIE 2457, 129-136, Oakland CA, June 1995.
4. Davis, A.G., J. G. Evans and B.H. Hertlein, 1997, "Nondestructive evaluation of concrete radioactive waste tanks," *Journal of Performance of Constructed Facilities,* ASCE, Vol. 11, No. 4, November 1997, pp. 161-167.
5. Davis, A.G., B.H. Hertlein, M. Lim and K. Michols, 1996, "Impact-Echo and Impulse Response stress wave methods: advantages and limitations for the evaluation of highway pavement concrete overlays," *Conf. Nondestructive Evaluation of Bridges and Highways*, Proc. SPIE 2946, 88-96, Scottsdale AZ, December 1996.
6. Davis, A.G. and J. Kennedy, 1998, "Impulse Response testing to evaluate the degree of alkali-aggregate reaction in concrete drilled-shaft foundations for electricity transmission towers", *Conf. Nondestructive Evaluation of Utilities and Pipelines II,* Proc. SPIE 3398, 178-185, San Antonio, TX, April 1998.
7. Sansalone, M. and W.B. Streett, 1997, "Impact-Echo: nondestructive evaluation of concrete and masonry," Bullbrier Press, Ithaca, NY.

APPENDIX B

STRESS WAVE NDT METHODS FOR CONCRETE DEEP FOUNDATIONS (APPLIED HERE TO THE PLUG/ROCK INTERFACE ASSESSMENT)

Since the 1960's, test methods based on stress wave propagation have been commercially available for the nondestructive testing of concrete deep foundations. Developed at first in France and Holland, they are now routinely specified as quality control tools for new pile and drilled pier construction in western Europe, northern Africa and parts of eastern Asia. Their present use on the North American continent is less widespread. Recent improvements in electronic hardware and portable computers have resulted in more reliable and faster testing systems, less subject to operator influence both in testing procedure and in the analysis of test results.

Two distinct groupings for deep foundation NDT methods are apparent:

- *Surface Reflection* techniques, and
- *Direct Transmission* through the concrete.

The Impulse Response method and its derivatives used at this site belong to the *Surface Reflection* family.

1. SONIC ECHO

The earliest of all NDT methods commercially available, this method is known variously as the Sonic Echo or Seismic Echo Test.

The Sonic Echo method uses a small impact delivered at the head of the pile shaft, and measures the time taken for the stress wave generated by the impact to travel down the shaft and to be reflected back to a transducer (either a geophone or an accelerometer) coupled to the pile head. The impact is typically from a small hammer with an electronic trigger. Both the moment of impact and the pile head vertical movement after impact are recorded by a digital data acquisition device, which records the data on a time base.

If the length of the pile shaft is known and the transmission time for the stress wave to return to the transducer is measured then its velocity can be calculated. Conversely, if the velocity is known, then the length can be deduced. Since the velocity of the stress wave is primarily a function of the dynamic elastic modulus and density of the concrete, the calculated velocity can provide information on concrete quality.

Where the stress wave has traveled the full length of the shaft, these calculations are based on the formula:

$$v_c = \frac{2l}{dt}$$

Where: v_c = Stress wave velocity in concrete
l = Shaft length
dt = Transit time of stress wave

Empirical data has shown that a typical range of values for v_c can be assumed where 3800 to 4000 m/s (12500 to 13200 ft/s) would be indicative of good quality concrete, with a crushing strength of the order of 30 - 35 N/mm^2 (4500 - 5250 psi). The actual correlation will vary according to aggregate type and mix, and these figures should be used only as a broad guide to concrete quality.

Where the length of the shaft is known, an early arrival of the reflected wave means that it has encountered an obstacle other than the toe of the shaft. This may be a break in the shaft, a significant change in shaft cross section, or the point at which the shaft is restrained by a stiffer soil layer. In certain cases, the polarity of the reflected wave (whether positive or negative with respect to the initial impact) can indicate whether the apparent defect is from an increase or decrease of support at the reflective point.

The energy imparted to the shaft by the impact is small, and the damping effect of the soils around the shaft will progressively dissipate that energy as the stress wave travels down and up the shaft. To increase information from the test, the signal response can be progressively amplified with time.

Depending on the stiffness of the lateral soils, a limiting length/diameter ratio exists beyond which all the wave energy is dissipated and no response is detected at the shaft head. In this situation, the only information that can be derived is that there are no significant defects in the upper portion of the shaft, since any defect closer to the head than the critical l/d ratio would reflect part of the wave. This limiting l/d ratio will vary according to the adjacent soils, with a typical value for medium stiff clays of 30/1.

2. IMPULSE RESPONSE (MOBILITY or TDR)

Originally developed as a steady state vibration test in France, where a controlled force was applied to the pile shaft head by a swept-frequency generator. Geophone velocity transducers recorded the vertical shaft response, and the input force from the vibrator was continuously monitored. The resulting response curve plotted the shaft Mobility (geophone velocity, v / vibrator force, F) against frequency, usually in the useful frequency range of 0 - 2000 Hz.

The evolution of data processing equipment over the last two decades has allowed the use of computers on site to transform the force from a hammer impact (similar to that used in the Sonic Echo method) into the frequency domain.

A blow on the shaft head by a small hammer equipped with a load cell generates a stress wave with a wide frequency content, which can vary from 0 - 1000 Hz for soft rubber-tipped hammers to 0 - 3000 Hz for metal-tipped hammers. The load cell measures the force input, and the vertical response of the shaft head is monitored by a geophone.

The force and velocity time-base signals are recorded by a digital acquisition device, and then processed by computer using the Fast Fourier Transform (FFT) algorithm to convert the data to the frequency domain. Velocity is then divided by force to provide the unit response, or transfer function, which is displayed as a graph of shaft Mobility against frequency.

This response curve consists of two major portions, which contain the following information:

- At low frequencies (<100 Hz), lack of inertia effects cause the pile/soil complex to behave as a spring, and this is shown as a linear increase in amplitude from zero with increasing frequency. The slope of this portion of the graph is known as the compliance, and the inverse of compliance is the dynamic stiffness. The dynamic stiffness is a property of the shaft/soil complex, and can therefore be used to assess a shaft population on a comparative basis, either to establish uniformity, or as an aid to selecting a representative shaft for full-scale load testing by either static or dynamic means.

- The higher frequency portion of the Mobility curve represents resonance of the shaft. The frequencies of these resonances are a function of the shaft length and the degree of shaft toe anchorage, and their relative amplitude is a function of the lateral soil damping. The mean amplitude of this resonating portion of the curve is a function of the impedance of the pile shaft, which depends in turn upon the shaft cross sectional area, the concrete density and the stress wave propagation velocity, v_c.

As with the sonic echo test, when the shaft length is known, a shorter length measurement will indicate the presence of an anomaly. The additional information available from the Mobility curve such as cross section and dynamic stiffness can help in differentiating between an increase or reduction in cross section, for example, even in relatively complex soils. The response curve also contains information on the phase of the reflected signals, shown as a shift of the peak frequencies along the frequency axis. The time-based Sonic Echo result gives signal phase as only positive or negative, with no graduation. The Mobility test makes it possible to quantify the phase shift caused by a change in support conditions, providing information on the quality of contact between the shaft and the lateral soil.

In common with the Sonic Echo test, a relatively small amount of energy is generated by the hammer impact, and soil damping effects limit the depth from which useful information may be obtained. However, even where no measurable shaft base response is present, the dynamic stiffness is still a parameter for comparative shaft assessment.

3. IMPEDANCE LOG

A recent approach to interpreting the responses from a combination of both Sonic Echo and Mobility surface reflection methods is the Impedance Log [Paquet, 1991; ACI Report 228.2R, 1998], where the information from the amplified time domain response of the Sonic Echo is combined with the characteristic impedance of the shaft measured with the Mobility test.

Even though the force applied to the head of the shaft by the Surface Reflection methods is transient, the wave generated by the blow is not. This wave contains signals from shaft changes as it proceeds downward, and these changes are reflected back to the shaft head. The reflectogram so obtained in the Sonic Echo test can not be quantified.

The amplitude of the reflected signal on the Reflectogram from a defect or from the pile base is a function of distance from the pile head, and of surrounding soil stiffness. In uniform soil conditions, this damping function is exponential, and the Reflectogram can be corrected using such a function to yield a uniform response over the total shaft length, as is frequently done in treatment of Sonic Echo test data.

The measured shaft impedance in the Mobility test can then be combined in the Reflectogram to give dimensions to the response amplitude. This final result is referred to as the Impedance Log of the pile shaft. This Impedance Log can be adjusted to eliminate varying soil reflections, and the profile so obtained is proportional to the pile cross sectional area.

The probe cables are withdrawn over an instrumented wheel that measures the cable length and thus probe depth. Continuous pulse measurements are made during withdrawal, at height increments ranging from 10 mm to 50 mm (0.4 inches to 2 inches), providing a series of hence

A summary of the test methods described above along with the advantages and disadvantages is noted in Table B.1.

TABLE B.1 INTEGRITY TEST METHODS

INTEGRITY TEST METHOD	SURFACE REFLECTION		
	SONIC ECHO	**IMPULSE RESPONSE (MOBILITY)**	**IMPEDANCE LOG**
PRINCIPLES & CONDITIONS	- Measurement of the propagation time of longitudinal stress waves. - The shaft head is struck with a hammer sending a compression wave down the shaft to the toe or any anomaly where it is reflected back to the surface. - Analysis is in the time domain.	- Measurement of the dynamic response of the shaft in the frequency domain - The shaft is struck with a hammer equipped with a load cell and the velocity or acceleration response of the shaft head is recorded. - Analysis is in the frequency domain.	Measurement of the response in both time and frequency. - The equipment and test procedure is as for the Mobility test. - Analysis is in both time and frequency domains.
ADVANTAGES	- No pre-placed tubes. - Portable equipment. - Rapid.	- No pre-placed tubes. - Stiffness measurement. - Portable equipment. - Rapid.	- As for Mobility test, plus: - Effective shape of shaft derived from analysis.
DISADVANTAGES	- Confuses necking and bulging. - Does not measure diameter. - Unable to determine defects in shafts > 100 ft or with l/d >30/1.	- Results interpretation is delicate. - Limitations on geometry of pile to be tested, as for sonic echo.	- Requires very good test data for accurate analysis. - Full analysis can not yet be completed on site at time of test.

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APPENDIX C

THE IMPACT-ECHO TEST

Like the Impulse Response test, the Impact-Echo (IE) test uses stress waves to detect flaws within concrete structures. However, the frequency range used is considerably higher in the IE test, since much shorter wavelengths are required to detect smaller anomalies. Surface displacements caused by reflecting stress waves can be viewed versus time as a displacement waveform. The amplitude spectrum of this waveform is computed by FFT, as for the Impulse Response. This spectrum has a periodic nature, which is a function of the depth to the reflective boundary (either the back of the element, or some anomaly such as a crack in the element under test). The depth of a concrete/air interface (internal void or external boundary) is determined by:

$$d = v_c / 2f \qquad (1)$$

d is the interface depth, v_c is the primary stress wave velocity and f is the frequency due to reflection of the P wave from the interface.

If the material beyond the reflective interface is acoustically stiffer than concrete (e.g. concrete/steel interface), then the following equation applies:

$$d = v_c / 4f \qquad (2)$$

The difference in the acoustic impedance of the two materials at an interface is important because it determines whether the presence of an interface will be detected by an I-E test. For example, a concrete/grout interface gives no reflection of the stress wave because the acoustic impedance of concrete and grout are nearly equal. In contrast, at a concrete/air interface, nearly all the energy is reflected, since the acoustic impedance of air is very much less than concrete.

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APPENDIX D

FIGURES

Figure D.1 Walker Mine Plug Stiffness (MN/mm)

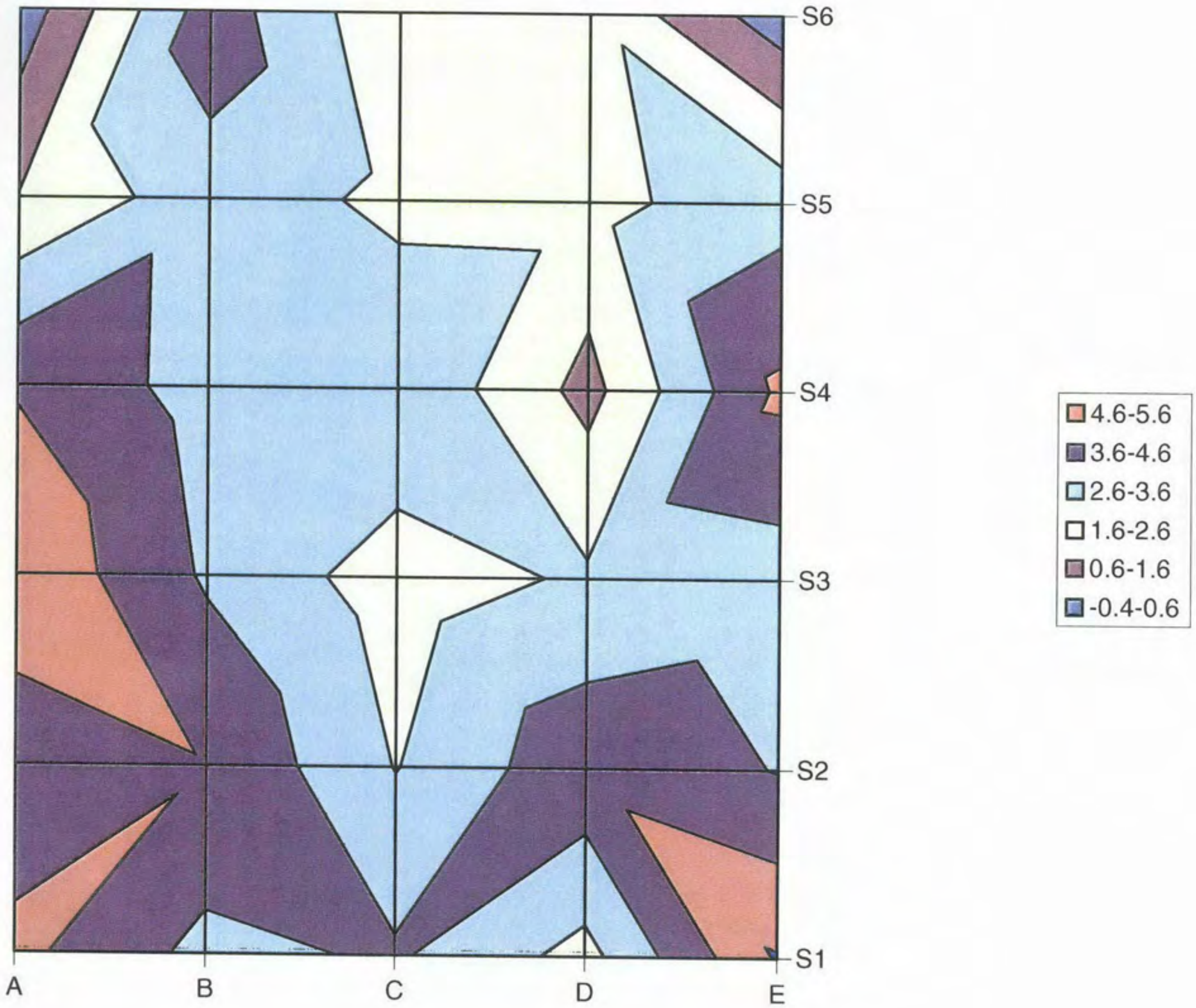
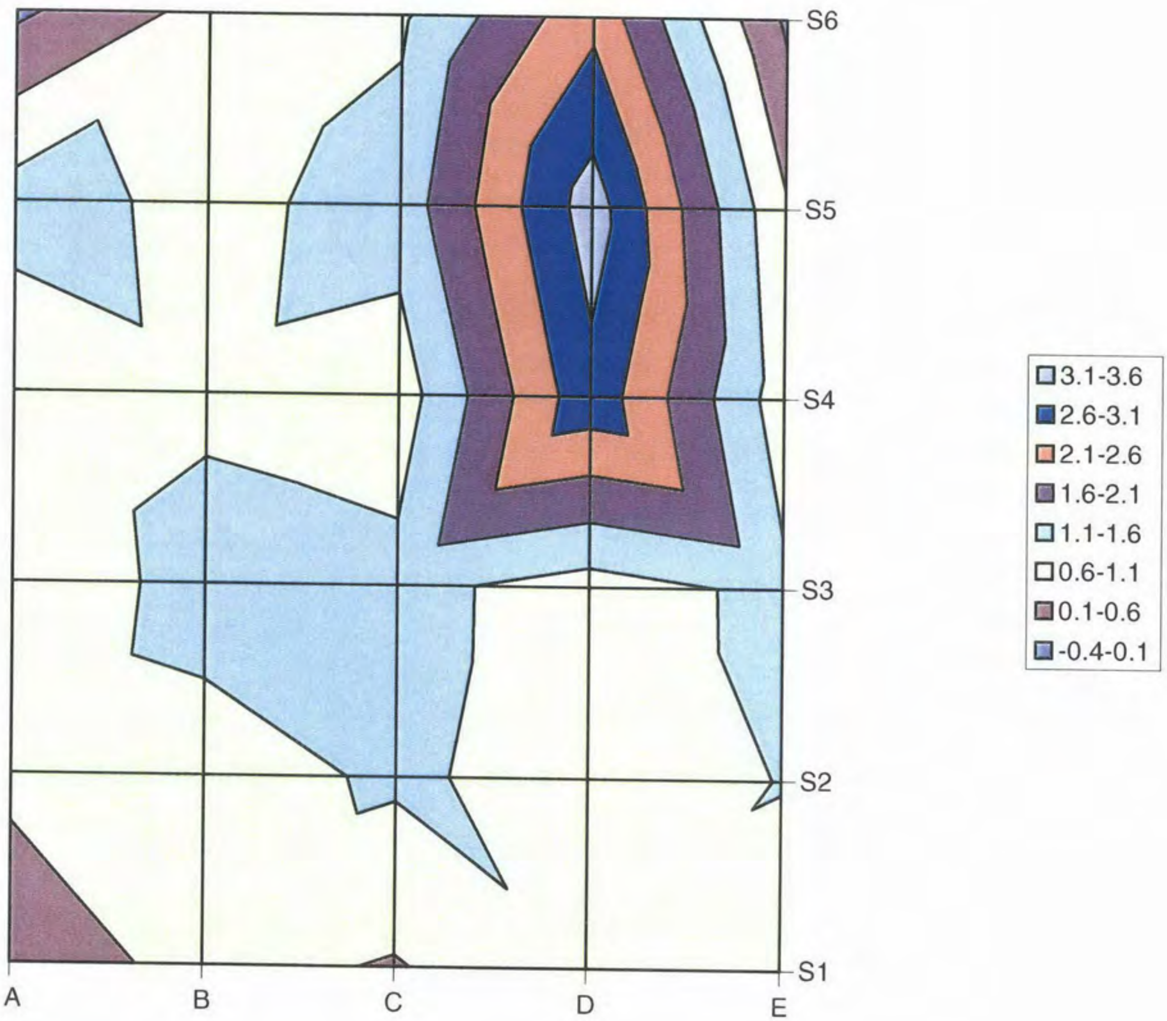


Figure D.2 Walker Mine Plug
Average Mobility



CTL

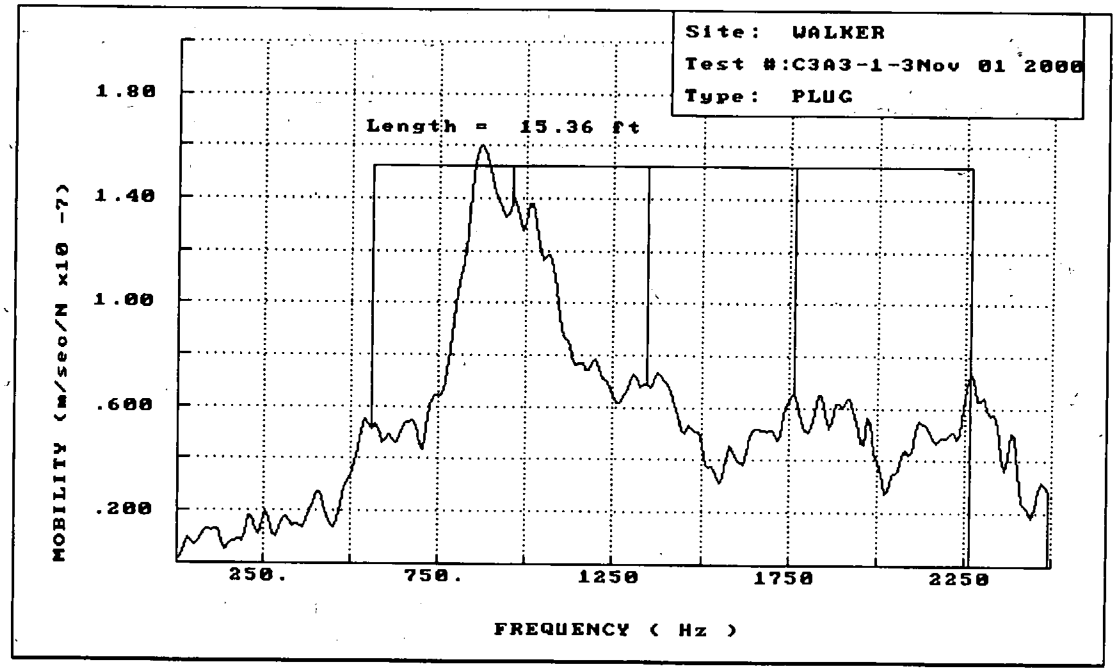


Figure D.3 IR test response – Plug Depth Measurement

CTL

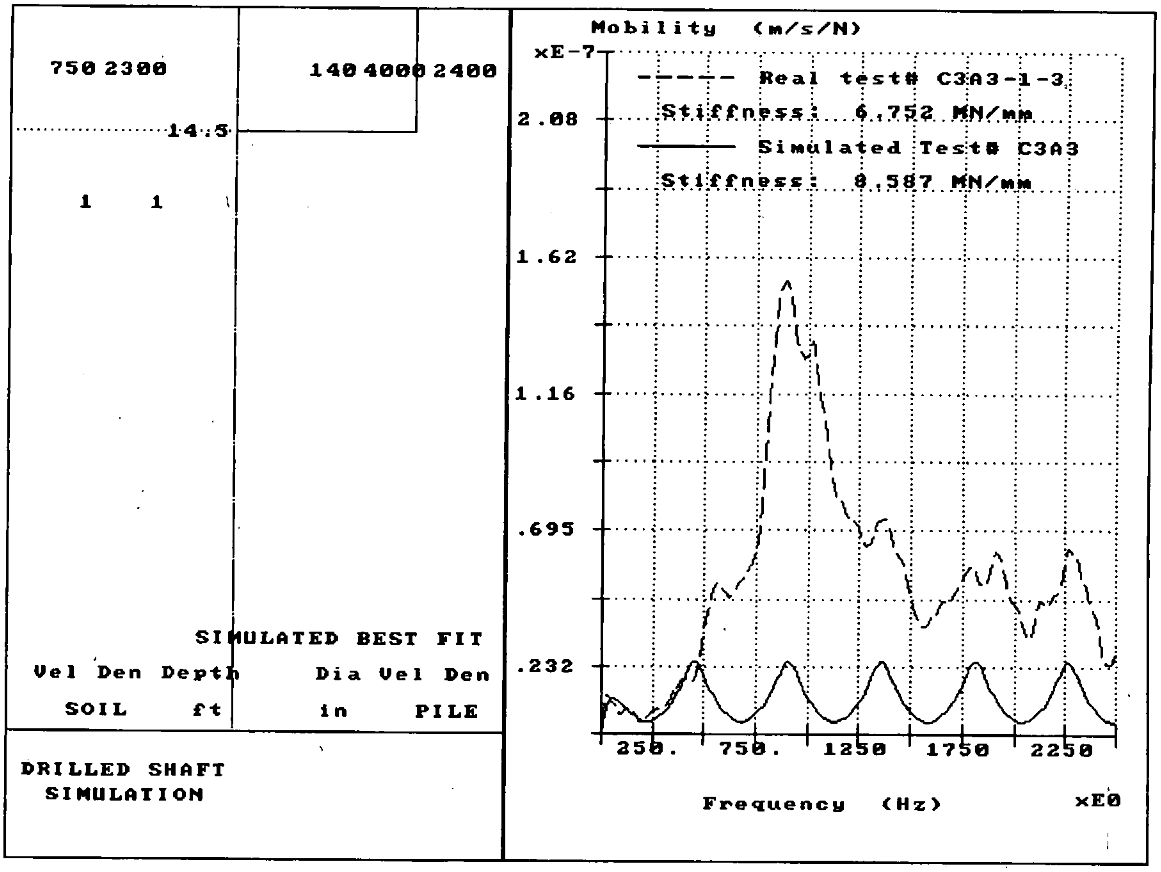
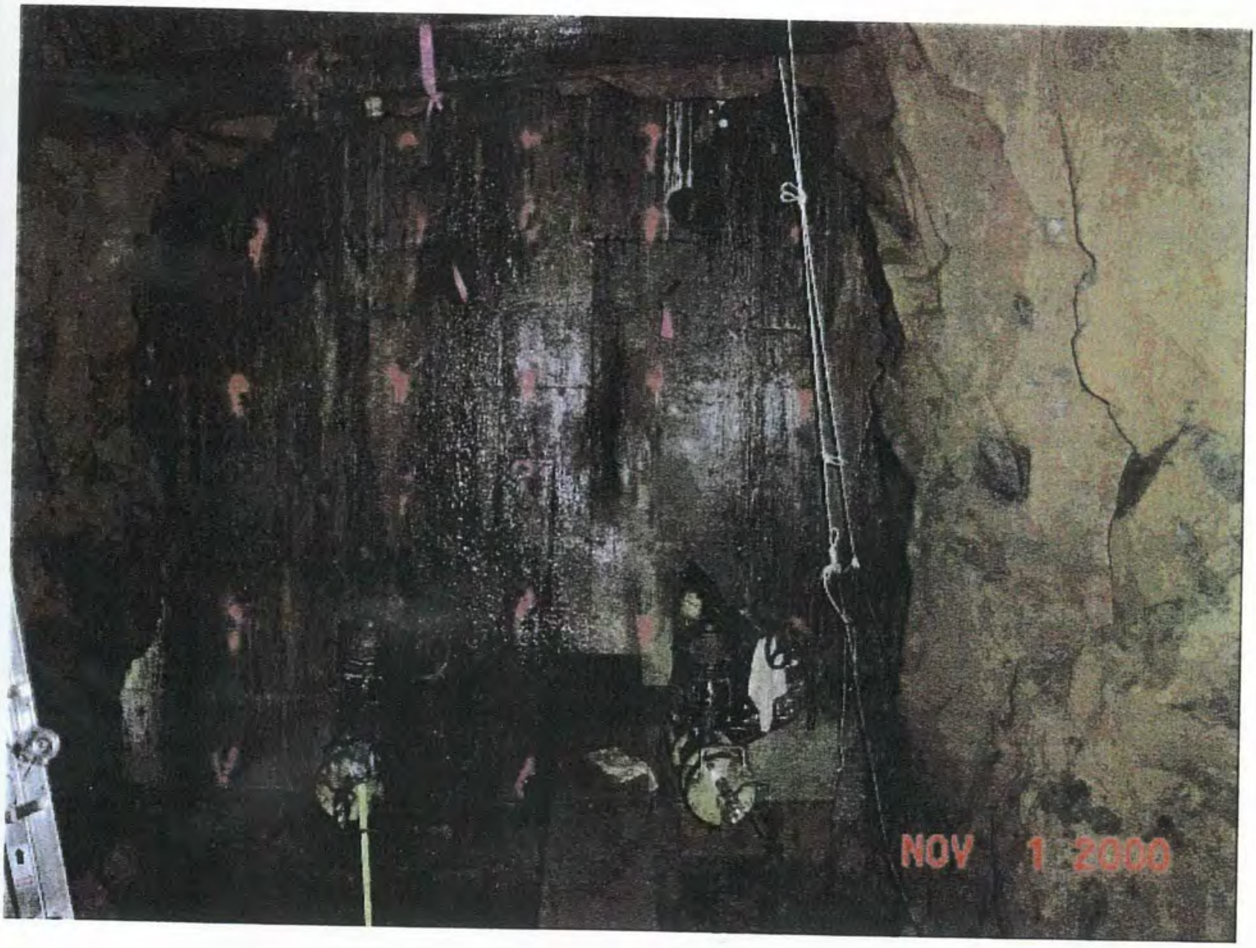


Figure D.4 Simulation Plot of IR Response

Plate 1: View of Plug Face



Appendix B.2

"Evaluation of Concrete Mine Seal Using Nondestructive Testing, Walker Mine, Portola, Plumas County, California, 2nd Visit"
CTL, October 31, 2001.

CTL
Construction Technology Laboratories, Inc.

www.ctlgroup.com
5420 Old Orchard Road
Skokie, Illinois 60077
800.522.2CTL (2285)

October 31, 2001

Mr. Alberto Pujol
GEI Consultants, Inc.
2201 Broadway Suite 321
Oakland, CA 94612

**RE: EVALUATION OF CONCRETE MINE SEAL USING NONDESTRUCTIVE TESTING
WALKER MINE, PORTOLA, PLUMAS COUNTY, CALIFORNIA
CALIFORNIA REGIONAL WATER QUALITY CONTROL BOARD
CENTRAL VALLEY REGION RESD-PSB-AD 2000-11
CTL PROJECT NO. 320098 – 2nd VISIT**

Dear Mr. Pujol:

GEI Consultants, Inc. (GEI) instructed Construction Technology Laboratories, Inc. (CTL) to carry out an evaluation of a mass concrete mine seal plug, located in the Walker Mine, Portola, Plumas County, California. The first phase of this evaluation comprised nondestructive testing from the downstream face of the plug, and was completed in November 2000 (see CTL Report No. 320098 dated November 16, 2000). The second phase reported here comprised nondestructive testing in core holes drilled in the plug to intersect the concrete/rock interface, as well as laboratory tests at CTL Headquarters on both concrete and rock core samples.

Nondestructive testing was performed on site on September 10 and 11, 2001. The nondestructive Cross-hole Sonic Logging (CSL) testing technique was used to evaluate the plug. This test method is fully described in Appendix A to this report.

CONCRETE SEAL PLUG DESCRIPTION

The mass concrete plug fills the original rock tunnel, and is approximately 9 ft wide by 12 ft high at the visible face, and approximately 15 ft deep. The granodioritic rock surrounding the plug appears to be hard and compact at the plug face. Water seepage is visible at the top face, and it is not clear whether this seepage is coming from the rock or from the concrete/rock interface, or both.

Plate 1 in the Appendix B shows the plug. Two drain outlet valves are visible on the plug face: at approximately 3 ft from the left and right edges respectively and approximately 3 ft from the base of the plug. In addition, a sealed pipe is apparent in the top right corner of Plate 1, which is thought to be the entry point of the original concrete tremie. The sheen on the concrete face is from the water seepage. Several 1-inch diameter grout pipes were also noted at the top right corner of the plug.

TEST PROGRAM

In order to evaluate the quality and the integrity of the concrete in the downstream 8-ft portion of the plug and to examine the interface between the plug and the rock where possible, seven 4-inch diameter core holes were drilled normal to the face of the plug. Figure 1 in Appendix B shows the location of the seven core holes drilled to between 8 and 10 ft into the plug face and subsequently used for CSL testing. Holes 1 to 6 were drilled at a slight inclination to the normal, so that the core hole would intercept the plug/rock interface at approximately 6 to 8 ft upstream of the plug face. Core 7 in the center of the plug was drilled with a slightly downward inclination, in order that the hole would retain water.

Continuous core samples were obtained by Jensen Drilling Company under subcontract to GEI Consultants and were logged by Jose Cercone of Washington Group International, project geologist. Alberto Pujol of GEI, in consultation with Dr. Allen Davis of CTL and Gary Mass, independent concrete consultant, selected samples for testing at CTL laboratories. When coring was completed, CSL tests were performed between core holes. The test comprises sending ultrasound pulses from a transmitter in one hole to a receiver in another hole, and measuring the pulse travel time between the two holes. A profile of transmitted signals with depth in the hole can be established, thereby obtaining a picture of the quality and uniformity of the concrete between the two holes.

For the CSL test to be successfully performed, it is essential that the core holes be full of water to ensure coupling of the transceivers to the surrounding concrete. Holes 1, 5, 6 and 7 were inclined slightly downward, so making it possible to retain water in these holes during testing. However, holes 2, 3, and 4 in the upper part of the plug were inclined upward in order to intercept the concrete/rock interface, and it was necessary to devise a valve system that would hold water in the hole while at the same time allowing the CSL probe and cable to be placed and operated in the hole. The system developed at the plug face to achieve the required seal was successful, and valid CSL test profiles were obtained for these upper holes.

TEST RESULTS

1. Cross-Hole Sonic Logging

Dr. A.G. Davis of CTL and Mr. Bernie Hertlein of STS Consultants, Ltd. (STS) performed the CSL testing. Figure 1, Appendix B shows the core locations on the plug face. Twelve CSL profiles were obtained, and are listed with the distances between each profile in Table 1.

The distance between core holes increases as the core hole advances into the plug from the face. This inter-hole distance cannot be calculated, but has an effect on the CSL profile obtained between cores.

The pulse travel time increase with increasing path length. This is shown clearly in the CSL plots for each of the 12 profiles given in Appendix B to this report.

Table 1. CSL Profile Spacing

CSL Profile	Distance (Inches)
1-2	58
1-6	50
1-7	60
2-3	46
2-7	46
3-4	39
3-7	47
4-5	58
4-7	44
5-6	40
5-7	47
6-7	44

Ten of the 12 profiles showed very clear CSL arrival signals over the total profile lengths. Poorer coupling of the probes in the top core holes caused by difficulty in maintaining these holes full of water during testing resulted in the need to greatly amplify the signal traces. This was the case for Profiles 2-3 and 3-4. Measurements of equivalent pulse velocities near the plug face gave between 12,000 and 12,500 ft/s. All test profiles show continuous concrete between test holes, with no breaks in arrival signals at any point.

In the cases where the core holes penetrated the rock, most of the pulse travel path at the end of the holes would be in concrete. However, if poor concrete consolidation or very fractured rock were to exist at the rock/concrete interface, a break in signal arrival time would be present in the CSL traces. This was not seen in any of the profiles.

Table 2. Core Test Results

Core Sample	Compression Strength (psi)	Core Unit Weight (lb/cu.ft)	UPV (ft/s)
BH-3: 0.0-0.9 ft (Concrete)	6290	137	14,269
BH-3: 8.8-9.5 ft (Concrete)	6110	146	13,382
BH-6: 0.0-1.0 ft (Concrete)	5800	139	13,211
BH-6: 5.3-5.7 ft (Concrete)	6750	137	14,889
BH-7: 5.0-6.1 ft (Concrete)	5780	142	13,667
BH-1: 9.5-10.0 ft (Rock)	16,240	166	19,118
BH-2 5.3-6.0 ft (Rock)	22,020	166	18,251

2. Laboratory Compression Tests on Cores

CTL performed compression tests on five concrete core samples and two rock core samples selected by Alberto Pujol of GEI from the seven full-depth cores taken in the plug. Core strengths and unit weights are summarized in Table 2 above. The full test results are presented in Appendix C to this report.

3. Laboratory Ultrasonic Pulse Velocity Tests on Cores

CTL performed ultrasonic pulse velocity (UPV) tests on the five concrete and two rock cores listed in Table 2 above, according to the methodology described in ASTM Standard C-597. The measured pulse velocities are given in Table 2. Figure 2, Appendix B shows the relationship between core compressive strength and core pulse velocity derived for the five concrete samples.

4. Petrographic Analysis

CTL performed petrographic studies on 3 core samples, and the full petrographic report is presented in Appendix D to this Report.

DISCUSSION OF TEST RESULTS

1. Concrete Quality

The concrete core compression strengths ranged between 5780 and 6750 psi, with four of the five tests between 5780 and 6290 psi. Core BH-6: 5.3-5.7 ft was a smaller core, taken in an area with obviously softer concrete, possibly due to segregation (see core photos). This is described in the petrographic examination section, Appendix D. The compressive strength was higher than expected, and does not reflect the concrete quality immediately below the tested core. The UPV results on the cores showed a regular increase with strength, and were in the range expected for well-consolidated concrete of this type.

The CSL tests showed very sound and integral concrete throughout the plug, with CSL velocities between 12,000 and 12,500 ft/s where they could be calculated at the plug face. No anomalies were observed in any of the CSL test results.

2. Rock Quality

The two cores tested gave very high compressive strengths and ultrasonic pulse velocities, to be expected for sound granitic rock. No signs of fracture or weathering were observed in the cores submitted for testing.

3. Plug/Rock Interface

The CSL tests showed regular transition between concrete and rock at the plug interface when intercepted by the test core holes. No loss of signal was observed at this interface in any of the CSL test profiles.

We sincerely appreciate the opportunity to work with GEI Consultants, Inc. on this project. If you have any questions, please call us at 1-800-522-2CTL.

Respectfully:

CONSTRUCTION TECHNOLOGY LABORATORIES, INC.

Malcolm K. Lim, P.E.
Project Manager

Steven H. Gebler, P.E.
Senior Principal Engineer

Attachments

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APPENDIX A

STRESS WAVE NDT METHODS FOR CONCRETE DEEP FOUNDATIONS (APPLIED HERE TO THE CONCRETE PLUG AND PLUG/ROCK INTERFACE ASSESSMENT)

Since the 1960's, test methods based on stress wave propagation have been commercially available for the nondestructive testing of concrete deep foundations. Developed at first in France and Holland, they are now routinely specified as quality control tools for new pile and drilled pier construction in Western Europe, northern Africa and parts of eastern Asia. Their present use on the North American continent is less widespread. Recent improvements in electronic hardware and portable computers have resulted in more reliable and faster testing systems, less subject to operator influence both in testing procedure and in the analysis of test results.

Two distinct groupings for deep foundation NDT methods are apparent:

- *Surface Reflection* techniques, and
- *Direct Transmission* through the concrete.

The Cross-hole Sonic Logging (CSL) method used during this visit to this site belongs to the *Direct Transmission* family. The Impulse Response and Impact-Echo methods used at the plug face during the first site visit belong to the *Surface Reflection* family.

THE SONIC LOGGING METHOD

The Sonic Logging method is designed for use on deep foundation shafts, mass concrete foundations such as slurry trench walls, dams and machinery bases. The method is described in the American Concrete Institute Report ACI 228.2R-98, "*Nondestructive Test Methods for Evaluation of Concrete in Structures*" (Reference 8).

The method uses the core holes in the concrete element to be tested, or pre-placed tubes. A transmitter probe placed at the bottom of one hole emits an ultrasonic pulse that is detected by a receiver probe at the bottom of a second hole. A recording unit measures the time taken for the ultrasonic pulse to pass through the concrete between the probes. The probes are sealed units, and the holes are filled with water to provide coupling between the probes and the concrete.
The probe cables are withdrawn over an instrumented wheel that measures the cable length and thus probe depth. Continuous pulse measurements are made during withdrawal, at increments ranging from 10 mm to 50 mm (0.4 inches to 2 inches), providing a series of measurements that can be printed out to provide a profile of the material between the holes.

The ultrasonic pulse velocity (UPV) is a function of the density and dynamic elastic modulus of the concrete. If the signal path length is known and the transit time is recorded, the apparent UPV can be calculated to provide a guide to the quality of the concrete. A reduction in modulus or density will result in a lower UPV. If the path length is not known, but the tubes are reasonably parallel, the continuous measurement profile will clearly show any sudden changes in transit time caused by a lower pulse velocity due to low modulus or poor quality material, such as contaminated concrete or inclusions. Voids will have a similar effect, by forcing the pulse to detour around them, thus increasing the path length and the transit time. By varying the geometric arrangement of the probes, the method can resolve the vertical and horizontal extent of such defects, and locate fine cracks or discontinuities.

The major advantage is that the method has no depth limitation, unlike surface reflection methods.

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3. Davis, A.G. and Robertson, S.A. (1975). "Economic pile testing." Ground Engng., 8(3), 40-43.
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APPENDIX B

FIGURES

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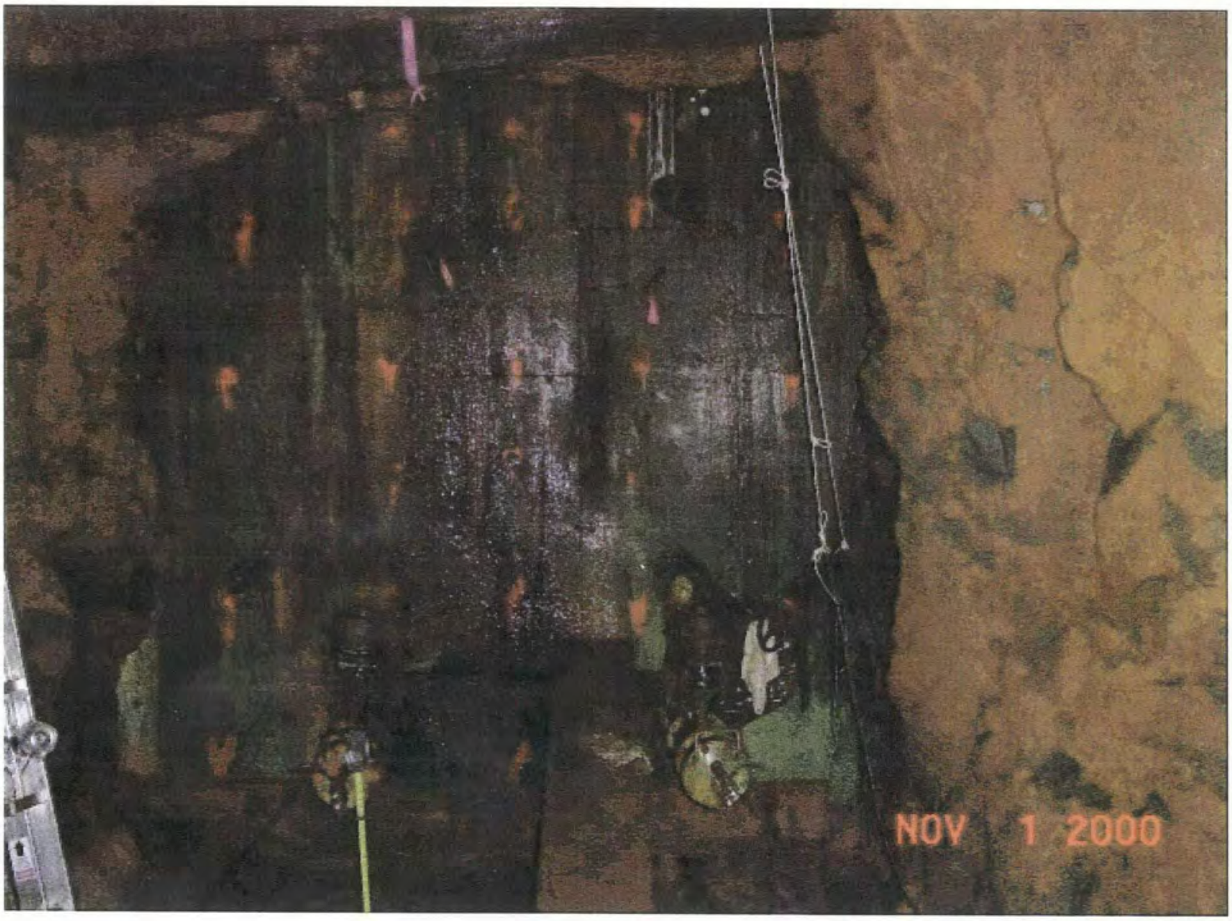
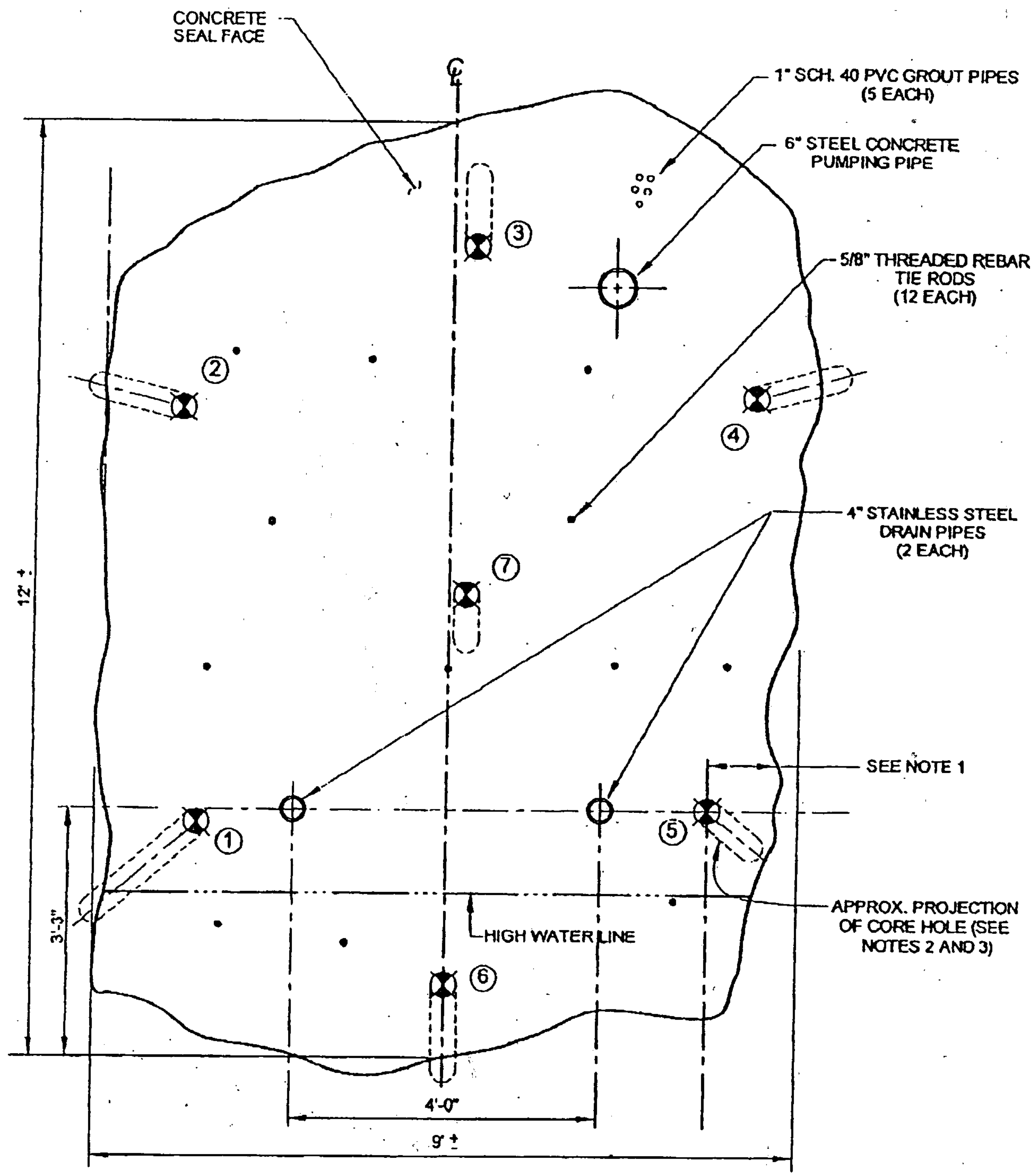
Plate 1: View of Plug Face





NOTES

1. Core hole entry point is about 10 to 19 inches from the rock face. Hole diameter is 4 inches.
2. Hole orientation projected on a vertical plane passing through the entry point:
   - Holes 1, 5, 6 and 7 are oriented at angles of 5, 7, 17 and 7 degrees downward, respectively.
   - Holes 2, 3 and 4 are oriented at angles of 6, 7 and 5 degrees upward, respectively.
3. Hole orientation projected on a horizontal plane passing through the entry point:
   - Holes 1 and 2 are oriented at angles of 8 and 6 degrees toward the left (toward the rock sidewall), respectively.
   - Holes 4 and 5 are oriented at an angle of 5 degrees toward the right (toward the rock sidewall).
   - Holes 3, 6 and 7 are parallel to the axis of the seal.

LEGEND

⑥ APPROX. CORE HOLE LOCATION AND NUMBER

0 1 2
SCALE, FEET

CTL
Construction Technology Laboratories, Inc.

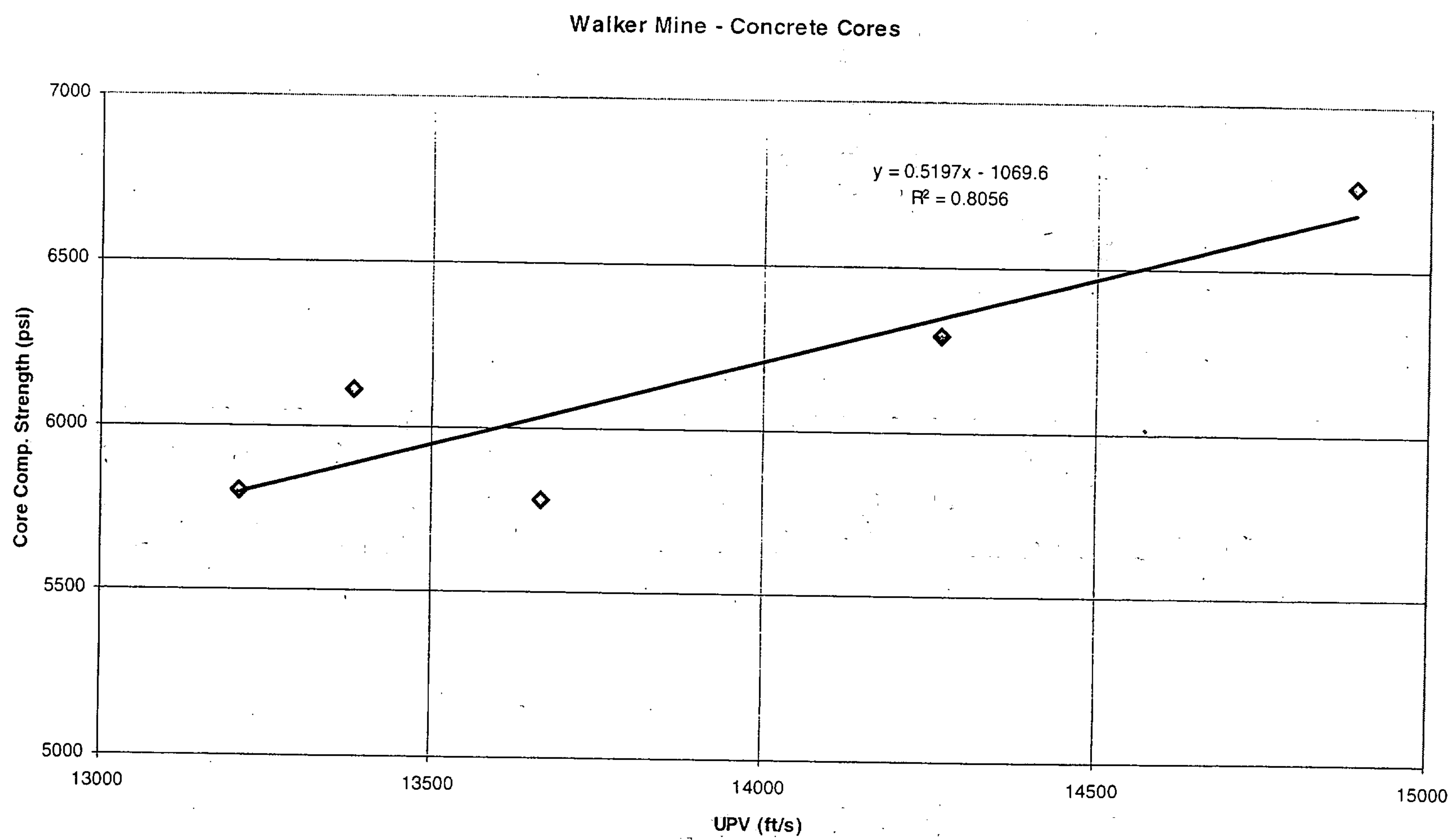
Approximate Borehole Location and Orientation

Regional Water Quality Control Board
Walker Mine
Portola, California

Date: 30OCT01

CTL Project No.: 320098

Figure No.: 1



Construction Technology Laboratories, Inc.	**Core Compressive Strength v. Ultrasonic Pulse Velocity**		
	Alberto Pujol GEI Consultants, Inc. Walker Mine, Portola, Plumas County, California	Date: 10/25/01 CTL Project No: 320098	**Fig. 2**

STS CONSULTANTS

CSL Profile

Date: 09/09/01

Test Site:

Walker Mine

Structure: Plug

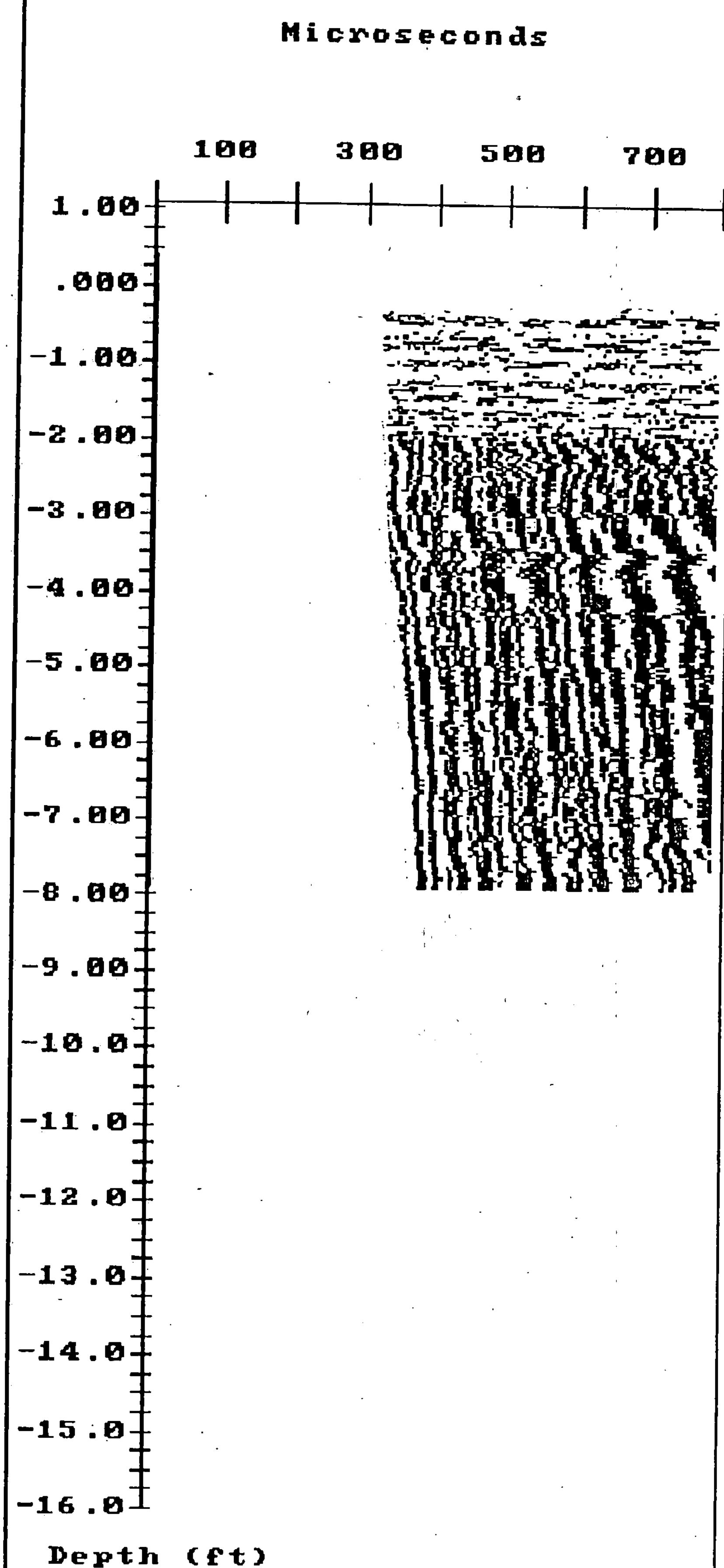
Profile: 4-7

0 time offset

Pulses: 251

Test Operator:

BHH



STS CONSULTANTS

CSL Profile

Date: 09/09/01

Test Site:

Walker Mine

Structure: Plug

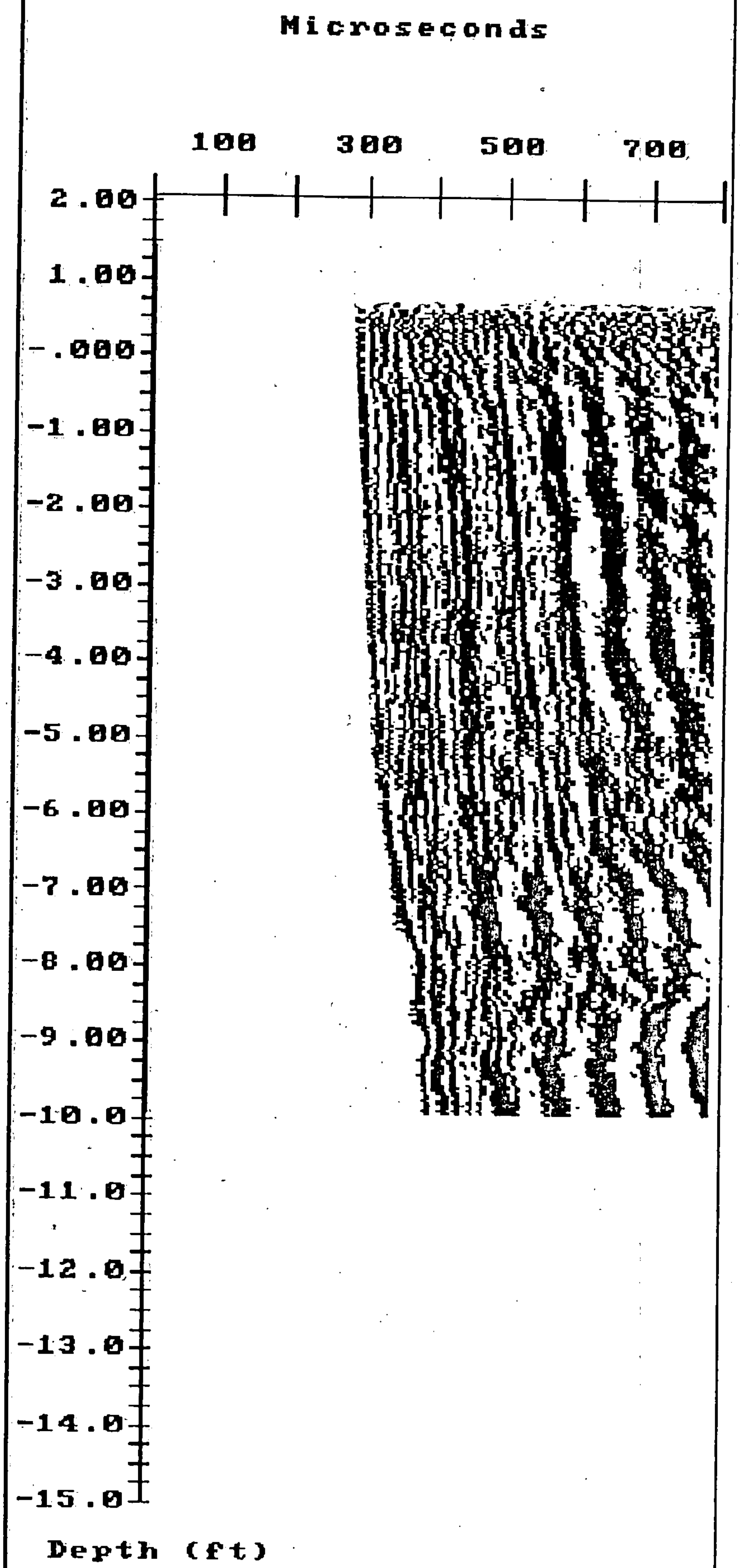
Profile: 5-6

0 time offset

Pulses: 344

Test Operator:

BHH



STS CONSULTANTS

CSL Profile

Date: 09/09/01

Test Site:

Walker Mine

Structure: Plug

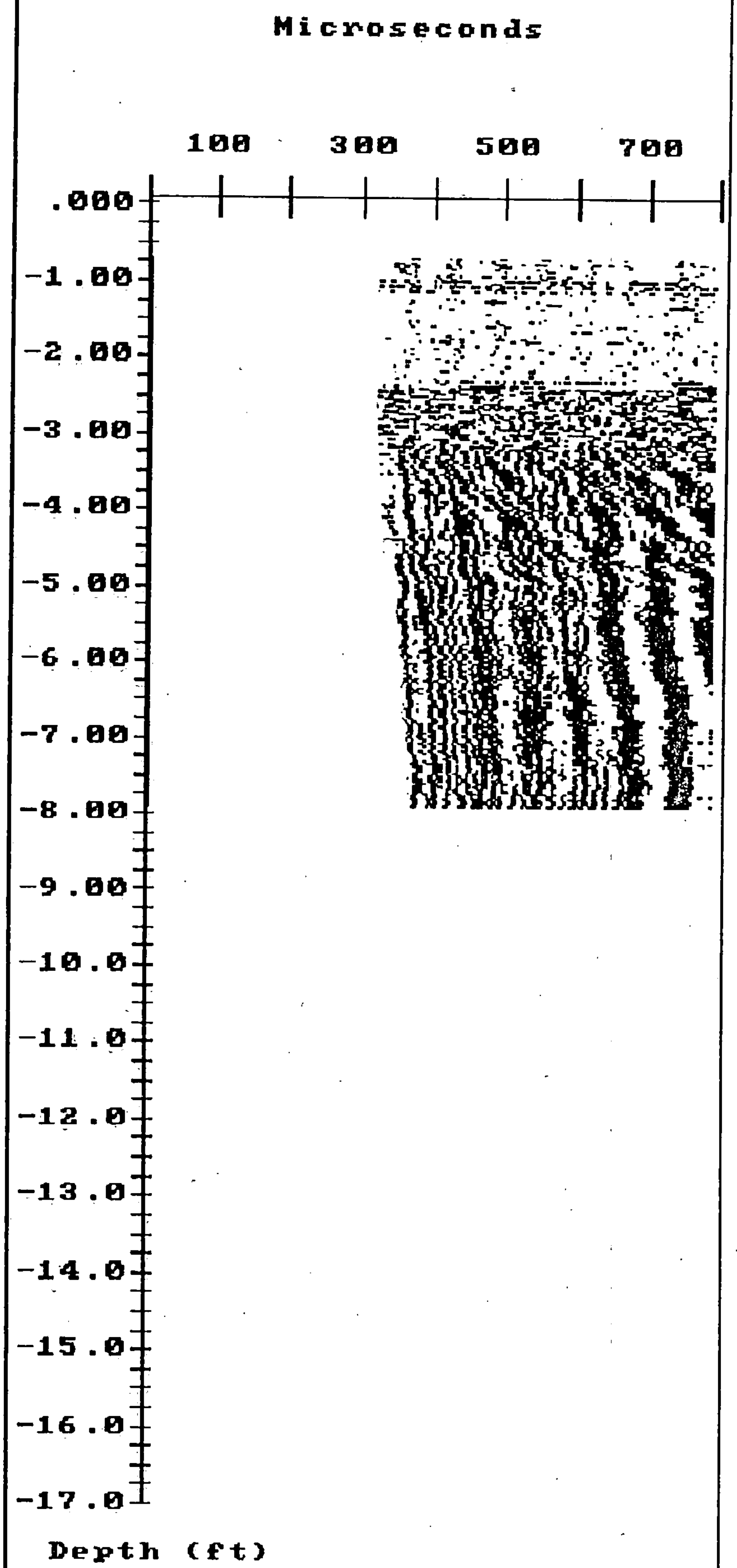
Profile: 5-7

0 time offset

Pulses: 210

Test Operator:

BHH



STS CONSULTANTS

CSL Profile

Date: 09/09/01

Test Site:

Walker Mine

Structure: Plug

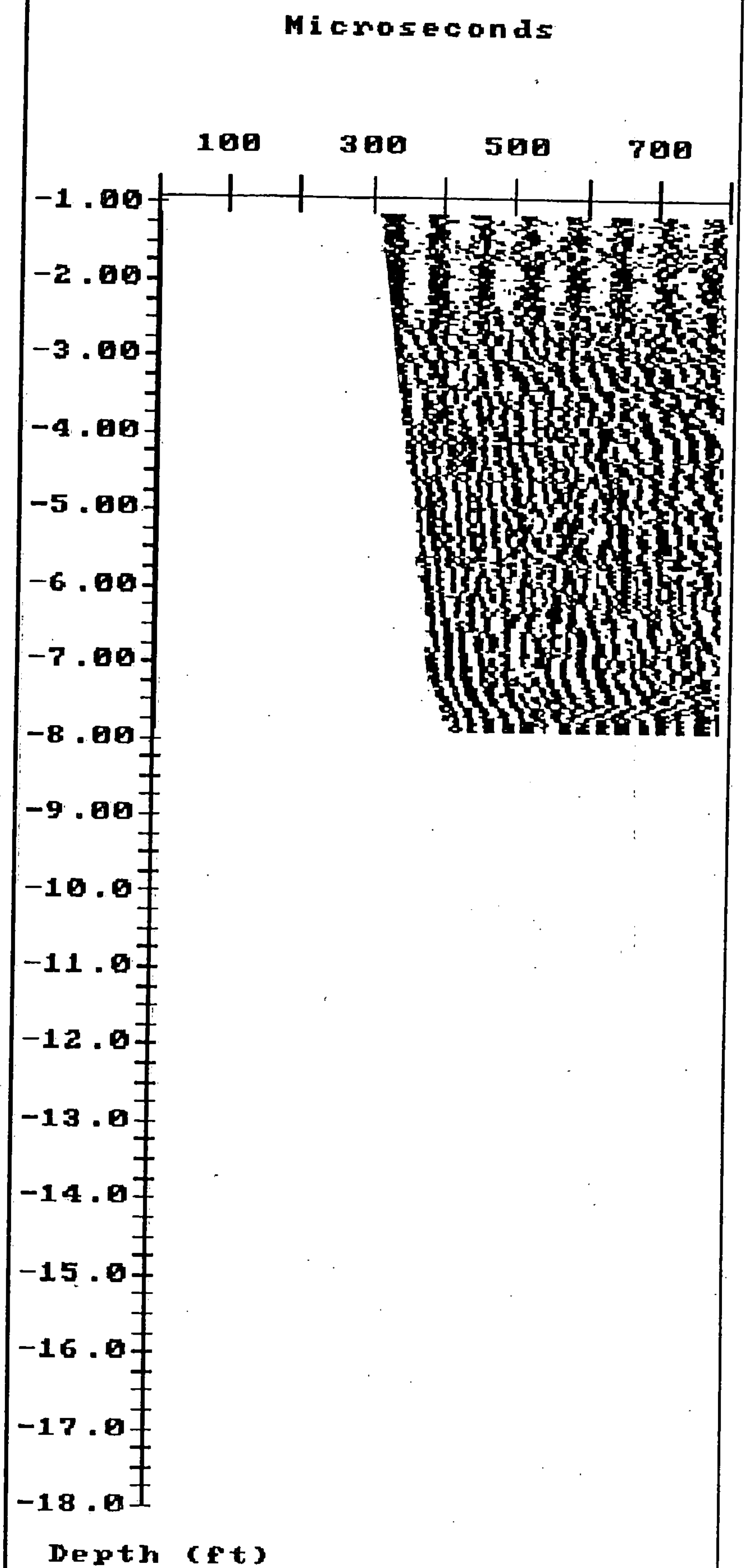
Profile: 6-7

0 time offset

Pulses: 204

Test Operator:

BHH



STS CONSULTANTS

CSL Profile

Date: 09/09/01

Test Site:

Walker Mine

Structure: Plug

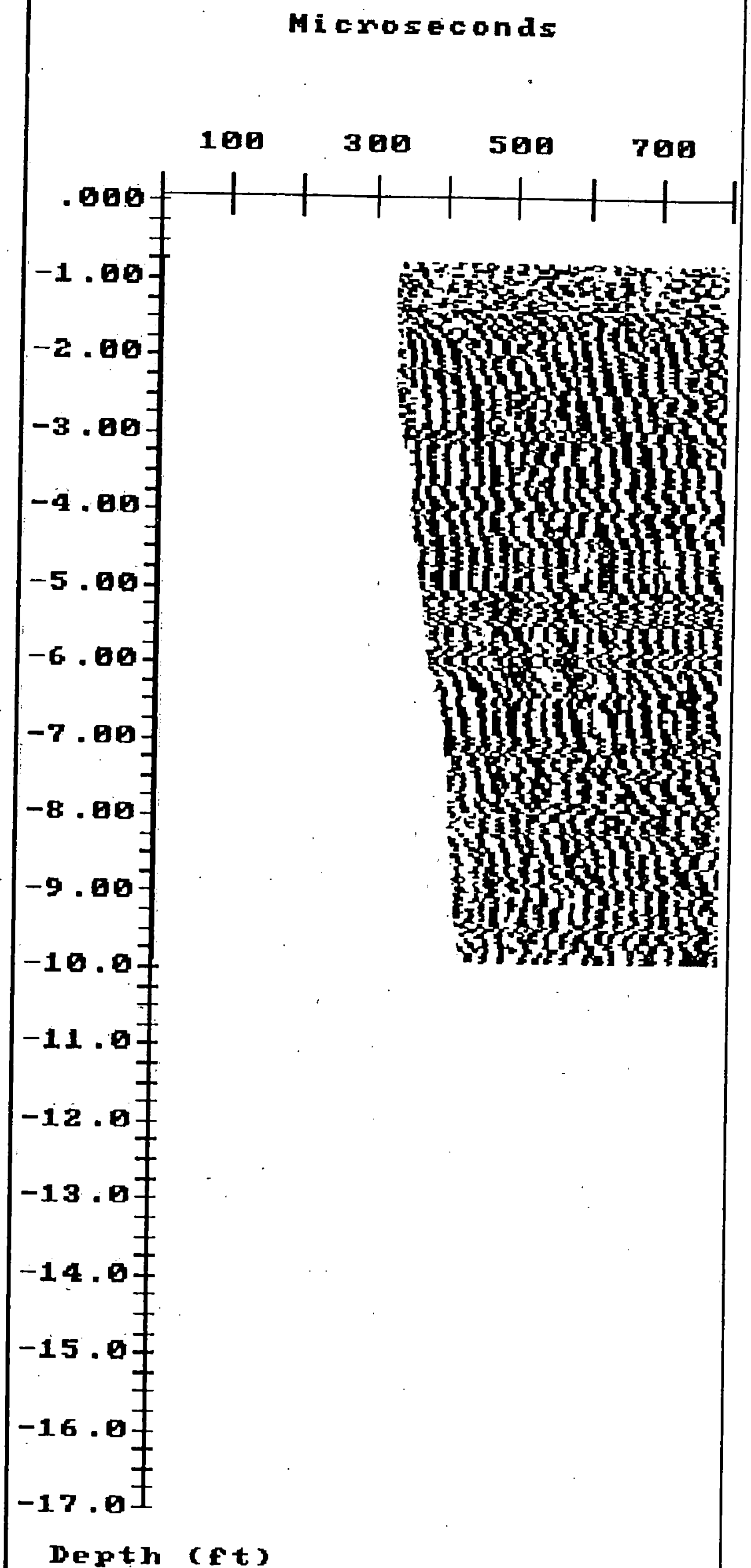
Profile: 1-6r

0 time offset

Pulses: 277

Test Operator:

BHH



STS CONSULTANTS

CSL Profile

Date: 09/09/01

Test Site:

Walker Mine

Structure: Plug

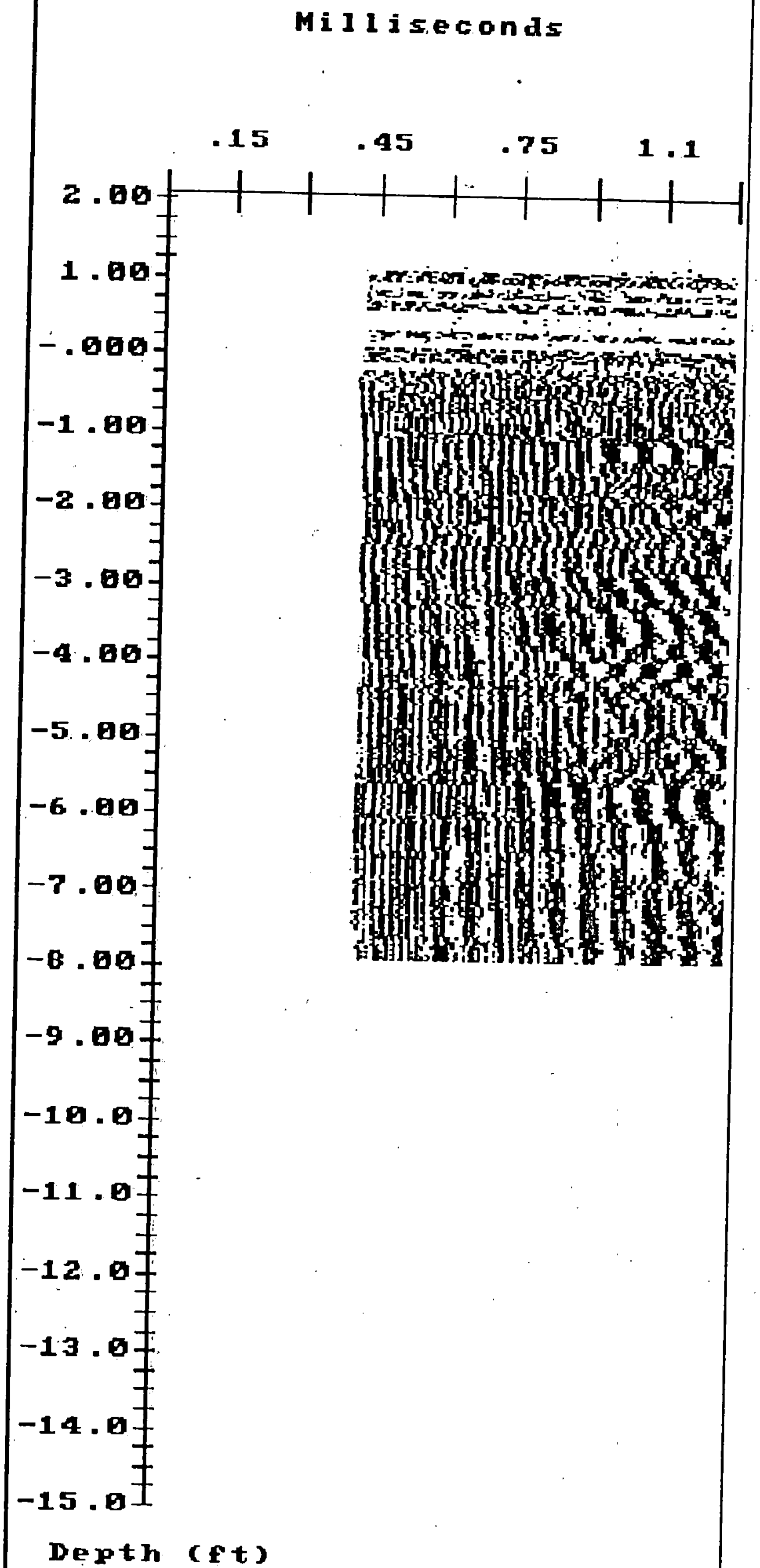
Profile: 1-7

0 time offset

Pulses: 276

Test Operator:

BHH



STS CONSULTANTS

CSL Profile

Date: 09/10/01

Test Site:

Walker Mine

Structure: Plug

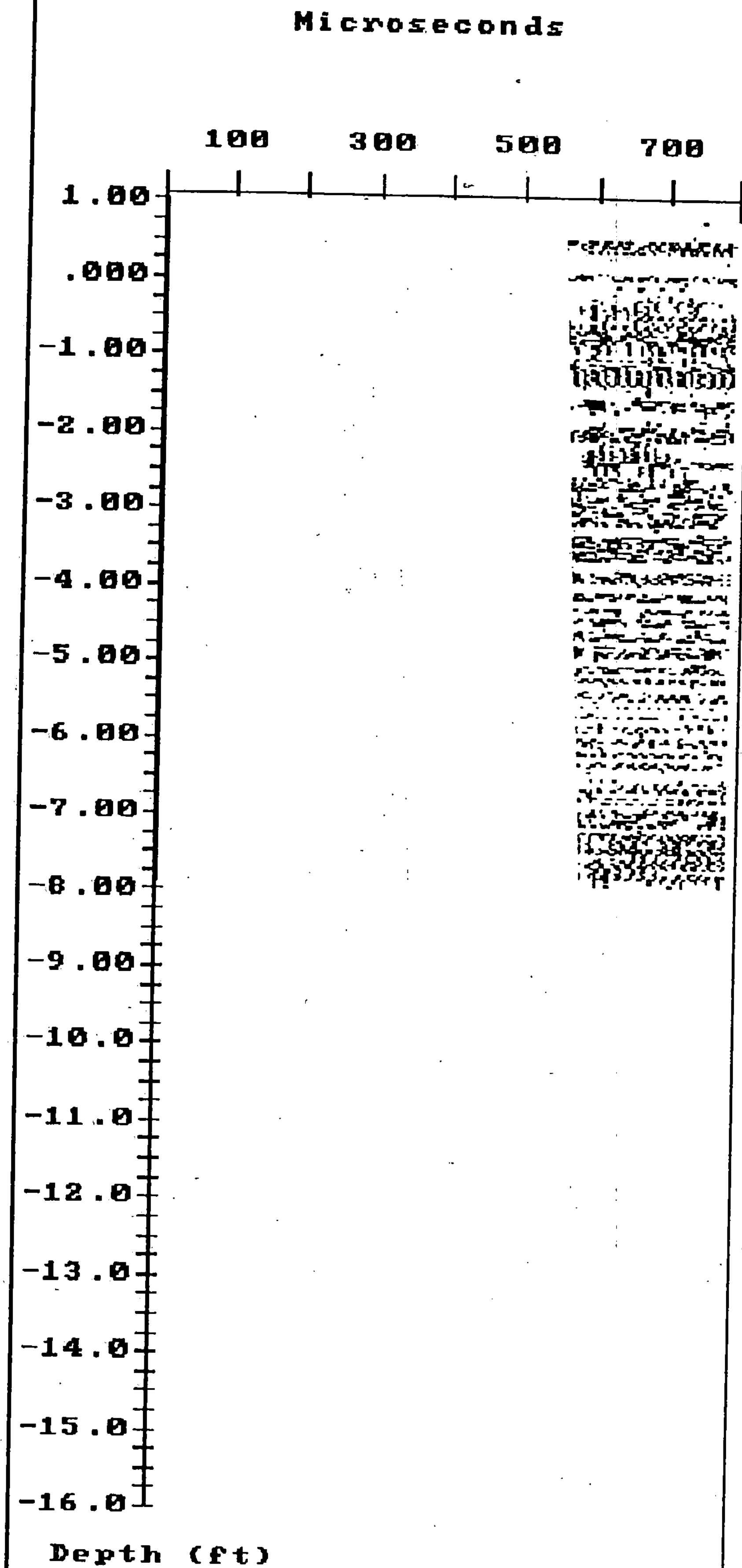
Profile: 2-3r

0 time offset

Pulses: 270

Test Operator:

BHH



STS CONSULTANTS

CSL Profile

Date: 09/10/01

Test Site:

Walker Mine

Structure: Plug

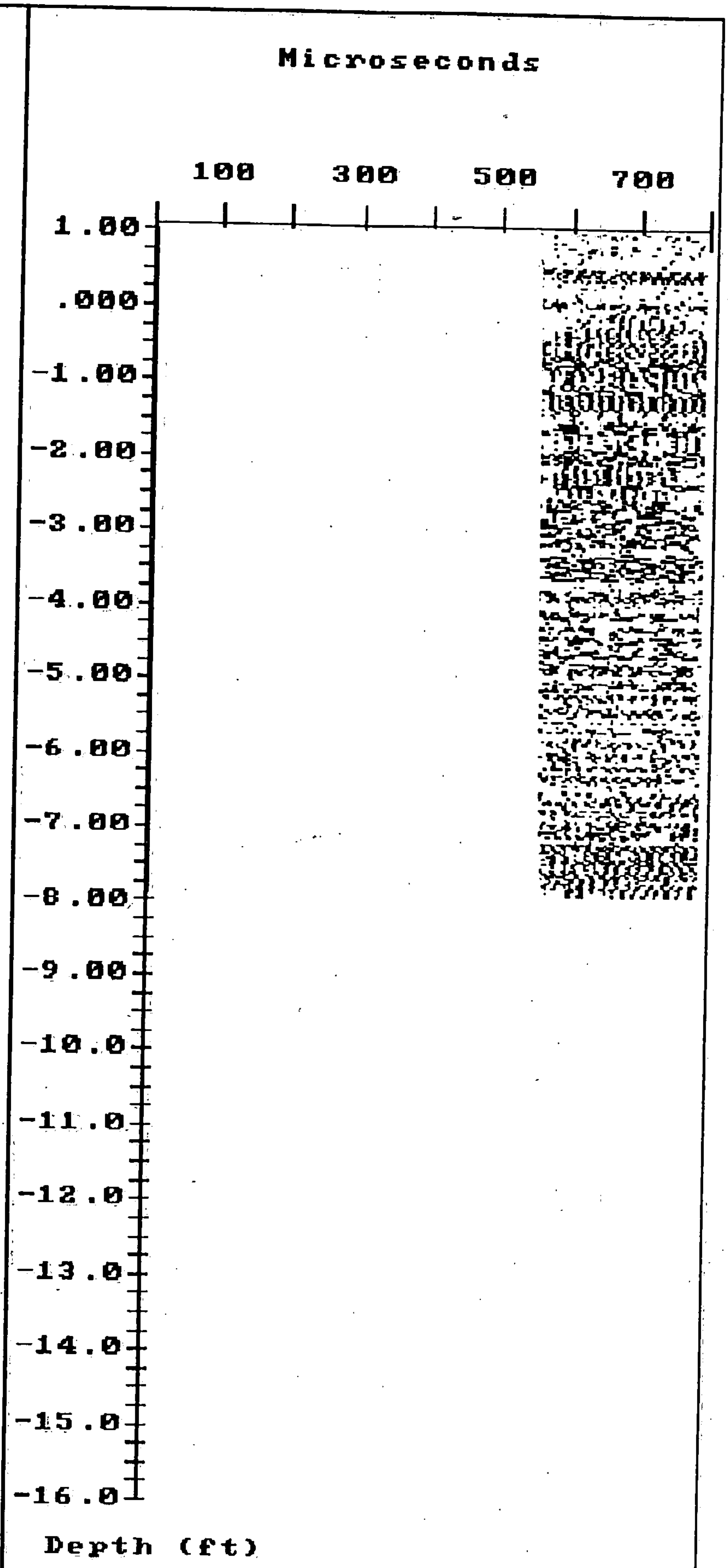
Profile: 2-3r

0 time offset

Pulses: 270

Test Operator:

BHH



STS CONSULTANTS

CSL Profile

Date: 09/09/01

Test Site:

Walker Mine

Structure: Plug

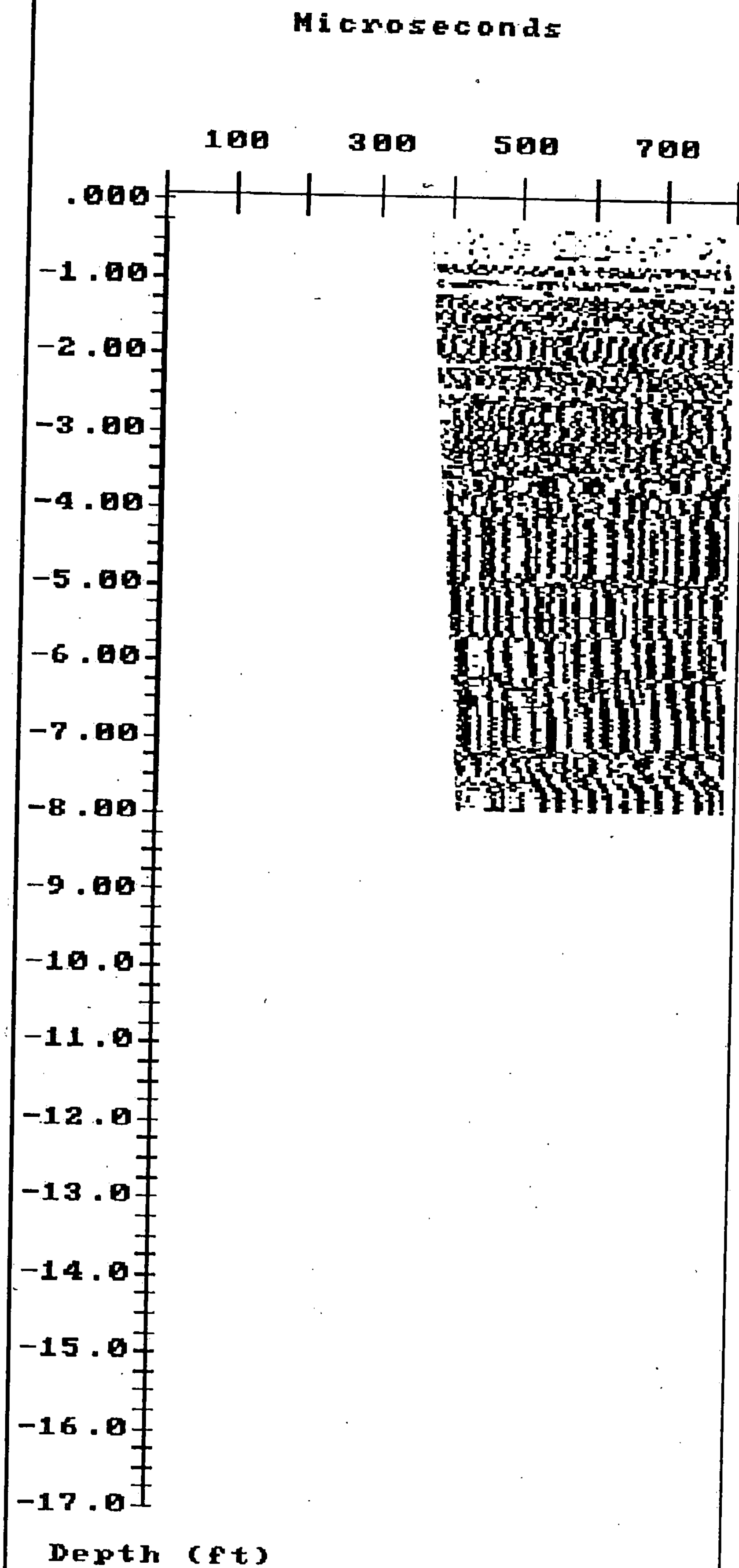
Profile: 2-7

0 time offset

Pulses: 230

Test Operator:

BHH



STS CONSULTANTS

CSL Profile

Date: 09/09/01

Test Site:

Walker Mine

Structure: Plug

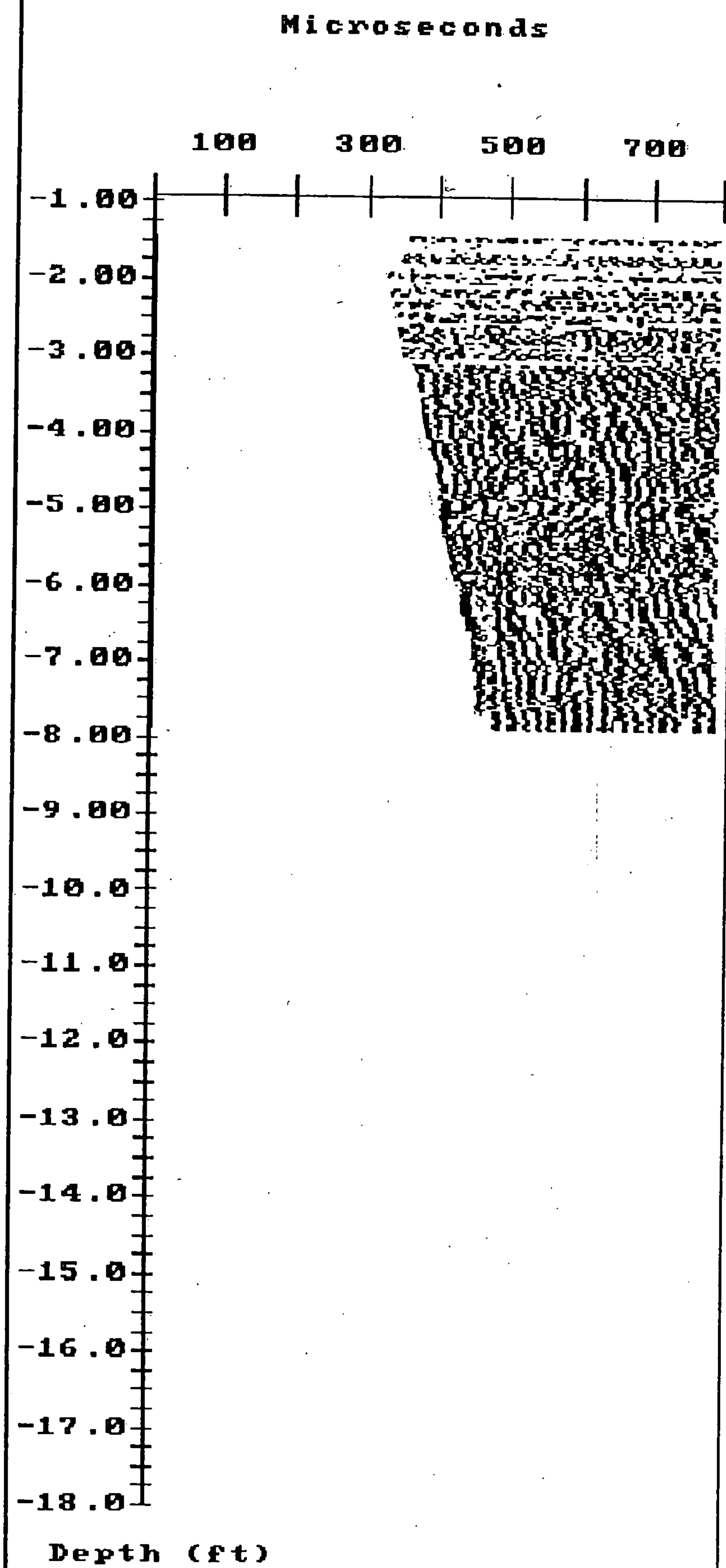
Profile: 2-7r

0 time offset

Pulses: 195

Test Operator:

BHH



STS CONSULTANTS

CSL Profile

Date: 09/09/01

Test Site:

Walker Mine

Structure: Plug

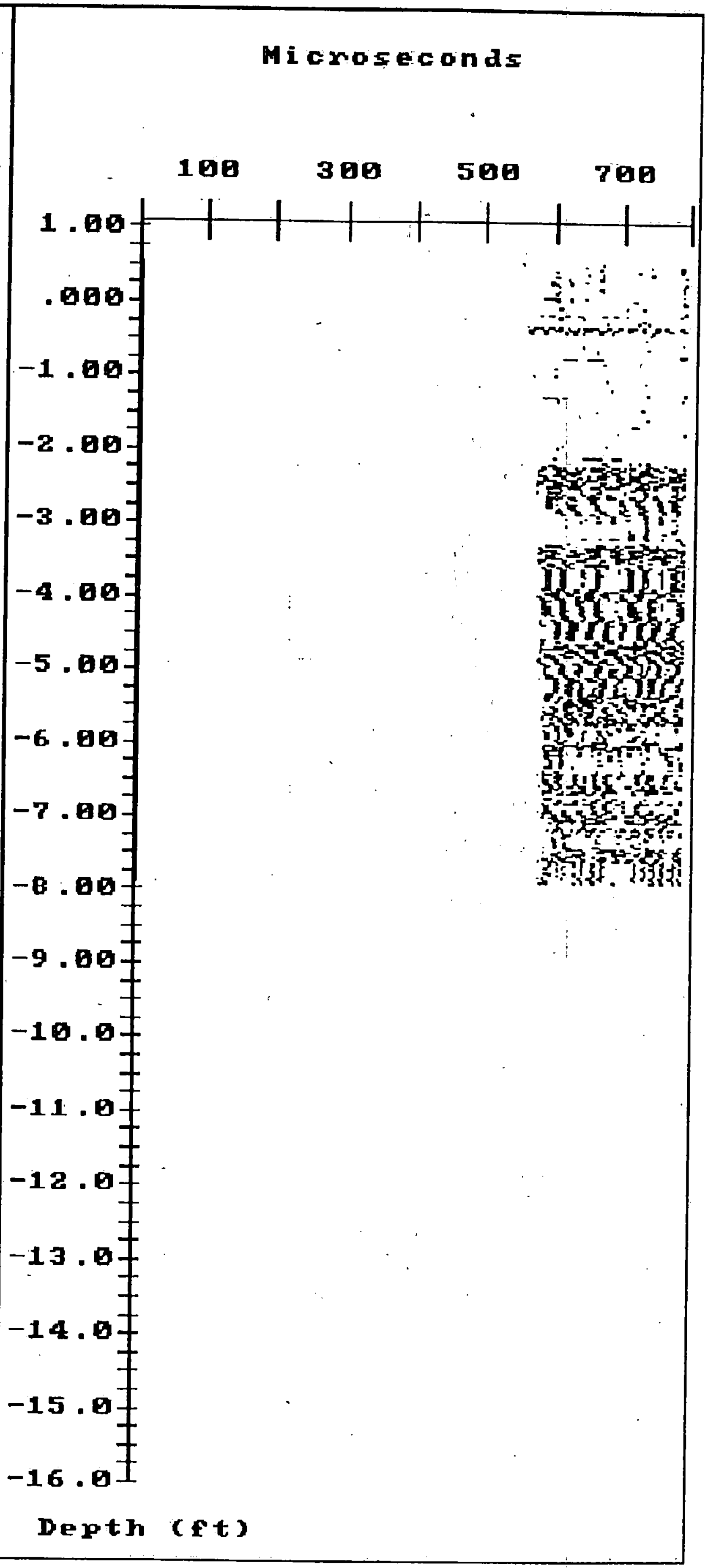
Profile: 3-4

0 time offset

Pulses: 255

Test Operator:

BHH



STS CONSULTANTS

CSL Profile

Date: 09/09/01

Test Site:

Walker Mine

Structure: Plug

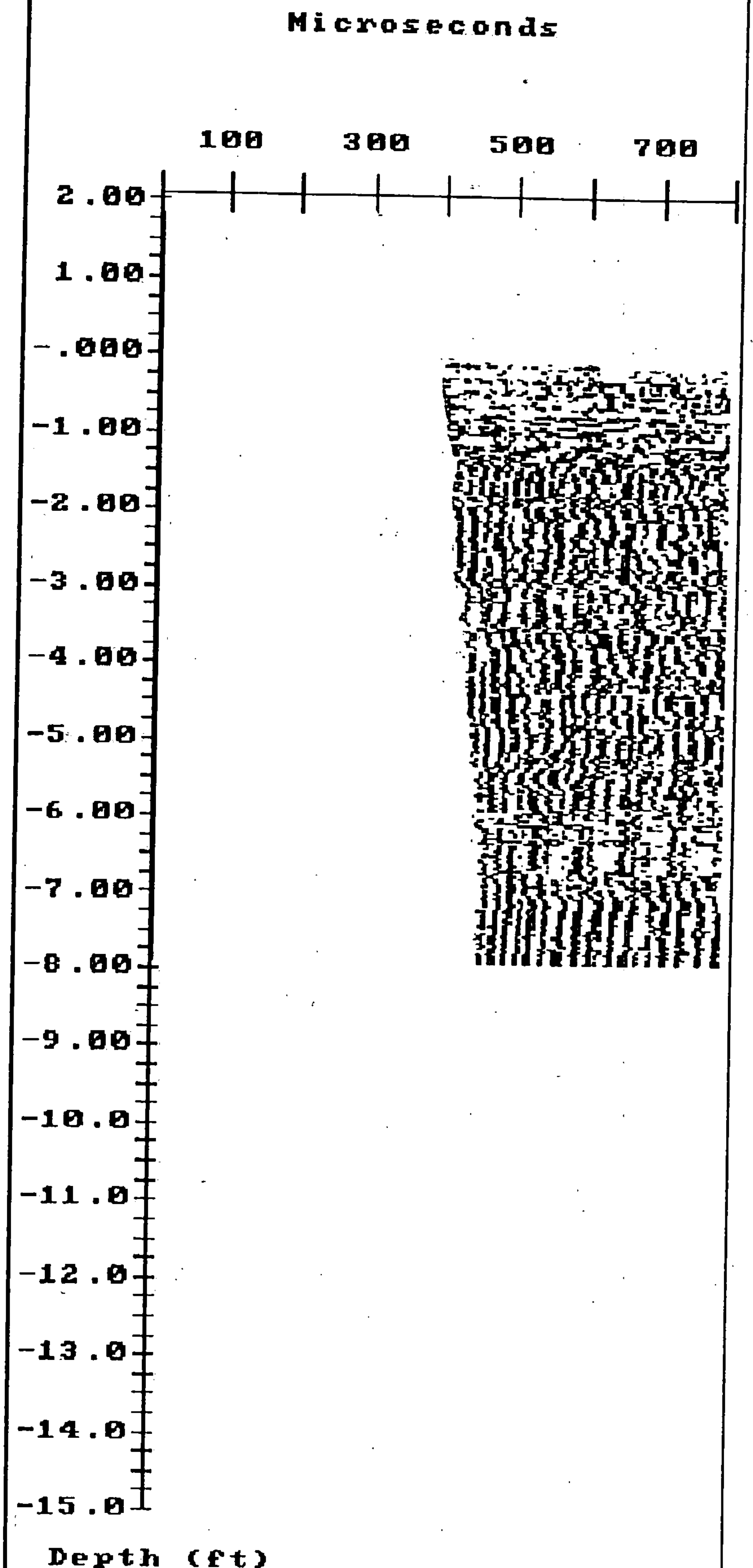
Profile: 3-7

0 time offset

Pulses: 295

Test Operator:

BHH



STS CONSULTANTS

CSL Profile

Date: 09/09/01

Test Site:

Walker Mine

Structure: Plug

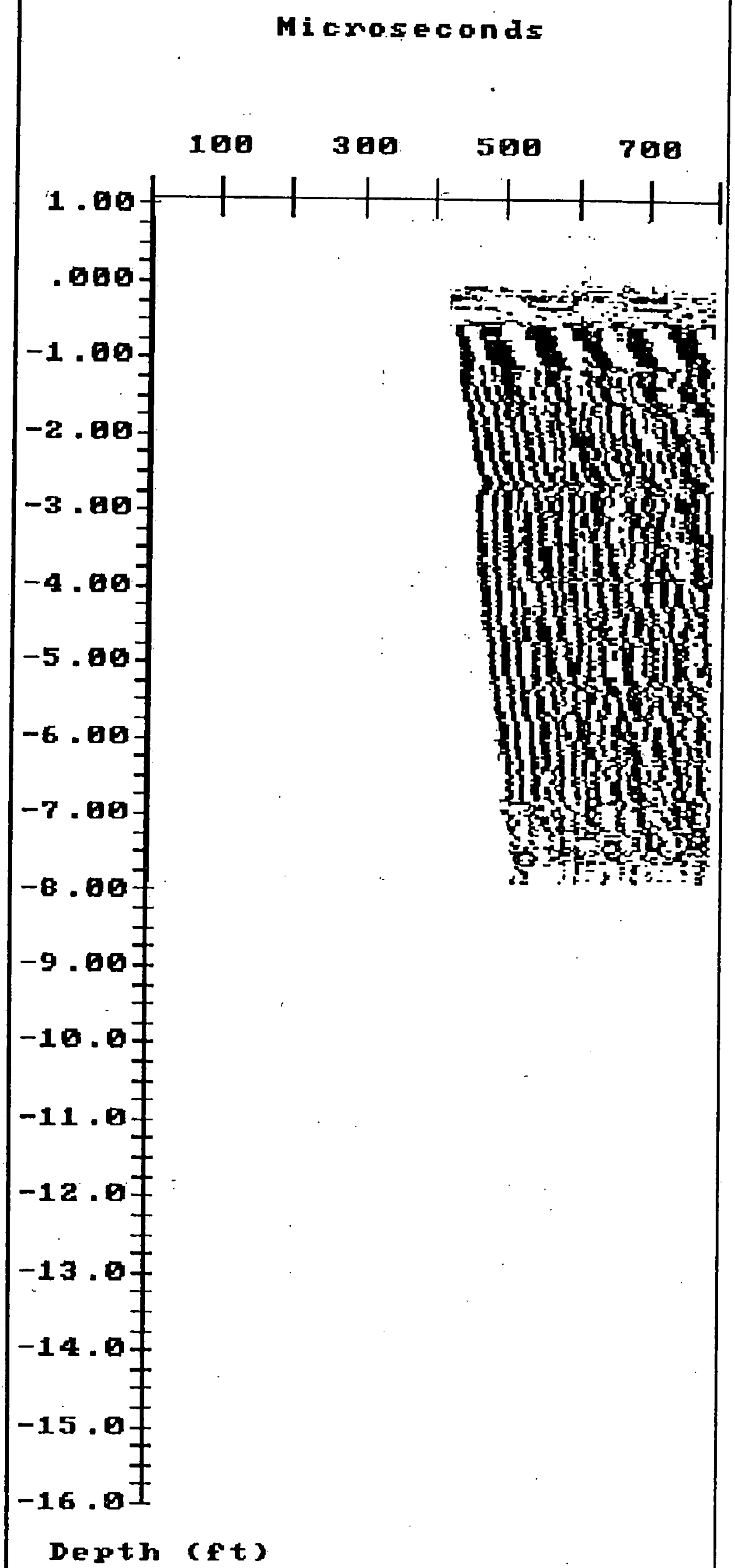
Profile: 4-5

0 time offset

Pulses: 264

Test Operator:

BHH



STS CONSULTANTS

CSL Profile

Date: 09/09/01

Test Site:

Walker Mine

Structure: Plug

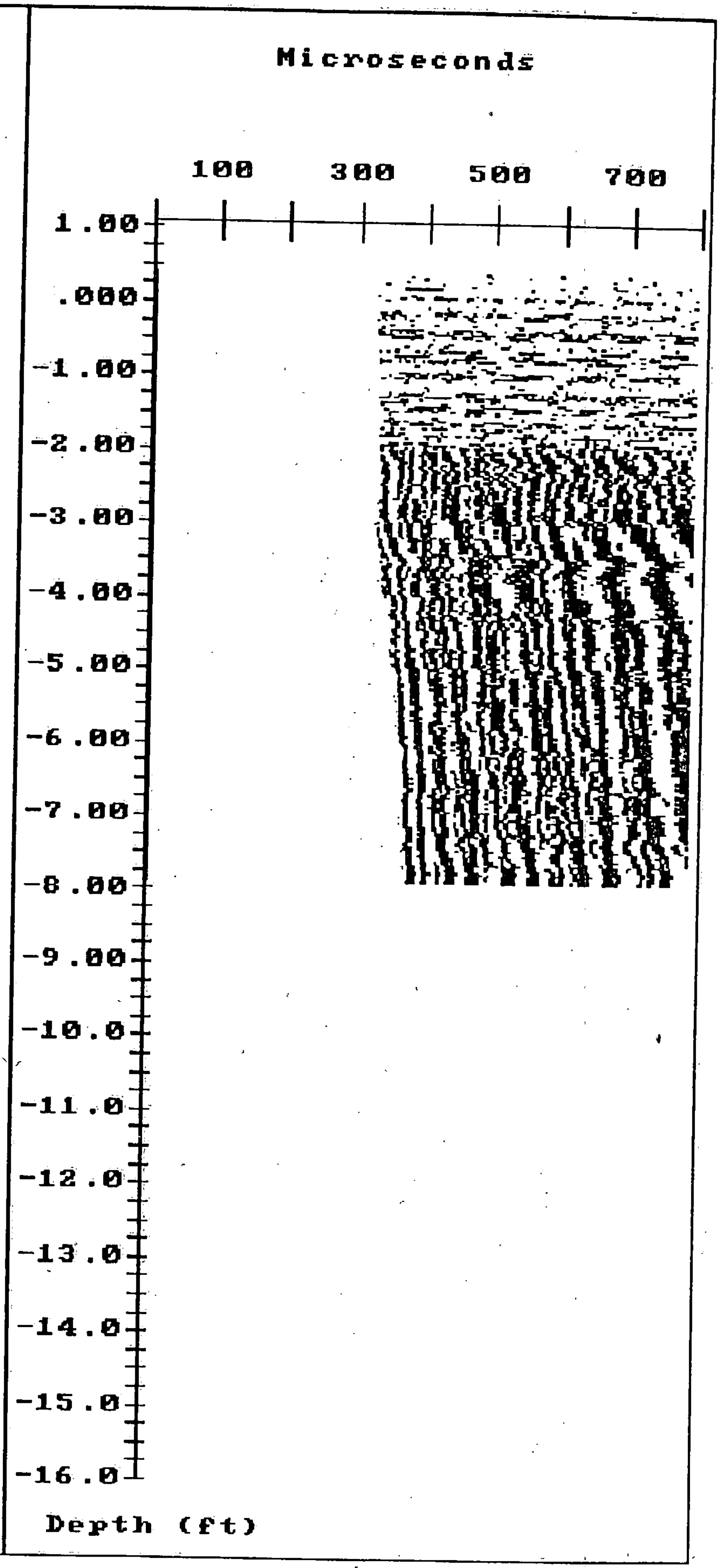
Profile: 4-7

0 time offset

Pulses: 251

Test Operator:

BHH



STS CONSULTANTS

CSL Profile

Date: 09/09/01

Test Site:

Walker Mine

Structure: Plug

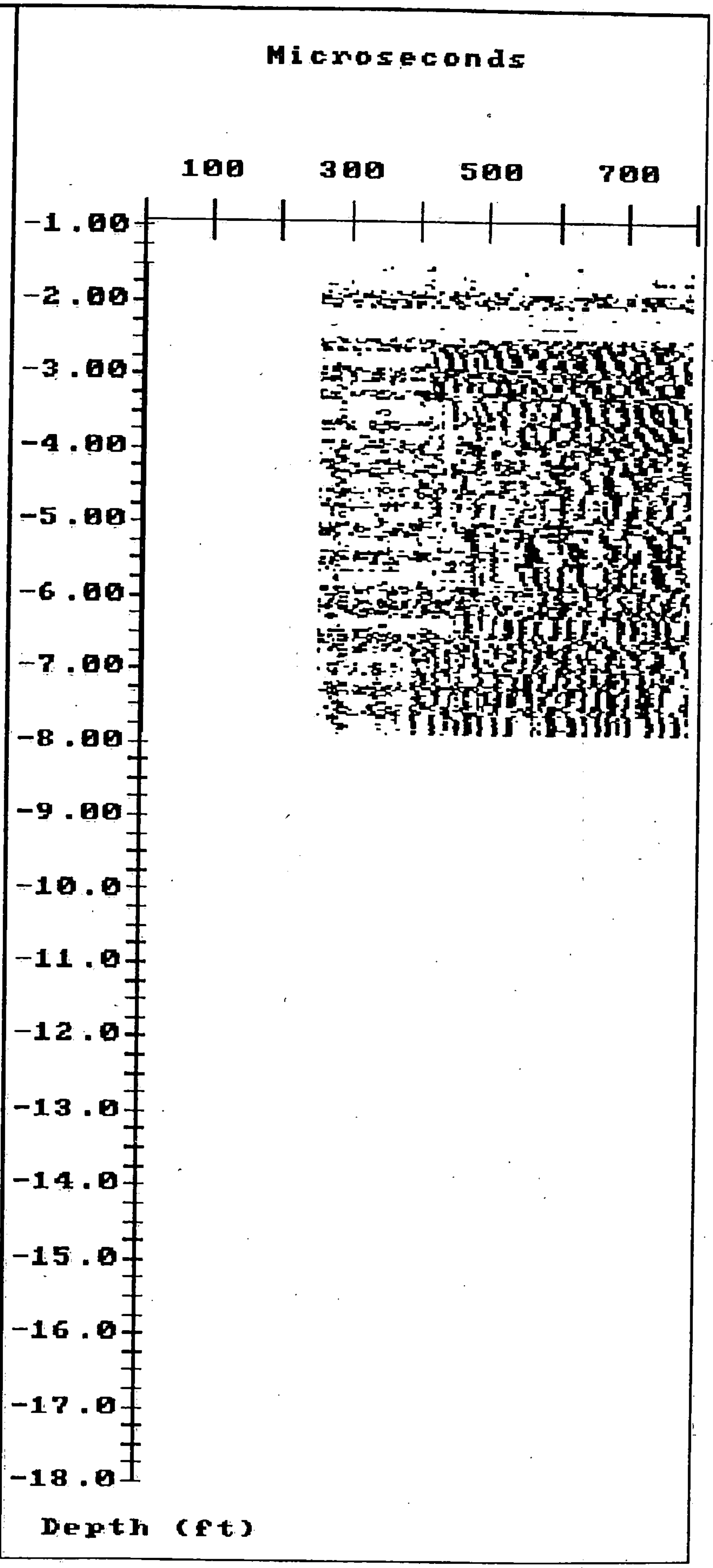
Profile: 1-2

0 time offset

Pulses: 192

Test Operator:

BHH



STS CONSULTANTS

CSL Profile

Date: 09/09/01

Test Site:

Walker Mine

Structure: Plug

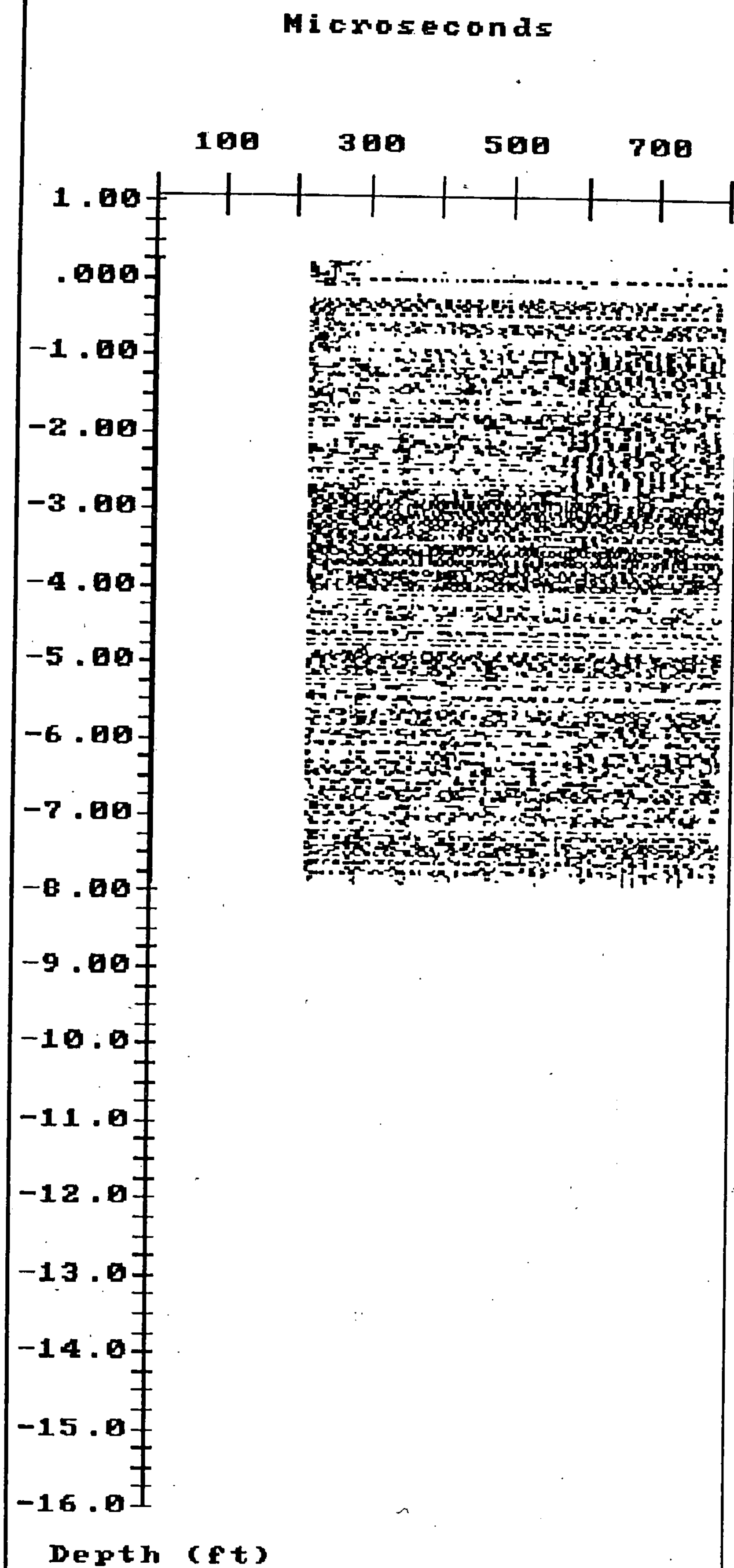
Profile: 2-3

0 time offset

Pulses: 246

Test Operator:

BHH



CTL

APPENDIX C

Client:	GEI Consultants, Inc.	CTL Project No.	320098
Project:	Walker Mine	CTL Project Mgr.:	M.Lim
Submitter:	M. Nagi	Technician:	T.Muresan
Contact:	Mr. Alberto Pujol	Date:	October 17, 2001

Test Results of ASTM C 42-94, "Standard Test Method for Compressive Strength of Drilled Cores"

Core Identification	**BH-3 (0.0-0.91)**	**BH-3 (8.8-9.5)**	**BH-6 (0.0-1.0)**	**BH-6 (5.3-5.7)**	**BH-7 (5.0-6.1)**	**BH-1 (9.5-10.0)**	**BH-2 (5.3-6.0)**
Nominal Maximum Aggregate Size (in.)							
Concrete Age at Test (days)							
Moisture Condition at Test	As rec'd dry	As rec'd dry	As rec'd dry	As rec'd dry	As rec'd dry	As rec'd dry	As rec'd dry
Orientation of Core Axis in Structure							
Diameter 1, (in.)	2.39	2.38	2.38	2.38	2.39	2.40	2.39
Diameter 2, (in.)	2.39	2.39	2.38	2.38	2.39	2.39	2.39
Average Diameter (in.)	2.39	2.38	2.38	2.38	2.39	2.39	2.39
Cross-Sectional Area (sq in.)	4.49	4.45	4.45	4.44	4.47	4.50	4.49
Length Trimmed (in.)	4.69	4.62	4.47	2.55	4.71	4.68	4.51
Length Capped (in.)	4.85	4.81	4.63	2.75	4.96	4.95	4.72
Unit Weight (lb/cu.ft)	137	146	139	137	142	166	166
Loading Rate, psi/sec	35	35	35	35	35	35	35
Maximum Load (lb)	28,200	27,200	25,800	33,000	25,800	73,000	98,800
Uncorrected Compressive Strength (psi)	6,290	6,110	5,800	7,430	5,780	16,240	22,020
Ratio of Capped Length to Diameter (L/D)	2.03	2.02	1.95	1.16	2.08	2.07	1.97
Correction Factor (ASTM C42)	1.00	1.00	1.00	0.91	1.00	1.00	1.00
Corrected Compressive Strength (psi)	**6,290**	**6,110**	**5,800**	**6,750**	**5,780**	**16,240**	**22,020**
Fracture Pattern	conical	conical	conical	conical	conical	collumnar	collumnar
Notes:							

Construction Technology Laboratories, Inc.

CTL

APPENDIX D

CTL
Construction Technology Laboratories, Inc.
www.ctlgroup.com
5420 Old Orchard Road
Skokie, Illinois 60077
800.522.2CTL (2285)

PETROGRAPHIC SERVICES REPORT

CTL Project No.: 320098 *Date:* November 29, 2001

Re: Petrographic Examination of Concrete Core Samples from Walker Mine

Three concrete core samples, labeled BH-3, BH-6 and BH-7 (Fig. 1) were received October 2, 2001 from Dr. Mohamad Nagi, CTL, on behalf of GEI, Consultants, Inc. The core samples were reportedly taken from the above-referenced structure to aid in evaluation of concrete quality. Petrographic examination was requested on the core samples to evaluate concrete properties.

FINDINGS AND CONCLUSIONS

The results of the petrographic examination are summarized below; additional details of which are presented in the attached data sheets.

Core Samples BH-3 and BH-7 appear to be similar in composition and quality (Fig. 2). The general quality of the concrete is good, no major abnormalities are observed. Distress is essentially limited to cracking. Core sample BH-7 was received fractured along the long axis of the core and somewhat diagonal (Figs. 1 and 2). The fracture surfaces pass through and around aggregate particles and are relatively clean; only small amounts of carbonate material are present on the fracture surface. A definitive cause for this crack was not determined. Randomly oriented microcracks are common in Core Samples BH-3 and BH-6, and abundant in Core Sample BH-7.

The concrete represented by Core Samples BH-3 and BH-7 was produced using a fairly well graded, 1/2 to 5/8 in. top size, crushed limestone coarse aggregate and a siliceous and calcareous sand uniformly dispersed in a non-air-entrained portland-cement paste. The concrete core samples are generally well consolidated. Air content, by volume of concrete, is estimated at 2 to 3% in Core Sample BH-3 and less than 2% in Core Sample BH-7. In Core Samples BH-3 and BH-7, the cement paste along surfaces of freshly fractured concrete is dark-brownish gray, moderately hard to hard and exhibits dull to subvitreous luster. Paste-aggregate bond is moderately tight to tight.

Core Sample BH-6 is different in composition and of lesser quality compared to that observed in Core Samples BH-3 and BH-7 (Fig. 2). The general quality of the concrete is poor. The core sample does not exhibit coarse aggregate, possibly due to segregation.

The concrete represented by Core Sample BH-6 was produced using a well graded, 1/4-in. top size, siliceous sand dispersed uniformly in a non-air-entrained portland-cement paste. The concrete core sample is well consolidated. Air content is estimated at less than 2% by volume of concrete (Fig. 3). In Core Sample BH-6, the cement paste is light gray, moderately soft to soft and friable and exhibits dull luster. Paste-aggregate bond is weak.

The microscopical examination of the submitted concrete samples did not detect mineral pozzolans in the cementitious paste matrix. However, it is reported that the cementitious material contained 31% pozzolan. The nondetection of pozzolan material by petrographic methods is most likely due to the extreme fineness of the particles.

The cement paste microstructure in each of the examined core samples exhibits a low degree of crystallinity (isotropic) (Fig. 4). The lack of calcium hydroxide (hydration product) in the cement paste is most likely due to its reaction with the reported pozzolan material in the cementitious paste. The lack of calcium hydroxide could also be an indication of moisture (water) passing through the concrete leaching out soluble hydrates. This occurrence does not appear to be adversely affecting the integrity of the paste represented by Core Samples BH-3 and BH-7. However, the integrity of the cement paste in Core Sample BH-6 could be further compromised by this occurrence due to the overall poor quality of the concrete.

Minor amounts of ettringite are observed lining some voids (Fig. 5). This occurrence suggests dissolution and recrystallization of soluble sulfates and calcium as secondary deposits in available void space.

METHODS OF TEST

Petrographic examination was performed in accordance with ASTM C 856-95, "Standard Practice for Petrographic Examination of Hardened Concrete." The core samples were cut longitudinally and one of the resulting surfaces of each was lapped and examined using a stereomicroscope at magnifications up to 45X. Surfaces of freshly broken concrete were also studied with the stereomicroscope.

A small rectangular block was cut from the midsection of each core sample, placed on individual glass microscope slides with epoxy, and reduced to a thickness of approximately 20 micrometers (0.0008-in.). These thin sections were studied using a polarized-light microscope at magnifications up to 400X to determine aggregate and paste mineralogy and microstructure.

David B. Vollmer
Senior Petrographer
Petrographic Services

DBV
320098
Attachments

Fig. 1 Concrete core samples, as received for examination.

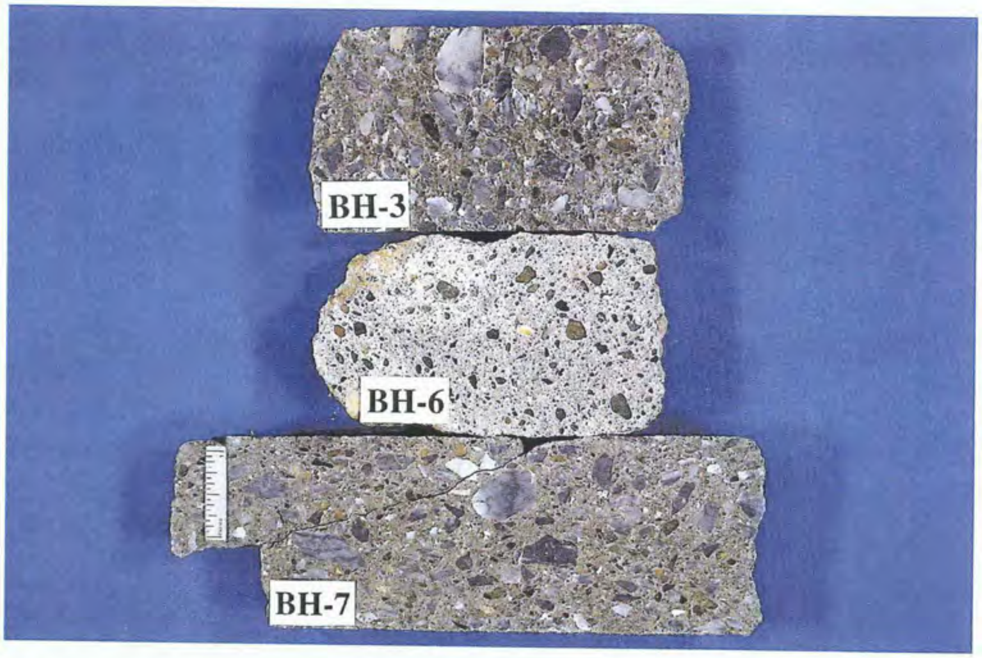


Fig. 2 Cut and lapped cross section of the core samples showing the generally uniform distribution of concrete constituents. Note the concrete appears well consolidated. Note difference in paste color and lack of coarse aggregate in Core Sample BH-6 compared to Core Samples BH-3 and BH-7.

Fig. 3 Photomicrograph of non-air-entrained concrete having an air content estimated to be less than 2%, by volume of concrete. Field of view left to right is approximately 0.3 in.

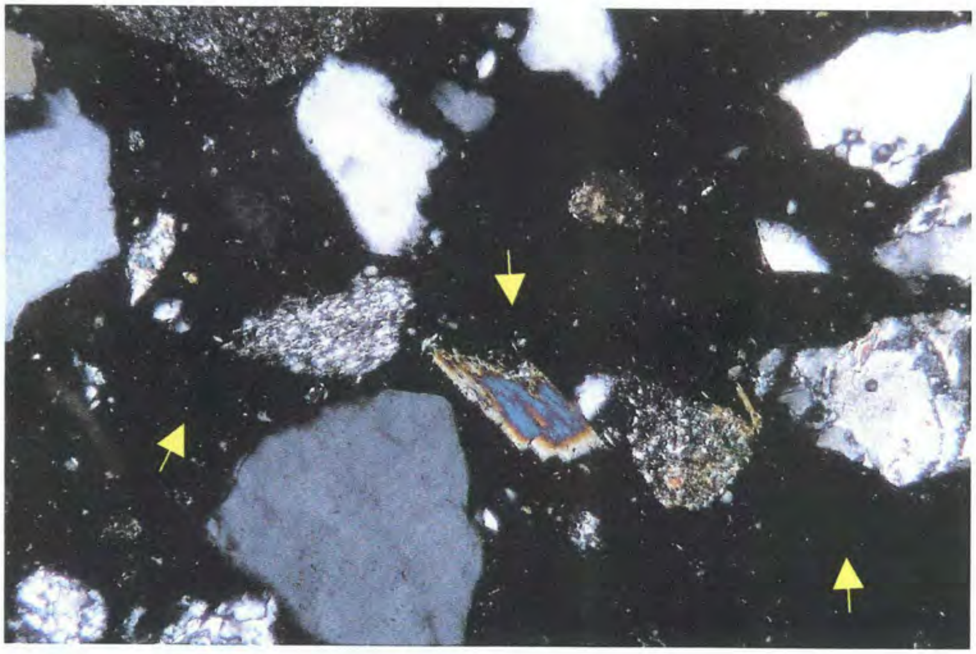


Fig. 4 Thin section photomicrograph in cross-polarized light showing cement paste microstructure exhibiting a low degree of crystallinity (arrows). Field of view left to right is approximately 0.07 in.

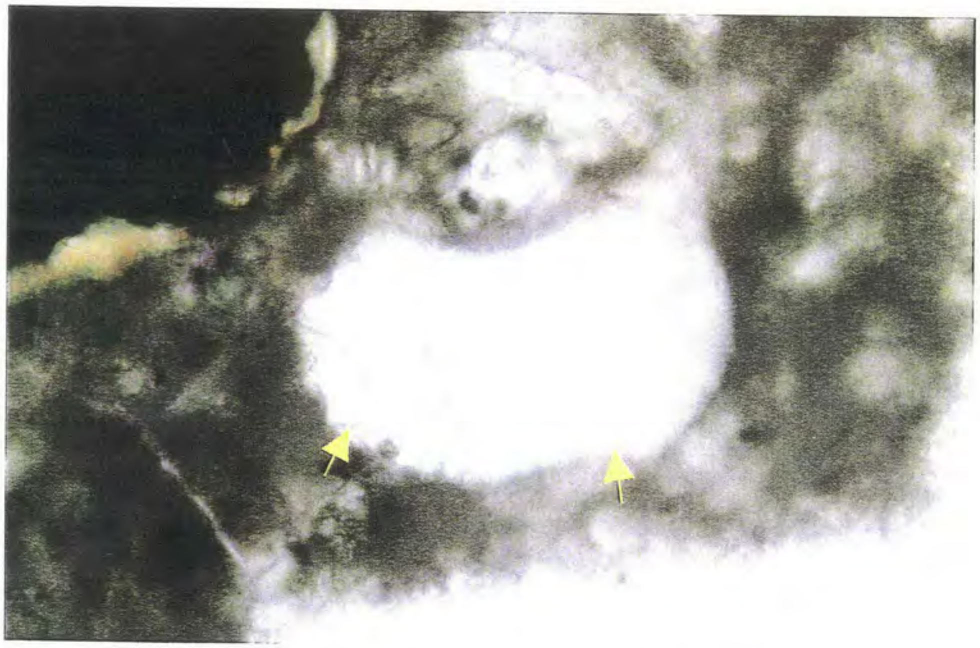


Fig. 5 **Inwardly projecting ettringite crystals partially filling void (arrows). Field of view left to right is approximately 0.01 in.**

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE, ASTM C 856

CTL PROJECT NO.: 320098
CLIENT: GEI Consultants Inc.
STRUCTURE: Walker Mine
LOCATION: California

DATE: October 25, 2001
REPORTED PROBLEM: None
EXAMINED BY: D. B. Vollmer

SAMPLE

Identification: Core Sample BH-3, 8.4 ft - 8.8 ft.

Dimensions: Diameter 2.4 in.; length 3.4 to 4.1 in.

Core Ends: Fracture surfaces pass through and around aggregate particles.

Cracks or Large Voids: No macrocracks or large voids observed.

Reinforcement: No reinforcement observed.

AGGREGATES

Coarse: Crushed calcareous rock composed mainly limestone.

Fine: Sand-size material composed mainly of a variety of calcareous and siliceous rocks (i.e. limestone, quartz, feldspar, chert and various igneous rocks).

Gradation & Top Size: Fairly well graded to a top size of 1/2 to 5/8 in.

Shape & Distribution: Coarse and fine aggregate particles are angular to subrounded, elongated to equant; aggregate distribution is relatively uniform.

PASTE

Color: Dark-brownish gray.

Hardness: Moderately hard to hard.

Luster: Dull to subvitreous.

Paste-Aggregate Bond: Moderately tight to tight.

Air Content: Estimated at 2 to 3%, by volume of concrete; concrete is not air entrained.

Depth of Carbonation: No applicable; no surface concrete present.

Calcium Hydroxide*: Estimated at 1 to 3%; the paste exhibits a low degree of crystallinity.

Unhydrated Portland-Cement Clinker Particles*: Estimated at 4 to 8%; pseudomorphic clinker particles (relics) are common to locally abundant.

Pozzolans*: No pozzolans observed.

Secondary Deposits: Inwardly projecting ettringite crystals partially line some voids.

MICROCRACKING: Randomly oriented microcracks are common in the body of the concrete.

ESTIMATED WATER-CEMENT RATIO: Due to the age of the concrete and continued hydration and alteration of the cement paste, estimating the original water-cement ratio would be speculative, therefore, a numeric estimation will not be reported.

MISCELLANEOUS: Sand streaks composed of calcite occur throughout the sample.

---

*percent by volume of paste

SAMPLE

Identification: Core Sample BH-6, 6.5 ft - 6.8 ft.

Dimensions: Diameter 2.4 in.; length approximately 4 in.

Core Ends: Eroded surfaces pass mainly around aggregate particles.

Cracks or Large Voids: No macrocracks or large voids observed.

Reinforcement: No reinforcement observed.

AGGREGATES

Coarse: No coarse aggregate present.

Fine: Natural siliceous sand composed mainly of quartz, feldspar, chert, a variety of igneous rocks and small amounts of various other rocks and minerals.

Gradation & Top Size: Sand is well graded to a top size of 1/4 in.

Shape & Distribution: The sand grains are angular to rounded, elongated to spherical; distribution is relatively uniform.

PASTE

Color: Light gray; predominately buff on exterior cylindrically cored surface due to the presence of what appears to be a fine sand material.

Hardness: Moderately soft to soft and friable.

Luster: Dull.

Paste-Aggregate Bond: Weak.

Air Content: Estimated at less than 2%, by volume of concrete; concrete is not air entrained.

Depth of Carbonation: Not applicable; no surface concrete present.

Calcium Hydroxide*: Estimated at 1 to 3% locally; the paste exhibits a low degree of crystallinity.

Unhydrated Portland-Cement Clinker Particles*: Estimated at 1 to 3%; pseudomorphic clinker particles (relics) are common.

Pozzolans*: No pozzolans observed.

Secondary Deposits: Inwardly projecting ettringite crystals partially line some voids.

MICROCRACKING: Randomly oriented microcracks are common in the body of the concrete

ESTIMATED WATER-CEMENT RATIO: Due to the age of the concrete and continued hydration and alteration of the cement paste, estimating the original water-cement ratio would be speculative, therefore, a numeric estimation will not be reported.

*percent by volume of paste

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE, ASTM C 856

CTL PROJECT NO.: 320098
CLIENT: GEI Consultants Inc.
STRUCTURE: Walker Mine
LOCATION: California

DATE: October 25, 2001
REPORTED PROBLEM: None
EXAMINED BY: D. B. Vollmer

SAMPLE

Identification: Core Sample BH-7, 3.1 ft - 3.7 ft.

Dimensions: Diameter 2.4 in.; length 6.7 in.

Core Ends: Fracture surfaces pass through and around aggregate particles.

Cracks or Large Voids: Core was received fractured along the long axis of the core and somewhat diagonal; fracture passes through and around aggregate particles. No large voids observed.

Reinforcement: No reinforcement observed.

AGGREGATES

Coarse: Crushed calcareous rock composed mainly limestone.

Fine: Sand-size material composed mainly of a variety of calcareous and siliceous rocks (i.e. limestone, quartz, feldspar, chert and various igneous rocks).

Gradation & Top Size: Fairly well graded to a top size of 1/2 to 5/8 in.

Shape & Distribution: Coarse and fine aggregate particles are angular to subrounded, elongated to equant; aggregate distribution is relatively uniform.

PASTE

Color: Dark-brownish gray.

Hardness: Moderately hard to hard.

Luster: Dull to subvitreous.

Paste-Aggregate Bond: Moderately tight to tight.

Air Content: Estimated at less than 2%, by volume of concrete; concrete is not air entrained.

Depth of Carbonation: Not applicable; no surface concrete present.

Calcium Hydroxide*: Estimated at 1 to 3% locally; the paste exhibits a low degree of crystallinity.

Unhydrated Portland-Cement Clinker Particles*: Estimated at 4 to 8%; pseudomorphic clinker particles (relics) are common to locally abundant.

Pozzolans*: No pozzolans observed.

Secondary Deposits: Inwardly projecting ettringite crystals partially line some voids.

MICROCRACKING: Randomly oriented microcracks are abundant in the body of the concrete.

ESTIMATED WATER-CEMENT RATIO: Due to the age of the concrete and continued hydration and alteration of the cement paste, estimating the original water-cement ratio would be speculative, therefore, a numeric estimation will not be reported.

---

*percent by volume of paste

Appendix B.3

Schmidt Hammer Test Results

Table 1 - Schmidt Hammer Test Results on Left Tunnel Wall
Walker Mine
Portola, CA

Rock Mass Surface							Joint Surface						
Measured R_N Value[1]	Test Angle[2] (deg)	Location of Test[3] X (ft)	Location of Test[3] Y (ft)	Test Angle Correction[4]	R_N Value, Corrected for Angle[5]	R_L Value, Corrected from Hammer Type N to L[6]	Measured R_N Value[1]	Test Angle[1] (deg)	Location of Test[3] X (ft)	Location of Test[3] Y (ft)	Test Angle Correction[3]	R_N Value, Corrected for Angle[5]	R_L Value, Corrected from Hammer Type N to L[6]
34	-45	7.5	3.5	2.2	36.2	41.5	32	-45	12.0	2.0	2.3	34.3	39.7
35	-45	17.0	4.5	2.2	37.2	42.4	34	-45	13.0	2.0	2.2	36.2	41.5
38	0	11.0	3.0	0.0	38.0	43.2	37	0	14.0	4.5	0.0	37.0	42.3
40	0	7.5	4.0	0.0	40.0	45.0	40	0	10.0	7.0	0.0	40.0	45.0
41	-45	10.0	7.0	2.0	43.0	47.7	40	-45	5.0	6.0	2.0	42.0	46.8
47	+45	16.0	6.0	-2.2	44.8	49.3	42	0	7.5	1.5	0.0	42.0	46.8
45	0	1.0	4.5	0.0	45.0	49.5	43	0	16.0	2.0	0.0	43.0	47.7
44	-45	4.5	3.5	1.9	45.9	50.3	45	0	4.0	4.5	0.0	45.0	49.5
44	-45	13.5	4.5	1.9	45.9	50.3	45	0	7.0	2.5	0.0	45.0	49.5
44	-45	15.5	5.0	1.9	45.9	50.3	45	0	7.5	1.0	0.0	45.0	49.5
46	0	2.0	5.0	0.0	46.0	50.4	48	+45	2.5	5.0	-2.1	45.9	50.3
46	0	4.0	4.0	0.0	46.0	50.4	48	0	10.5	3.5	0.0	48.0	52.2
48	-45	16.0	1.5	1.6	49.6	53.6	48	0	17.0	2.0	0.0	48.0	52.2
52	+45	2.0	6.0	-2.1	49.9	53.8	48	-45	2.5	2.5	1.6	49.6	53.6
50	0	6.0	5.5	0.0	50.0	53.9	50	0	10.0	4.0	0.0	50.0	53.9
52	0	1.0	5.0	0.0	52.0	55.6	52	0	6.0	6.0	0.0	52.0	55.6
52	-45	16.0	4.0	1.6	53.6	57.0	54	+45	4.0	7.0	-1.8	52.2	55.8
55	0	12.0	2.5	0.0	55.0	58.2	55	+45	2.5	6.0	-1.8	53.2	56.6
54	-45	6.0	4.5	1.5	55.5	58.6	54	0	18.0	5.5	0.0	54.0	57.3
58	0	7.0	2.5	0.0	58.0	60.6	58	-45	4.5	6.5	1.3	59.3	61.7

	Rock Mass Surface	Joint Surface
Average Corrected R Value for L Hammer (R_L Value) =	51.1	50.4
Compressive strength estimated from average R_L value in psi[7] =	22,248	21,424
Use a rounded compressive strength in psi =	22,000	21,000
Unconfined compressive strength in Mpa =	152	145

Notes:

1) R Value measured with a Type N Schmidt Hammer
2) Angle of instrument from horizontal, positive upwards, negative downwards.
3) Coordinate X=0 and Y=0 is on left wall at invert level where tunnel wall contacts seal, left side is relative to tunnel drive direction (facing seal).
X is distance from seal wall along tunnel axis. Y is vertical distance above invert.
4) Correction based on the angle of the test relative to horizontal and the correction from Table 1 of the Operating Instructions for the N Type and L Type Schmidt Hammer (attached).
5) Measured R Value plus test angle correction.
6) Conversion of R Value from N Type Hammer (R_N) to L Type Hammer (R_L), based on expression from Swiss Manufacturer, as follows; $R_L = 4.5 + 1.113 \times R_N - 0.0025\ (R_N)^2$.
7) Compressive strength was estimated using the expression in Figure 6.18 of Deere and Miller (1966) "Engineering Classification and Index Properties for Intact Rock",
where Compressive strength, in pounds per square inch, $= 10^{\wedge\,(0.00014 \times \text{Rock unit weight} \times \text{RL Value} + 3.16)}$, with a unit weight of rock of 166 pounds per cubic foot.

Table 2 - Schmidt Hammer Test Results on Tunnel Roof
Walker Mine
Portola, CA

Rock Mass Surface							Joint Surface						
Measured R_N Value[1]	Test Angle[2] (deg)	Location of Test[3] X (ft)	Location of Test[3] Y (ft)	Test Angle Correction[4]	R_N Value, Corrected for Angle[5]	R_L Value, Corrected from Hammer Type N to L[6]	Measured R_N Value[1]	Test Angle[1] (deg)	Location of Test[3] X (ft)	Location of Test[3] Y (ft)	Test Angle Correction[3]	R_N Value, Corrected for Angle[5]	R_L Value, Corrected from Hammer Type N to L[6]
30	+45	14.0	3.0	-3.1	26.9	32.6	34	+45	5.0	2.0	-2.9	31.1	36.7
32	+45	4.0	3.0	-3.1	28.9	34.6	36	+45	5.0	3.5	-2.8	33.2	38.7
33	+45	9.0	8.0	-2.8	30.2	35.8	38	+45	9.0	1.0	-2.6	35.4	40.8
34	0	17.0	4.0	0.0	34.0	39.5	38	+45	12.0	3.0	-2.6	35.4	40.8
34	0	15.5	4.0	0.0	34.0	39.5	39	+45	8.5	1.0	-2.6	36.4	41.7
32	-45	9.0	1.0	2.3	34.3	39.7	42	+45	4.5	2.5	-2.6	39.4	44.5
38	+45	4.5	3.5	-2.6	35.4	40.8	38	-45	10.0	1.0	2.0	40.0	45.0
40	0	17.0	3.0	0.0	40.0	45.0	46	+45	10.5	6.0	-2.4	43.6	48.3
42	0	10.0	2.5	0.0	42.0	46.8	47	+45	14.5	5.0	-2.4	44.6	49.2
42	0	10.0	6.0	0.0	42.0	46.8	48	+45	5.0	1.5	-2.1	45.9	50.3
42	0	17.5	3.5	0.0	42.0	46.8	50	+45	6.0	4.0	-2.1	47.9	52.1
45	0	13.0	3.5	0.0	45.0	49.5	51	+45	18.0	4.0	-2.1	48.9	52.9
48	+45	4.0	4.0	-2.1	45.9	50.3	51	+45	17.5	4.5	-2.1	48.9	52.9
50	+45	9.5	2.0	-2.1	47.9	52.1	52	+45	13.0	4.5	-2.1	49.9	53.8
48	0	10.5	2.5	0.0	48.0	52.2	55	+45	10.5	4.0	-1.8	53.2	56.6
48	0	9.5	4.0	0.0	48.0	52.2	55	+45	18.0	6.0	-1.8	53.2	56.6
49	-45	18.0	8.0	1.6	50.6	54.4	56	+45	13.0	5.5	-1.8	54.2	57.5
54	+45	8.5	5.5	-2.0	52.0	55.6	58	+45	10.5	2.0	-1.6	56.4	59.3
55	+45	14.0	4.0	-1.8	53.2	56.6	60	+45	6.0	5.0	-1.6	58.4	61.0
58	+45	4.0	2.5	-1.6	56.4	59.3	60	+45	17.5	6.5	-1.6	58.4	61.0

	Rock Mass Surface	Joint Surface
Average Corrected R Value for L Hammer (R_L Value) =	46.5	50.0
Compressive strength estimated from average R_L value in psi[7] =	17,414	20,972
Use a rounded compressive strength in psi =	17,000	21,000
Unconfined compressive strength in Mpa =	117	145

Notes:

1) R Value measured with a Type N Schmidt Hammer
2) Angle of instrument from horizontal, positive upwards, negative downwards.
3) Coordinate X=0 and Y=0 is on roof at the contact with the seal on the right side of the roof relative to tunnel drive direction.
   X is distance from seal wall along tunnel axis. Y is horizontal distance across tunnel, measured from the right wall.
4) Correction based on the angle of the test relative to horizontal and the correction from Table 1 of the Operating Instructions for the N Type and L Type Schmidt Hammer (attached).
5) Measured R Value plus test angle correction.
6) Conversion of R Value from N Type Hammer (R_N) to L Type Hammer (R_L), based on expression from Swiss Manufacturer, as follows; $R_L = 4.5 + 1.113 \times R_N - 0.0025 (R_N)^2$.
7) Compressive strength was estimated using the expression in Figure 6.18 of Deere and Miller (1966) "Engineering Classification and Index Properties for Intact Rock",
   where Compressive strength, in pounds per square inch, = $10^{\wedge (0.00014 \times \text{Rock unit weight} \times \text{RL Value} + 3.16)}$, with a unit weight of rock of 166 pounds per cubic foot.

Table 3 - Schmidt Hammer Test Results on Right Tunnel Wall
Walker Mine
Portola, CA

Rock Mass Surface							Joint Surface						
Measured R_N Value[1]	Test Angle[2] (deg)	Location of Test[3] X (ft)	Location of Test[3] Y (ft)	Test Angle Correction[4]	R_N Value, Corrected for Angle[5]	R_L Value, Corrected from Hammer Type N to L[6]	Measured R_N Value[1]	Test Angle[1] (deg)	Location of Test[3] X (ft)	Location of Test[3] Y (ft)	Test Angle Correction[3]	R_N Value Corrected for Angle[5]	R_L Value, Corrected from Hammer Type N to L[6]
32	0	8.0	6.0	0.0	32.0	37.6	36	-45	18.5	5.0	-2.1	33.9	39.4
34	-45	9.5	1.0	2.2	36.2	41.5	34	0	8.0	2.0	0.0	34.0	39.5
34	-45	9.5	0.5	2.2	36.2	41.5	37	-45	9.5	2.5	-2.1	34.9	40.3
41	+45	2.0	6.0	-2.6	38.4	43.6	38	-45	6.0	4.5	-2.0	36.0	41.3
41	0	1.0	3.0	0.0	41.0	45.9	38	0	0.5	2.0	0.0	38.0	43.2
39	-45	6.0	5.0	2.0	41.0	45.9	40	-45	12.0	1.5	-2.0	38.0	43.2
40	-45	2.0	2.5	2.0	42.0	46.8	44	+45	4.5	4.0	-2.3	41.7	46.6
40	-45	17.0	3.0	2.0	42.0	46.8	44	+45	9.5	3.5	-2.3	41.7	46.6
42	-45	17.0	6.0	2.0	44.0	48.6	44	-45	15.0	2.5	-1.8	42.2	47.0
43	-45	6.0	3.0	1.9	44.9	49.4	44	-45	16.0	2.0	-1.8	42.2	47.0
44	-45	9.0	3.0	1.8	45.8	50.2	44	0	2.0	4.0	0.0	44.0	48.6
44	-45	11.0	1.5	1.8	45.8	50.2	48	+45	15.0	7.0	-2.1	45.9	50.3
47	0	10.0	4.0	0.0	47.0	51.3	48	-45	16.0	1.5	-1.6	46.4	50.8
46	-45	13.5	6.0	1.8	47.8	52.0	49	-45	5.5	4.5	-1.6	47.4	51.6
48	-45	15.0	4.0	1.7	49.7	53.6	49	0	18.5	4.5	0.0	49.0	53.0
51	0	2.0	6.5	0.0	51.0	54.8	50	0	13.0	2.0	0.0	50.0	53.9
53	+45	13.0	5.5	-2.0	51.0	54.8	51	0	5.5	2.0	0.0	51.0	54.8
50	-45	14.5	1.5	1.6	51.6	55.3	56	+45	13.5	6.5	-1.8	54.2	57.5
58	+45	8.0	2.5	-1.7	56.3	59.2	57	+45	2.5	4.0	-1.8	55.2	58.3
59	0	13.0	3:5	0.0	59.0	61.5	62	+45	18.0	6.0	-1.6	60.4	62.6

Rock Mass Surface:

Average Corrected R Value for L Hammer (R_L Value) = 49.5
Compressive strength estimated from average R_L value in psi[7] = 20,469
Use a rounded compressive strength in psi = 20,000
Unconfined compressive strength in Mpa = 138

Joint Surface:

Average Corrected R Value for L Hammer (R_L Value) = 48.8
Compressive strength estimated from average R_L value in psi[7] = 19,653
Use a rounded compressive strength in psi = 20,000
Unconfined compressive strength in Mpa = 138

Notes:

1) R Value measured with a Type N Schmidt Hammer
2) Angle of instrument from horizontal, positive upwards, negative downwards.
3) Coordinate X=0 and Y=0 is on right wall at invert level where tunnel wall contacts seal, right side is relative to tunnel drive direction. X is distance from seal wall along tunnel axis. Y is vertical distance above invert.
4) Correction based on the angle of the test relative to horizontal and the correction from Table 1 of the Operating Instructions for the N Type and L Type Schmidt Hammer (attached).
5) Measured R Value plus test angle correction.
6) Conversion of R Value from N Type Hammer (R_N) to L Type Hammer (R_L), based on expression from Swiss Manufacturer, as follows; $R_L = 4.5 + 1.113 \times R_N - 0.0025 (R_N)^2$.
7) Compressive strength was estimated using the expression in Figure 6.18 of Deere and Miller (1966) "Engineering Classification and Index Properties for Intact Rock", where Compressive strength, in pounds per square inch, $= 10^{\wedge (0.00014 \times \text{Rock unit weight} \times \text{RL Value} + 3.16)}$, with a unit weight of rock of 166 pounds per cubic foot.

Table 4 - Schmidt Hammer Test Results on Concrete Seal
Walker Mine
Portola, CA

Measured R Value[1]	Test Angle[2] (deg)	Location of Test[3] X (ft)	Location of Test[3] Y (ft)	Test Angle Correction[4]	Corrected R Value[5]
11	0	4.5	6.0	0.0	11.0
15	0	4.5	0.0	0.0	15.0
17	0	5.0	4.0	0.0	17.0
22	0	4.5	8.0	0.0	22.0
24	0	2.0	6.0	0.0	24.0
24	0	4.5	2.0	0.0	24.0
24	0	8.0	6.0	0.0	24.0
25	0	6.0	6.5	0.0	25.0
25	0	6.0	2.0	0.0	25.0
30	0	6.0	0.5	0.0	30.0
31	0	8.0	8.0	0.0	31.0
31	0	8.0	4.5	0.0	31.0
33	0	2.0	8.0	0.0	33.0
33	0	2.0	2.0	0.0	33.0
35	0	6.0	4.5	0.0	35.0
35	0	8.0	3.0	0.0	35.0
36	0	2.0	10.0	0.0	36.0
37	0	2.0	0.0	0.0	37.0
37	0	4.5	10.0	0.0	37.0
39	0	2.0	4.0	0.0	39.0

Average Corrected R Value = 28.2
Compressive strength from average corrected R Value, in MPa[6] = 183
Unconfined compressive strength in psi = 2,600

Notes:
1) R Value measured with a Type N Schmidt Hammer on the face of the concrete seal.
2) Angle of instrument from horizontal.
3) Coordinate X=0 and Y=0 is on left side of concrete seal at invert level where tunnel wall contacts seal, left side is relative to tunnel drive direction.
X measures horizontal distance across tunnel, Y measures vertical distance from invert.
4) Correction based on the angle of the test relative to horizontal and the correction from Table 1 of the Operating Instructions for the Schmidt Hammer.
5) Measured R Value plus test angle correction.
6) Compressive strength estimated from Fig. 3b of Operating Instructions for Type N Schmidt Hammer.

Attachment B

II. Operating Instructions for the Concrete Test Hammer (Fig. 2)

1. By lightly pressing on the head of the impact plunger (1) the plunger is released and will slide out of the housing (3) by itself.
2. The plunger (1) is pressed against the spot of the concrete surface (2) to be tested. Just before it disappears completely in the housing (3) the hammer is released. Release must be effected by **slowly** increasing the pressure on the housing. At the moment of impact the hammer must be held exactly at right angle to the surface (2). Do not touch the push-button (6)!
3. After the impact the hammer mass (14) rebounds by a certain amount which is indicated on the scale (19) by the rider (4). The reading of the rider position gives the rebound value in percent of the forward movement of the hammer mass.
4. By simply removing the hammer from the spot tested, it is reset for a further test and at the same time the indication is cancelled. The rider never returns quite to zero. In its extended position the plunger (1) is slightly out of the longitudinal axis of the hammer.
5. After having finished the tests, the plunger (1) together with its guide bar (7) and guide disk (8) is locked in its rear position by means of the push-button (6). Locking should always be effected after releasing the impact, i.e. with untensioned impact spring (16). The lock also serves for fixing the rebound reading after impact tests in dark or not easily accessible locations.
6. The test hammer is calibrated for horizontal impact direction, i.e. for testing vertical surfaces. When using it on inclined or horizontal surfaces, the rebound value R_α must be corrected as per table I.

Table I Correction of the Test Hammer Indications for Non-horizontal Impacts

Rebound value R_α	Correction for inclination angle α upwards + 90°	+ 45°	downwards − 45°	− 90°
10			+ 2.4	+ 3.2
20	− 5.4	− 3.5	+ 2.5	+ 3.4
30	− 4.7	− 3.1	+ 2.3	+ 3.1
40	− 3.9	− 2.6	+ 2.0	+ 2.7
50	− 3.1	− 2.1	+ 1.6	+ 2.2
60	− 2.3	− 1.6	+ 1.3	+ 1.7

LA-0350-01 Impact Plunger
LA-0350-03 Housing
LA-0350-04 Rider with Guide Rod
LA-0350-06 Push-Button Complete
LA-0350-07 Hammer Guide Bar
LA-0350-08 Disc
LA-0350-09 Cap
LA-0350-10 Two-Part Ring
LA-0350-11 Ring Cover
LA-0350-12 Compression Spring
LA-0350-13 Pawl
LA-0350-14 Hammer Mass—"N" and "NA" Hammer
LA-0350-15 Retaining Spring
LA-0350-16 Impact Spring
LA-0350-17 Guide Sleeve
LA-0350-18 Felt Washer
LA-0350-19 Window & Scale
LA-0350-20 Trip Screw
LA-0350-21 Lock Nut
LA-0350-22 Pin
LA-0350-23 Pawl Spring
LA-0350-24 Screw-Window
LA-0350-25 Calib. Chart Cyl.
LA-0350-26 Carrying Case
LA-0350-27 Calib. Chart Cube kg/cm

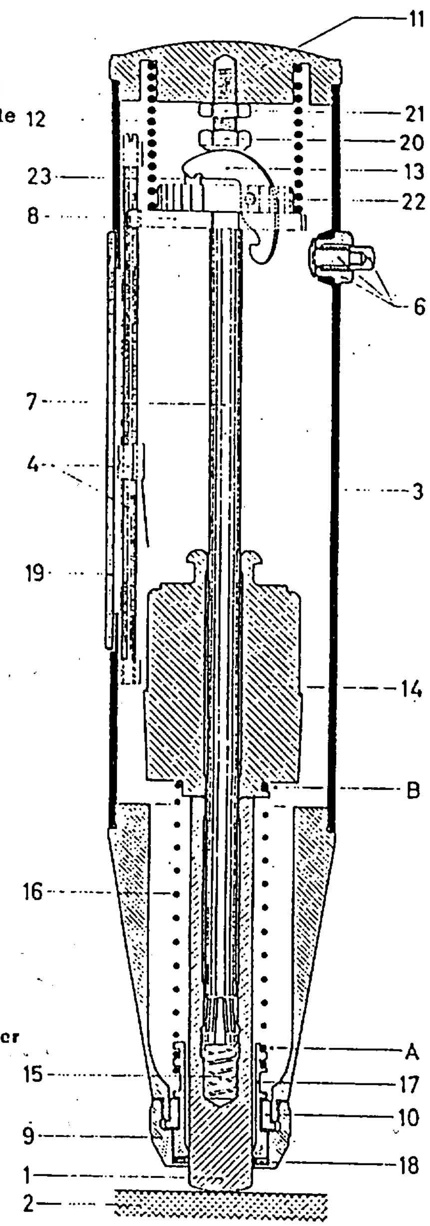


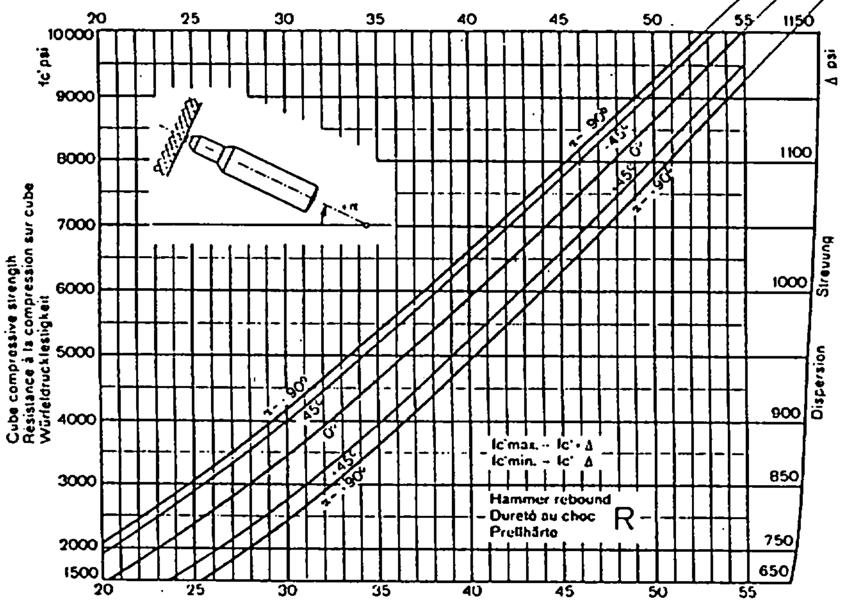
Fig. 2
Longitudinal Section of the Type N Concrete Test Hammer

Condition on Impact

When ordering spare parts, please state No. of part and serial No. of test hammer!

Fig. 3a **Cube Compressive Strength in psi plotted against the Rebound Number**

Type N Test Hammer



The curves apply to compact Portland cement with good-quality gravel/sand aggregate. Age 14 to 56 days. Smooth and dry concrete surface.

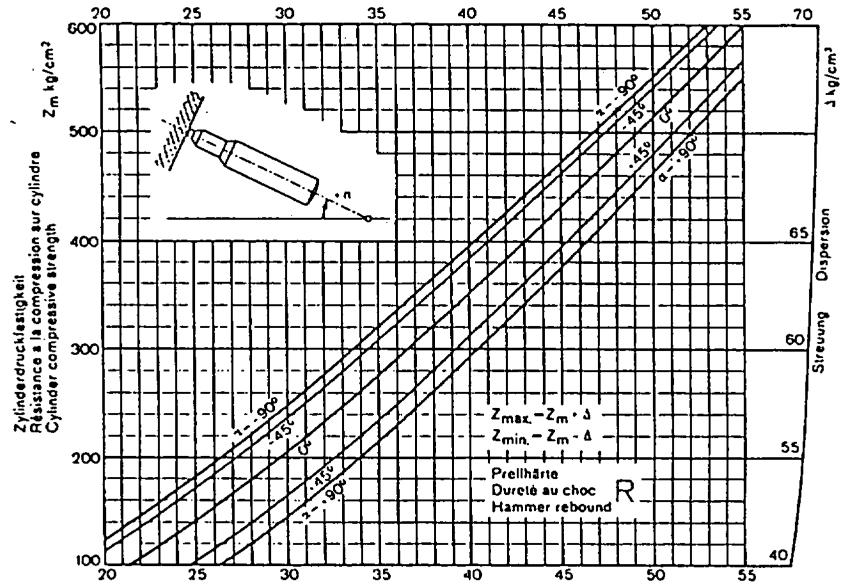
fc' = most likely value of the cube compressive strength in psi.

The dispersion limits fc'max. and fc'min. are so defined that they include 80 percent of all the test results.

Note under Section IV "Limits of Validity of the Calibration Curves"!

Fig. 3b **Cylinder Compressive Strength in kg/cm² plotted against the Rebound Number**

Type N Test Hammer



The curves apply to compact Portland cement with good-quality gravel/sand aggregate. Age 14 to 56 days. Smooth and dry concrete surface.

Z_m = most likely value of the cylinder compressive strength in kg/cm².

The dispersion limits $Z_{max.}$ and $Z_{min.}$ are so defined that they include 80 percent of all the test results.

Note under Section IV "Limits of Validity of the Calibration Curves"!

Appendix B.4

"Ultrasonic Thickness Evaluation, Walker Mine Seal"
Kleinfelder, Inc., October 17, 2001.

KLEINFELDER
An employee owned company

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BY:

December 14, 2001
File: 26-2988-01
Revision: 1

Alberto Pujol
GEI Consultants, Inc.
2201 Broadway, Suite 321
Oakland, California 94612

Subject: **ULTRASONIC THICKNESS EVALUATION**
WALKER MINE SEAL
PORTOLA, CALIFORNIA

Dear Mr. Pujol:

At your request Kleinfelder performed ultrasonic thickness evaluation on the east (right) and west (left) drain pipe at Walker Mine located in Portola, California.

On September 11, 2001 Kleinfelder's ASNT Level II operator performed nondestructive testing at locations chosen by Alberto Pujol using a Panametrics model 26DL thickness gauge to evaluate in-place thickness measurements. Following is a summary of the test results.

WALKER MINE SEAL THICKNESS TESTING AND EVALUATION			
Test locations shown on bottom 1/2 on attached fig.1			
East (Right) Drain Pipe			
Test No.	**Ultrasonic Thickness Reading (inches)**	**Test No. cont.**	**Ultrasonic Thickness Reading (inches) Cont.**
1 Underneath Side	.221	7	.221
2	.224	8	.221
3	.221	9 Underneath Side	.221
4	.226	10	.213
5 Underneath Side	.219	11	.220
6	.219	12	.220

KLEINFELDER 9530 Crossroads Drive, Redding, CA 96003 (530) 222-7203 (530) 222-3053 fax

WALKER MINE SEAL THICKNESS TESTING AND EVALUATION			
Test locations shown on upper 1/2 on attached fig.1			
West (Left) Drain Pipe			
Test No.	**Ultrasonic Thickness Reading (inches)**	**Test No. cont.**	**Ultrasonic Thickness Reading (inches) Cont.**
1	.113	16	.370
2	.116	17	.377
3	.106	18 Underneath Side	.240
4	.107	19	.231
5 Underneath Side	.232	20	.224
6	.227	21	.230
7	.229	22 Underneath Side	.231
8	.228	23	.232
9 Underneath Side	.236	24	.231
10	.227	25	.230
11	.229	26 Underneath Side	.231
12	.228	27	.229
13	.376	28	.233
14	.371	29	.230
15 Underneath Side	.403		

WALKER MINE SEAL THICKNESS TESTING AND EVALUATION			
Test results shown on attached fig. 2			
East (Right) Drain Pipe Elevation A-A			
Test No.	**Ultrasonic Thickness Reading (inches)**	**Test No. cont.**	**Ultrasonic Thickness Reading (inches) Cont.**
1	.110	12 Inside Reading	.405
2	.105	13 Inside Reading	.408
3	.119	14	.104
4	.107	15	.100
5 Inside	.232	16	.290
6 Outside	.238	17	.155
7 Top Side	.229	18	.151
8 Underneath Side	.230	19	.222
9 Wall Side	.398	20	.222
10 Wall Side	.398	21	.221
11	.408	22	.221

KLEINFELDER 9530 Crossroads Drive, Redding, CA 96003 (530) 222-7203 (530) 222-3053 fax

All nondestructive test equipment as well as reference standards used by Kleinfelder during ultrasonic testing met or exceeded the equipment specifications of ASTM E797.

If you have any questions regarding this information, please contact Rob DeArmond or Corky Metcalf at (530) 222-7203 at your convenience.

Very truly yours,

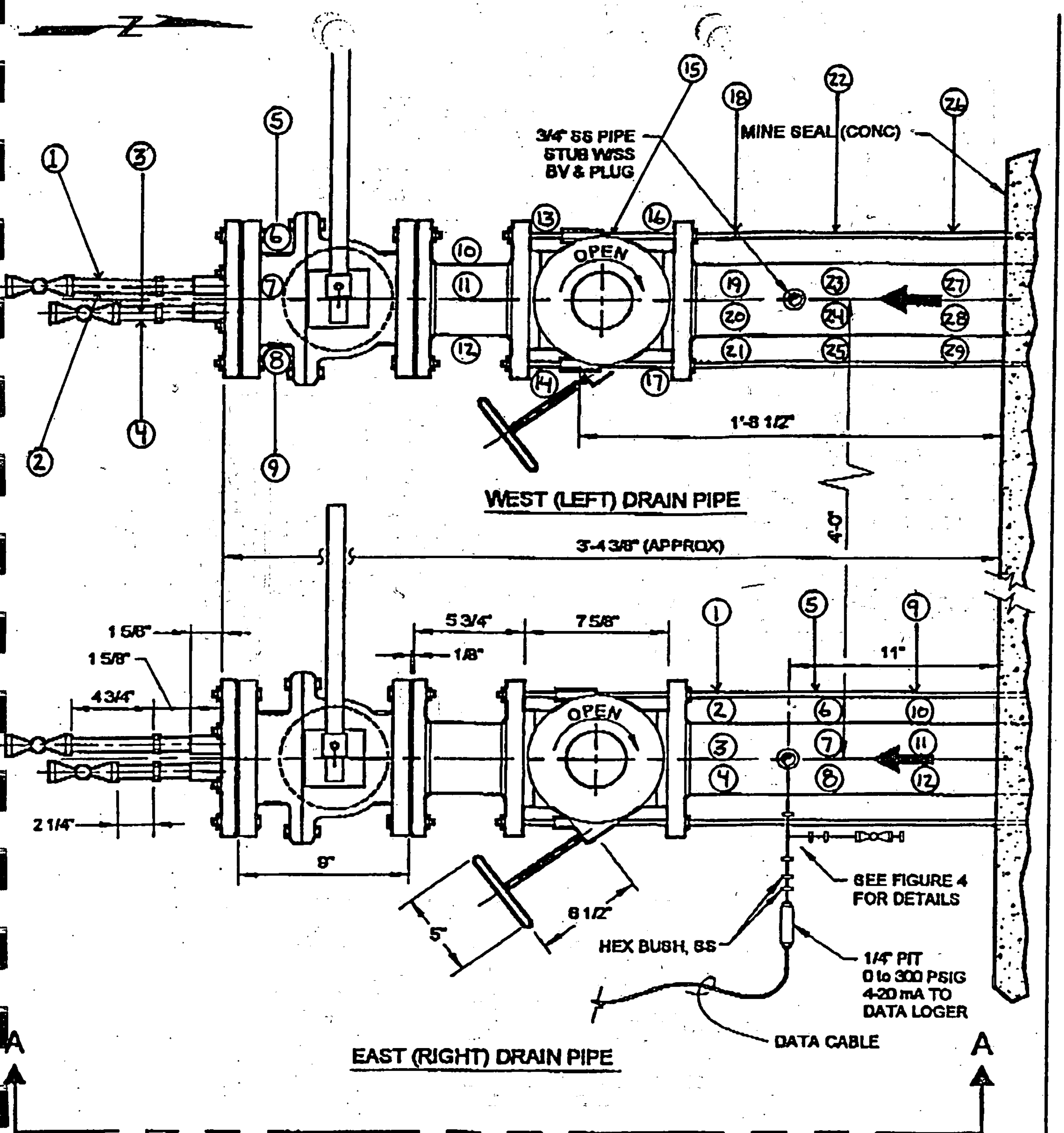
KLEINFELDER, Inc.

Rob DeArmond
Project Manager

for

Traver E. Corky Metcalf, Jr., P.E.
Area Manager

RAD:TEM:nec



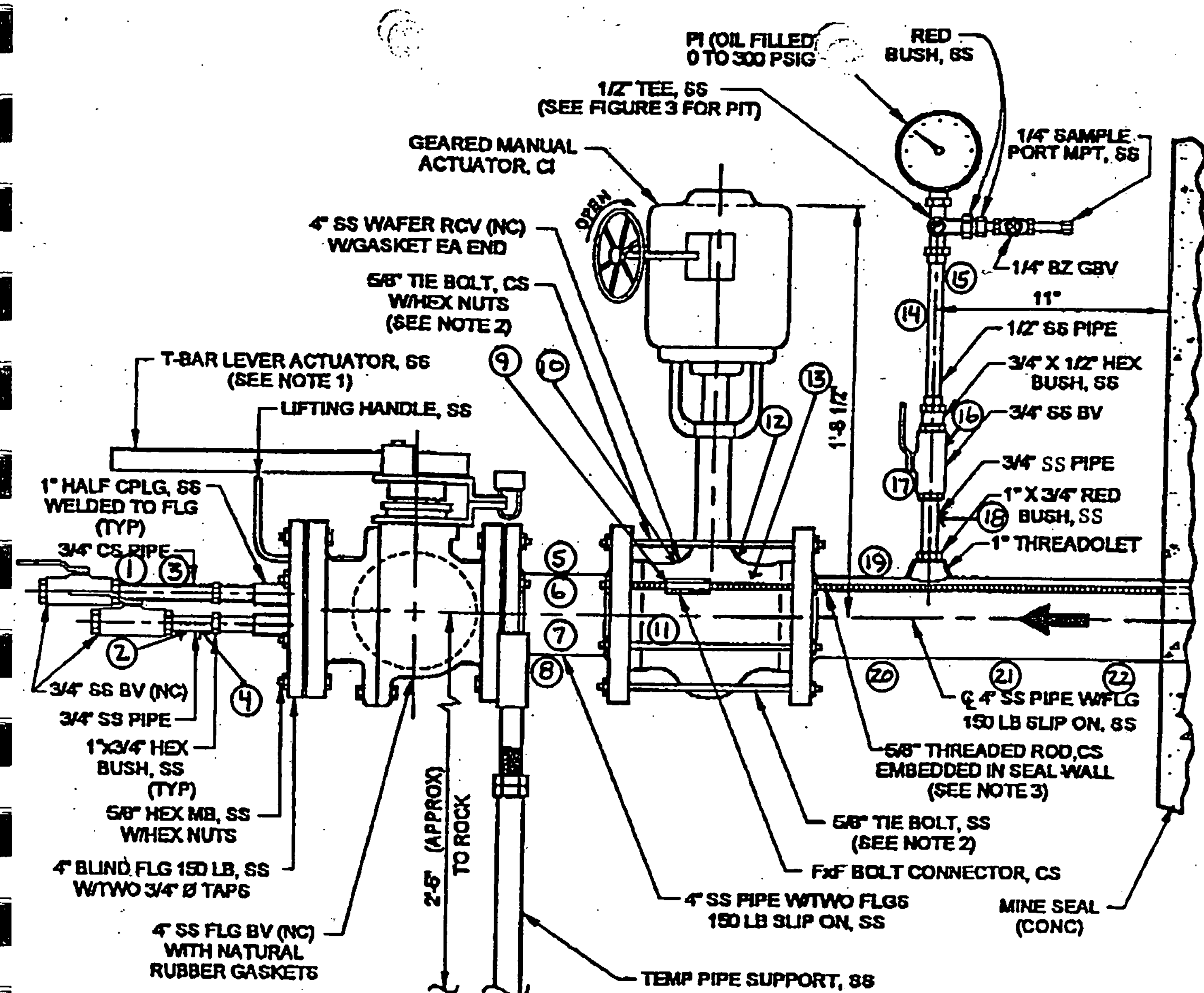
NOTES

1. SEE FIGURE 4 FOR LEGEND OF ABBREVIATIONS.

0 6 12

1 1/2" = 12"

Regional Water Quality Control Board, Central Valley Region Washington Group International, Inc.	Walker Mine Seal Testing & Evaluation Portola, CA	4-INCH DRAIN PIPES PLAN (AFTER MODIFICATIONS)
GEI Consultants, Inc.	Project 00387	June 2001 | Figure 1



LEGEND

BZ	BRONZE	BV	BALL VALVE
CI	CAST IRON	GBV	GLOBE VALVE
CS	CARBON STEEL	RCV	ROTARY CONTROL VALVE
GS	GALVANIZED STEEL	PI	PRESSURE INDICATOR
SS	STAINLESS STEEL	NC	NORMALLY CLOSED
BUSH	BUSHING	NO	NORMALLY OPEN
FLG	FLANGE	PIT	PRESSURE INDICATOR TRANSMITTER
CONC	CONCRETE	MB	MACHINE BOLT
F	FEMALE	EA	EACH
M	MALE	MPT	MALE PIPE THREADS
THK	THICK	RED	REDUCING

NOTES

1. ACTUATOR LOCKED IN CLOSED POSITION WITH PADLOCK.
2. SOME FLANGE FASTENERS ARE CARBON STEEL.
3. EMBEDDED ROD POSITION VARIES. RIGHT SIDE ROD ON EAST PIPE IS ABOVE 4-INCH PIPE CENTERLINE. ALL OTHERS ARE BELOW PIPE CENTERLINE.

0 6 12

1 1/2" = 12"

Regional Water Quality Control Board, Central Valley Region Washington Group International, Inc. GEI Consultants, Inc.	Walker Mine Seal Testing & Evaluation Portola, CA	EAST (RIGHT) DRAIN PIPE ELEVATION A-A (AFTER MODIFICATIONS)
	Project 00387	June 2001 — Figure 2

Appendix C

Appendix C

Product Information for Valves, Piping and Fittings

C.1 Rotary Control Valves (Leslie Controls)

C.2 Manual Gear Actuators (DeZurik)

C.3 Flanged Ball Valve (Apollo)

C.4 Pressure Transducer (Druck)

C.5 Pressure Gauge (Ashcroft)

C.6 Stainless Steel Information (DeZurik, Enduro Stainless Steel, Allegheny Ludlum Steel Corp.)

C.7 Piping Information (Crane)

C.8 Fittings (Flowline Corp., Grinnell)

Appendix C.1

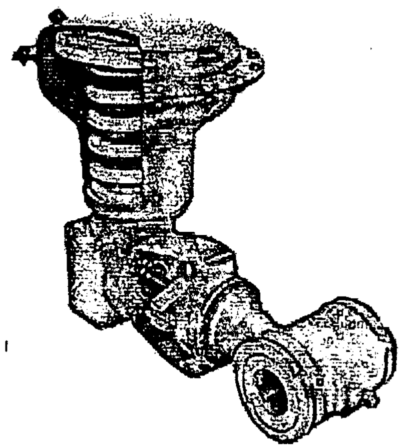
Rotary Control Valves
(Leslie Controls)

Rotary Control Valves

Class GKM Rotary Control Valves

- 1 - 8" Valve Sizes
- ANSI 150 - 600
- Body Mat'l: WCB, CF8M, Exotic Alloys
- Cv Range: 0.25 - 800
- End Conn.: RFF, Wafer

The Leslie K-Max is designed and engineered to handle nearly all industrial process control requirements. For over 15 years, the K-Max has been in successful service in thousands of applications worldwide including high and low pressure steam, clean, dirty, and corrosive liquids and gases, and erosive and abrasive slurries.



Superior Features

Other features include reduced port trim options, class V shutoff option, Alloy 6® trim hardening option, bi-directional flow capability (while maintaining shut-off class in either direction), and triple bearing large diameter precision splined shaft for tortional and flexural rigidity with excellent radial support.

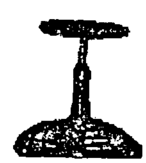
Greater Efficiency

Efficient straight-through flow design allows for a much lower cost per Cv than conventional globe style valves with the same degree of quality people have come to expect from Leslie.

RVK and RVB Three-Way Rotary Valves

- 4 - 16" Valve Sizes
- ANSI CL 125 - 150, DIN PN10 - 16
- Cv Range: 145 - 3500
- End Conn.: RF
- Body Mat'l: Cast Iron, Ductile Iron, Bronze

These relatively low-cost valves are designed for bypass temperature control systems in industrial, utility and marine applications. They require very low operating forces and can be used with smaller actuators, further reducing system cost. Leslie rotary valves allow nearly twice the overall flow



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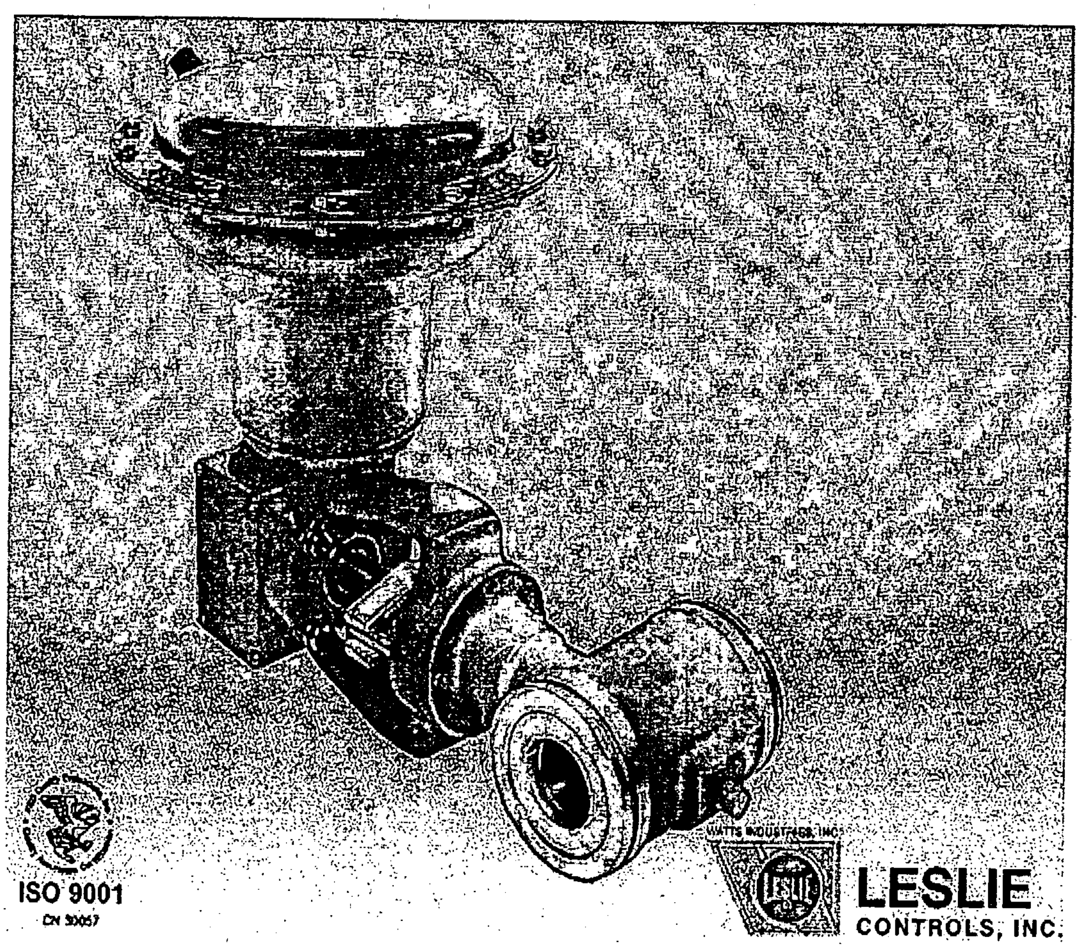
Rev. 1

PRODUCT DATA BULLETIN

K-MAX
ROTARY CONTROL VALVE
ANSI 150-600 CLASS



Contents/Hyperlinks

The K-Max Control Valve

Return to Contents

The K-Max is a high performance rotary plug control valve designed and engineered to handle nearly all industrial process control requirements. For over 15 years, the K-Max valve has been successfully applied on thousands of applications worldwide including:

- High and low pressure steam
- Clean, dirty, and corrosive gas
- Clean, dirty, and corrosive liquids
- Erosive and abrasive slurries

Economical Design
The inherent versatility of the K-Max offers the advantage of using one valve style for many applications, allowing for plant standardization and minimal stocking requirements. In addition, the efficient straight-through flow design allows for a much lower cost per Cv than conventional globe style control valves.

Choice of End Connections
The K-Max is offered in flangeless, separable flanged and integral flanged body styles.

Eccentric Rotary Plug Action
The K-Max plug is offset to the shaft centerline. This allows the plug to break free of the seat ring immediately upon initial rotation of the shaft. Since there is no sliding contact between the plug and the seat ring throughout travel, seat ring life and shut-off integrity are greatly enhanced.

Self-aligning Orbital Seat Ring
This innovative design allows orbital movement of the seat ring to provide self-alignment with the plug at assembly. Once seat ring to plug alignment is made, the seat is locked in place by the seat ring retainer. The seat ring and plug rigidly mate with every closure of the valve, maintaining excellent shut-off capability.

Material Offerings

A wide selection of body and trim materials is readily available to assure compatibility with even the most highly corrosive fluids. Materials offered range from carbon steel and stainless steels to the high alloys including Hastelloy and Titanium.

Trim Options
The K-Max offers full and reduced port trim options for all body sizes. Trim size changes are easily accomplished in the field by replacing only the seat ring. The ability to match valve Cv with the required application Cv provides exceptional control of the process media.

Rangeability
Rangeability of the K-Max valve is 100:1, allowing precise throttling over a wide range of flows.

Flow Characteristic
The inherent flow characteristic of the K-Max valve is linear and can be modified to equal percentage with a simple positioner cam adjustment.

Bi-directional Flow Capability
The normal flow direction for clean liquids, gases and steam is flow to open (flow into the face of the plug). The recommended flow direction for erosive and slurry service is flow to close (flow into the backside of the plug). Shut-off class is maintained in either flow direction.

Hardened Trim Availability
Hardened trims utilizing Alloy 6 are readily available for all sizes of the K-Max valve. Alloy 6 provides increased trim life due to its erosion resistance and high strength at elevated temperatures.

Shaft Features

- Large shaft diameters machined of high strength materials provide the torsional and flexural rigidity required under high operating pressures.
- Three bearings provide the shaft with the radial support required for smooth throttling control.
- All shaft connections are precision splined to assure minimal lost motion.
- A shaft access plug provides easy removal of the valve shaft during maintenance.

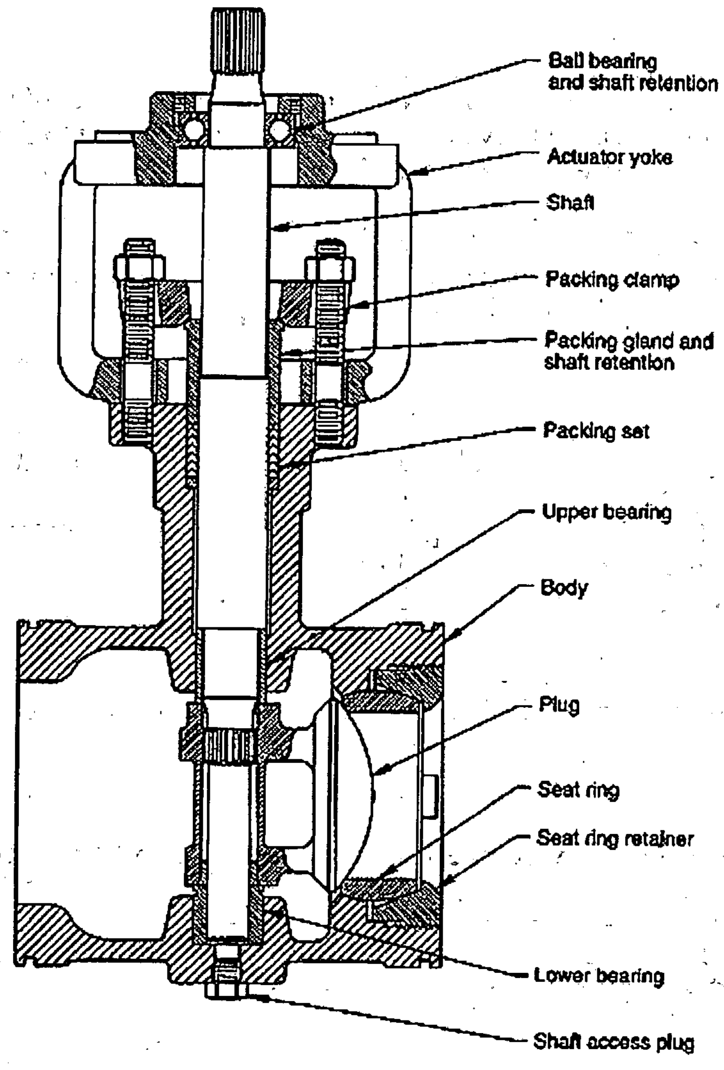
Slurry Trim Construction
As an option for slurry applications, bearing seals are available to provide protection from abrasive particles entering the bearing area.

NACE Construction
When specified as an option, the K-Max rotary control valve meets the requirements of NACE MR1075 providing resistance to sulfide stress cracking.

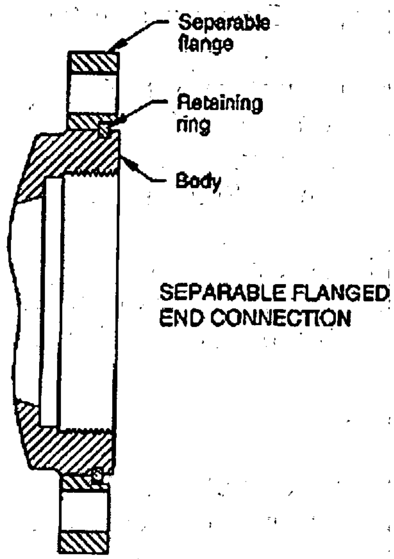
K-Max Rotary Control Valve

The K-Max high performance rotary control valve is available in flangeless, separable flanged and integral flanged end connections.

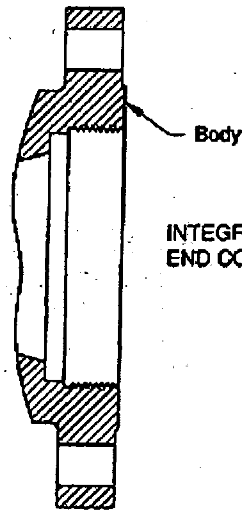
- Flangeless end connections provide added economy in materials savings and versatility of end connection installations.
- Separable flanged end connections utilize one common body which can be fitted with different flange class ratings. In addition, when high alloy flanged bodies are required, non-wetted separable flanges are provided in economical carbon steel or stainless steel.
- Integral flanged end connections are available when required by process conditions or piping specifications.



FLANGELESS END CONNECTION



SEPARABLE FLANGED END CONNECTION



INTEGRAL FLANGED END CONNECTION

K-Max Valve Technical Specifications

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Valve Style:
High performance eccentric rotary plug control valve.

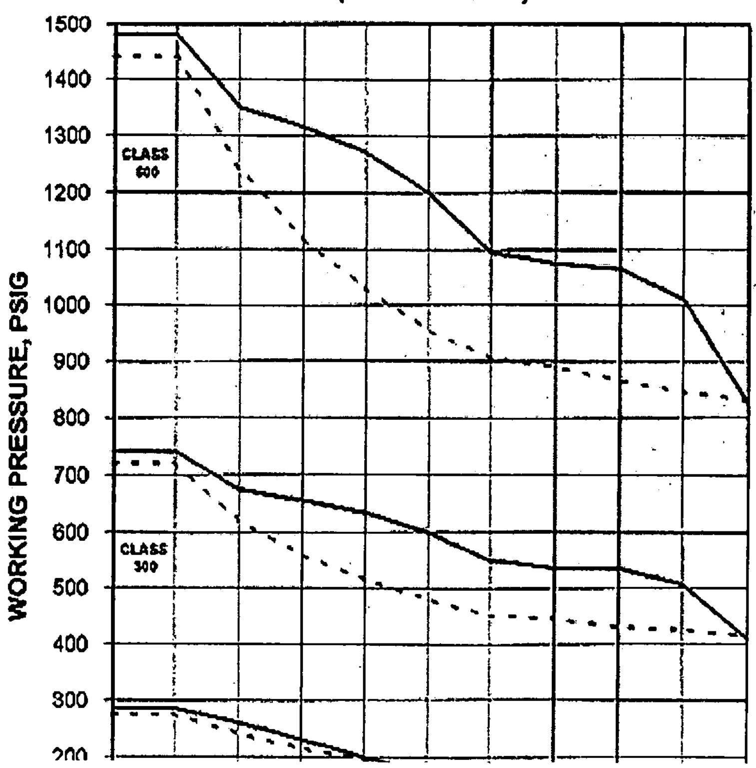
Valve Size:
Sizes 1" through 8" (25mm-200mm) with full or reduced port trim.

End Connection:
Flangeless ANSI class 150, 300 or 600, sizes 1" - 8".
Separable flanged ANSI class 150 or 300, sizes 1" - 6".
Integral flanged ANSI class 150, 300 or 600, sizes 1" - 8".

Note: Serrated raised face flanges are standard. Smooth raised face flanges, DIN and JIS flanges, available on application.

Body Material:
Carbon steel, ASTM A216 grade WCB
316 stainless steel, ASTM A351 grade CF8M
317 stainless steel, ASTM A743 grade CG8M
Alloy 20 Cb3, ASTM A351 grade CN7M
Hastelloy C4C, ASTM A494 grade CW2M
Titanium C3, ASTM B367 grade C-3

(PER ANSI B16.34)



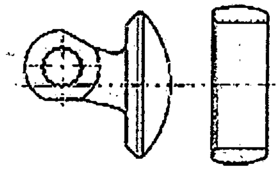
- - - - - Stainless Steel CF8M
-------- Carbon Steel WCB

Packing:

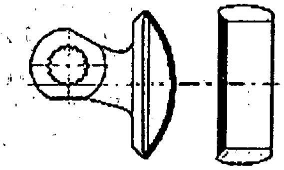
Packing Type	Temperature Range
PTFE V-Ring	-40°F through +450°F
Carbon graphite	-300°F through +800°F
Kalrez V-Ring	-40°F through +450°F

Trim Material:
See K-Max valve material specifications on page 5.
Note: Other trim combinations available on application.

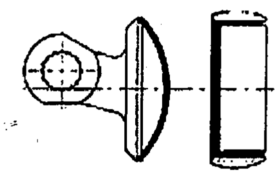
Alloy 6 Trim Options:



No Alloy 6
No Alloy 6 on seat ring or plug.



Partial Alloy 6
Alloy 6 on seat ring and plug seating surfaces.



Full Alloy 6
Alloy 6 on seat ring bore in addition to seat ring and plug seating surfaces.

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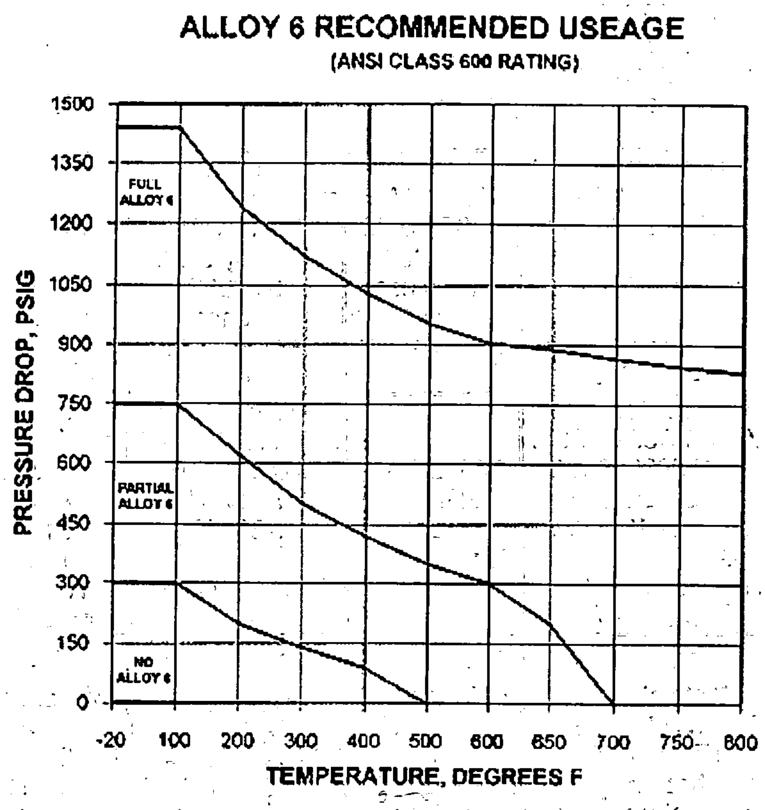

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Trim Material-Alloy 6 Code	Body Material	Plug Material	Seat Ring Material: size 2" - 8" full trim valves	Seat Ring Material: size 1" - 1.5" full trim valves and size 1" - 8" reduced trim valves	Shaft Material	Bearing Material
S2 - N	Carbon steel ASTM A216 WCB UNS J03002	316 stainless steel ASTM A351 CF8M UNS J92900 with hardened electroless nickel coating	316 stainless steel ASTM A351 CF8M UNS J92900 Hardness Brinell 150	316 stainless steel ASTM A479 316 UNS S31600 Hardness Brinell 150	17-4 PH stainless steel ASTM A564 S17400 UNS S17400 condition H900 Hardness Rockwell C 40	440C stainless steel ASTM A276 S44004 UNS S44004 Hardness Rockwell C 58
	316 stainless	316 stainless steel ASTM A351 CF8M	316 Stainless Steel ASTM	316 stainless steel ASTM A479	17-4 PH stainless steel ASTM A564	317 stainless steel ASTM

	steel ASTM A351 CF8M UNS J92900	UNS J92900 with hardened electroless nickel coating	A351 CF8M UNS J92900 Hardness Brinell 150	316 UNS S31600 Hardness Brinell 150	S17400 UNS S17400 condition H900 Hardness Rockwell C 40	A276 317 UNS S31700 with carbon graphite liner
S2 - P or S2 - F	Carbon steel ASTM A216 WCB UNS J03002	316 stainless steel ASTM A351 CF8M UNS J92900 with Alloy 6 hard overlay AWS A5.13 RCoCr - A UNS R30006 Hardness Rockwell C 38 - 47	Alloy 6 AMS 5387B UNS R30006 Hardness Rockwell C 37 - 41	316 stainless steel ASTM A479 316 UNS S31600 with Alloy 6 UNS R30006 Hardness Rockwell C 38 - 47	17-4 PH stainless steel ASTM A564 S17400 UNS S17400 condition H900 Hardness Rockwell C 40	440C stainless steel ASTM A276 S44004 UNS S44004 Hardness Rockwell C 58
	316 stainless steel ASTM A351 CF8M UNS J92900	316 stainless steel ASTM A351 CF8M UNS J92900 with Alloy 6 hard overlay AWS A5.13 RCoCr - A UNS R30006 Hardness Rockwell C 38 - 47	Alloy 6 AMS 5387B UNS R30006 Hardness Rockwell C 37 - 41	316 stainless steel ASTM A479 316 UNS S31600 with Alloy 6 UNS R30006 Hardness Rockwell C 38 - 47	17-4 PH stainless steel ASTM A564 S17400 UNS S17400 condition H900 Hardness Rockwell C 40	Alloy 6 AMS 5387B UNS R30006 Hardness Rockwell C 37 - 41
S3 - N	317 stainless steel ASTM A743 CG8M UNS J93000 or 316 stainless steel ASTM A351 CF8M UNS J92900	317 stainless steel ASTM A743 CG8M UNS J93000 with hardened electroless nickel coating	317 stainless steel ASTM A743 CG8M UNS J93000 Hardness Brinell 160	317 stainless steel ASTM A276 317 UNS S31700 Hardness Brinell 200 maximum	317 stainless steel ASTM A276 317 UNS S31700 Hardness Brinell 200 maximum	317 stainless steel ASTM A276 317 UNS S31700 with carbon graphite liner
S3 - P or S3 - F	317 stainless steel ASTM A743 CG8M UNS J93000 or 316 stainless steel ASTM A351 CF8M UNS J92900	317 stainless steel ASTM A743 CG8M UNS J93000 with Alloy 6 hard overlay AWS A5.13 RCoCr - A UNS R30006 Hardness Rockwell C 38 - 47	Alloy 6 AMS 5387B UNS R30006 Hardness Rockwell C 37 - 41	317 stainless steel ASTM A276 317 UNS S31700 with Alloy 6 UNS R30006 Hardness Rockwell C 38 - 47	317 stainless steel ASTM A276 317 UNS S31700 Hardness Brinell 200 maximum	Alloy 6 AMS 5387B UNS R30006 Hardness Rockwell C 37 - 41
AA - N	Alloy 20 ASTM A351 CN7M UNS N08007	Alloy 20 ASTM A351 CN7M UNS N08007 Hardness Brinell 130	Alloy 20 ASTM A351 CN7M UNS N08007 Hardness Brinell 130	Alloy 20 Cb3 ASTM B473 N08020 UNS N08020 Hardness Brinell 183	Titanium 5 ASTM B348 Grade 5 UNS R56400 Hardness Rockwell C 36	Hastelloy ASTM B574 N10276 UNS N10276 Hardness Brinell 184
HC - N	Hastelloy C4C ASTM A494 CW2M	Hastelloy C4C ASTM A494 CW2M Hardness Brinell 200	Hastelloy C4C ASTM A494 CW2M Hardness Brinell 200	Hastelloy C276 ASTM B574 N10276 UNS N10276 Hardness Brinell 184	Titanium 5 ASTM B348 Grade 5 UNS R56400 Hardness Rockwell C 36	Hastelloy ASTM B574 N10276 UNS N10276 Hardness Brinell 184
T3 - N	Titanium C3 ASTM B367 C - 3 UNS R50550	Titanium C3 ASTM B367 C -3 UNS R50550 Hardness Brinell 235 maximum	Titanium C3 ASTM B367 C -3 UNS R50550 Hardness Brinell 235 maximum	Titanium 5 ASTM B348 Grade 5 UNS R56400 Hardness Rockwell C 36	Titanium 5 ASTM B348 Grade 5 UNS R56400 Hardness Rockwell C 36	Ceramic Partially stabilized zirconium Grade MS

Note: Seat ring retainer material is the same as the base plug material.

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K-Max Valve Technical Specifications

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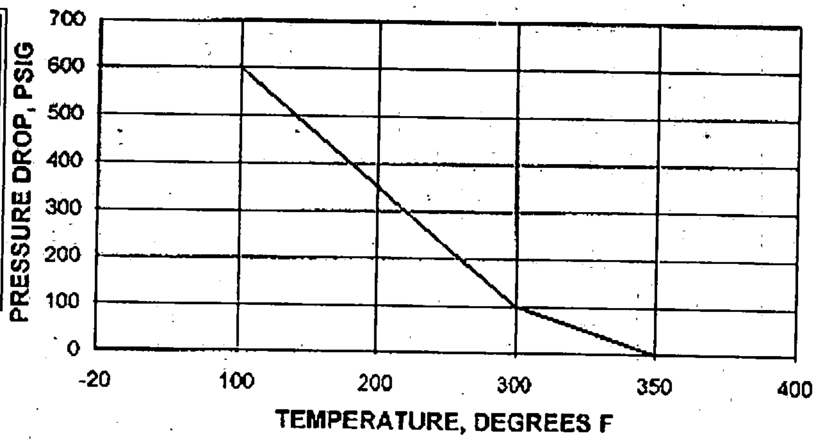
Seat Seal

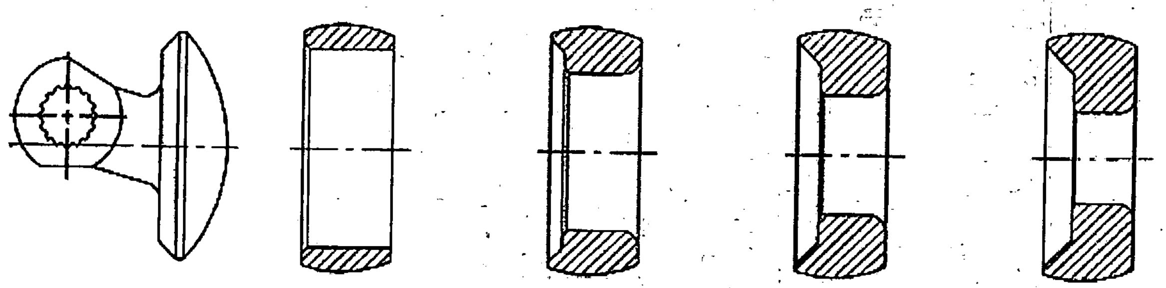
Metal to metal seat (standard)	- ANSI class IV
Metal to metal seat (optional)	- ANSI class V
PTFE soft seat	- ANSI class VI

PTFE SOFT SEAT



Trim Size

Full size trim - 100% capacity
.6 reduced trim - 60% of full capacity
.4 reduced trim - 40% of full capacity
.2 reduced trim - 20% of full capacity
Note: Other trim sizes available on application.



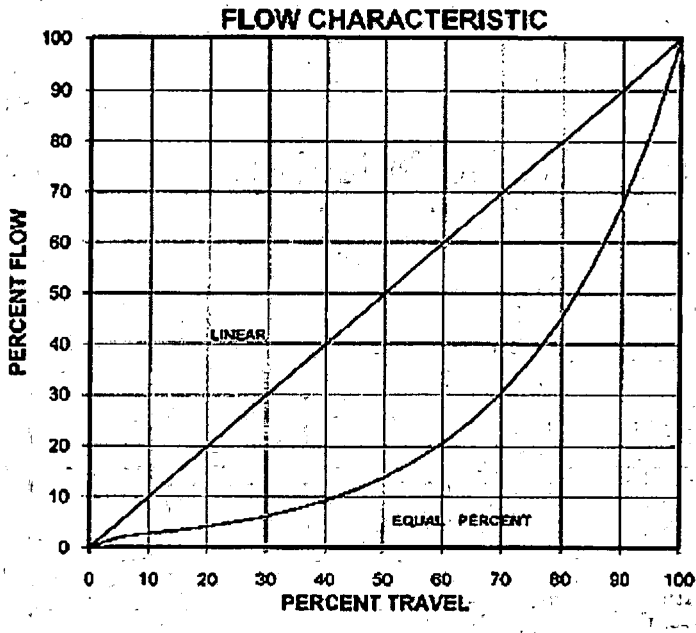
COMMON PLUG FOR ALL TRIM SIZES | FULL PORT SEAT (100% CAPACITY) | .6 REDUCED PORT SEAT (60% CAPACITY) | .4 REDUCED PORT SEAT (40% CAPACITY) | .2 REDUCED PORT SEAT (20% CAPACITY)

Options

- Fluroelastomer beraring seal for slurry devices
- Kalrez bearing seal for slurry service
- 316 stainless steel seperable flanges and retaining rings
- 316 stainless steel valve to actuator bolting

- NACE trim - conforms to NACE MR0175

The inherent flow characteristic of the K-Max valve is linear. When required, the valve travel can be modified with a positioner cam adjustment to provide equal percent flow characteristic.



Recovery Coefficients FL2 (all fluids) and Cavitation Index Kc (liquids)

Valve Opening %	Flow to open		Flow to close	
	FL2	Kc	FL2	Kc
100	0.77	0.6	0.56	0.56
90	0.79	0.61	0.55	0.49
80	0.8	0.62	0.54	0.49
70	0.81	0.63	0.54	0.49
60	0.8	0.62	0.57	0.5
50	0.79	0.61	0.61	0.51
40	0.78	0.61	0.64	0.52
30	0.77	0.6	0.67	0.53
20	0.76	0.59	0.7	0.55
10	0.76	0.59	0.72	0.55

Note:

For calculating the pressure drop at which cavitation will begin, ΔPc, multiply Kc by the quantity P1-Pv, where P1= upstream pressure (PSIA), and Pv= vapor pressure (PSIA). $\Delta Pc = Kc\,(P1-Pv)$.

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K-Max Valve Flow Capacity Specifications

Flow Coefficients (Cv), Linear Characteristic

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Flow to Open		Percent Travel (60° Rotation)									
Valve Size	Trim Size	10%	20%	30%	40%	50%	60%	70%	80%	90%	100%
1"	Full	1.3	2.8	4.2	5.9	7.8	9.1	11	12	13	14
	.6 reduced	0.76	1.6	2.5	3.5	4.8	5.5	6.4	7.4	7.9	8.4
	.4 reduced	0.50	1.1	1.7	2.4	3.2	3.7	4.3	4.9	5.3	5.6
	.2 reduced	0.25	0.53	0.84	1.2	1.6	1.8	2.1	2.5	2.6	2.8
1.5"	Full	2.9	6.1	9.6	13	18	21	24	28	30	32
	.6 reduced	2.7	5.7	9.0	11	12	13	15	17	18	20
	.4 reduced	1.2	2.5	3.9	5.5	7.4	8.6	9.8	11	12	13
	.2 reduced	0.59	1.2	2.0	2.8	3.7	4.3	4.9	5.7	6.1	6.5
2"	Full	4.6	9.7	15	21	29	34	39	45	48	51
	.6 reduced	2.7	5.7	9.0	13	17	20	23	26	28	30
	.4 reduced	1.8	3.8	6.0	8.4	11	13	15	18	19	20
	.2 reduced	0.90	1.9	3.0	4.2	5.7	6.6	7.6	8.8	9.4	10
3"	Full	14	29	45	63	86	99	114	132	141	150
	.6 reduced	8.1	17	27	38	51	59	68	79	85	90
	.4 reduced	5.4	11	18	25	34	40	46	53	56	60
	.2 reduced	2.7	5.7	9.0	13	17	20	23	26	28	30
4"	Full	22	47	74	104	141	163	187	217	232	247
	.6 reduced	13	28	44	62	84	97	112	129	138	147
	.4 reduced	8.8	19	29	41	56	65	75	86	92	98
	.2 reduced	4.4	9.3	15	21	28	32	37	43	46	49
6"	Full	47	99	156	218	296	343	395	458	489	520
	.6 reduced	28	59	89	125	170	206	225	275	294	312
	.4 reduced	19	40	59	83	113	137	150	183	196	208
	.2 reduced	9.4	20	30	42	57	69	75	92	98	104
8"	Full	78	165	261	365	496	574	661	766	818	870

	.6 reduced	47	99	156	219	297	345	396	459	491	522
	.4 reduced	31	66	104	146	198	230	264	306	327	348
	.2 reduced	16	33	52	73	99	115	132	153	164	174

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Flow to Close		Percent Travel (60° Rotation)									
Valve Size	Trim Size	10%	20%	30%	40%	50%	60%	70%	80%	90%	100%
1"	Full	1.4	2.9	4.5	6.3	8.6	9.9	11	13	14	15
	.6 reduced	0.81	1.7	2.7	3.8	5.1	5.9	6.8	7.9	8.5	9.0
	.4 reduced	0.54	1.1	1.8	2.5	3.4	4.0	4.6	5.3	5.6	6.0
	.2 reduced	0.27	0.57	0.90	1.3	1.7	2.0	2.3	2.6	2.8	3.0
1.5"	Full	3.1	6.5	10	14	19	22	26	30	32	34
	.6 reduced	1.9	4.0	6.3	8.8	12	14	16	19	20	21
	.4 reduced	1.3	2.7	4.2	5.9	8.0	9.2	11	12	13	14
	.2 reduced	0.63	1.3	2.1	2.9	4.0	4.6	5.3	6.2	6.6	7.0
2"	Full	5.0	11	17	23	31	36	42	48	52	55
	.6 reduced	2.7	5.7	9.0	13	17	20	24	29	30	33
	.4 reduced	1.8	3.8	6.0	8.4	11	13	16	19	20	22
	.2 reduced	0.90	1.9	3.0	4.2	5.7	6.5	8.0	9.0	10	11
3"	Full	14	30	47	65	88	102	118	136	146	155
	.6 reduced	8.4	18	28	39	53	61	71	81	87	93
	.4 reduced	5.6	12	19	26	35	41	47	54	58	62
	.2 reduced	2.8	5.9	9.3	13	15	21	24	27	29	31
6"	Full	24	51	80	112	152	176	202	234	250	266
	.6 reduced	14	30	48	67	90	95	120	140	149	159
	.4 reduced	9.5	20	32	45	60	63	80	93	99	106
	.2 reduced	4.8	10	16	22	30	32	40	47	50	53
8"	Full	43	91	144	202	273	316	364	422	451	480
	.6 reduced	26	55	86	120	164	189	219	254	270	288
	.4 reduced	17	37	58	80	109	126	146	169	180	192
	.2 reduced	8.6	18	29	40	55	63	73	85	90	96
	Full	72	152	240	336	456	528	608	704	752	800
	.6 reduced	43	90	144	201	273	317	365	422	450	480
	.4 reduced	29	60	96	134	182	211	243	281	300	320
	.2 reduced	14	30	48	67	91	106	122	141	150	160

K-Max Valve Torque Specifications

Torque Requirements to Achieve ANSI Class IV, V, or VI shut off
(foot pounds)

Valve Size	Shut off Pressure Drop (PSIG)													
	30	50	100	200	300	400	500	600	700	800	900	1000	1200	1440
1"	10.8	11.0	11.5	12.2	12.9	13.8	14.6	15.4	16.2	16.9	17.8	18.6	20.2	21.8
1.5"	19.5	19.9	21.2	23.4	25.8	28.0	30.3	32.7	34.9	37.3	39.6	42.4	46.5	51.1
2"	28.7	29.5	31.4	35.3	39.3	43.3	47.3	51.3	55.2	59.2	63.1	67.4	74.9	78.8
3"	61.3	63.7	69.7	81.7	93.6	106	117	129	142	153	165	177	193	210
4"	110	116	130	160	189	218	248	277	306	336	365	394	-	-
6"	229	252	299	393	487	582	676	770	-	-	-	-	-	-
8"	397	442	552	773	994	-	-	-	-	-	-	-	-	-

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Maximum Allowable Differential Pressure (PSIG) based on Torsional Shear Strength of Shaft

Valve Size	Shaft Material			
	17-4 SST	Titanium	317 SST	Hastelloy C
1"	1440	1440	1440	1440
1.5"	1440	1440	1440	1440
2"	1440	1440	700	1440
3"	1440	1440	350	800
4"	1000	1000	30	175
6"	1000	1000	200	400
8"	550	550	30	100

Note: Hastalloy C shaft material on Application.

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Maximum Allowable Shaft Torques
(foot pounds)

Valve Size	Shaft Material			
	17-4 SST	Titanium	317 SST	Hastelloy C
1"	210	210	55	80
1.5"	210	210	55	80

2"	210	210	55	80
3"	390	390	100	150
4"	390	390	100	150
6"	1550	1550	405	550
8"	1550	1550	405	550

Note: Hastalloy C shaft material on Application.

Diaphragm Actuator Output Torques

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Actuator Size	Actuator Action	Actuator Spring	Output Torque(Ft-lbs)
40	Fail closed	20 psi	31
	Fail open	20 psi	19
	Fail closed	35 psi	58
	Fail open	35 psi	58
	Fail closed	60 psi	95
	Fail open	60 psi	102
55	Fail closed	20 psi	88
	Fail open	20 psi	76
	Fail closed	35 psi	152
	Fail open	35 psi	124
	Fail closed	60 psi	263
	Fail open	60 psi	224
85	Fail closed	20 psi	130
	Fail open	20 psi	130
	Fail closed	35 psi	220
	Fail open	35 psi	221
	Fail closed	60 psi	389
	Fail open	60 psi	389
145	Both actions	20 psi	264
	Both actions	35 psi	487
	Both actions	60 psi	867
250	Both actions	20 psi	451
	Both actions	35 psi	842
	Both actions	60 psi	1497

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K-Max Valve Dimensional Data

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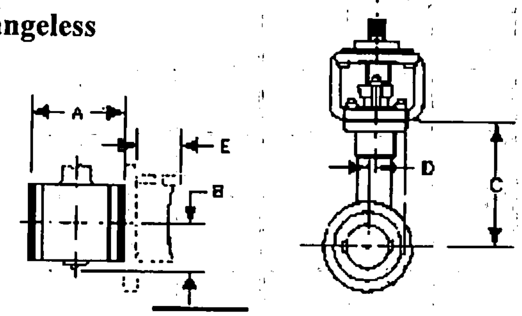
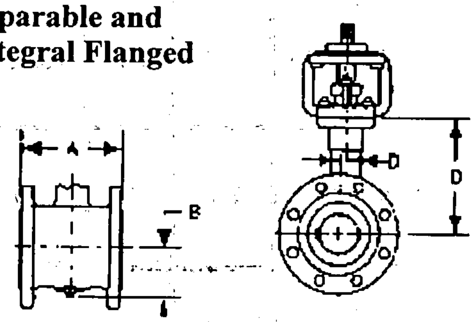
Flangeless



Separable and Integral Flanged



Valve Size		Dimensions (inches/millimeters)												
Inches	MM	A	B	C	D	E	F	G	H	J	K	L	M	N
1		4	2.44	4.44	0.16	7.62	1.00	1.5	1.06	N/A	0.94	N/A	0.44	3.25
	25	102	62.0	113	4.06	194	25.4	38.1	26.9		23.9		11.2	82.6
1.5		4.5	2.75	5.12	0.25	8.75	1.00	1.5	1.06	N/A	0.94	N/A	0.44	3.25
	40	114	69.8	130	6.35	222	25.4	38.1	26.9		23.9		11.2	82.6
2		4.88	2.81	4.88	0.22	9.12	1.00	1.5	1.06	N/A	0.94	N/A	0.44	3.25

	50	124	71.4	124	5.59	232	25.4	38.1	26.9		23.9		11.2	82.6
3		6.50	3.56	5.75	0.31	11.50	1.25	2.44	1.19	0.88	0.75	4.995	0.56	6.50
	80	165	90.4	146	7.87	292	31.8	62.0	30.2	22.4	19	126.9	14.2	165
4		7.62	4.03	7.00	0.44	13.38	1.25	2.44	1.19	0.88	0.75	4.995	0.56	6.50
	100	194	102	178	11.2	340	31.8	62.0	30.2	22.4	19	126.9	14.2	165
6		9.00	5.06	9.59	0.66	15.75	1.75	2.31	0.94	0.44	0.66	4.995	0.56	6.50
	150	229	129	244	16.8	400	44.4	58.7	23.9	11.2	16.8	126.9	14.2	165
8		9.56	6.00	11.00	0.88	17.75	1.75	2.31	0.94	0.44	0.66	4.995	0.56	6.50
	200	243	152	279	22.4	438	44.4	58.7	23.9	11.2	16.8	126.9	14.2	165

Note:
1. All dimension are subject to change without notice. Request certified drawings for use in preparing piping layouts.
2. Flange dimensions conform to ANSI B16.5.
3. Face-to-face dimensions conform to ISA S75.04.

External Involute Spline Data Table Fillet Root Side Fit

(Dimensions in inches)

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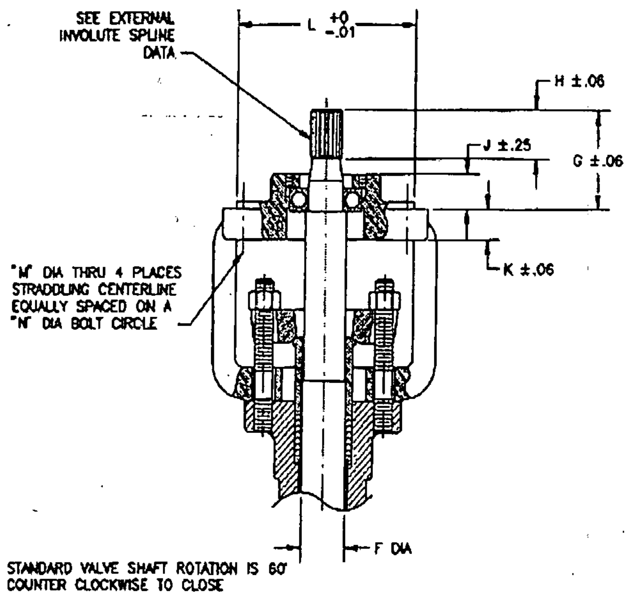
Feature	Valve Size

	1, 1.5 & 2	3 & 4	6 & 8
Number of teeth	28	28	52
Pitch	40/80	32/64	40/80
Pressure angle	30°	30°	30°
Base diameter	0.6062	0.7578	1.1258
Pitch diameter	0.7000	0.8750	1.3
Major diameter	0.725/0.722	0.906/0.903	1.325/1.322
Form diameter	0.671	0.839	1.271
Minor diameter	0.638	0.800	1.236
Circular Tooth Thickness			
Max effective	0.0393	0.0491	0.0393
Min actual	0.0366	0.0464	0.0361



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Appendix C.2

Manual Gear Actuators

- 2/08/02 Memo from C. Fox (Washington Group) to A. Pujol (GEI)
- 02/04/02 Telephone conversation record from C. Fox (Washington Group) to J. Contant (Bay Valve)
- Photograph of Actuator
- Leslie Controls, Inc. diagram for the K-Max-Actuator Parts for the 55 and 85 Actuator
- Leslie Controls, Inc. Class K RVC valve parts information
- 05/11/01 Telephone conversation record from C. Fox (Washington Group) to J. Ketchen (CFM Controls)
- DeZurik Manual Gear Actuator

WALKER MINE DRAIN PIPING

Memo

RE: Walker Mine W. O. # 23480-03
From: Cynthia Fox, Washington Group International, Inc.
To: Alberto Pujol, GEI Consultants, Inc.
Date: 02/08/02

<u>Valve Actuators</u>

In the report, we recommended that the RWQCB protect and service the manual, geared, valve actuators on the drain valves to prevent them from becoming unserviceable in future years. The RWQCB wants to know in more detail what they should do.

The valve actuators should be protected from the constant dripping of acidic water. The actuators could be cleaned of deposits using a non-abrasive methods, such as a plastic scrub brush or pad, that will not damage the paint. The actuators could then be dried and covered with a removable plastic tent. The "tent" can be simple such as a heavy-duty PVC garbage bag, although the plastic needs to be restrained using elastic cord, waterproof tape, etc. to prevent puddling water from dragging the thin material down.. A more rigid "tent" might be a fabricated from a 2-foot-long section of plastic pipe such as HDPE pressure pipe with an inside diameter greater than about 14 inches. An end cap or plate could be welded to one end the rim of the open end could be notched to rest on the main drain pipe. An off-the-shelf item could be modified to make a serviceable tent, such as a plastic garbage can, so long as it was waterproof, acid resistant, and light weight.

The company Bay Valve Service & Engineering was contacted regarding the long-term maintenance requirements of the valve actuators. See the attached 2/4/02 Telecom C. Fox (WGINT) to Joe Contant (Bay Valve). J. Contant, (Bay Valve) stated that with the proper care and maintenance, the valve actuators could operate indefinitely. As with the valves, it is a good idea to exercise the actuators periodically. J. Contant stated that the typical geared actuator has a grease-filled case, and that, after years, the grease can get old and hard, especially in a cold wet environment. He felt that eventually, the actuators might not operate easily and smoothly and at that point the actuators would need to be removed from the mine, refurbished, and the grease replaced.

Typically, an actuator of this type can be removed by unscrewing the four screw bolts that thread upwards through the top plate of the yoke into the bottom of the actuator. (See attached photograph of actuator.) When the actuator is unbolted, it can then be slid up off the splined valve stem and removed. Consequently, the actuators, yokes, and bolting should be monitored for corrosion, and the actuators' screw bolts should be replaced before the bolting connections become frozen from corrosion. J. Contant suggested that, if the existing screw bolts are carbon steel, they might be replaced with stainless steel.

(The actuator screw bolt appears to be a hex head, zinc-plated, carbon steel screw bolt, similar to the screw bolt labeled as SCR HX HD 1/2-13 x 1-1/2_ZP. See Part B34 on the Leslie Controls, Inc. diagram for the *K-MAX – Actuator Parts for the 55 and 85 Actuator*, also attached. Note that the exact installed part is not known and the screw bolt dimensions may differ on the installed actuators. The actuator mounting yoke holds the valve's packing gland in place and should not be loosened or removed. Leslie Controls Class K RVC valve information shows the valve components at the packing gland on pages A 11-A 12, also attached.)

WALKER MINE DRAIN PIPING

Telephone Conversation Record

RE: Walker Mine Drain Piping Maintenance - Valve Actuators
Date: 02/04/02
To: Joe Contant, (925) 228-0665 at Bay Valve Service & Engineering Company (referenced by Jay Ketchen, of CFM Controls, the valve representative for DeZurik Valves)
Jcontant@Bay-Valve.com
From: Cynthia Fox, (415) 442-7555 at Washington Group International, Inc (for Alberto Pujol, GEI Consultants, Inc.)
Cynthia.Fox@WGINT.com

I called Joe Contant to discuss the proper maintenance and service requirements needed to preserve the useful life of the geared manual actuators mounted on the 4-inch K-Max shutoff valves in the Walker Mine. (Bay Valve is in the business of refurbishing valves and actuators with services that include rebuilding, checking for cracks and wear, paint and corrosion removal and recoating with primer, paint, or any specified coatings. Joe has about 25 years of field service and now is Bay-Valves' plant service representative.) I explained that we did not have documentation or nameplate information for the Mine 4-inch shutoff valve actuators, but that they appeared to be standard, painted cast iron , worm-gear, manual actuators with 6-inch hand wheels. I added that the actuators were now more than 13 years old and that they might be difficult to replace for the following reasons:

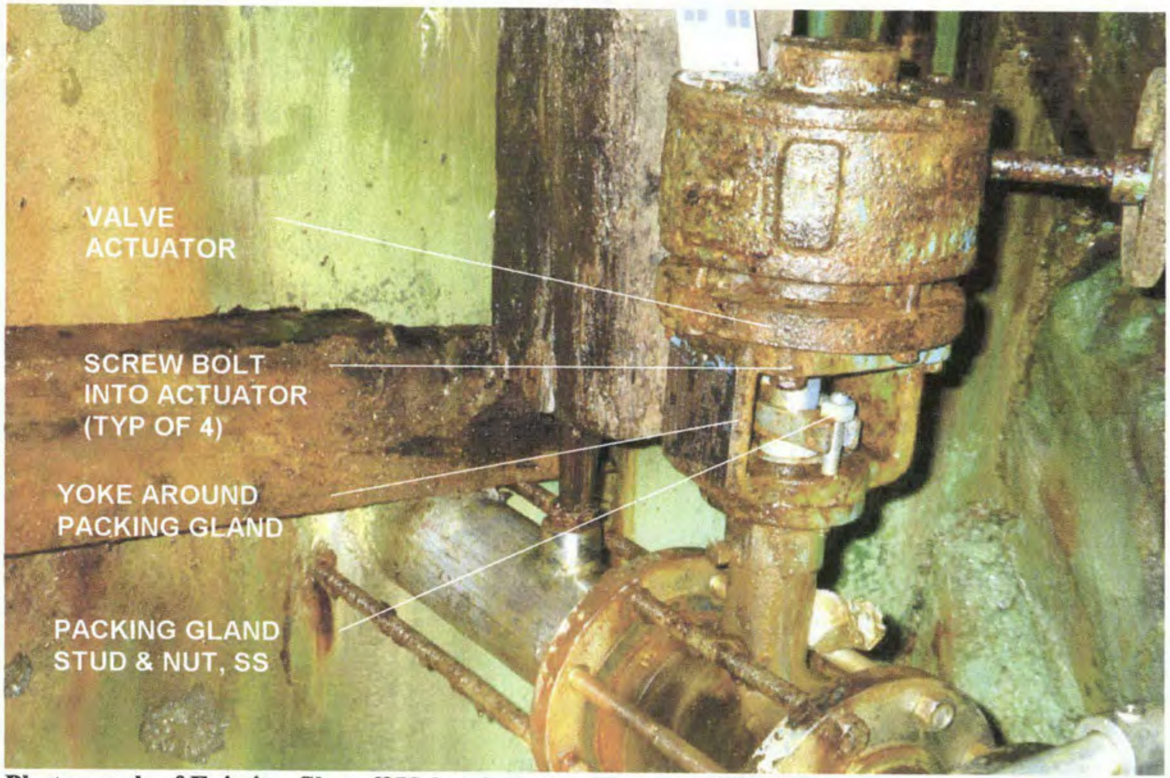
- The shutoff valves are special control valves with a valve stem that rotates clockwise to open. They are almost always fitted with a pneumatic or electric actuator.
- The valve has a tapered spline at the top of the valve stem, rather than a standard square nut-type valve stem.
- the actuators were fitted more than 13 years ago and bolt spacing and mount dimensions for actuator connections have been standardized (ISO bolting) since the mine valves were installed. It would need to be verified that a standard actuator could be mounted directly on the existing valves without modifications.

I sent Joe three photos of the Mine piping via E-mail so that we could discuss the environment with respect to the maintenance of the actuators. I mentioned that the actuators look corroded in the photos because they are covered with oxide deposits from the dripping water, but that, in fact, the cases are not corroded and the paint is still fairly good condition. I also said that the actuators had recently been operated and that they worked smoothly and easily. I asked Joe what was the useful lifespan we could expect from these actuators, and what components may require maintenance.

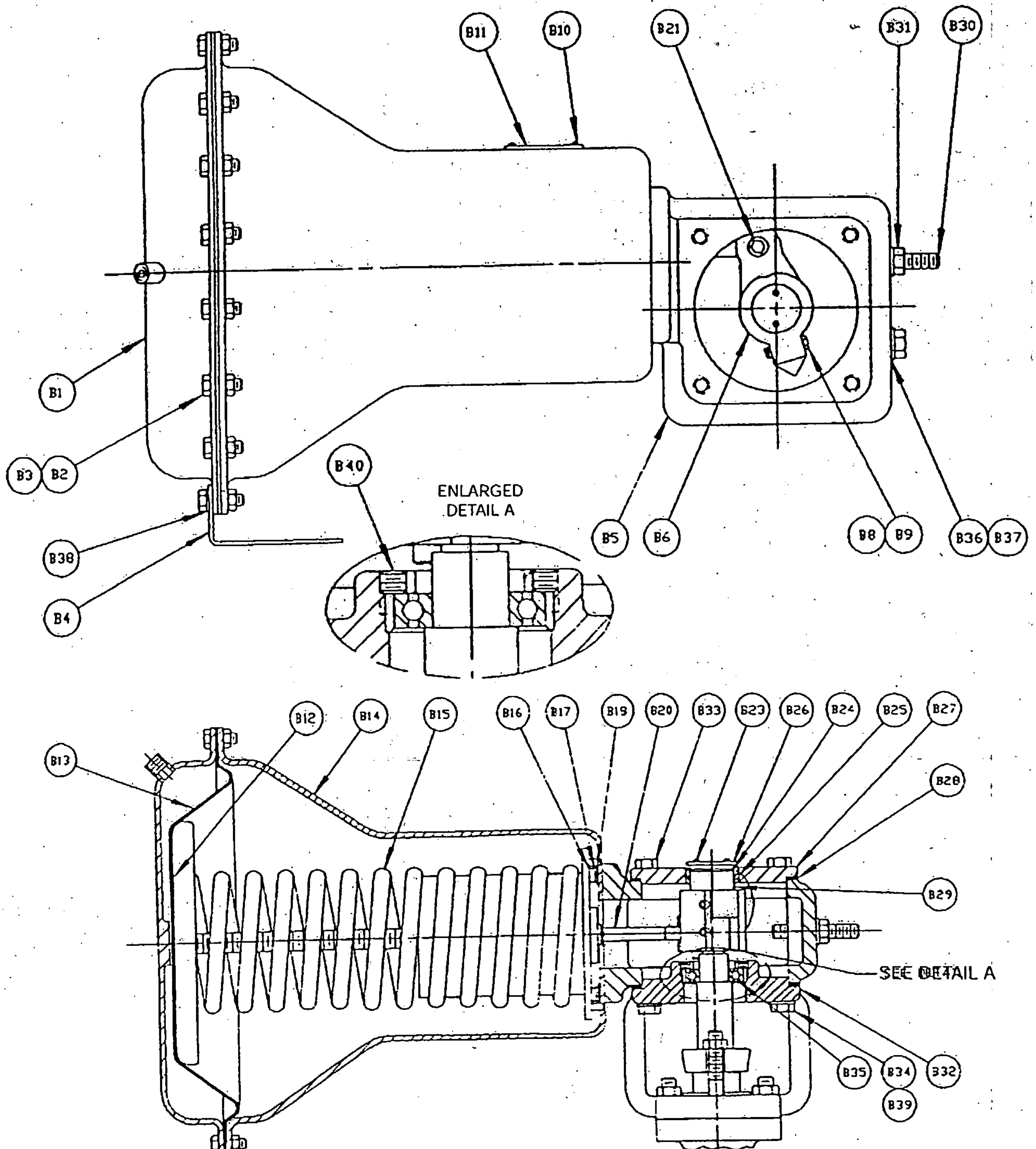
Joe said that with the proper care and maintenance, these actuators could last indefinitely. Like the valves, he indicated that it is a good idea to exercise the actuator gears periodically. He said these were probably grease-filled cases, and that the grease could get old and hard in the cold wet environment. He felt that eventually, the actuators might need to be refurbished and the grease replaced.

We discussed how to remove the actuator from the valve. I mentioned that there is a harp-shaped bracket (yoke) between the valve and the actuator. He said that it was likely that the actuator can be removed just by unscrewing the four screw bolts that thread upwards through the top plate of the yoke into the bottom of the actuator. (See photograph of actuator.) When unbolted, he expects that the actuator can then be slid up off the valve shaft and removed. Joe agreed that the actuators should be monitored for corrosion, and that the actuators' bolting should be replaced before the bolt connections become frozen from corrosion. He suggested that, if the existing bolting is carbon steel, it might be replaced with stainless steel.

Joe said that if the actuators needed to be serviced, they could be removed and delivered to Bay Valve in Martinez, CA. At Bay Valve, the actuators would be disassembled, examined for cracks and wear, rebuilt, refilled with a modern grease product, grit-blasted to white metal, primed and painted. The owner can specify or supply special coatings for severe service. Joe thought that the labor would be a maximum of two days, or about $1,000 per valve, but probably less. Special coatings, specified by the owner, would be extra for the materials.



Photograph of Existing Shutoff Valve Actuator (June 14, 2001).



PARTS IDENTIFICATION FOR 55 & 85 ACTUATOR
FIGURE 8

K-MAX - ACTUATOR PARTS

55 ACTUATOR

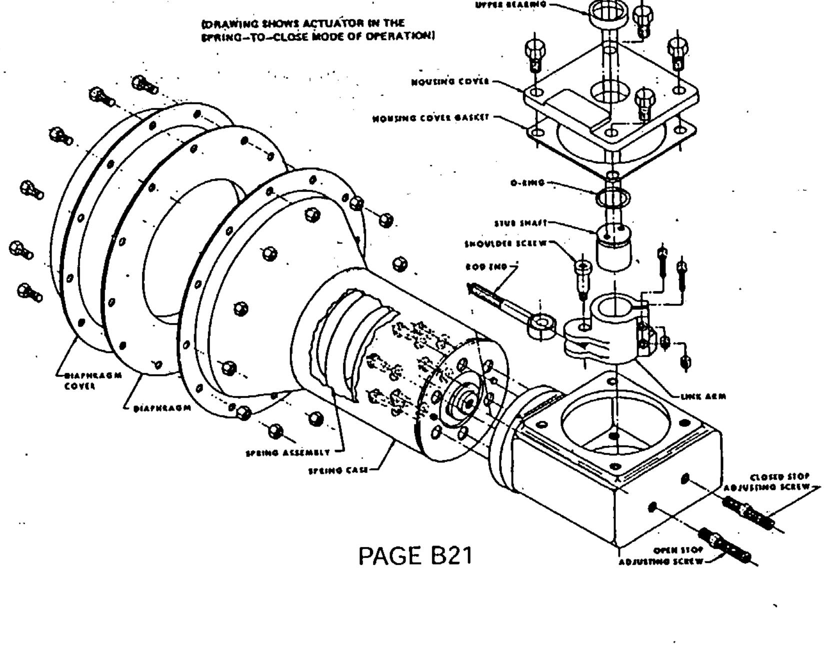
REF	PART #	DESCRIPTION	QTY PER VALVE
B01	KM1204998	COVER ASSY DIAPH LIN/ROT	1
B02	KM1008224	SCR HX HD 3/8-16X1 18-8	14
B03	KM1031950	NUT HX 3/8-16 T316	14
B04	KM1012920	TAG BRASS RED CAUTION	1
B05	KM1111114	HOUSING DIAPH S5&85	1
B06	KM1140372	LINKARM	1
B08	KM1046647	SCR HX HD 3/8-16X1-1/4 G5	2
B09	KM1000085	NUT HX 3/8-16 STL ZP	2
B10	KM1141194	SCR DR U6 X 1/4	2
B11	KM1186847	DATA PLATE K-MAX BRASS	1
B12	KM1105228	LABEL CAUTION DIA	1
B13	KM1093469	DIAPHRAGM 55	1
B14	KM1204999	CASE SPRING DIA ACT 55	1
B15(A)	KM1141584	ASSY,SPRG,DR55,20PSI"	1
B15(B)	KM1141583	ASSY,SPRG,DR55,35PSI	1
B15(C)	KM1140351	ASSY,SPRG,DR55,60 PSI	1
B16	KM1002018	SCR SOCHD 3/8-16X1 C36	6
B17	KM1004901	WASHER LOCK SPR 3/8 ZP	6
B20	KM1105216	BRG ROD END	2
B21	KM1037834	SCR SHLD 1/2X3/8-16X1	1
B23	KM1001955	SCR RH SL 10-24X1/2 HP	2
B24	A74277	ORNG, ,BUNAN, -028	1
B25	KM1051666	BRG SLV 1.503X1.754X1/2	1
B26	KM1122260	POINTER DR55/8B	1
B27	KM1140371	COVER, HSG DIA 55/85	1
B28	KM1110889	GSKT DIA ACT HOUS COVER	2
B29	KM1211328	STUB SHAFT	1
B30	KM5000513	SCR STSOC 1/2-13X2-3/4 18-8	1
B31	KM1031949	NUT HX JAM 1/2-13 316	1
B32	RM1140346	YOKE DIAPH 55	1
B33	KMI000267	SCR HX HD 1/2-13X1 ZP	4
B34	KM1000269	SCR HX HD 1/2-13X1-1/2 ZP	4
B35	KM1140353	BRG BALL 25MMX52MMX15MM	1
B36	KM1000266	SCR HX HD 1/2-13X3/4 ZP	1
B37	KM1031657	WASHER LOCK SPR 1/2 18-8	1
B38	KM1048344	WASHER F A 3/8 N 18-8	1
B39	KM10002505	WASHER LK EXT T 1/2 ZP	4
B40	KM1071706	SCR STLK 3/8-16X1/2 KN18	2

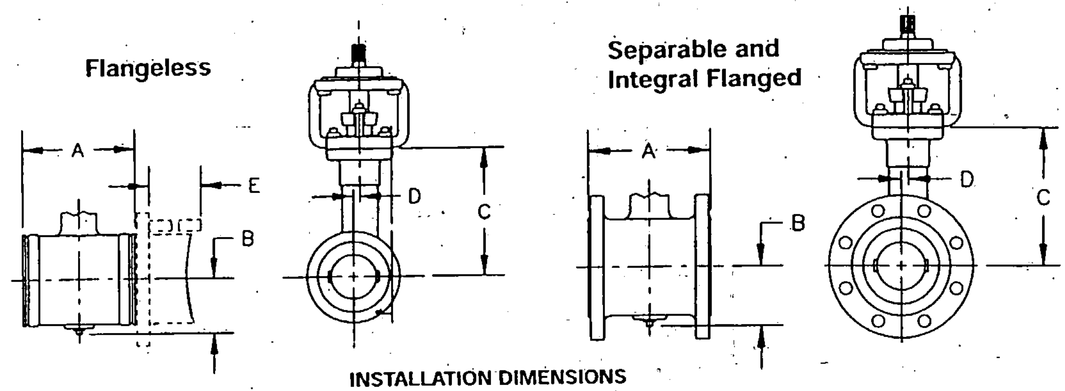
85 ACTUATOR

REF	PART #	DESCRIPTION	QTY PER VALVE
B01	KM1105195	COVER ASSY DIA85	1
B02	KM1008224	SCR HX HD 3/8-16x7/8 316	16
B03	KM1008224	NUT HEX 3/8-16 316SS	16
B04	KM1012920	TAG BRASS RED CAUTION	1
B05	KM1111114	HOUSING DIAPH 55&85	1
B06	KM1140373	LINKARM	1
B08	KM1046647	SCR HX HD 3/8-16x1-1/4 G5	2
B09	KM1000085	NUT HX 3/8-16 STL ZP	2
B10	KM1141194	SCR DR UE X 1/4	2
B11	KM1186847	DATA PLATE K-MAX BRASS	1
B12	KM1105228	LABEL CAUTION DIA	1
B13*	KM1093470	DIAPHRAGM 85	1
B14	KM1205604	CASE SPRING DIA ACT85	1
B15(A)	KM1141582	SPRING ASSY DIAPH 85,20 PSI	1
B15(B)	KM1205611	SPRING ASSY DIAPH 85,35 PSI	1
B15(C)	KM1140350	SPRING ASSY DIAPH 85,60 PSI	1
B16	KM1002017	SCR SOCHD 3/8-16X3/4 C36	6
B17	KM1004901	WASHER LOCK SPR 3/8 ZP	6
B18	KM1002510	WASHER F A 3/8 W EP	6
B20	KM1105216	BRG ROD END	1
B21	KM1037834	SCR SHLD 1/2X3/8-16X1	1
B23	KM1001955	SCR RH SL 10-24X1/2 ZP	2
B24*	A74277	ORNG, BUNAN, -028	1
B25*	KM1051666	BRG SLV 1.BO3X1. 754x1/2	1
B26	KM1122260	POINTER DR55/85	1
B27	KM1140371	COVER, HSG DIA 55/85	1
B28*	KM1110889	GSKT DIA ACT HOUS COVER	2
B29	KM1211328	STUB SHAFT	1
B30	KM1105217	SCR STSOC 1/2-13x2 3/4 FLZ	2
B31	KM1105227	NUT HX HVY JAM 1/2-13 HP	2
B32	KM1140347	YOKE DIAPH 85	1
B33	KM1000267	SCR HX HD 1/2-13X1 ZP	4
B34	KM1000269	SCR HX HD 1/2-13X1-1/2 ZP	4
B35*	KM1140356	BRG BALL 40MHX80MHX18MM	1
B36	KM1000266	SCR HEX HD 1/2-13X3/4 ZP	1
B37	KM1004921	WASHER LOCK SPR 1/2 ZP	1
B38			
B39	KM1002505	WASHER LK EXT T 1/2 ZP	4
B40	KM1071706	SCR STLK 3/8-16X1/2 KN18	2

*Recommended spare parts

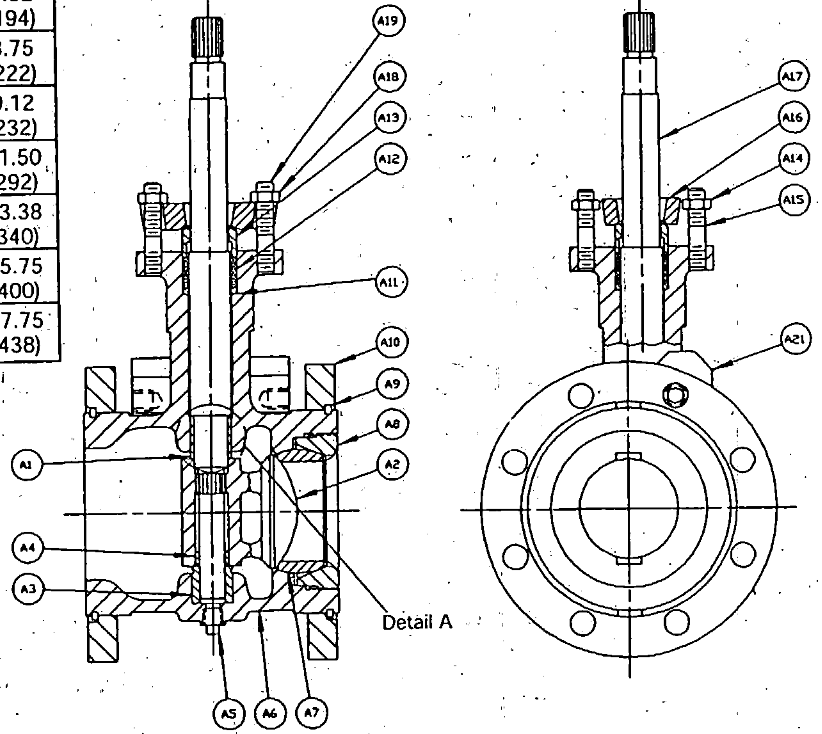
EXPLODED DRAWING OF
MODEL 55 AND 85 DIAPHRAGM ACTUATOR
FIGURE 10

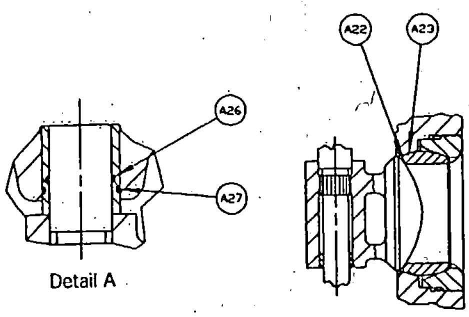




INSTALLATION DIMENSIONS
FIGURE 4A

Valve Size	Dimensions - Inches (mm)				
	A	B	C	D	E
1 (25)	4 (102)	27/16 (62)	47/16 (113)	5/32 (4.06)	7.62 (194)
1.5 (40)	41/2 (114)	23/4 (69.8)	51/8 (130)	1/4 (6.35)	8.75 (222)
2 (50)	47/8 (124)	213/16 (71.4)	47/8 (124)	7/32 (5.59)	9.12 (232)
3 (80)	61/2 (165)	39/16 (90.4)	53/4 (146(	5/16 (7.87)	11.50 (292)
4 (100)	75/8 (194)	4 (102)	7 (178)	7/16 (11.2)	13.38 (340)
6 (150)	9 (229)	51/16 (129)	99/16 (244)	11/16 (16.8)	15.75 (400)
8 (200)	99/16 (243)	6 (152)	11 (279)	7/8 (22.4)	17.75 (438)





PARTS IDENTIFICATION
FIGURE 4B

Parts List

Ref. No.	Description	Material	Qty	1"	1.5"	2"	3"	4"	6"	8"
A1 •	Upper Guide Bearing	BMtl CS	1	KM1140146	KM1140148	KM1140150	KM1140152	KM1140152	KM1140154	KM1140154
A1 •	Upper Guide Bearing	BMtl S2	1	KM1192604	KM1192605	KM1192606	KM1192607	KM1192607	KM1192608	KM1192608
A1 •	Upper Guide Bearing	BMtl HC	1	KM1199622	KM1199623	KM1199624	KM1199625	KM1199625	KM1199626	KM1199626
A1 •	Upper Guide Bearing	BMtl T3	1	KM1216042	KM5000730	KM1207967	KM1203953	KM1203953	KM1203629	KM1203629
A2 •	Plug: Full 316 No Stel	S2-N	1	KM1202473	KM1202477	KM1202481	KM1202485	KM1202489	KM1202493	KM1202497
A2 •	Plug: Full 316 Part. S	S2-P	1	KM1202501	KM1202502	KM1202503	KM1202504	KM1202505	KM1202506	KM1202507
A2 •	Plug: Full 316 Full St	S2-F	1	KM1202501	KM1202502	KM1202503	KM1202504	KM1202505	KM1202506	KM1202507
A2 •	Plug: Full Hastelloy	HC-N	1	KM1202476	KM1202480	KM1202484	KM1202488	KM1202492	KM1202496	KM1202500
A2 •	Plug: Full Titanium	T3-N	1	KM5000208	Consult Leslie	KM1219430	KM1205155	KM1203954	—	—

• Recommended spare parts
• Always reference serial number when ordering parts

• Consult factory for o-ring type guide bushings, Ref. A1 & A3. Also for Teflon-insert seatring Ref. A23.

Ref. No.	Description	Material	Qty	1"	1.5"	2"	3"	4"	6"	8"
A3 *	Lower Guide Bearing	BMtl CS	1	KM1140136	KM1140138	KM1140140	KM1140142	KM1140142	KM1140144	KM1140144
A3 *	Lower Guide Bearing	BMtl S2	1	KM1192599	KM1192600	KM1192601	KM1192602	KM1192602	KM1192603	KM1192603
A3 *	Lower Guide Bearing	BMtl HC	1	KM1199628	KM1199629	KM1199630	KM1199631	KM1199631	KM1199632	KM1199632
A3 *	Lower Guide Bearing	BMtl T3	1	KM1216056	KM5000731	KM1207971	KM1203956	KM1203956	KM1203630	KM1203630
A4 *	Bushing (Plug)	Trim CS	1	—	—	—	KM1147238	KM1147238	KM1147241	KM1147241
A4 *	Bushing (Plug)	Trim S2	1	—	—	—	KM1147238	KM1147238	KM1147241	KM1147241
A4 *	Bushing (Plug)	Trim HC	1	—	—	—	KM1147239	KM1147239	KM1147242	KM1147242
A4 *	Bushing (Plug)	Trim T3	1	—	—	—	KM1203957	KM1203957	KM1203632	Consult Leslie
A5	Pipe Plug	BMtl CS	1	KM1192699	KM1192699	KM1192699	KM1192699	KM1192699	KM1192699	KM1192699
A5	Pipe Plug	BMtl S2	1	KM1192698	KM1192698	KM1192698	KM1192698	KM1192698	KM1192698	KM1192698
A5	Pipe Plug	BMtl HC	1	KM1190387	KM1190387	KM1190387	KM1190387	KM1190387	KM1190387	KM1190387
A5	Pipe Plug	BMtl T3	1	KM1203633	KM1203633	KM1203633	KM1203633	KM1203633	KM1203633	KM1203633
A6	Body (W1,W2,W3,L1,L2)	BMtl CS	1	KM1199301	KM1199302	KM1199303	KM1199304	KM1199305	KM1199306	KM1199307
A6	Body (W1,W2,W3,L1,L2)	BMtl S2	1	KM1199439	KM1199440	KM1199441	KM1199442	KM1199443	KM1199444	KM1199445
A6	Body (W1,W2,W3,L1,L2)	BMtl HC	1	KM1199561	KM1199562	KM1199563	KM1199564	KM1199565	KM1199566	KM1199567
A6	Body (W1,W2,W3,L1,L2)	BMtl T3	1	KM5000225	KM5000022	KM1207972	KM1215100	KM1206349	KM1203635	Consult Leslie
A6	Body (F1)	BMtl CS	1	KM1199308	KM1199309	KM1199310	KM1199311	KM1199312	KM1199313	KM1199314
A6	Body (F1)	BMtl S2	1	KM1199446	KM1199447	KM1199448	KM1199449	KM1199450	KM1199451	KM1199452
A6	Body (F1)	BMtl HC	1	KM1199575	KM1199576	KM1199577	KM1199578	KM1199579	KM1199580	KM1199581
A6	Body (F1)	BMtl T3	1	KM5000749	Consult Leslie	Consult Leslie	KM5000398	KM5000400	Consult Leslie	Consult Leslie
A6	Body (F2)	BMtl CS	1	KM1199322	KM1199323	KM1199324	KM1199325	KM1199326	KM1199327	KM1199328
A6	Body (F2)	BMtl S2	1	KM1199460	KM1199461	KM1199462	KM1199463	KM1199464	KM1199465	KM1199466
A6	Body (F2)	BMtl HC	1	KM1199589	KM1199590	KM1199591	KM1199592	KM1199593	KM1199594	KM1199595
A6	Body (F2)	BMtl T3	1	Consult Leslie	Consult Leslie	KM5000396	Consult Leslie	Consult Leslie	Consult Leslie	Consult Leslie
A6	Body (F3)	BMtl CS	1	KM1199336	KM1199337	KM1199338	KM1199339	KM1199340	KM1199341	KM1199342
A6	Body (F3)	BMtl S2	1	KM1199473	KM1199474	KM1199475	KM1199476	KM1199477	KM1199478	KM1199479
A6	Body (F3)	BMtl HC	1	KM1199603	KM1199604	KM1199605	KM1199606	KM1199607	KM1199608	KM1199609
A6	Body (F3)	BMtl T3	1	Consult Leslie	Consult Leslie	Consult Leslie	Consult Leslie	Consult Leslie	Consult Leslie	Consult Leslie
A7 *	Seat Ring: Full 316 No Stel.	S2-N	1	KM1203101	KM1203121	KM1203142	KM1203164	KM1203186	KM1203208	KM1203230
A7 *	Seat Ring: Full 316 P. Stel.	S2-P	1	KM1203102	KM1203122	KM1203144	KM1203166	KM1203188	KM1203210	KM1203232
A7 *	Seat Ring: Full 316 F. Stel.	S2-F	1	KM1203102	KM1203122	KM1203144	KM1203166	KM1203188	KM1203210	KM1203232
A7 *	Seat Ring: Hastelloy	HC-N	1	KM1203105	KM1203125	KM1203150	KM1203172	KM1203194	KM1203216	KM1203238
A7 *	Seat Ring: Titanium	T3-N	1	Consult Leslie	KM5000024	KM1207724	KM1210492	KM1203960	KM5000084	Consult Leslie
A7 *	Seat Ring: .6 No Stellite	S2-N	1	KM1203106	KM1203126	KM1203151	KM1203173	KM1203195	KM1203217	KM1203239
A7 *	Seat Ring: .6 Part. Stellite	S2-P	1	KM1203107	KM1203127	KM1204787	KM1204793	KM1204799	KM1204805	KM1204811
A7 *	Seat Ring: .6 Full Stellite	S2-F	1	KM1203107	KM1203127	KM1204788	KM1204794	KM1204800	KM1204806	Consult Leslie
A7 *	Seat Ring: .6 Hastelloy	HC-N	1	KM1203110	KM1203130	KM1203154	KM1203176	KM1203198	KM1203220	KM1203243
A7 *	Seat Ring: .6 Titanium	T3-N	1	KM5000745	Consult Leslie	KM5000854	KM1215803	KM5000395	KM1203636	Consult Leslie
A7 *	Seat Ring: .4 No Stellite	S2-N	1	KM1203111	KM1203131	KM1203155	KM1203177	KM1203199	KM1203221	KM1203244
A7 *	Seat Ring: .4 Part. Stellite	S2-P	1	KM1203112	KM1203132	KM1204785	KM1204791	KM1204797	KM1204803	KM1204809
A7 *	Seat Ring: .4 Full Stellite	S2-F	1	KM1203112	KM1203132	KM1204786	KM1204792	KM1204798	KM1204804	Consult Leslie
A7 *	Seat Ring: .4 Hastelloy	HC-N	1	KM1203115	KM1203135	KM1203158	KM1203180	KM1203202	KM1203224	KM1203248
A7 *	Seat Ring: .4 Titanium	T3-N	1	KM5000746	Consult Leslie	KM1207974	KM1205158	KM5000083	KM5000085	Consult Leslie
A7 *	Seat Ring: .2 No Stellite	S2-N	1	KM1203116	KM1203136	KM1203159	KM1203181	KM1203203	KM1203225	KM1203249
A7 *	Seat Ring: .2 Part. Stellite	S2-P	1	KM1203117	KM1203137	KM1204783	KM1204789	KM1204795	KM1204801	KM1204807
A7 *	Seat Ring: .2 Full Stellite	S2-F	1	KM1203117	KM1203137	KM1204784	KM1204790	KM1204796	KM1204802	Consult Leslie
A7 *	Seat Ring: .2 Hastelloy	HC-N	1	KM1203120	KM1203140	KM1203162	KM1203184	KM1203206	KM1203228	KM1203255
A7 *	Seat Ring: .2 Titanium	T3-N	1	KM5000747	Consult Leslie	KM1219434	Consult Leslie	Consult Leslie	Consult Leslie	Consult Leslie
A8	Seat Retaining Ring	Trim-S2	1	KM1139636	KM1139640	KM1139644	KM1139648	KM1139652	KM1139656	KM1199350
A8	Seat Retaining Ring	Trim-HC	1	KM1201697	KM1201696	KM1201693	KM1201692	KM1201694	KM1201695	KM1199617
A8	Seat Retaining Ring	Trim-T3	1	KM5000748	KM5000313	KM1208950	KM1205160	KM1203962	KM5000855	Consult Leslie
A9	Flange Retaining Ring	Standard	4	KM1139629	KM1139630	KM1139631	KM1139632	KM1139633	KM1139634	—
A9	Flange Retaining Ring	Option S	4	KM1150019	KM1150041	KM1154190	KM1150078	KM1150089	KM1154146	—
A10	Separable Flanges (L1)	Standard	2	KM1140101	KM1140104	KM1140107	KM1140110	KM1140113	KM1140116	—
A10	Separable Flanges (L1)	Option S	2	KM1151699	KM1195975	KM1195976	KM1159874	KM1195977	KM1195978	—
A10	Separable Flanges (L2)	Standard	2	KM1200133	KM1200134	KM1200135	KM1140111	KM1140114	KM1140117	—
A10	Separable Flanges (L2)	Option S	2	KM1200136	KM1200137	KM1200138	KM1154189	KM1195984	KM1195985	—
A11	Back-Up Ring	Trim-S2	1	KM1139687	KM1139687	KM1139687	KM1139688	KM1139688	KM1139689	KM1139689
A11	Back-Up Ring	Trim-HC	1	KM1139690	KM1139690	KM1139690	KM1139691	KM1139691	KM1139692	KM1139692
A11	Back-Up Ring	Trim-T3	1	KM1207977	KM1207977	KM1207977	KM1203964	KM1203964	KM1203639	KM1203639
A12*	Packing Set	TC	1	KM83000057	KM83000057	KM83000057	KM83000002	KM83000002	KM83000020	KM83000020
A12	Packing Set	G1	1	KM83000058	KM83000058	KM83000058	KM83000019	KM83000019	KM83000029	KM83000029
A13	Gland	Trim-S2	1	KM1199369	KM1199369	KM1199369	KM1199370	KM1199370	KM1199371	KM1199371
A13	Gland	Trim-HC	1	KM1199619	KM1199619	KM1199619	KM1199620	KM1199620	KM1199621	KM1199621
A13	Gland	Trim-T3	1	KM1207979	KM1207979	KM1207979	KM1204060	KM1204060	KM1206326	KM1206326
A14	Nut (Steel)		2	KM1147010	KM1147010	KM1147010	KM1141512	KM1141512	KM1141512	KM1141512
A14	Nut (316 SST)		2	KM1155548	KM1155548	KM1155548	KM1144321	KM1144321	KM1144321	KM1144321
A15	Stud (Steel)		2	KM1140219	KM1140219	KM1140219	KM1140220	KM1140220	KM1140221	KM1140221
A15	Stud (316 SST)		2	KM1192587	KM1192587	KM1192587	KM1192588	KM1192588	KM1192589	KM1192589
A16	Gland Clamp (Steel)		1	KM1139759	KM1139759	KM1139759	KM1139760	KM1139760	KM1139761	KM1139761
A17*	Shaft	Trim-S2	1	KM1199351	KM1199352	KM1199353	KM1199354	KM1199355	KM1206246	KM1206249
A17*	Shaft	Trim-HC	1	KM1199430	KM1199431	KM1199432	KM1199433	KM1199434	KM1206247	KM1206250
A17*	Shaft	Trim-T3	1	Consult Leslie	Consult Leslie	Consult Leslie	Consult Leslie	Consult Leslie	Consult Leslie	Consult Leslie
A18	Packing Nut (316 SST)		2	KM1155548	KM1155548	KM1155548	KM1144321	KM1144321	KM1144321	KM1144321
A19	Packing Stud (316 SST)		2	KM1139762	KM1139762	KM1139762	KM1139763	KM1139763	KM1139764	KM1139764
A21	Stud Block (W1)	WCB	1	N/A	N/A	N/A	N/A	KM1199360	KM1199363	KM1199366
A21	Stud Block (W2)	WCB	1	N/A	N/A	KM1199358	KM1199359	KM1199361	KM1199364	KM1199367
A21	Stud Block (W3)	WCB	1	N/A	N/A	KM1199358	KM1199359	KM1199362	KM1199365	KM1199368
A22	O-Ring (PTFE Seat Seal †	PTFE	1	Consult Leslie	Consult Leslie	Consult Leslie	Consult Leslie	Consult Leslie	Consult Leslie	Consult Leslie
A23	O-Ring (PTFE Seat Seal †	PTFE	1	Consult Leslie	Consult Leslie	Consult Leslie	Consult Leslie	Consult Leslie	Consult Leslie	Consult Leslie
A26	O-Ring (Viton) †	M1	1	KM1151557	KM1151557	KM1151557	KM1151563	KM1151563	KM1156484	KM1156484
A26	O-Ring (Kalrez) †	M2	1	KM1203420	KM1203420	KM1203420	KM1203422	KM1203422	KM1203424	KM1203424
A27	O-Ring (Viton) †	M1	1	KM1151560	KM1151560	KM1151560	KM1151566	KM1151566	KM1151636	KM1151636
A27	O-Ring (Kalrez) †	M2	1	KM1203421	KM1203421	KM1203421	KM1203423	KM1203423	KM1203425	KM1203425

† Not Shown

*Recommended spare parts

Telephone Conversation Record

RE: Walker Mine Valve Maintenance and Operational Testing – Replacement Valve Actuators
Date: 5/11/01
From: Jay Ketchen, (925) 370-1500 at CFM-SF (Valve Representative for DeZurik Valves) www.CFMControls.com
To: Cynthia Fox, (415) 442-7555 at Washington Group International, Inc Cynthia.Fox@WGINT.com

Jay was returning my call after checking with the DeZurik Company to find out if DeZurik could supply geared manual actuators that would fit on the existing 4-inch K-Max shutoff valves in the Walker Mine. Jay found out that DeZurik did not make actuators that would work on these valves for the following reasons:

1. The K-Max valves are special control valves with a valve stem that rotates clockwise to open. They are almost always fitted with a pneumatic or electric actuator.
2. The valve has a tapered spline at the top of the valve stem. DeZurik's actuators are made to fit a standard square nut-type valve stem.
3. The bolt spacing and mount dimensions for the actuator connection to the valve have been standardized (ISO bolting) since the mine valves were installed. It would need to be verified that a standard actuator could be mounted directly on the existing valves without modifications.

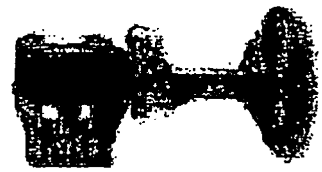
In addition, DeZurik would be hesitant to provide an actuator for a valve line that is now owned by Leslie Controls [(813) 978-1000], for contractual reasons, even though the original valves were furnished without actuators.

Jay said that if the actuators needed to be serviced, they could be removed and taken to a service company such as Bay Valve Service & Engineering (Martinez, CA (925) 228-0665). Jay thought that the most likely problem with the existing valves would not be the actuators, but might be that the valve stems were frozen from not being actuated. Forcing the actuator might cause the stem to break. He suggested the valve service vendor should be sure to have spare packing (Teflon) on hand for the existing valves.

DeZURIK
COPES-VULCAN

Products

Manual Gear Actuators



Product Description [Back to top]

Manual Gear Actuators feature a cast iron housing with sintered bronze bearings on each end of the input shaft for durability and performance. The ductile iron gear provides strength for robust applications and a long service life without maintenance. Manual Gear Actuators are available with handwheel, chainwheel, or a 2" (50mm) square nut input option. All Manual Gear Actuators feature external position indication and are available with safety lockout devices.

Actuator Turns [Back to top]

Actuator Type	No. of Turns (approx.)
MG-30-_	7.5
MG-30Z-_	7.5
MG-31-_	8
MG-46-_	11.5
MG-64-_	16

Literature [Back to top]

Manual Gear Actuators Bulletin 72.00-1.

Manual Gear Actuators Sizing Sheets Bulletin 72.00-2. This bulletin contains sizing information for BHP, BRS, and BGS Butterfly valves, VPB V-port ball valves, RCV Rotary Control Valves, FPB Full Port ball valves and Permaseal plug valves.

Appendix C.3

Flanged Ball Valve
(Apollo)

#2

87/88-200 Series

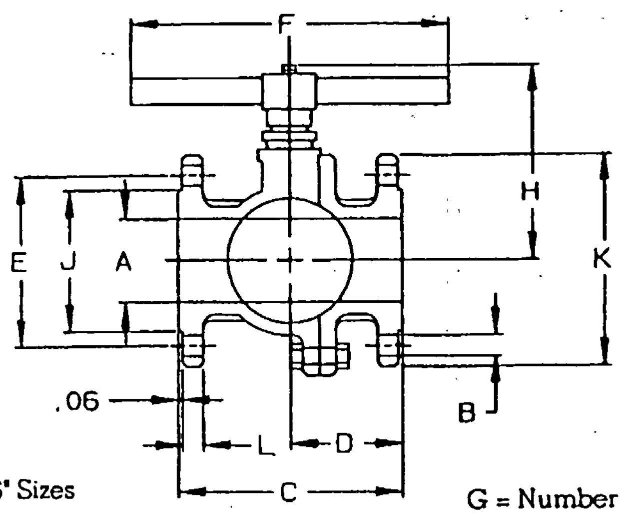
ANSI Class 150 Full Port Flanged Apollo®

FEATURES

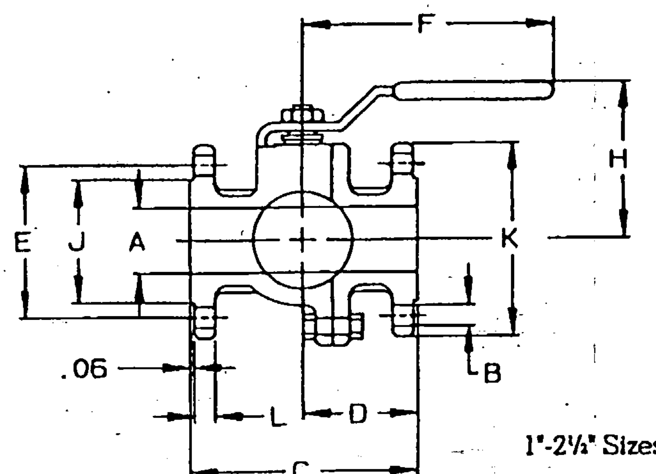
- Blow-out-proof stem
- Reinforced TFE seats and seals
- Corrosion resistant
- Meets NACE MR-01-75 (CS valves must be ordered with S.S. trim)
- Positive shut-off
- Meets WW-V-35C
- Certified to API standard 607, 3rd Edition (1" to 2½" valves must be ordered with graphite stem packing)
- CS valves available with SS trim
- Field repairable
- Bolted body design with Grade 8 (carbon steel bodies) or 300 series Stainless steel (Stainless bodies) bolting hardware
- Optional virgin TFE seats and seals available (thru 4" size)

Stainless Steel Valve Shown

Contact factory for new actuator mounting dimensions



3"-6" Sizes



1"-2½" Sizes

G = Number of holes in flanges
NOTE: 1" valve is of unibody design

See P/T Chart No. 21 on Page 65.

FLANGED ENDS

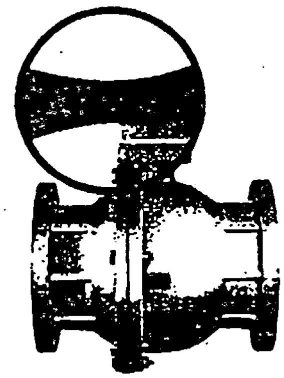
316 SS	STEEL	SIZE	A	B	C	D	E	F	G	H	J	K	L	Cv*
87-205	88-205	1	1.00	.62	5.00	1.94	3.12	8.00	4	3.62	2.00	4.25	.50	100
87-207	88-207	1½	1.50	.62	6.50	3.25	3.87	8.00	4	4.31	2.87	5.00	.68	265
87-208	88-208	2	2.00	.75	7.00	3.50	4.75	8.00	4	4.75	3.62	6.00	.68	495
87-209	88-209	2½	2.50	.75	7.50	3.75	5.50	8.00	4	5.12	4.12	7.00	.81	781
87-200	88-200	3	3.00	.75	8.00	4.00	6.00	18.00	4	7.25	5.00	7.50	.75	1320
87-20A	88-20A	4	4.00	.75	9.00	5.25	7.50	18.00	8	8.87	6.18	9.12	.93	2550
87-20C	88-20C	6	6.00	.87	15.50	7.75	9.50	18.00	8	9.87	8.50	11.06	1.00	5590

*The Cv factor is the gallons of water per minute that the valve will pass with 1 PSIG pressure drop.

87/88-200 Series (Cont'd.)

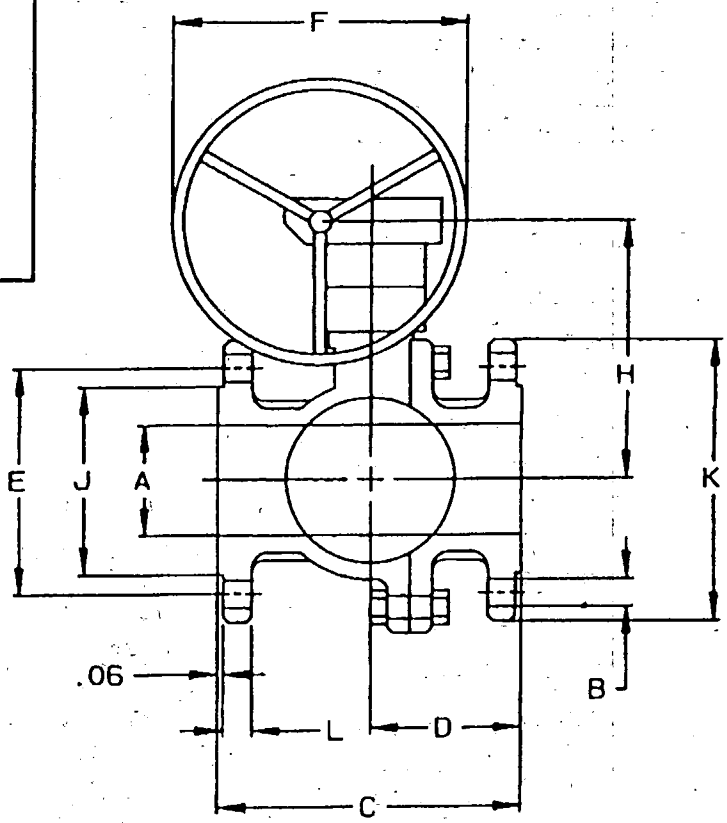
#3

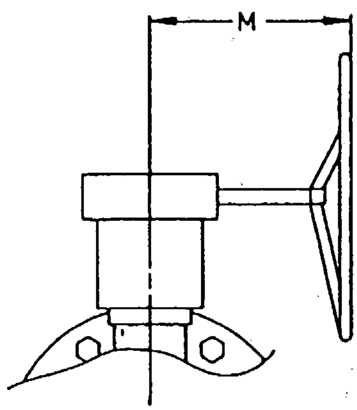
ANSI Class 150 Full Port Flanged Apollo®



FEATURES

- Blow-out-proof stem
- Reinforced TFE seats and seals
- Corrosion resistant
- Meets NACE MR-01-75 (CS valves must be ordered with S.S. trim)
- Positive shut-off
- Standard unit has manual worm gear operator
- Meets WW-V-35C
- Certified to API standard 607, 3rd Edition
- CS valves available with SS trim
- Field repairable
- Graphite stem packing
- Bolted body design with Grade 8 (carbon steel bodies) or 300 series Stainless steel (Stainless bodies) bolting hardware





See P/T Chart No. 21 on Page 65.

G=Number of holes in flanges.

FLANGED ENDS

316 SS	STEEL	SIZE	A	B	C	D	E	F	G	H	J	K	L	M	Cv*
87-20E	88-20E	8	8.00	.87	18.00	9.00	11.75	13.00	8	13.06	10.62	16.25	1.18	9.75	10000
87-20G	88-20G	10	10.00	1.00	21.00	10.50	14.25	24.00	12	15.50	13.00	20.00	1.31	12.00	16390
87-20H	88-20H	12	12.00	1.00	24.00	12.00	17.00	24.00	12	17.00	15.25	24.00	1.43	12.00	23780

*The Cv factor is the gallons of water per minute that the valve will pass with 1 PSIG pressure drop.

Conbraco Industries, Inc.

Customer Service Department
Post Office Box 247
Matthews, NC 28106
Phone 704-841-6000
Fax 704-841-6020

Sales Department
Post Office Box 247
Matthews, NC 28106
Phone 704-847-9191
Fax 704-841-6021

Engineering Department
Post Office Box 125
Pageland, SC 29728
Phone 803-672-6161
Fax 803-672-6747

APOLLO DIVISION

Sales Representatives

California, Anaheim 92805
VALVES AND CONTROLS, INC.
1410 E. Katella
Phone: 714-935-9533 Fax: 714-935-9503

California, Pleasanton 94566
SPECIFIED PROCESS EQUIPMENT CO.
1040 Serpentine Lane, Suite 207
Phone: ~~510-484-4211~~ Fax: 510-484-4213

Canada, Montreal, Quebec H4RIC9
ROMATEC
6535 Henri Bourassa Blvd., West
Phone: 514-332-9302 Fax: 514-332-0578

Canada, Sarnia, Ontario N7T 7H5
ROMATEC
461 Scott Rd.
Phone: 519-337-2329 Fax: 519-336-0998

Canada, Willowdale, Ontario M2H 2E1
ROMATEC
452 McNicoll Ave.
Phone: 416-499-0222 Fax: 416-499-0352

Canada, Dartmouth, Nova Scotia B3B-1L2
ROMATEC
100 Wright Ave., Unit 24
Burnside Industrial Park
Phone: 902-465-2505 Fax: 902-464-0127

Colorado, Denver 80216
THOMAS J. RODENO & ASSOC.
4238 York St.
Phone: 303-296-4600 Fax: 303-296-6724

Florida, St. Petersburg 33710
HENRY & ASSOCIATES
4950 1st Ave. North
Phone: 813-327-7911 Fax: 813-327-6911

Georgia, Conyers 30207
VALVES & FITTINGS, INC..
1560 Farmer Road
Phone: 404-483-3057 Fax: 404-483-3110

Illinois, Bolingbrook 60440
NEW TECH MARKETING, INC.
500 E. Frontage Rd. North
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HEBCO, INC.
5310 College Blvd., P.O. Box 7188, 66207
Phone: 913-491-0797 Fax: 913-491-5126

Kentucky, Louisville 40206
L.J. WHITFILL & ASSOCIATES, INC.
2843 Brownsboro Rd., Suite 206
Phone: 502-896-6661 Fax: 502-893-2546

Louisiana, Baton Rouge 70806
KNOX & ASSOCIATES, INC.
5646 Berkshire Ave.
Phone: 504-923-1416 Fax: 504-923-1481

Massachusetts, Watertown 02272
URELL, INC.
P.O. Box 321, 86 Coolidge Ave., 02272
Phone: 617-923-9500 Fax: 617-926-9414

Michigan, St. Clair Shores 48081
VAN EGMOND SALES CO.
25200 Jefferson Ave.
Phone: 313-774-7760 Fax: 313-774-1490

Minnesota, Chanhassen 55317
NORTHSTAR VALVE & FITTING, INC.
80 W. 78th St., Suite 130-B
Phone: 612-937-0108 Fax: 612-937-0803

Mississippi, Ridgeland 39158
SOUTHERN MARKETING GROUP
P.O. Box 1093
Phone 601-829-2935 Fax: 601-829-2941

Missouri, St. Louis 63110
R.W. HASTIE & ASSOCIATES
5100 Oakland Ave.
Phone: 314-531-4205 Fax: 314-531-4521

Nebraska, Omaha 68112
WILLCO, INC.
6405 John J. Pershing Dr.
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Netherlands
VAPO TECHNIEK B.V.
Gestelsestraat 28, 5582 HH WAALRE
Phone: 31-4904-15455 Fax: 31-4904-15565

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Phone: 718-937-8400
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Ohio, Canfield 44406
JOSEPH D. MORRISSEY ASSOCIATES
111 S. Broad St.
Phone: 216-533-2221 Fax: 216-533-2748

Pennsylvania, Bethel Park 15102
JOSEPH D. MORRISSEY ASSOCIATES
Box 374, 3927 Mimosa Drive
Phone: 412-831-3436 Fax: 412-831-7555

Pennsylvania, Newton Square 19073
J.A. LAYDEN CO.
3602 Winding Way
Phone: 215-353-2311 Fax: 215-359-1299

Puerto Rico/Santurce 00914
RAFAEL RODRIGUEZ BARRIL INC.
P.O. Box 12007
470 Francia Street, Hato Rey, PR 00917
Phone: 809-759-8680 Fax: 809-759-8986

Singapore 0511
Compact Distributors Pte. Ltd.
BLK 1, Wholesale Centre
#02-10, Pasir Panjang Rd.
Phone 65-7782022 Fax: 65-7778668

Tennessee, Memphis 38115
SOUTHERN MARKETING GROUP
6074 Apple Tree Dr., Suite 2
P.O. Box 752018, 38175-2018
Phone: 901-367-2912 Fax: 901-367-1591

Texas, Euless 76040
B. GATES CO., INC.
1010 Pamela Dr.
Phone: 817-267-8755 Fax: 817-545-8454

Texas, Houston 77041
KNOX & ASSOCIATES, INC.
10002 Grover #1, P.O. Box 40493, 77240
Phone: 713-462-7766 Fax: 713-690-6228

Venezuelaq, Maracaibo
E.M.L. INTERNATIONAL
Calle 79 Ave. 19, Edif. LaFrida, Local 2.P.B.
Phone: 011-58-61-548803
Fax: 011-58-61-539204

Virginia, Ruckersville 22968
MID SOUTH MARKETING, INC.
Mail: Rt. 2, Box 16-D
Ship: Village on the Green
Hwy. 29 North
Phone: 804-985-6084 Fax: 804-985-6087

Washington, Seattle 98134
LOU FALKENHAGEN CO.
2400 6th Ave. South, Room 255
Phone: 206-624-3011 Fax: 206-624-3012

Washington, Vancouver 98682
LOU FALKENHAGEN, CO.
12714 N.E. 95th St.
Vancouver, WA 98682
Phone: 206-944-8457 Fax: 206-944-8459

CONBRACO

Appendix C.4

Pressure Transducer (Druck)

athena

ve - VENDOR CATALOGS

FULL DESCRIPTION

Viewing **1** of **1** for

DRUCK

Previous Next Go Back Start Over Help

Look Left Look Right

Copies	1, No status info available
Call	13400
Author	DRUCK INC.
Title	Transducers, transmitters, depth level, field calibrators controllers/ indicators, primary standards 1999
Summary	Rep. Address: RDP Corp. / 4900 Brecksville Road / POB 506 / Richfield OH 44286 / 1-800-676-7631 or 1-330-659-3138 / Fax: 1-330-659-6969
Notes	Company Address: 4 Dunham Drive / New Fairfield CT 06812 / 1-203-746-0400 / Fax: 1-203-746-2494 / E-mail: sales@druckinc.com
Publisher	1999.
URL	http://www.druckinc.com

PTX 500 Series
Industrial pressure transmitters

Druck Pressure Measurement

- Ranges from 1 psig to 10,000 psig and 5 psia to 10,000 psia
- Output 2-wire, 4-20mA
- Accuracy ±0.15% FS BSL
- Stability ±0.1%FS per year
- TEB ±1%FS from -5° to +176°F
- 400% FS overpressure
- NACE compatible Hastelloy and SS wetted parts
- RFI protected
- Intrinsically safe version available

Sensor-Groups

Output | Application

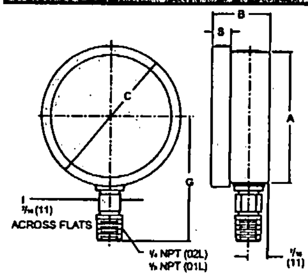
Appendix C.5

Pressure Gauge
(Ashcroft)

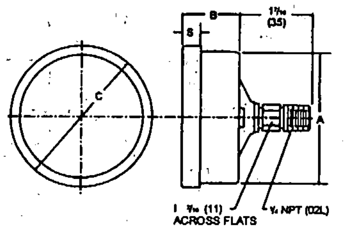
Dimensions

Ashcroft® Stainless Steel Case Pressure Gauges

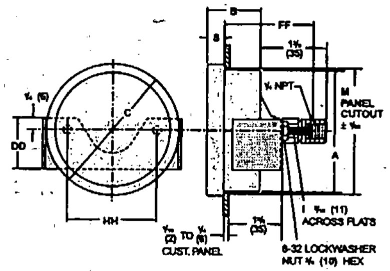
Case Type 1009 – 2½ & 3½



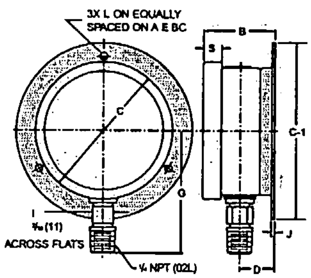
2½" & 3½" 1009 lower connected



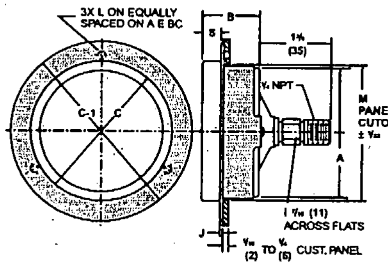
2½" & 3½" 1009 back connected



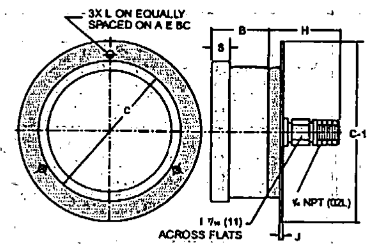
2½" & 3½" 1009 back connected (XUC)



2½" & 3½" 1009 lower connected (XFW)

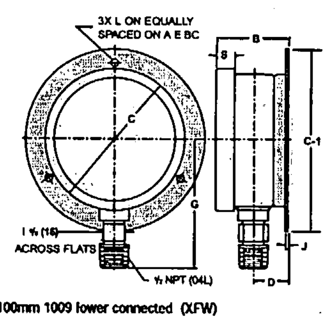


2½" & 3½" 1009 back connected (XFF)

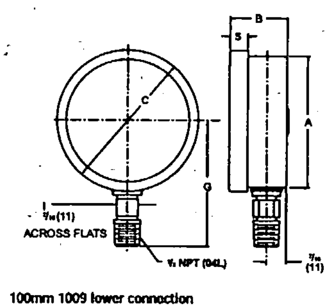


2½" & 3½" back connected (XFW)

Case Type 1009 – 100mm



100mm 1009 lower connected (XFW)



100mm 1009 lower connection

Gauge Size	A	B	C	C-1	D	DD	E	FF	G	H	HH	I	J	L	M	S	Weight Dry	Weight LF
2½ (63)	2 21/32 (67)	1 3/16 (30)	2 7/8 (73)	3 11/16 (94)	3/4 (19)	1 1/8 (29)	3 1/8 (79)	1 11/16 (43)	2 9/16 (65)	1 3/32 (28)	2 1/16 (52)	7/16 (11)	1/16 (2)	5/32 (4)	2 13/16 (71)	3/8 (10)	.26# .12kg	.50# .32kg
3½ (100)	3 19/32 (91)	1 9/32 (33)	3 31/32 (100)	5 7/32 (133)	7/8 (22)	15/32 (12)	4 9/16 (106)	1 29/32 (48)	3 (76)	3 (76)	2 13/32 (61)	7/16 (11)	5/32 (4)	7/32 (6)	3 13/16 (97)	15/32 (12)	.44# .20kg	.88# .40kg
(100)	3 19/32 (91)	1 19/32 (40)	3 31/32 (100)	5 7/32 (133)	7/8 (22)		4 9/16 (116)		3 11/16 (94)			7/16 (11)	5/32 (4)	7/32 (6)		15/32 (12)	.50# .23kg	.97# .45kg

Note: Dimensions in brackets ( ) are millimeters.

How to Order

Ashcroft® Stainless Steel Case Pressure Gauges

Table A – Case selection and mounting

Dial Size	Ordering Code	Case Type	Case: Finish & Material	Ring: Style, Finish & Material	Mounting/Connection
2½", 3½"	(25) (35)	1009	Polished 304 SS	Bayonet, Lock ring	Stem – Lower or back
100mm	(10)	1009	Polished 304 SS	Polished 304 SS	Surface – Lower or back: specify (XFW) back flange Flush – Back: specify front flange (FF) or U-clamp (UC)
4½", 6"	(45) (60)	1009	Polished 304 SS	Bayonet, Lock ring Polished 304 SS	Stem – Lower or back Surface – Lower or back, wall mount bracket (BF) Flush – Back: specify front flange (FF) or U-clamp (UC)
40mm, 50mm	(40) (50)	1008	Polished 304 SS	Push-In, Polished 304 SS	Stem – Lower or back Flush – Back: specify front flange (FF) or U-clamp (UC)
63mm 100mm	(63) (10)	1008	304 SS	Crimped 304 SS	Stem – Lower or back Flush – Back: specify front flange (FF), U-clamp (UC) or retrofit flange (RF)

Table B – System, connection and location

Dial Size	Case Type	Tube and Socket Code	Tube and Socket Material	NPT Conn. and Code	Conn. Location and Code	Range Selection Limits (psi)
(2½", 3½" 100mm)	1009	(AW)	Welded 316 SS tube, bronze socket	(02) ¼ standard (04) ½ optional[1]	(L) lower (B) back	Vac/6000
		(SW)	Welded 316 SS tube and socket			Vac/15,000
(4½", 6")		(A)	Grade A phosphor bronze tube, brass tip sliver brazed Brass socket	(02) ¼ standard (04) ½ optional		Vac/1000
		(B)	4130 alloy steel tube, 1019 steel socket			Vac/5000
		(R)	316 SS tube, 1019 steel socket			Vac/20,000
		(S)	316 SS tube and socket			Vac/20,000
(40/50mm)	1008	(S)	316 SS tube and socket	(01) ⅛ std, 40mm (02) ¼ std, 50mm		Vac/5000
(63/100mm)		(A)	Phosphor bronze tube, brass socket, soldered	(02) ¼ std,		Vac/6,000
		(S)	Welded 316 SS tube and socket	(02) ¼ std. (04) ½ opt.[1]		Vac/15,000

NOTES:
(1) 3½"/100mm 1009SW, 100mm 1008S lower connect only
(2) Not available with ⅛" NPT

To order an Ashcroft Stainless Steel Case Pressure Gauge (sample coding shown)

Select: 25 1009 SWL 02L XGV 160 psi

1. Dial size–2½" — 25
2. Case type–1009 — 1009
3. Bourdon tube and socket 316SS — SWL
4. Connection–¼ NPT Lower — 02L
5. Optional features-Silicone filled — XGV
6. Pressure range (see range tables on pages 12 through 14) — 160 psi

Range Tables

Ashcroft® 1009 Stainless Steel Case
2½", 3½", 4½", 6", and 100mm Pressure Gauges

Standard Ranges

Pressure

psi	Figure Interval	Minor graduation
0/15	1	0.2
0/30	5	0.5
0/60	5	1
0/100	10	1
0/160	20	2
0/200	20	2
0/300	30	5
0/400	50	5
0/600	50	10
0/800	100	10
0/1000	100	10
0/1500	200	20
0/2000	200	20
0/3000	300	50
0/5000	500	50
0/6000	1000	100
0/7500	1000	100
0/10,000	1000	100
0/15,000	2000	200
0/20,000	2000	200
0/30,000	3000	500

Compound

Range	Figure Interval		Minor graduation	
	in. Hg	psi	in. Hg	psi
30" Hg/15 psi	5	3	1	0.5
30" Hg/30 psi	10	5	1	1
30" Hg/60 psi	10	10	2	1
30" Hg/100 psi	10	10	2	1
30" Hg/150 psi	10	20	5	2
30" Hg/300 psi	30	25	5	5

Vacuum

Range	Figure Interval	Minor graduation
30/0 in. Hg	5 in.	0.5 in.

Metric Ranges

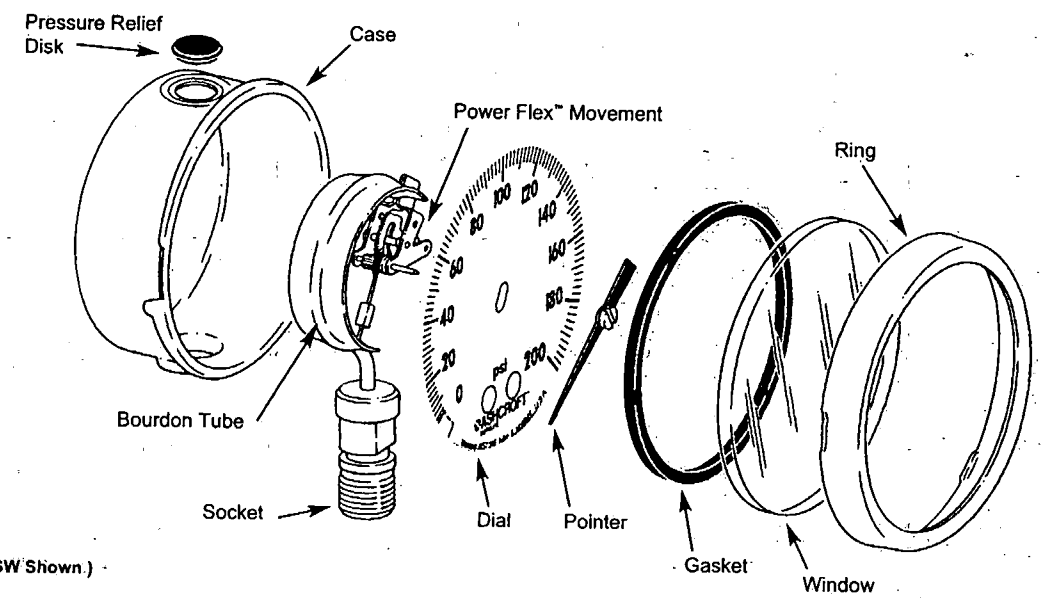
kg/cm² (kilograms per sq. centimeter)	bar	Figure interval	Minor graduation	kPa (kilopascal)	Figure interval	Minor graduation	psi outer scale of dual range*
Pressure							
0/1	0/1	0.1	0.01	0/100	10	1	0/14
0/1.6	0/1.6	0.2	0.02	0/160	20	2	0/22
0/2.5	0/2.5	0.5	0.05	0/250	50	5	0/35
0/4	0/4	0.5	0.05	0/400	50	5	0/55
0/6	0/6	0.5	0.1	0/600	50	10	0/85
0/10	0/10	1	0.1	0/1000	100	10	0/140
0/16	0/16	2	0.2	0/1600	200	20	0/220
0/25	0/25	5	0.5	0/2500	500	50	0/350
0/40	0/40	5	0.5	0/4000	500	50	0/550
0/60	0/60	5	1	0/6000	500	100	0/850
0/100	0/100	10	1	0/10,000	1000	100	0/1,400
0/160	0/160	20	2	0/16,000	2000	200	0/2,200
0/250	0/250	50	5	0/25,000	5000	200	0/3,500
0/400	0/400	50	5	0/40,000	5000	200	0/3,500
0/600	0/600	50	10	0/60,000	5,000	500	0/5,500
0/1000	0/1000	100	10	0/100,000	10,000	1000	0/14,000
0/1600	0/1600	200	20	0/160,000	20,000	2000	0/22,000
Vacuum							
-1/0	-1/0	0.1	0.01	-100/0	10	1	30" Hg
Compound							
-1/0/1.5	-1/0/1.5	0.05	0.05	-100/0/150	50	5	30" Hg/0/20
-1/0/3	-1/0/3	0.05	0.05	-100/0/300	50	5	30" Hg/0/40
-1/0/5	-1/0/5	0.5	0.1	-100/0/500	50	10	30" Hg/0/70
-1/0/9	-1/0/9	1	0.1	-100/0/900	100	10	30" Hg/0/125
-1/0/15	-1/0/15	2	0.2	-100/0/1500	200	20	30" Hg/0/215
-1/0/24	-1/0/24	5	0.2	-100/0/2400	500	20	30" Hg/0/340

*Inner scale is dominant

Case and ring options	Code	Comments
Hermetically Sealed or Weatherproof Liquid-Fillable Case	LJ	Gauge furnished dry or for liquid filling. Includes a solid fill plug and throttle plug for ranges 30 psi and above.
U-Clamp	UC	Used for panel mounting back-connect gauges.
Front Flange	FF	Includes 3 holes for panel mounting gauges. (back-connect only)
Wall Mounting Bracket	BF	Available on 4½" 1009 lower or back-connect.
Back Flange for Wall Mounting	FW	Available on 2½" and 3½" 1009 lower or back-connect.
Retrofit Flange	RF	Available on 63mm and 100mm 1008 back-connect gauges for panel mounting. (Includes U-clamp.)
Metric Gauge	MG	Available on 3½" 1009 with ½ NPT lower connection. Gauge supplied with micrometer pointer, white dial and safety glass.
Bourdon tube and system assembly options		
SS Throttle Plug – (restrictor) Push-in Design	TU	SS push-in type with a 0.013" orifice for 2½", 3½", 100mm 1009, 63mm, 100mm 1008. Throttle plug standard on all 2½", 3½" 1009, 40mm thru 100mm 1008 liquid filled gauges 30 psi—1000 psi
SS Throttle Plug (restrictor) Helical Design	TS	Standard on all 2½", 3½", 100mm 1009s, 40mm, thru 100mm 1008 liquid filled gauges with ranges of 1500 psi and above. 4½", 6" 1009 furnished with thread-in design with a .031" orifice.
Liquid Filled Gauge Without Throttle Plug	WP	Required when the process may clog a throttle plug on the gauge.
Cleaning for Gaseous Oxygen	6B	If gauge is liquid filled specify Halocarbon as the fill.
Liquid filling options		
Silicone Fill	GV	Not available on 40mm and 50mm 1008 gauges.
Halocarbon Fill	GX	Not available on 40mm and 50mm 1008. For oxidizing media. Examples; chlorine, oxygen, nitric acid and sulfuric acid.
Pointer options		
Red Set-Hand (Single)	SH	Available on 1009 only. Single stationary set-hand used to indicate a specific pressure.
Red Set-Hand (Double)	SJ	Available on 1009 only. Double stationary set-hand used to indicate 2 specific pressures.
Red Set-Hand (Adjustable)	EU	Available on 1009 only.
Maximum Pointer	EP	Available on 4½" and 6" 1009 only. Externally reset by a knob on outside of an acrylic window.
Minimum Pointer	EQ	Available on 4½" and 6" 1009 only. Externally reset by a knob on outside of an acrylic window.
Window options		
Polycarbonate Window	PD	40mm, 50mm 1008 only. Ambient temperature limits -50/270°F. XPD standard on 2½", 3½" 1009 and 63mm and 100mm 1008.
Acrylic Window	PD	4½" and 6" 1009 only. Ambient temperature limits -50/180°F.
Shatterproof Glass	SG	Not available on 63mm, 100mm 1008. Ambient temperature limits -50/200°F.
External Zero Adjustable Pointer (Easy Zero™)	EA	Available in 3½" 1009 only dry or liquid filled.
Marking and tagging options		
Dial Marking	DA	Service marking printed on dial.
Paper Tagging of Carton and Gauge	NN	Tag is bonded to gauge case and carton.
Stainless Steel Tagging of Gauge Case	NH	300 series stainless steel tag is wired to gauge case.
Calibration options		
Accuracy 0.5% full scale	AN	4½" and 6" 1009 only.
Test and certificate options		
Certificate of Conformance	CD-1	Conformance to specifications and/or drawings.
Individual Certified Calibration Chart	CD-4	
Special connection options		
¼" NPT	01	Available on 2½", 3½" 1009SW.
SAE 7/16" and 20 straight thread	RW	Not available on 40mm, 50mm 1008
7/16" and 20 UNF-3A 37° Flare	EJ	Not available on 40mm, 50mm 1008
¼" straight JIS, BSP	KJ	Not available on 40mm, 50mm 1008
¼" tapered JIS, BSP	KA	Not available on 40mm, 50mm 1008
⅜" straight JIS, BSP	KP	3½", 100mm 1009SW lower connection only
½" straight JIS, BSP	KN	3½", 100mm 1009SW lower connection only
⅜" tapered JIS, BSP	KR	3½", 100mm 1009SW lower connection only
½" tapered JIS, BSP	KQ	3½", 100mm 1009SW lower connection only
G ¼" DIN	13	Not available on 40mm, 50mm 1008

Product Selection Information

Ashcroft® Stainless Steel Case Pressure Gauges



(2½" 1009SW Shown.)

Consult ASME B40.1 for guidance in gauge selection

WARNING: To prevent misapplication, pressure gauges should be selected considering media and ambient operating conditions. Improper application can be detrimental to the gauge, causing failure and possible personal injury or property damage. The information contained in this catalog is offered as a guide to assist in making the proper selection of a pressure gauge. Additional information is available from Dresser Instrument Division.

Pressure Ranges:
As recommended by ASME B40.1, select a gauge with a full scale pressure range of approximately twice the normal operating pressure. The maximum operating pressure should not exceed approximately 75% of the full scale range. Failure to select a gauge range within these criteria may ultimately result in fatigue failure of the bourdon tube.

Operating Conditions:
The operating conditions to which a gauge will be subjected must be considered. If the gauge will be subjected to severe vibration or pressure pulsation, liquid filling the gauge will be necessary to obtain normal product life.

Other than discoloration of the dial and hardening of the gasketing that may occur as ambient temperatures exceed 150°F, stainless steel gauges (that are not liquid filled) can withstand continuous ambient temperatures as high as 350°F. All gauges can withstand ambient temperatures up to 250°F. Accuracy will be affected by approximately 1.5% per 100°F.

Gauges with welded joints will withstand 750°F (450°F with silver brazed joints) for short times without rupture, although other parts of the gauge will be destroyed and calibration will be lost.

Proper selection of the bourdon system material is dependent on the process fluid to which the system will be subjected. If the correct material is not available, the use of a diaphragm seal may be necessary to protect the system from the process fluid. Liquid filled gauges with throttle plugs are recommended for the discharge side of positive displacement pumps.

Pressure Elements:
Available in a wide variety of materials, including: phosphor bronze, alloy steel, 316 stainless steel and K Monel.

Cases:
Ashcroft stainless steel case gauges have 304 stainless steel cases. The 2½", 3½", 100mm 1009 and the 63mm and 100mm 1008 are field convertible. These gauges can be converted to hermetically sealed, weatherproof or liquid filled by changing the fill plug and adding a throttle plug. The 40mm and 50mm 1008 gauges can be furnished from the factory hermetically sealed, weatherproof or liquid fillable. Specify the XLJ variation. With the exception of 40mm and 50mm 1008 gauges, all **dry** stainless steel gauges come standard with a vented pressure relief disc.These gauges with the vented plug **are not** weatherproof or hermetically sealed. If a weatherproof or hermetically sealed gauge is required, specify the XLJ variation and your gauge will be shipped with a solid nonventing plug.

Rings:
The ring, which retains the window, is push-in, crimped or bayonet (cam) depending on the type number.

Movements:
Movements are designed and materials of construction selected to reduce friction and extend wear life.

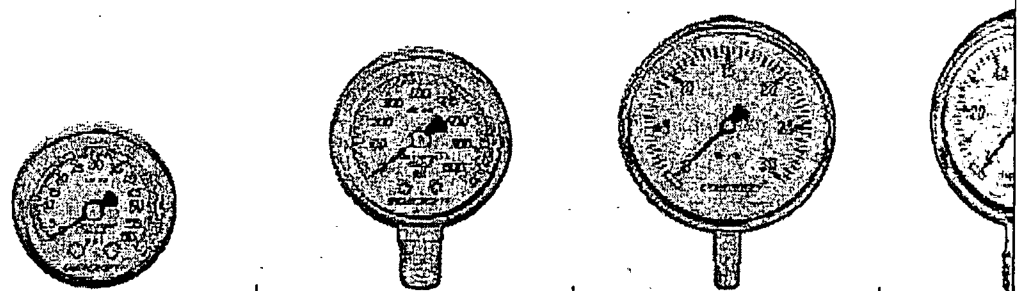
Dials:
Dials are uniformly graduated and have highly legible black markings. All 1009 gauges, with the exception of 1009 XMG, have a brushed aluminum dial with black markings. Type 1008 gauges have a white dial with black markings.

Windows:
Depending on the size and type, Ashcroft stainless steel case gauges are available with polycarbonate, acrylic, shatterproof glass or glass windows.

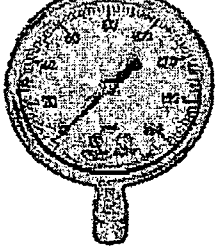
Pointers:
Depending on the type, Ashcroft stainless steel gauges are available with adjustable or fixed pointers.

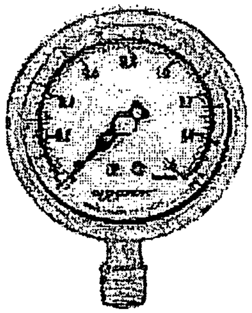
Specification Matrix

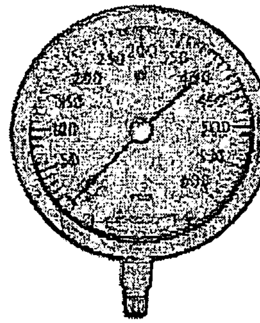
Ashcroft® Stainless Steel
Case Pressure Gauges

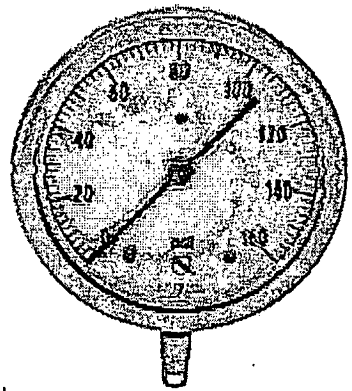


Specifications	Type 1008, 40mm	Type 1008, 50mm	Type 1008A, 63-100mm	Type 1008S
Accuracy	3-2-3% ASME Grade B	3-2-3% ASME Grade B	3-2-3% ASME Grade B	3-2-3% AS
Case Style	Open Front		Open Front	Open
Case Material	304 Stainless Steel		304 Stainless Steel	304 Stai
Dial Size (Code)	40mm (40)	50mm (50)	63mm (63), 100mm (10)	63mm (63);
Dial Material & Color	Aluminum, white background w/black markings			
Ring Type	304 Stainless Steel Push-In		304 Stainless Steel Crimped	304 Stainless
Bourdon Tube (Code)	316 Stainless Steel (S)		Bronze (A)	316 Stainle
Socket Material	316 Stainless Steel		Brass (socket O ring standard)	316 SS, Socke
Range Limits	Vac/5000 psi-(40mm)	Vac/5000 psi-(50mm)	Vac/6000 psi	Vac/15
Connection Size (Code)	⅛ NPT (01)	¼ NPT (02)	¼ NPT (02)	¼ NP
Connection Location	Lower (L), Back (B)		Lower (L), Back (B)	Lower (L
Mounting	Stem, Flush		Stem, Flush	Stem
Movement	300 Series SS (conventional)		Brass (Power Flex™) with polyester segment	300 Series SS with polyes
Pointer	Nonadjustable (Aluminum)		Nonadjustable (Aluminum)	Nonadjustabl
Window	Glass		Polycarbonate	Polyca
Warranty	One Year		One Year	One
Options Code				
Glycerin Fill (L)	Standard		Standard	Stan
Silicone Fill (GV)	N/A		Available	Avai
Halocarbon Fill (GX)	N/A		N/A	Avai
Weatherproof, hermetic seal (LJ)	Available	Available	Available	Avai
U-Clamp (UC)	Available		Available	Avai
Front Flange Ring (FF)	Available		Available	Avai
Retrofit Flange (RF)	N/A		Available	Avai
Back Flange (FW)	N/A		N/A	N
Wall Mounting Bracket (BF)	N/A		N/A	N
Acrylic Window (PD)	N/A		N/A	N
Polycarbonate Window	Standard on liquid filled gauge		Standard	Stan
Shatterproof Glass (SG)	Available		N/A	N









63-100mm	Type 1009, 2½"-3½"	Type 1009, 100mm (XMG)	Type 1009, 4½"	Type 1009, 6"
E Grade B	1% ASME Grade A	1% ASME Grade A	1% ASME Grade A	1% ASME Grade A
ont	Open Front	Open Front	Open Front	Open Front
ss Steel	304 Stainless Steel	304 Stainless Steel	304 Stainless Steel	304 Stainless Steel
0mm (10)	2½" (25), 3½" (35)	100mm (10)	4½" (45)	6" (60)
	Brushed Aluminum, w/black markings	Aluminum, white background w/black markings	Brushed Aluminum, w/black markings	
eel Crimped	304 Stainless Steel Bayonet	304 Stainless Steel Bayonet	304 Stainless Steel Bayonet	304 Stainless Steel Bayonet
Steel (S)	316 Stainless Steel	316 Stainless Steel (SW)	Bronze Tube, Brass Socket (A)	4130 Alloy Steel Tube & Socket (B)
			316 SS Tube, Steel Socket (R)	316 SS Tube & Socket (S)
			Monel Tube & Socket (P)	
Weld to Case	Bronze Socket Weld to Case (AW) 316 SS Socket Weld to Case (SW)	316 SS Socket Welded to Case		
0 psi	Vac/15,000 psi	Vac/15,000 psi	Vac/30,000 psi	Vac/30,000 psi
02)	¼ NPT (02), ½ NPT (04)	½ NPT (04)	¼ NPT (02), ½ NPT (04)	¼ NPT (02), ½ NPT (04)
Back (B)	Lower (L), Back (B)	Lower (L)	Lower (L), Back (B)	Lower (L), Back (B)
ush	Stem, Surface, Flush	Stem, Surface	Stem, Surface, Flush	Stem, Surface, Flush
ower Flex™) segment	300 Series SS (Power Flex™)	300 Series SS (Power Flex™)	400 Series SS (Conventional)	400 Series SS (Conventional)
Aluminum)	Adjustable (Aluminum)	Micrometer Adjustable (Aluminum)	Adjustable (Aluminum)	Adjustable (Aluminum)
onate	Polycarbonate	Shatterproof Glass	Glass	Glass
ear	Five Years	Five Years	One Year	One Year
ard	Standard	Standard	Standard	Standard
le	Available	Available	Available	Available
ble	Available	Available	Available	Available
ble	Available	Available	Available	Available
le	Available	Available	Available	Available
ble	Available	Available	Available	Available
le	N/A	N/A	N/A	N/A
	Available	Available	N/A	N/A
	N/A	N/A	Available	Available
	N/A	N/A	Available	Available
ard	Standard	Available	N/A	N/A
	Available	Available	Available	Available

Appendix C.6

Stainless Steel Information
(DeZurik, Enduro Stainless Steel, Allegheny Ludlum Steel Corp.)

DeZURIK
250 Riverside Ave. North
Sartell, MN 56377 – USA
E-mail: customerservice@dezurik.com
Web Page: www.dezurik.com

FAX Message

Sent by: TOM SCHOENBERG
Fax: 320 / 259 – 2131
Phone: 320 / 259 – 2273
E-mail: tom.schoenberg@dezurik.gensig.com

Fax # 415-442-7673 Phone X-7555

To: Morrison Knutson Attention: Cynthia Fox

Subject: RCV Date: Nov. 17, 2000

Stainless Steels Page: 1 of 3

Cynthia,

Thank you for your interest and phone call in regards to the Dezurik RCV (rotary control valve). As discussed, if you require updated "hardcopy" literature, CFM-SF is your local Dezurik manufactures representative.

The attached Dezurik Application Data 16.01-7, may be of interest to you. Other soures for information on stainless steels are:

1. Stainless Steel Market Development Office, sponsored by the Specialty Steel Industry of North America (SSINA), has a toll free number for technical assistance: 800 982 0355.
2. Nickel Development Institute

location	Phone	Fax
West Virginia	304-733-1516	X-1655
Toronto	416-591-7999	X-7987

Fax Copy to: CFM-SF, Inc.

Regards,
Tom

MAXUM™ ROTARY CONTROL VALVES
2205 Duplex Stainless Steel

Application Data 16.01-7
March, 1997
First Issue

DeZURIK
A UNIT OF GENERAL SIGNAL
DeZURIK, SARTELL, MINNESOTA 56377 USA
DeZURIK CANADA, CAMBRIDGE, ONTARIO, CANADA

This application sheet provides a summary description of duplex stainless steel alloy 2205 and compares the mechanical and corrosion resistance properties to other commonly used stainless steel alloys.

Alloy 2205 is known as second generation duplex stainless steels. This alloy offers excellent pitting and crevice corrosion resistance and significantly better chloride stress-corrosion-cracking resistance than 300 series austenitic stainless steels. In addition alloy 2205 has yield strengths two to three times higher than those of 304, 316, or 317 stainless steels.

First generation duplexes, e.g. 329, have existed for many years. Duplex stainless steels have a microstructure that is a mixture of austenite and ferrite. This blend produces alloys that exhibit the best features of both types of stainless steels. One problem with the first generation duplexes is they tend to lose some of their corrosion resistance when welded, which can only be restored by a postweld heat treatment.

Second generation duplex alloys were developed by adding 0.15 to 0.25% nitrogen, which reduces the chromium partitioning between the austenitic and ferritic phases of the alloys. This change enhances the pitting and crevice corrosion resistance and, when properly welded, retains the full corrosion resistance of the alloy. The mixture of austenite and ferrite is usually 50-50 in the second generation duplex alloys.

The following tables compare the compositions and the mechanical properties of several commonly used stainless steel alloys. The alloys are listed in the order of their general corrosion resistance. Corrosion resistance for specific applications may vary, and must be considered when choosing an alloy for a specific environment.

Stainless Steel Alloy	Alloy Description
304	Austenitic
17-4PH	Precipitation Hardenable
316	Austenitic
XM-19	Nitrogen Strengthened Austenitic (Nitronic 50)
317	Austenitic
904L	Austenitic
329	Duplex (first generation)
2205	Duplex (second generation)

MAXUM ™ ROTARY CONTROL VALVES
2205 Duplex Stainless Steel

Table I: Chemical Composition of Stainless Steel Alloys

Alloy	Cr	Ni	Mo	Cu	N	Other
304	18	8				
17-4 PH	16.5	4		4		
316	17	12	2.5			
XM-19	22	12.5	2		0.3	5 Mn
317	19	13	3.5			
904L	20	25	4.5	1		
329	26	4.5	1.5			
2205	22	5	3		0.15	

Table II: Mechanical Properties of Stainless Steel Alloys

Alloy	Condition	Yeild Strength		Tensile Strength		Elongation
304	A	30,000 psi	(206,700 kPa)	75,000 psi	(516,750 kPa)	40
17-4PH	A	75,000 psi	(516,750 kPa)	115,000 psi	(792,350 kPa)	18
17-4PH	H900	170,000 psi	(1,171300 kPa)	190,000 psi	(1,309,100 kPa)	10
316	A	30,000 psi	(206,700 kPa)	75,000 psi	(516,750 kPa)	40
316	B	45,000 psi	(310,050 kPa)	95,000 psi	(654,550 kPa)	28
XM-19	A	55,000 psi	(378,950 kPa)	100,000 psi	(689,000 kPa)	35
317	A	30,000 psi	(206,700 kPa)	75,000 psi	(516,750 kPa)	35
904L	A	31,000 psi	(213,590 kPa)	71,000 psi	(489,190 kPa)	35
329	A	70,000 psi	(482,300 kPa)	90,000 psi	(620,100 kPa)	15
2205	A	65,000 psi	(447,850 kPa)	90,000 psi	(620,100 kPa)	25

In conclusion, Alloy 2205 provides a stainless steel with twice the yield strength of the standard austenitic alloys and an upgrade in general corrosion resistance without any reduction of corrosion resistance in parts that are welded. Alloy 329 provides similar property advantages for parts that are not going to be welded.

ENDURO® Stainless Steel

Republic Steel Corporation
Pacific Coast District Sales
Suite 404
1499 Huntington Drive
South Pasadena CA 91030

Republicsteel

THE 300 SERIES

Type 304 and 304L (Austenitic)

Available as:
Bar, Sheet, Strip, Plate

Commonly called the "all purpose" stainless steel, Type 304 has properties desirable for many applications. For welded construction of light sections where annealing is not practical or possible, but good corrosion resistance is needed, Type 304 is recommended. Where heavier gage material is used (over ¼-inch) it is suggested that you use Type 304L, which contains less carbon than the standard Type 304. Other desirable properties of Type 304 are its satisfactory service at high temperatures (800-1600° F), and the deep drawing properties and good mechanical properties.

However, if Type 304 is exposed to high temperatures (in the 800°-1600° F range) for prolonged periods carbide precipitation may occur. Provided that corrosive conditions are not severe, Type 304 will give satisfactory service life even in high temperature applications. Because the incidence of carbide precipitation decreases as the carbon content decreases, Type 304, which has a maximum carbon content of 0.08-percent (as compared to 0.15 percent in Type 302) is considered a good material for most welding applications.

Harmful carbide precipitation should not ordinarily occur when welding light sections (up to ¼-inch thick) of Type 304. When welding thicker sections, (over ¼-inch thick) and the exposure to welding temperatures is longer, Type 304L, with a carbon content of 0.03 maximum, is recommended.

In welding applications where carbide precipitation occurs and annealing is possible, harmful carbides can be eliminated by annealing followed by rapid cooling. Annealing also relieves the residual stresses at the weld area.

Typical Applications, Type 304 and 304L:

Atomic Reactor Equipment, Chemical Processing Equipment, Food Processing and Handling Equipment, Heat Exchangers, Pharmaceutical Equipment, Still Tubes, Valves and Fittings, Beverage Equipment, Dairy Equipment, Hospital Equipment, Pulp and Paper Equipment, Textile Dyeing Equipment.

Chemical Analysis, Percent

Carbon	0.08 Max.
Chromium	18.00-20.00
Nickel	8.00-10.50
Manganese	2.00 Max.
Silicon	1.00 Max.
Phosphorus	0.045 Max.
Sulphur	0.030 Max.

Typical Mechanical Properties (Annealed)

	Sheet	Bar
Yield Strength (Offset: 0.2%)	42,000 psi	35,000 psi
Tensile Strength	84,000 psi	85,000 psi
Elongation in 2 inches	55%	60%
Reduction of Area	—	70%
Hardness	Rв 80	150 Bhn
Olsen Value, inches	0.400-0.450	—

Type 316
(Austenitic)

Available as:
Bar, Sheet, Strip, Plate

Type 316 has a 2.0 to 3.0 percent molybdenum addition which improves the corrosion resistance of austenitic stainless steels and imports hot strength characteristics. This type has, in general, better corrosion resistance to most chemicals, salts and acids. It is also more resistant to marine atmospheres.

Type 316 has better resistance to pitting corrosion than the other chromium-nickel stainless steels where brines, sulphur-bearing waters or halogen salts, such as chlorides, are present.

A valuable property of Type 316 is high creep strength at elevated temperatures. Other mechanical properties and fabricating characteristics of Type 316 are similar to Type 302 or Type 304.

Type 316L may be preferred where extensive welding is to be done. Type 318, a stabilized modification of Type 316 containing columbium, should be considered when service temperatures of about 800 to 1600° F. are employed for long periods of time.

Type 316 has extensive use in chemical processing equipment when better corrosion resistance is required than is afforded by the regular chromium-nickel types. In some cases, this type is specfied for use with high purity products where product contamination must be held to a minimum.

Typical Applications, Type 316:
Architectural Trim (Marine Exterior), Chemical Processing Equipment, Food Processing Equipment, Petroleum Refining Equipment, Pharmaceutical Equipment, Photographic Equipment, Pulp and Paper Processing Equipment, Textile Finishing Equipment.

Chemical Analysis, Percent

Carbon	0.08 Max.
Chromium	16.00-18.00
Nickel	10.00-14.00
Manganese	2.00 Max.
Silicon	1.00 Max.
Phosphorus	0.045 Max.
Sulphur	0.030 Max.
Molybdenum	2.00-3.00

Typical Mechanical Properties (Annealed)

	Sheet	Bar
Yield Strength (Offset: 0.2%)	42,000 psi	35,000 psi
Tensile Strength	84,000 psi	80,000 psi
Elongation in 2 inches	50%	60%
Reduction of Area	—	70%
Hardness	RB 80	150 Bhn
Olsen Value, inches	0.400-0.500	—

Creep Strength

Temperature °F.	Load for 1% Elongation in 10,000 hours, psi
1000	22,400
1100	16,800
1200	11,200
1300	6,900
1400	3,800
1500	2,000

Type 317
(Austenitic)

Available as:
Bar, Sheet, Strip, Plate

Type 317 is a modification of Type 316 with increased chromium, nickel and molybdenum ranges for improved corrosion resistance in special chemical applications. Its other properties are similar to Type 316.

This type is sometimes used for textile dyeing equipment and ink manufacture. It has the best corrosion resistance to body acids and blood and is recommended for surgical bone applications.

Typical Applications, Type 317:
Surgical Bone Screws. Plates. and Wire.
Pharmaceutical Equipment.
Pulp and Paper Equipment.
Ink Manufacturing Equipment.
Dyeing Equipment.
Chemical Processing Equipment.

Chemical Analysis, Percent

Carbon	0.08 Max.
Chromium	18.00-20.00
Nickel	11.00-15.00
Manganese	2.00 Max.
Silicon	1.00 Max.
Phosphorus	0.045 Max.
Sulphur	0.030 Max.
Molybdenum	3.00-4.00

Typical Mechanical Properties (Annealed)

	Sheet	Bar
Yield Strength (Offset: 0.2%)	40,000 psi	40,000 psi
Tensile Strength	90,000 psi	85,000 psi
Elongation in 2 inches	45%	50%
Reduction of Area	—	65%
Hardness	R_B 85	160 Bhn

Stainless Steel Handbook

ALLEGHENY LUDLUM STEEL CORPORATION

Pittsburgh 22, Pennsylvania

STAINLESS STEEL FINDER

Group	CHROMIUM-NICKEL AUSTENITIC GROUP					
Type Number	*201*	*202*	*301*	*302*	*304*	*304L*
Analyses — percent:						
Chromium	16.0–18.0	17.0–19.0	16.00–18.00	17.00–19.00	18.00–20.00	18.00–20.00
Nickel	3.5–5.5	4.0–6.0	6.00–8.00	8.00–10.00	8.00–12.00	8.00–12.00
Other elements (Note 6)	N_2 .25 max	N_2 .25 max				
Carbon	.15 max	.15 max	.15 max	.15 max	.08 max	.03 max
Manganese	5.5/7.5	7.5/10.0	2.00 max	2.00 max	2.00 max	2.00 max
Silicon	1.00 max	1.00 max	1.00 max	1.00 max	1.00 max	1.00 max
Physical data:						
Melting range — °F			2550–2590	2550–2590	2550–2650	2550–2650
Density — lb/in.³	.28	.28	0.29	0.29	0.29	0.29
Specific heat — Btu/°F/lb (32–212 F)	0.12	0.12	0.12	0.12	0.12	0.12
Thermal conductivity — Btu/ft²/hr/°F/ft:						
212 F			9.4	9.4	9.4	9.4
932 F			12.4	12.4	12.4	12.4
Mean coefficient of thermal expansion — in/in/°F x 10⁻⁶:						
68–212 F	9.2	9.4	9.2	9.2	9.2	9.2
68 to indicated temperature — °F	11.3 (1600)	10.9 (1600)	11.0 (1600)	11.0 (1600)	11.0 (1600)	11.0 (1600)
Electrical properties:						
Magnetic permeability at 200 H annealed	1.02 max	1.02 max	1.02	1.02	1.02	1.02
Electrical resistivity — microhm–cm:						
68 F	69.0	69.0	72.0	72.0	72.0	72.0
1200 F			116.0	116.0	116.0	116.0
Heat resistance:						
Maximum operating temperature — °F:						
Intermittent service (Note 1)	1500	1500	1600	1600	1600	1600
Continuous service	1550	1550	1700	1700	1700	1700
Temperatures—working and treating—°F:						
Forging — start	2300	2300	2200	2200	2200	2200
Forging — finish	1700	1700	1700	1700	1700	1700
Annealing — ranges (Note 2)	1850–2000	1850–2000	1950–2050	1850–2050	1800–1950	1800–1950
Annealing — cooling (Note 3)	WQ (AC)	WQ (AC)	WQ(AC)	WQ(AC)	WQ(AC)	A.C.
Hardening — ranges	(Note 7)	(Note 7)	(Note 7)	(Note 7)	(Note 7)	(Note 7)
Quenching						
Tempering — for intermediate hardness						
Drawing — for relieving stresses						
Mechanical properties — annealed:						
Structure annealed	A	A	A	A	A	A
Yield strength — lb/in.² — min	40 000	40 000	35 000	30 000	30 000	25 000
Ultimate strength — lb/in.² — min	115 000	100 000	100 000	80 000	80 000	70 000
Elongation — % in 2 inches — min	40.0	40.0	50.0	50.0	50.0	40.0
Reduction in area — % — min			60.0	60.0	60.0	60.0
Modulus of elasticity in tension — lb/in.² x 10⁶	29.0	29.0	29.0	29.0	29.0	29.0
Hardness — Brinell	210 max	210 max	180 max	180 max	180 max	180 max
Hardness — Rockwell	B95 max	B95 max	B90 max	B90 max	B90 max	B90 max
Impact values — Izod — ft-lb	85 min	85 min	85 min	85 min	85 min	80 min
Mechanical properties — heat treated:						
Yield strength — lb/in.²						
Ultimate strength — lb/in.²	(Note 8)	(Note 8)	(Note 8)	(Note 8)	(Note 8)	(Note 8)
Elongation — % in 2 inches						
Hardness — Brinell						
Hardness — Rockwell						
Creep strength — lb/in.² at 1000°F:						
1% Flow in 10,000 hr			19 000	19 000	19 000	19 000
1% Flow in 100,000 hr			13 000	13 000	13 000	13 000

R-2756

See page 5 for alternate grades.
See page 6 for notes.

STAINLESS STEEL FINDER

Group	CHROMIUM-NICKEL AUSTENITIC GROUP					
Type Number	316	316L	317	347	309	310
Analyses — percent:						
Chromium	16.00–18.00	16.00–18.00	18.00–20.00	17.00–19.00	22.00–24.00	24.00–26.00
Nickel	10.00–14.00	10.00–14.00	11.00–15.00	9.00–12.00	12.00–15.00	19.00–22.00
Other elements (Note 6)	Mo 2.00–3.00	Mo 2.00-3.00	Mo 3.00–4.00	Cb10xC min	...	...
Carbon	.08 max	.03 max	.08 max	.08 max	.20 max	.25 max
Manganese	2.00 max	2.00 max	2.00 max	2.00 max	2.00 max	2.00 max
Silicon	1.00 max	1.00 max	1.00 max	1.00 max	1.00 max	1.50 max
Physical data:						
Melting range — °F	2500–2550	2550–2650	2500–2550	2550–2600	2550–2650	2550–2650
Density — lb/in.^{3}	0.29	0.29	0.29	0.29	0.29	0.29
Specific heat — Btu/°F/lb (32-212 F)	0.12	0.12	0.12	0.12	0.12	0.12
Thermal conductivity — Btu/ft^2/hr/°F/ft:						
212 F	9.4	9.4	9.4	9.3	9.0	8.0
932 F	12.4	12.4	...	12.8	10.8	10.8
Mean coefficient of thermal expansion — in/in/°F x 10^{-6}:						
68–212 F	9.2	9.2	9.2	9.2	8.7	8.0
68 to indicated temperature — °F	10.7 (1600)	10.7 (1600)	10.7 (1600)	10.7 (1600)	10.9 (2100)	10.9 (2100)
Electrical properties:						
Magnetic permeability at 200 H annealed	1.02	1.02	1.02	1.02	1.02	1.01
Electrical resistivity — microhm–cm:						
68 F	74.0	72.0	74.0	73.0	78.0	78.0
1200 F	116.0	116.0	...	...	114.8	...
Heat resistance:						
Maximum operating temperature — °F:						
Intermittent service (Note 1)	1600	1600	1600	1600	1800	1900
Continuous service	1700	1700	1700	1700	2000	2100
Temperatures — working and treating — °F:						
Forging — start	2200	2200	2200	2200	2150	2150
Forging — finish	1700	1700	1700	1700	1800	1800
Annealing — ranges (Note 2)	1975–2150	1800-2000	1975–2150	1800–2000	2050–2150	2050–2150
Annealing — cooling (Note 3)	WQ(AC)	A.C.	WQ(AC)	WQ(AC)	WQ(AC)	WQ(AC)
Hardening — ranges	(Note 7)	(Note 7)	(Note 7)	(Note 7)	(Note 7)	(Note 7)
Quenching	...	...	...	...	...	...
Tempering — for intermediate hardness	...	...	...	...	...	...
Drawing — for relieving stresses	...	...	...	...	...	...
Mechanical properties — annealed:						
Structure annealed	A	A	A	A	A	A
Yield strength — lb/in.^{2} — min	30 000	30 000	30 000	30 000	30 000	30 000
Ultimate strength — lb/in.^{2} — min	75 000	70 000	75 000	80 000	75 000	75 000
Elongation — % in 2 inches — min	40.0	40.0	40.0	40.0	40.0	40.0
Reduction in area — % — min	50.0	60.0	50.0	50.0	50.0	50.0
Modulus of elasticity in tension — lb/in.^{2} x 10^6	29.0	29.0	29.0	29.0	29.0	30.0
Hardness — Brinell	200 max	180 max	200 max	200 max	200 max	180 max
Hardness — Rockwell	B95 max	B90 max	B95 max	B95 max	B95 max	B90 max
Impact values — Izod — ft-lb	70 min	80 min	70 min	80 min	80 min	80 min
Mechanical properties — heat treated:						
Yield strength — lb/in.^{2}						
Ultimate strength — lb/in.^{2}	(Note 8)	(Note 8)	(Note 8)	(Note 8)	(Note 8)	(Note 8)
Elongation — % in 2 inches						
Hardness — Brinell						
Hardness — Rockwell						
Creep strength — lb/in.^{2} at 1000°F:						
1% Flow in 10,000 hr	24 000	24 000	24 000	32 000	22 000	32 000
1% Flow in 100,000 hr	15 000	15 000	15 000	27 000	12 000	17 000

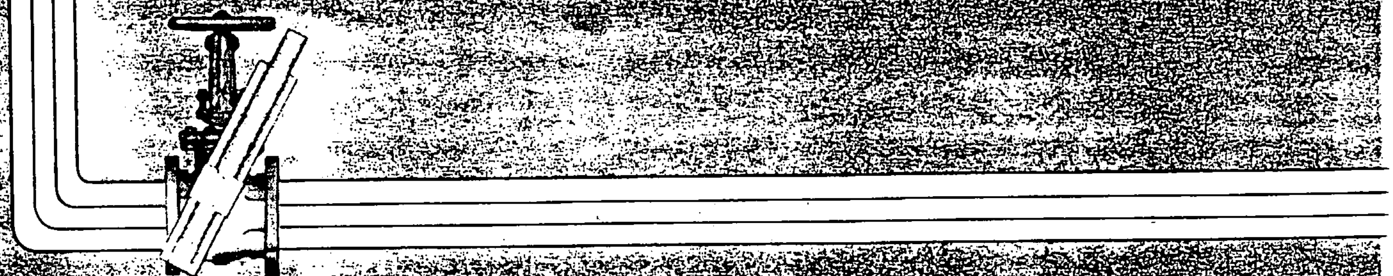
See page 5 for alternate grades.
See page 6 for notes.

Appendix C.7

Piping Information
(Crane)

CRANE

ENGINEERING DATA CATALOG



PIPE DATA
Carbon and Alloy Steel — Stainless Steel

(also see next three pages)

Nominal Pipe Size, Inches	Outside Diam., Inches	Identification: Steel, Iron Pipe Size	Identification: Steel, Sched. No.	Identification: Stainless Steel Sched. No.	Wall Thickness (t), Inches	Inside Diameter (d), Inches	Area of Metal, Square Inches	Transverse Internal Area (a), Square Inches	Transverse Internal Area (A), Square Feet	Moment of Inertia (I), Inches[4]	Weight Pipe, Pounds per foot	Weight Water, Pounds per foot of pipe	External Surface, Sq. Ft. per foot of pipe	Section Modulus $\left(2\frac{I}{O.D.}\right)$
1/8	0.405	...	...	10S	.049	.307	.0548	.0740	.00051	.00088	.19	.032	.106	.00437
		STD	40	40S	.068	.269	.0720	.0568	.00040	.00106	.24	.025	.106	.00523
		XS	80	80S	.095	.215	.0925	.0364	.00025	.00122	.31	.016	.106	.00602
1/4	0.540	...	...	10S	.065	.410	.0970	.1320	.00091	.00279	.33	.057	.141	.01032
		STD	40	40S	.088	.364	.1250	.1041	.00072	.00331	.42	.045	.141	.01227
		XS	80	80S	.119	.302	.1574	.0716	.00050	.00377	.54	.031	.141	.01395
3/8	0.675	...	...	10S	.065	.545	.1246	.2333	.00162	.00586	.42	.101	.178	.01736
		STD	40	40S	.091	.493	.1670	.1910	.00133	.00729	.57	.083	.178	.02160
		XS	80	80S	.126	.423	.2173	.1405	.00098	.00862	.74	.061	.178	.02554
1/2	0.840	...	...	5S	.065	.710	.1583	.3959	.00275	.01197	.54	.172	.220	.02849
		...	...	10S	.083	.674	.1974	.3568	.00248	.01431	.67	.155	.220	.03407
		STD	40	40S	.109	.622	.2503	.3040	.00211	.01709	.85	.132	.220	.04069
		XS	80	80S	.147	.546	.3200	.2340	.00163	.02008	1.09	.102	.220	.04780
		...	160	...	.187	.466	.3836	.1706	.00118	.02212	1.31	.074	.220	.05267
		XXS	...	...	.294	.252	.5043	.050	.00035	.02424	1.71	.022	.220	.05772
3/4	1.050	...	...	5S	.065	.920	.2011	.6648	.00462	.02450	.69	.288	.275	.04667
		...	...	10S	.083	.884	.2521	.6138	.00426	.02969	.86	.266	.275	.05655
		STD	40	40S	.113	.824	.3326	.5330	.00371	.03704	1.13	.231	.275	.07055
		XS	80	80S	.154	.742	.4335	.4330	.00300	.04479	1.47	.188	.275	.08531
		...	160	...	.219	.612	.5698	.2961	.00206	.05269	1.94	.128	.275	.10036
		XXS	...	...	.308	.434	.7180	.148	.00103	.05792	2.44	.064	.275	.11032
1	1.315	...	...	5S	.065	1.185	.2553	1.1029	.00766	.04999	.87	.478	.344	.07603
		...	...	10S	.109	1.097	.4130	.9452	.00656	.07569	1.40	.409	.344	.11512
		STD	40	40S	.133	1.049	.4939	.8640	.00600	.08734	1.68	.375	.344	.1328
		XS	80	80S	.179	.957	.6388	.7190	.00499	.1056	2.17	.312	.344	.1606
		...	160	...	.250	.815	.8365	.5217	.00362	.1251	2.84	.230	.344	.1903
		XXS	...	...	.358	.599	1.0760	.282	.00196	.1405	3.66	.122	.344	.2136
1¼	1.660	...	...	5S	.065	1.530	.3257	1.839	.01277	.1038	1.11	.797	.435	.1250
		...	...	10S	.109	1.442	.4717	1.633	.01134	.1605	1.81	.708	.435	.1934
		STD	40	40S	.140	1.380	.6685	1.495	.01040	.1947	2.27	.649	.435	.2346
		XS	80	80S	.191	1.278	.8815	1.283	.00891	.2418	3.00	.555	.435	.2913
		...	160	...	.250	1.160	1.1070	1.057	.00734	.2839	3.76	.458	.435	.3421
		XXS	...	...	.382	.896	1.534	.630	.00438	.3411	5.21	.273	.435	.4110
1½	1.900	...	...	5S	.065	1.770	.3747	2.461	.01709	.1579	1.28	1.066	.497	.1662
		...	...	10S	.109	1.682	.6133	2.222	.01543	.2468	2.09	.963	.497	.2598
		STD	40	40S	.145	1.610	.7995	2.036	.01414	.3099	2.72	.882	.497	.3262
		XS	80	80S	.200	1.500	1.068	1.767	.01225	.3912	3.63	.765	.497	.4118
		...	160	...	.281	1.338	1.429	1.406	.00976	.4824	4.86	.608	.497	.5078
		XXS	...	...	.400	1.100	1.885	.950	.00660	.5678	6.41	.42	.497	.5977
2	2.375	...	...	5S	.065	2.245	.4717	3.958	.02749	.3149	1.61	1.72	.622	.2652
		...	...	10S	.109	2.157	.7760	3.654	.02538	.4992	2.64	1.58	.622	.4204
		STD	40	40S	.154	2.067	1.075	3.355	.02330	.6657	3.65	1.45	.622	.5606
		XS	80	80S	.218	1.939	1.477	2.953	.02050	.8679	5.02	1.28	.622	.7309
		...	160	...	.344	1.687	2.190	2.241	.01556	1.162	7.46	.97	.622	.979
		XXS	...	...	.436	1.503	2.656	1.774	.01232	1.311	9.03	.77	.622	1.104
2½	2.875	...	...	5S	.083	2.709	.7280	5.764	.04002	.7100	2.48	2.50	.753	.4939
		...	...	10S	.120	2.635	1.039	5.453	.03787	.9873	3.53	2.36	.753	.6868
		STD	40	40S	.203	2.469	1.704	4.788	.03322	1.530	5.79	2.07	.753	1.064
		XS	80	80S	.276	2.323	2.254	4.238	.02942	1.924	7.66	1.87	.753	1.339
		...	160	...	.375	2.125	2.945	3.546	.02463	2.353	10.01	1.54	.753	1.638
		XXS	...	...	.552	1.771	4.028	2.464	.01710	2.871	13.69	1.07	.753	1.997
3	3.500	...	...	5S	.083	3.334	.8910	8.730	.06063	1.301	3.03	3.78	.916	.7435
		...	...	10S	.120	3.260	1.274	8.347	.05796	1.822	4.33	3.62	.916	1.041
		STD	40	40S	.216	3.068	2.228	7.393	.05130	3.017	7.58	3.20	.916	1.724
		XS	80	80S	.300	2.900	3.016	6.605	.04587	3.894	10.25	2.86	.916	2.225
		...	160	...	.438	2.624	4.205	5.408	.03755	5.032	14.32	2.35	.916	2.876
		XXS	...	...	.600	2.300	5.466	4.155	.02885	5.993	18.58	1.80	.916	3.424

Identification, wall thickness and weights are extracted from ANSI B36.10 and B36.19. The notations STD, XS, and XXS indicate Standard, Extra Strong, and Double Extra Strong pipe respectively.

Transverse internal area values listed in "square feet" also represent volume in cubic feet per foot of pipe length.

PIPE DATA — cont.

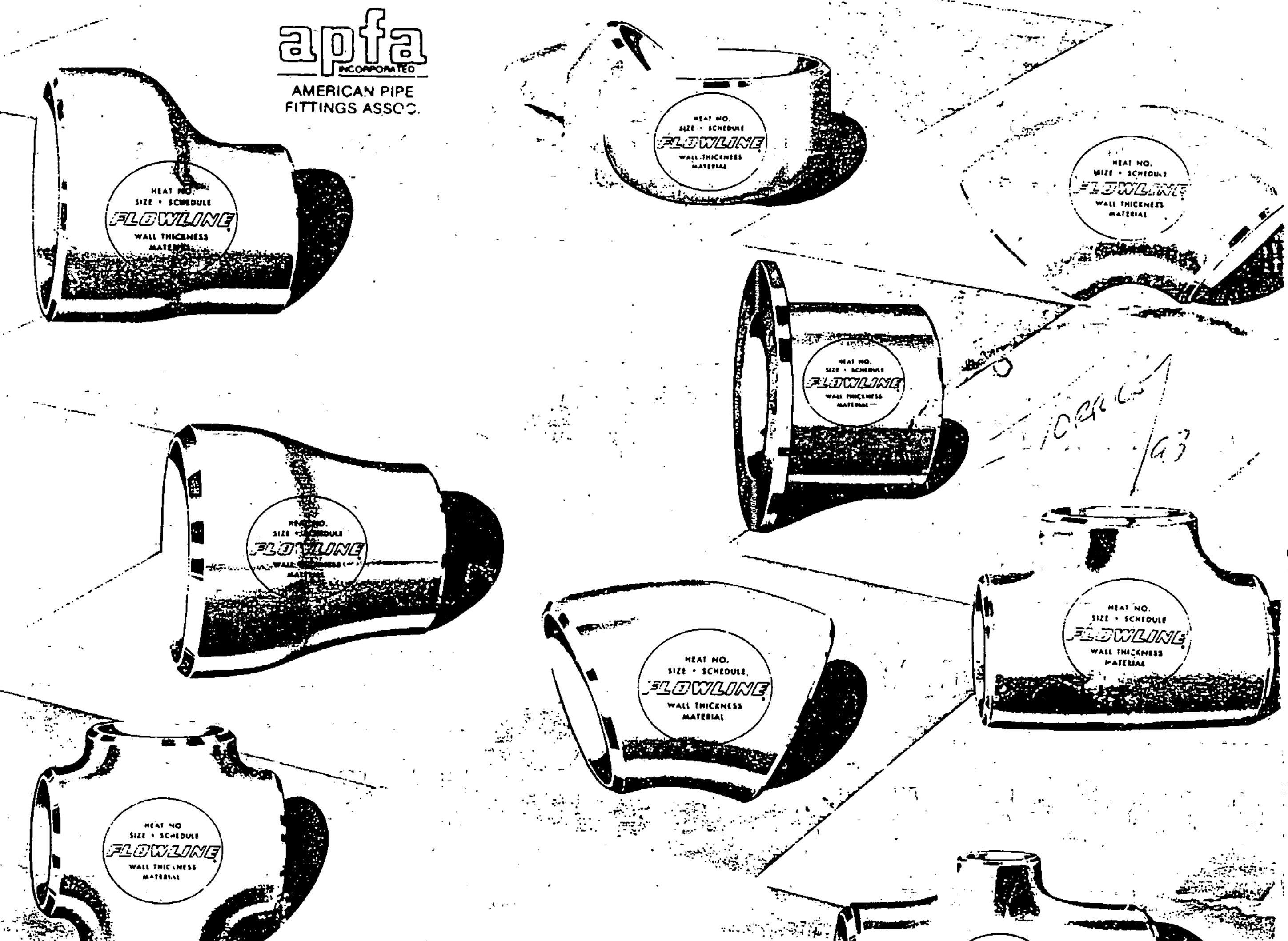
Nominal Pipe Size (Inches)	Outside Diam. (Inches)	Identification: Steel: Iron Pipe Size	Identification: Steel: Sched. No.	Identification: Stainless Steel Sched. No.	Wall Thickness (t) Inches	Inside Diameter (d) Inches	Area of Metal Square Inches	Transverse Internal Area (a) Square Inches	Transverse Internal Area (A) Square Feet	Moment of Inertia (I) Inches[4]	Weight Pipe Pounds per foot	Weight Water Pounds per foot of pipe	External Surface Sq. Ft. per foot of pipe	Section Modulus $\left(2\frac{I}{O.D.}\right)$
3½	4.000	...	...	5S	.083	3.834	1.021	11.545	.08017	1.960	3.48	5.00	1.047	.9799
		...	...	10S	.120	3.760	1.463	11.104	.07711	2.755	4.97	4.81	1.047	1.378
		STD	40	40S	.226	3.548	2.680	9.886	.06870	4.788	9.11	4.29	1.047	2.394
		XS	80	80S	.318	3.364	3.678	8.888	.06170	6.280	12.50	3.84	1.047	3.140
4	4.500	...	...	5S	.083	4.334	1.152	14.75	.10245	2.810	3.92	6.39	1.178	1.249
		...	...	10S	.120	4.260	1.651	14.25	.09898	3.963	5.61	6.18	1.178	1.761
		STD	40	40S	.237	4.026	3.174	12.73	.08840	7.233	10.79	5.50	1.178	3.214
		XS	80	80S	.337	3.826	4.407	11.50	.07986	9.610	14.98	4.98	1.178	4.271
		...	120	...	.438	3.624	5.595	10.31	.0716	11.65	19.00	4.47	1.178	5.178
		...	160	...	.531	3.438	6.621	9.28	.0645	13.27	22.51	4.02	1.178	5.898
		XXS	...	...	.674	3.152	8.101	7.80	.0542	15.28	27.54	3.38	1.178	6.791
5	5.563	...	...	5S	.109	5.345	1.868	22.44	.1558	6.947	6.36	9.72	1.456	2.498
		...	...	10S	.134	5.295	2.285	22.02	.1529	8.425	7.77	9.54	1.456	3.029
		STD	40	40S	.258	5.047	4.300	20.01	.1390	15.16	14.62	8.67	1.456	5.451
		XS	80	80S	.375	4.813	6.112	18.19	.1263	20.67	20.78	7.88	1.456	7.431
		...	120	...	.500	4.563	7.953	16.35	.1136	25.73	27.04	7.09	1.456	9.250
		...	160	...	.625	4.313	9.696	14.61	.1015	30.03	32.96	6.33	1.456	10.796
		XXS	...	...	.750	4.063	11.340	12.97	.0901	33.63	38.55	5.61	1.456	12.090
6	6.625	...	...	5S	.109	6.407	2.231	32.24	.2239	11.85	7.60	13.97	1.734	3.576
		...	...	10S	.134	6.357	2.733	31.74	.2204	14.40	9.29	13.75	1.734	4.346
		STD	40	40S	.280	6.065	5.581	28.89	.2006	28.14	18.97	12.51	1.734	8.496
		XS	80	80S	.432	5.761	8.405	26.07	.1810	40.49	28.57	11.29	1.734	12.22
		...	120	...	.562	5.501	10.70	23.77	.1650	49.61	36.39	10.30	1.734	14.98
		...	160	...	.719	5.187	13.32	21.15	.1469	58.97	45.35	9.16	1.734	17.81
		XXS	...	...	.864	4.897	15.64	18.84	.1308	66.33	53.16	8.16	1.734	20.02
8	8.625	...	...	5S	.109	8.407	2.916	55.51	.3855	26.44	9.93	24.06	2.258	6.131
		...	...	10S	.148	8.329	3.941	54.48	.3784	35.41	13.40	23.61	2.258	8.212
		...	20	...	.250	8.125	6.57	51.85	.3601	57.72	22.36	22.47	2.258	13.39
		...	30	...	.277	8.071	7.26	51.16	.3553	63.35	24.70	22.17	2.258	14.69
		STD	40	40S	.322	7.981	8.40	50.03	.3474	72.49	28.55	21.70	2.258	16.81
		...	60	...	.406	7.813	10.48	47.94	.3329	88.73	35.64	20.77	2.258	20.58
		XS	80	80S	.500	7.625	12.76	45.66	.3171	105.7	43.39	19.78	2.258	24.51
		...	100	...	.594	7.437	14.96	43.46	.3018	121.3	50.95	18.83	2.258	28.14
		...	120	...	.719	7.187	17.84	40.59	.2819	140.5	60.71	17.59	2.258	32.58
		...	140	...	.812	7.001	19.93	38.50	.2673	153.7	67.76	16.68	2.258	35.65
		XXS	...	...	.875	6.875	21.30	37.12	.2578	162.0	72.42	16.10	2.258	37.56
		...	160	...	.906	6.813	21.97	36.46	.2532	165.9	74.69	15.80	2.258	38.48
10	10.750	...	...	5S	.134	10.482	4.36	86.29	.5992	63.0	15.19	37.39	2.814	11.71
		...	...	10S	.165	10.420	5.49	85.28	.5922	76.9	18.65	36.95	2.814	14.30
		...	20	...	.250	10.250	8.24	82.52	.5731	113.7	28.04	35.76	2.814	21.15
		...	30	...	.307	10.136	10.07	80.69	.5603	137.4	34.24	34.96	2.814	25.57
		STD	40	40S	.365	10.020	11.90	78.86	.5475	160.7	40.48	34.20	2.814	29.90
		XS	60	80S	.500	9.750	16.10	74.66	.5185	212.0	54.74	32.35	2.814	39.43
		...	80	...	.594	9.562	18.92	71.84	.4989	244.8	64.43	31.13	2.814	45.54
		...	100	...	.719	9.312	22.63	68.13	.4732	286.1	77.03	29.53	2.814	53.22
		...	120	...	.844	9.062	26.24	64.53	.4481	324.2	89.29	27.96	2.814	60.32
		XXS	140	...	1.000	8.750	30.63	60.13	.4176	367.8	104.13	26.06	2.814	68.43
		...	160	...	1.125	8.500	34.02	56.75	.3941	399.3	115.64	24.59	2.814	74.29
12	12.75	...	...	5S	.156	12.438	6.17	121.50	.8438	122.4	20.98	52.65	3.338	19.2
		...	...	10S	.180	12.390	7.11	120.57	.8373	140.4	24.17	52.25	3.338	22.0
		...	20	...	.250	12.250	9.82	117.86	.8185	191.8	33.38	51.07	3.338	30.2
		...	30	...	.330	12.090	12.87	114.80	.7972	248.4	43.77	49.74	3.338	39.0
		STD	...	40S	.375	12.000	14.58	113.10	.7854	279.3	49.56	49.00	3.338	43.8
		...	40	...	.406	11.938	15.77	111.93	.7773	300.3	53.52	48.50	3.338	47.1
		XS	...	80S	.500	11.750	19.24	108.43	.7528	361.5	65.42	46.92	3.338	56.7
		...	60	...	.562	11.626	21.52	106.16	.7372	400.4	73.15	46.00	3.338	62.8
		...	80	...	.688	11.374	26.03	101.64	.7058	475.1	88.63	44.04	3.338	74.6
		...	100	...	.844	11.062	1:53	96.14	.6677	561.6	107.32	41.66	3.338	88.1
		XXS	120	...	1.000	10.750	36.91	90.76	.6303	641.6	125.49	39.33	3.338	100.7
		...	140	...	1.125	10.500	41.08	86.59	.6013	700.5	139.67	37.52	3.338	109.9
		...	160	...	1.312	10.126	47.14	80.53	.5592	781.1	160.27	34.89	3.338	122.6

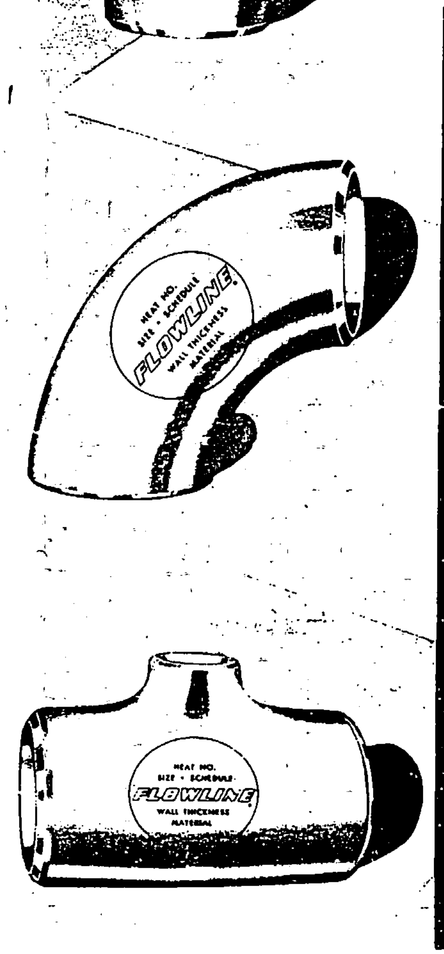
Identification, wall thickness and weights are extracted from ANSI B36.10 and B36.19. The notations STD, XS, and XXS indicate Standard, Extra Strong, and Double Extra Strong pipe respectively.

Transverse internal area values listed in "square feet" also represent volume in cubic feet per foot of pipe length.

Appendix C.8

Fittings
(Flowline Corp., Grinnell)





FLOWLINE®

STAINLESS STEEL
NICKEL, NICKEL ALLOY
ALUMINUM ALLOY

BUTT WELD FITTINGS AND FLANGES

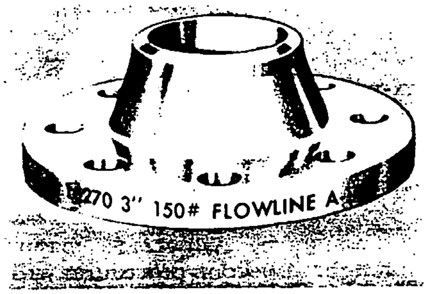
•

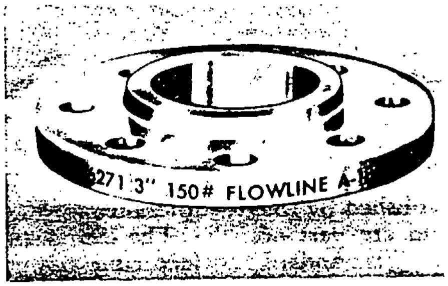
FLOWLINE CORP.

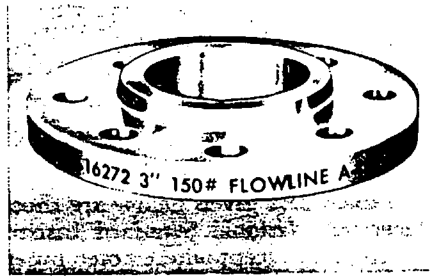
World's Largest Manufacturer of Corrosion-Resistant Fittings

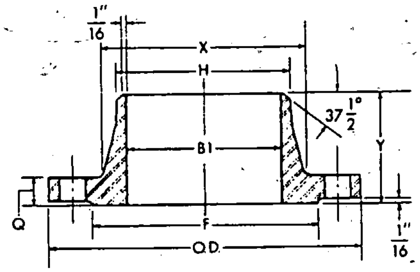
P.O. BOX 7027, NEW CASTLE, PENNSYLVANIA 16107-7027
Phone: Area Code 412/658-3711 Cable Code FLOWLINE, NCAS
Fax 412/658-6117 TELEX-81-2304 TWX-510/451/0752

CRANFORD, NJ 07016	663 Raritan	Tel. 201/709-0320	Fax 201/709-03
ARLINGTON HEIGHTS, IL 60006	P.O. Box 398	Tel. 708/392-5100	Fax 708/392-83
HOUSTON, TX 77083	15406 El Padre Drive	Tel. 713/879-7844	Fax 713/879-48
MATTHEWS, NC 28106	4705 Pioneer Lane	Tel. 704/821-9316	Fax 704/821-97
TEMECULA, CA 92390	41741 Gilwood Ct.	Tel. 714/699-1580	Fax 714/699-78
VERBENA, AL 36091	P.O. Box 128	Tel. 205/755-9100	Fax 205/755-30
WEST MIDDLESEX, PA 16159	Box 420	Tel. 412/528-1100	Fax 412/528-1

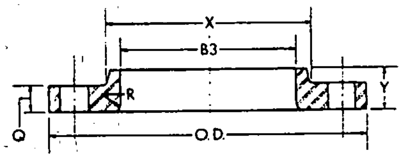




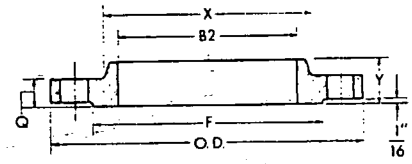




WELDING NECK



LAP JOINT



SLIP-ON

CLASS 150 CORROSION RESISTANT

NOM. PIPE SIZE	FLANGE DIAMETER (O.D.)	FLANGE THICKNESS (Q) MIN. (1)	HUB DIA. AT BASE (X)	RAISED FACE DIA. (F)	HUB DIA. AT WELDING POINT (H)	BORE DIAMETER (B): (B1) WELDING NECK AND SOCKET WELD (2)	BORE DIAMETER (B): (B2) SLIP-ON AND SOCKET WELD MIN.	BORE DIAMETER (B): (B3) LAP JOINT MIN.	COUNTER BORE OF THREADED FLANGE (C)	DEPTH OF SOCKET	LENGTH THROUGH HUB (Y): WELDING NECK	LENGTH THROUGH HUB (Y): SLIP-ON SOCKET WELD AND THREADED	LENGTH THROUGH HUB (Y): LAP JOINT
½	3.50	0.44	1.19	1.38	0.84	0.62	0.88	0.90	NO COUNTER BORE REQUIRED ON 150# THREADED FLANGES	0.38	1.88	0.62	0.62
¾	3.88	0.50	1.50	1.69	1.05	0.82	1.09	1.11		0.44	2.06	0.62	0.62
1	4.25	0.56	1.94	2.00	1.32	1.05	1.36	1.38		0.50	2.19	0.69	0.69
1 ¼	4.62	0.62	2.31	2.50	1.66	1.38	1.70	1.72		0.56	2.25	0.81	0.81
1 ½	5.00	0.69	2.56	2.88	1.90	1.61	1.95	1.97		0.62	2.44	0.88	0.88
2	6.00	0.75	3.06	3.62	2.38	2.07	2.44	2.46		0.69	2.50	1.00	1.00
2 ½	7.00	0.88	3.56	4.12	2.88	2.47	2.94	2.97		0.75	2.75	1.12	1.12
3	7.50	0.94	4.25	5.00	3.50	3.07	3.57	3.60		0.81	2.75	1.19	1.19
3 ½	8.50	0.94	4.81	5.50	4.00	3.55	4.07	4.10		0.88	2.81	1.25	1.25
4	9.00	0.94	5.31	6.19	4.50	4.03	4.57	4.60		0.94	3.00	1.31	1.31
5	10.00	0.94	6.44	7.31	5.56	5.05	5.66	5.69		0.94	3.50	1.44	1.44
6	11.00	1.00	7.56	8.50	6.63	6.07	6.72	6.75		1.06	3.50	1.56	1.56
8	13.50	1.12	9.69	10.62	8.63	7.98	8.72	8.75		1.25	4.00	1.75	1.75
10	16.00	1.19	12.00	12.75	10.75	10.02	10.88	10.92		1.31	4.00	1.94	1.94
12	19.00	1.25	14.38	15.00	12.75	12.00	12.88	12.92		1.56	4.50	2.19	2.19
14	21.00	1.38	15.75	16.25	14.00	13.25	14.14	14.18		1.63	5.00	2.25	3.12
16	23.50	1.44	18.00	18.50	16.00	15.25	16.16	16.19		1.75	5.00	2.50	3.4[illegible]
18	25.00	1.56	19.88	21.00	18.00	17.25	18.18	18.20		1.94	5.50	2.69	3.81
20	27.50	1.69	22.00	23.00	20.00	19.25	20.20	20.25		2.12	5.69	2.88	4.06
24	32.00	1.88	26.12	27.25	24.00	23.25	24.25	24.25		2.50	6.00	3.25	4.38

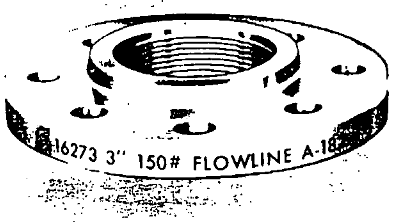
Dimensions are in inches and conform to ANSI B16.5. For dimensional tolerances, see page 65.

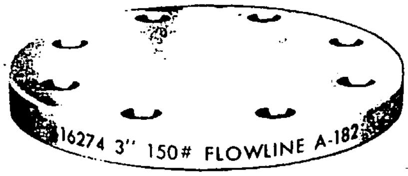
(1) Flange Thickness (Q) includes $\frac{1}{16}$" raised face.

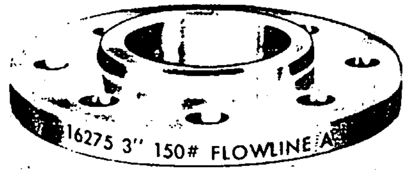
(2) Bore Diameter (B) of welding neck flanges corresponds to matching dimension of Standard Wall/Schedule 40S pipe. Flanges can be bored to match Extra Strong/Schedule 80S pipe.

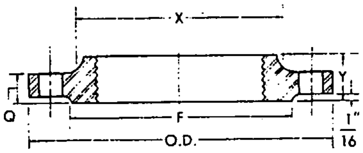
(3) Every flange is marked to show the ***FLOWLINE*** trademark, type of metal, pressure rating, and pipe size in accordance with ANSI B16.5.

(continued on next page)

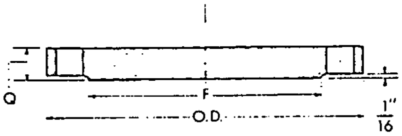




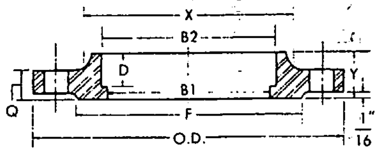




THREADED



BLIND



SOCKET WELD

FORGED FLANGES ANSI B16.5

STAINLESS STEEL
INCO® ALLOY
ALUMINUM

			DRILLING				BOLTING		
							MACHINE BOLT LENGTH	STUD BOLT LENGTH	
NOM. PIPE SIZE	MIN. THREAD LENGTH	RADIUS (R) LAP JOINT	DIAMETER OF BOLT CIRCLE	NO. OF HOLES	DIA. OF HOLES	DIA. OF BOLTS	Raised Face .06"	Raised Face .06"	Ring Joint
½	0.62	0.12	2.38	4	0.62	½	2.00	2.50	—
¾	0.62	0.12	2.75	4	0.62	½	2.25	2.50	—
1	0.69	0.12	3.12	4	0.62	½	2.25	2.75	3.25
1¼	0.81	0.19	3.50	4	0.62	½	2.50	2.75	3.25
1½	0.88	0.25	3.88	4	0.62	½	2.50	3.00	3.50
2	1.00	0.31	4.75	4	0.75	⅝	2.75	3.25	3.75
2½	1.12	0.31	5.50	4	0.75	⅝	3.00	3.50	4.00
3	1.19	0.38	6.00	4	0.75	⅝	3.25	3.75	4.25
3½	1.25	0.38	7.00	8	0.75	⅝	3.25	3.75	4.25
4	1.31	0.44	7.50	8	0.75	⅝	3.25	3.75	4.25
5	1.44	0.50	8.50	8	0.88	¾	3.25	4.00	4.50
6	1.56	0.50	9.50	8	0.88	¾	3.50	4.00	4.50
8	1.75	0.50	11.75	8	0.88	¾	3.75	4.25	4.75
10	1.94	0.50	14.25	12	1.00	⅞	4.00	4.75	5.25
12	2.19	0.50	17.00	12	1.00	⅞	4.25	4.75	5.25
14	2.25	0.50	18.75	12	1.12	1	4.50	5.25	5.75
16	2.50	0.50	21.25	16	1.12	1	4.75	5.50	6.00
18	2.69	0.50	22.75	16	1.25	1⅛	5.00	6.00	6.50
20	2.88	0.50	25.00	20	1.25	1⅛	5.50	6.25	6.75
24	3.25	0.50	29.50	20	1.38	1¼	6.00	7.00	7.50

	APPROXIMATE WEIGHT EACH—POUNDS‡			
NOM. PIPE SIZE	WELDING NECK	SLIP-ON, SOCKET WELD AND THREADED	LAP JOINT	BLIND
½	1¼	1	1	1¼
¾	2	1½	1½	1½
1	2½	2	2	2
1¼	3	3	3	3
1½	4¼	3	3½	4
2	6	5¼	5	6½
2½	10	8	7	9½
3	12	9	9¼	12½
3½	12	11	11	13
4	16½	12½	12½	17
5	19	15	15	20
6	25	19	19	26½
8	39	30	30	45
10	52	43	43	70
12	80	64	64	110
14	110	90	105	140
16	140	98	140	180
18	150	130	160	220
20	180	165	195	285
24	260	220	275	430

(4) Length Through Hub (Y) does include 1/16" raised face.

(5) Length of stud bolt does not include the height of the points.

(6) For flange facing details see pages 84 through 87.

‡Weights shown are for Stainless Steel. Approximate Nickel and Nickel Alloy weights are obtained by multiplying by 1.12. Approximate Aluminum weights are obtained by multiplying by .33.

PIPE FITTINGS

Catalog PF-91

Grinnell is the leading manufacturer and distributor of iron pipe fittings in North America, having produced cast iron screwed pressure and drainage fittings and cast iron flanged fittings since the early 1900's. The company has produced malleable iron pipe fittings for several decades.

Grinnell fittings may be found on most piping systems throughout the United States and Canada, this popularity is due to the company's reputation for producting high quality products combined with a nationwide distribution system.

Grinnell pipe fittings are manufactured to conform with applicable standards and this conformance is rigorously monitored by the Grinnell Quality Department.

WARNING

Pipe fittings included in this catalog are intended for installation and service as described herein.

We are aware that these pipe fittings have been used successfully for purposes other than for which they were designed and we also know that on occasion these products have failed when so misused. Examples of misapplication which can result in failure and in personal or property damage include: overtightening; using too much torque in "making on"; re-use of fittings which may have been damaged in removing; tightening in-line under pressure causing possible damage to the threads and weakening the joint; the use of fittings in load bearing structures such as handrails; the use of plain untested or drainage fittings in pressure applications; using pressure fittings in systems beyond the listed pressure and/or temperature limitations.

Our customers should exercise care to use these products properly so as to avoid any possible on-the-job accident.

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A. S. T. M. PRODUCT CROSS INDEX

Metal	Type	Pipe	Tubing	Welding Fittings[1]	Flanges	Welding Rod
Stainless Austenitic Steel	Type 304 18 Cr-8 Ni	A312-TP304 A358-304 A376-TP304	A213-TP304 A249-TP304 A269-TP304 A271-TP304	A403-WP304	A182-F304	308
		312-TP304H 376-TP304H	A213-TP304H A249-TP304H A271-TP304H	A403-WP304H	A182-F304H	308
		A312-TP304L	A213-TP304L A249-TP304L A271-TP304L	A403-WP304L	A182-F304L	308-L
	Type 309 25 Cr-12 Ni	A312-TP309 A358-309	A249-TP309	A403-WP309	A314-309	309
	Type 310 25 Cr-20 Ni	A312-TP310 A358-310	A213-TP310 A249-TP310	A403-WP310	A182-F310	310
	Type 316 16 Cr-13 Ni with 2½ Mo	A312-TP316 A358-316 A376-TP316	A213-TP316 A249-TP316 A269-TP316	A403-WP316	A182-F316	316
		A312-TP316H A376-TP316H	A213-TP316H A249-TP316H	A403-WP316L	A182-F316H	316
		A312-TP316L	A213-TP316L A249-TP316L A269-TP316L	A403-WP316L	A182-F316L	316-L
	Type 317 16 Cr-13 Ni with 3½ Mo	A312-TP317	A249-TP317 A269-TP317	A403-WP317	A314-317	317
	Type 321 18 Cr-8 Ni with Ti	A312-TP321 A358-321 A376-TP321	A213-TP321 A249-TP321 A269-TP321 A271-TP321	A403-WP321	A182-F321	347
		A312-TP321H A376-TP321H	A213-TP321H A249-TP321H A271-TP321H	A403-WP321H	A182-F321H	347
	Type 347 18 Cr-8 Ni with Ta-Cb	A312-TP347 A358-347 A376-TP347	A213-TP347 A249-TP347 A269-TP347 A271-TP347	A403-WP347	A182-F347	347
		A312-TP347H A376-TP347H	A213-TP347H A249-TP347H A271-TP347H	A403-WP347H	A182-F347H	347
	Type 348 18 Cr-8 Ni with Cb	A312-TP348 A358-348 A376-TP348	A213-TP348 A249-TP348 A269-TP348 A271-TP348	A403-WP348	A182-F348	347
		A312-TP348H	A213-TP348H A249-TP348H A271-TP348H	A403-WP348H	A182-F348H	
Nickel and Nickel Base Alloys	**Nickel—200**	B161	B161	B366-WPN	(2)	Nickel #61
	Nickel—201 (low carbon)	B161	B161	B366-WPNL	(2)	
	Monel—400 Ni-Cu	B165	B165	B366-WPNC	(2)	Monel #60
	Inconel—600 Ni-Cr-Fe	B167	B167	B366-WPNCI	(2)	Inconel #62
	Alloy B—(Hastelloy) Ni-Mo	(2)	(2)	B366-WPHB	(2)	
	Alloy C—(Hastelloy) Ni-Mo-Cr	(2)	(2)	B366-WPHC	(2)	
Aluminum	3003F	B241	B210 B221 B234	B361	(2)	1100 or 404
	5083-0	B241	B210 B221 B234	B361	(2)	5356
	6061-T6	B241	B210 B221 B234	B361	(2)	5356 or 404

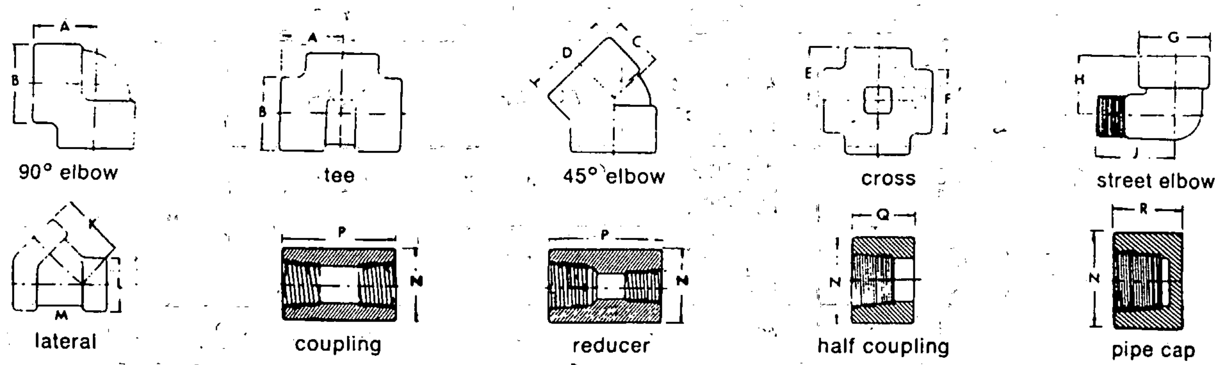
1. When fittings are of welded construction, the fitting manufacturer shall supplement the grade symbol marking with the letter "W".

2. No ASTM specification has been written. However, materials having chemical and physical properties comparable to the other materials listed may be used.

forged steel threaded

Pressure ratings, psi (non shock)				
Schedule	40	80	XXS	
Class	2000	3000	6000	
Cold water, oil, gas, air	2000	3000	6000	
900° F steam, hot oil, vapor	615	925	1855	(carbon steel)

Forged steel threaded fittings conform to ANSI B16.11.



dimensions (inches)

Class 2000

size	1/8	1/4	3/8	1/2	3/4	1	1 1/4	1 1/2	2	2 1/2	3	3 1/2	4
A	13/16	13/16	31/32	1 1/8	1 5/16	1 1/2	1 3/4	2	2 3/8	3	3 3/8	....	4 3/16
B	7/8	7/8	1	1 5/16	1 1/2	1 13/16	2 3/16	2 7/16	2 31/32	3 5/8	4 5/16	....	5 3/4
C	11/16	11/16	3/4	7/8	1	1 1/8	1 5/16	1 3/8	1 11/16	2 1/16	2 1/2	....	3 1/8
D	7/8	7/8	1	1 5/16	1 1/2	1 13/16	2 3/16	2 7/16	2 31/32	4	4 5/8	....	5 3/4
E	13/16	13/16	31/32	1 1/8	1 5/16	1 1/2	1 3/4	2	2 3/8	3 1/4	3 3/8	....	4 3/16
F	7/8	7/8	1	1 5/16	1 1/2	1 13/16	2 3/16	2 7/16	2 31/32	4	4 5/8	....	5 3/4

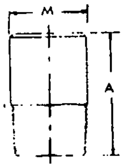
Class 3000

size	1/8	1/4	3/8	1/2	3/4	1	1 1/4	1 1/2	2	2 1/2	3	3 1/2	4
A	13/16	31/32	1 1/8	1 5/16	1 1/2	1 3/4	2	2 3/8	2 1/2	3 1/4	3 3/4	4 1/2	4 1/2
B	7/8	1	1 5/16	1 1/2	1 13/16	2 3/16	2 7/16	2 31/32	3 5/16	4	4 3/4	6	6
C	11/16	3/4	7/8	1	1 1/8	1 5/16	1 3/8	1 11/16	1 23/32	2 1/16	2 1/2	3 1/8	3 1/8
D	7/8	1	1 5/16	1 1/2	1 13/16	2 3/16	2 7/16	2 31/32	3 5/16	4	4 5/8	5 3/4	5 3/4
E	13/16	31/32	1 1/8	1 5/16	1 1/2	1 3/4	2	2 3/8	2 1/2	3 1/4	3 3/8	4 3/16	4 3/16
F	7/8	1	1 5/16	1 1/2	1 13/16	2 3/16	2 7/16	2 31/32	3 5/16	4	4 5/8	5 3/4	5 3/4
G	15/16	15/16	1 5/16	1 1/2	1 3/4	2	2 7/16	2 3/4	3 5/16	....	....	....	....
H	7/8	7/8	1 1/8	1 1/8	1 3/8	1 3/4	2	2 1/8	2 1/2	....	....	....	....
J	1 1/4	1 1/4	1 1/2	1 5/8	1 7/8	2 1/4	2 5/8	2 13/16	3 5/16	....	....	....	....
K	....	1 3/4	1 3/4	1 3/4	2 1/8	2 1/2	2 13/16	3 3/16	4 3/8	....	....	....	....
L	....	1 1/2	1 1/2	1 1/2	1 3/4	2	2 7/16	2 3/4	3 5/16	....	....	....	....
M	....	2 1/2	2 1/2	2 1/2	3	3 1/2	4	4 1/2	6	....	....	....	....
N	5/8	3/4	7/8	1 1/8	1 3/8	1 3/4	2 1/4	2 1/2	3	3 5/8	4 1/4	4 3/4	5 1/2
P	1 1/4	1 3/8	1 1/2	1 7/8	2	2 3/8	2 5/8	3 1/8	3 3/8	3 5/8	4 1/4	4 1/2	4 3/4
Q	5/8	11/16	3/4	15/16	1	1 3/16	1 5/16	1 9/16	1 11/16	1 13/16	2 1/8	2 1/4	2 3/8
R	3/4	1	1	1 1/4	1 7/16	1 5/8	1 3/4	1 3/4	1 7/8	2 3/8	2 9/16	2 5/8	2 11/16

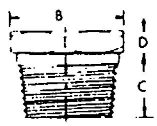
Class 6000

size	1/8	1/4	3/8	1/2	3/4	1	1 1/4	1 1/2	2	2 1/2	3	3 1/2	4
A	31/32	1 1/8	1 5/16	1 1/2	1 3/4	2	2 3/8	2 1/2	3 1/4	3 3/4	4 3/16	4 1/2	....
B	1	1 5/16	1 1/2	1 13/16	2 3/16	2 7/16	2 31/32	3 5/16	4	4 3/4	5 3/4	6	....
C	3/4	7/8	1	1 1/8	1 5/16	1 3/8	1 11/16	1 23/32	2 1/16	2 1/2	3 1/8	3 1/8	....
D	1	1 5/16	1 1/2	1 13/16	2 3/16	2 7/16	2 31/32	3 5/16	4	4 5/8	5 3/4	5 3/4	....
E	31/32	1 1/8	1 5/16	1 1/2	1 3/4	2	2 3/8	2 1/2	3 1/4	3 3/8	4 3/16	4 3/16	....
F	1	1 5/16	1 1/2	1 13/16	2 3/16	2 7/16	2 31/32	3 5/16	4	4 5/8	5 3/4	5 3/4	....
G	15/16	1 5/16	1 1/2	1 3/4	2	2 7/16	2 3/4	3 5/16	....	....	....	....	....
H	7/8	1 1/8	1 1/8	1 3/8	1 3/4	2	2 1/8	2 1/2	....	....	....	....	....
J	1 1/4	1 1/2	1 5/8	1 7/8	2 1/4	2 5/8	2 13/16	3 5/16	....	....	....	....	....
K	....	1 3/4	1 3/4	2 1/8	2 1/2	2 13/16	3 3/16	4 5/8	....	....	....	....	....
L	....	1 1/2	1 1/2	1 3/4	2	2 7/16	2 3/4	3 5/16	....	....	....	....	....
M	....	2 1/2	2 1/2	3	3 1/2	4	4 1/2	6	....	....	....	....	....
N	7/8	1	1 1/4	1 1/2	1 3/4	2 1/4	2 1/2	3	3 5/8	4 1/4	5	5 3/4	6 1/4
P	1 1/4	1 3/8	1 1/2	1 7/8	2	2 3/8	2 5/8	3 1/8	3 3/8	3 5/8	4 1/4	4 1/2	4 3/4
Q	5/8	11/16	3/4	15/16	1	1 3/16	1 5/16	1 9/16	1 11/16	1 13/16	2 1/8	2 1/4	2 3/8
R	3/4	1	1 3/16	1 3/8	1 1/2	1 13/16	1 7/8	1 7/8	2	2 1/2	2 11/16	2 7/8	2 15/16

forged steel threaded

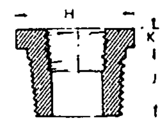


round head plug



hex head plug

square head plug



hexagon bushing

flush bushing

dimensions (inches)

size	1/8	1/4	3/8	1/2	3/4	1	1 1/4	1 1/2	2	2 1/2	3	3 1/2	4
A	1 3/8	1 5/8	1 5/8	1 3/4	1 3/4	2	2	2	2 1/2	2 3/4	2 3/4	3	3
B	7/16	5/8	11/16	7/8	1 1/16	1 7/16	1 13/16	2	2 1/2	3	3 3/4	4 1/4	5
C	9/16	5/8	11/16	3/4	15/16	1	1	1	1 1/16	1 5/16	1 1/2	1 7/16	1 7/8
D	1/4	1/4	5/16	5/16	3/8	3/8	9/16	5/8	11/16	3/4	11/16	7/8	1
E	9/32	3/8	7/16	9/16	5/8	13/16	15/16	1 1/8	1 5/16	1 1/2	1 11/16	1 7/8	2 1/2
F	3/8	7/16	1/2	9/16	5/8	3/4	13/16	13/16	7/8	1 1/16	1 1/8	1 3/16	1 1/4
G	1/4	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	1 1/4
H	....	5/8	11/16	7/8	1 1/16	1 7/16	1 13/16	2	2 1/2	3	3 3/4	4 1/4	5
J	....	1/2	9/16	11/16	3/4	13/16	7/8	15/16	1	1 3/16	1 9/16	1 3/8	1 7/8
K	....	1/8	3/16	3/16	1/4	1/4	3/8	3/8	7/16	1/2	11/16	1/2	1
L	....	7/16	7/16	1/2	9/16	5/8	11/16	3/4	13/16	15/16	1 1/8	1 1/8	1 1/4
M	13/32	17/32	11/16	27/32	1 1/16	1 5/16	1 11/16	1 29/32	2 3/8	2 7/8	3 1/2	4	4 1/2

weights (lb each)

size	1/8	1/4	3/8	1/2	3/4	1	1 1/4	1 1/2	2	2 1/2	3	3 1/2	4
Class 2000													
90° elbow, fig. 2101	0.21	0.17	0.28	0.49	0.70	1.03	1.63	2.04	3.38	6.56	10.00	....	22.50
45° elbow, fig. 2102	0.18	0.14	0.23	0.45	0.59	0.90	1.40	1.65	2.63	7.63	12.00	....	19.75
tee, fig. 2103	0.28	0.26	0.34	0.69	0.95	1.35	2.10	2.75	4.25	9.06	13.50	....	32.50
cross, fig. 2104	0.59	0.50	0.40	0.80	1.05	1.65	2.35	3.28	5.00	17.20	22.10	....	38.60
Class 3000													
90° elbow, fig. 2111	0.21	0.31	0.60	0.91	1.43	2.28	2.88	4.88	5.44	10.00	17.13	37.12	29.25
45° elbow, fig. 2112	0.18	0.26	0.54	0.75	1.18	2.03	2.13	4.05	4.25	7.63	12.00	26.03	19.75
street elbow, fig. 2113	0.20	0.25	0.36	0.53	0.85	1.38	2.25	2.81	5.09	....	....	....	....
tee, fig. 2114	0.28	0.43	0.84	1.23	1.85	3.00	3.63	6.83	7.00	13.75	21.00	49.62	38.00
cross, fig. 2115	0.59	0.50	0.96	1.43	2.30	3.73	4.40	8.13	8.31	16.94	20.75	45.70	34.13
lateral, fig. 2116	....	1.10	1.06	0.93	1.50	2.62	2.94	4.50	8.25	....	....	....	....
coupling, fig. 2117	0.11	0.10	0.13	0.28	0.42	0.85	1.50	2.19	3.02	4.56	6.79	8.03	12.00
half coupling, fig. 2119	0.06	0.05	0.07	0.14	0.21	0.43	0.75	1.10	1.51	2.28	3.40	4.02	6.00
reducer, fig. 2118	0.11	0.10	0.13	0.28	0.42	0.85	1.50	2.19	3.02	4.56	6.79	8.03	12.00
cap, fig. 2120	0.08	0.09	0.11	0.24	0.39	0.72	1.32	1.54	2.34	4.05	5.84	7.09	10.08
Class 6000													
90° ell, fig. 2131	0.37	0.66	1.00	1.59	2.54	3.56	5.88	7.06	13.00	21.78	36.22	37.35	....
45° ell, fig. 2132	0.25	0.59	0.85	1.34	2.25	2.59	4.56	5.75	9.63	15.46	31.21	26.21	....
street elbow, fig. 2133	....	0.40	0.99	1.00	1.63	2.75	3.86	7.23	....	....	....	....	....
tee, fig. 2134	0.44	0.92	1.38	2.16	3.63	4.83	7.75	9.75	17.38	28.90	49.60	52.00	....
cross, fig. 2135	0.50	1.12	1.55	2.59	4.21	5.64	9.58	11.39	21.37	28.32	55.88	46.16	....
lateral, fig. 2136	....	1.31	1.18	1.84	4.10	4.23	5.62	11.84	....	....	....	....	....
coupling, fig. 2137	0.18	0.14	0.40	0.69	0.90	1.88	2.31	4.00	7.50	9.25	13.44	18.53	22.13
half coupling, fig. 2141	0.09	0.07	0.20	0.35	0.45	0.94	1.16	2.00	3.75	4.63	6.72	9.27	11.07
reducer, fig. 2138	0.18	0.14	0.40	0.69	0.90	1.88	2.31	4.00	7.50	9.25	13.44	18.53	22.13
cap, fig. 2143	0.13	0.13	0.19	0.31	0.44	0.75	1.31	1.69	3.25	....	....	....	....
plugs and bushings • Class 2000, 3000, 6000													
plugs													
square head, fig. 2122	0.02	0.03	0.06	0.11	0.19	0.36	0.60	0.84	1.38	2.12	3.38	4.76	8.44
hex head, fig. 2142	0.03	0.06	0.10	0.16	0.30	0.55	1.03	1.36	2.29	3.81	4.75	8.37	13.00
round head, fig. 2121	0.05	0.10	0.16	0.26	0.43	0.74	1.21	1.58	3.11	4.88	7.19	10.34	13.25
hex bushing, fig. 2139	....	0.02	0.03	0.06	0.11	0.19	0.39	0.36	0.84	1.19	2.56	5.50	7.06
flush bushing, fig. 2140	....	0.03	0.03	0.06	0.07	0.12	0.14	0.14	0.34	0.45	1.18	1.35	1.70

Appendix D

Appendix D

Coring and Grouting Services

D.1 Scope of Work and Pricing Schedule

D.2 Technical Specifications

Appendix D.1

Scope of Work and Pricing Schedule

ATTACHMENT I

SCOPE OF WORK AND PRICING SCHEDULE

CORING AND GROUTING SERVICES
FOR
WALKER MINE SEAL TESTING AND EVALUATION PROJECT
PLUMAS COUNTY, CALIFORNIA

1. BACKGROUND

The Walker Mine is an inactive mine located in Plumas County, about 23 miles northwest of Portola, California. The general site location is shown on Figure 1. Directions to the site are shown on Figure 2. The approximate topography of the site is shown on Figure 3. Four-wheel-drive vehicles may be needed to reach the site depending on road conditions.

In 1987, the California Regional Water Quality Control Board (RWQCB) constructed a concrete plug, or seal, in the 700 Level Adit, with the purpose of stopping Acid Mine Drainage (AMD) discharges that issued from the adit and impacted aquatic resources downstream of the mine. The seal is located about 2,700 feet from the portal and is seated in granodiorite rock (see Figure 4). The seal is about 9 feet wide by 12 feet high and is about 15 feet thick. Two 4-inch-diameter stainless steel pipes are embedded in the seal to allow draining of the impounded water. The pipes are controlled by valves. A sampling port with a pressure transducer is mounted on one of the drain pipes. The transducer is connected to a data logger which is monitored by the RWQCB.

The water pressure behind the seal typically ranges from 60 to 90 psi. The water is mildly acidic, with a pH of about 4. Some leakage occurs through the rock-concrete interface and the rock around the seal. The leakage rate is estimated to be in the order of 0.2 gpm. The seepage collects in a pool at the toe of the seal and in a ditch along the left sidewall (looking toward the seal). The pool has a depth of up to 2 feet and a volume in the range of 1,000 to 2,000 gallons.

The adit is nearly horizontal, with a grade generally less than one percent. Access to the adit is through a heavy steel door. The floor of the adit is covered with soil (mine muck) which supports a narrow gauge track. The track ends within 20 feet from the seal.

The adit begins with a 150-foot-long cut-and-cover section supported by corrugated metal pipe lining. This section, constructed recently, is in good condition and reasonably dry. The next 1,100 feet of the adit is heavily timbered and very wet. The surrounding ground is decomposed or highly weathered granodiorite. Water drains into the adit, dripping through the roof and pooling on the ground to a depth of up to about 6 inches. The water overflows into the ditch that runs along the left side of the adit.

Beyond the first 1,300 feet or so from the portal, the adit runs through generally fresh or slightly weathered granodiorite. The opening is unsupported and essentially dry. Water from leakage through the seal is found in the ditch along the sidewall beginning about 200 feet from the seal.

A 24-inch-diameter ventilation duct runs along the left side of the adit from the portal to within 20 feet of the seal. The duct hangs from the roof in the timbered section and lies on the ground in the unsupported section. The pressure monitoring cable runs along the right side of the adit. It too hangs from the roof in the timbered section and lies on the ground on the unsupported section.

The timbered section has the minimum cross-sectional dimensions along the adit, with clear height as low as about 6 feet and width of about 4 feet. The section in sound rock is wider and taller, with typical dimensions of 8 feet wide by 10 feet high. (Please note that dimensions listed herein are very rough and need to be verified by the bidder during the site visit to ensure clearance for equipment.)

2. SCOPE

2.1 General

The general scope of the coring and grouting program is to (1) drill approximately six angled core holes into the concrete seal and adjacent rock and one horizontal hole into the concrete seal, (2) allow for nondestructive testing of the seal by others using the holes, (3) water pressure test the concrete-rock contact and the surrounding rock, (4) pressure grout the concrete-rock contact and the surrounding rock, and (5) tightly backfill the holes with grout. The purposes of the core holes are to (1) observe concrete and rock conditions, (2) collect continuous concrete and rock core samples for laboratory testing, (3) allow for nondestructive testing of the concrete seal and concrete-rock interface using the holes to insert the testing probes, and (4) pressure grout the rock mass surrounding the seal to fill open seepage features and thus reduce seepage flows along the concrete-rock interface. Approximate boring locations and orientations are shown on Figure 5. The work shall be performed in accordance with the attached Technical Specifications (Attachment II).

2.2 Scope of Work

The Subcontractor shall furnish all equipment, personnel, and supplies necessary for performing and completing this Scope of Work in a timely manner in accordance with the approved Work Plan and schedule.

2.2.1 Mobilization (Bid Item No. 1)

Mobilization includes but is not limited to the following:

- Submittals: Subcontractor shall provide the submittals specified in the Technical Specifications and shall obtain the Engineer's approval of said submittals before mobilizing equipment or beginning on-site work.
- Procurement and mobilization to site of all equipment, materials, supplies and personnel necessary to execute the work.
- Site preparation including access improvements if needed by Subcontractor.
- Installation of water, power and/or compressed air lines to the mine seal as necessary to perform the work.
- Installation of sanitary and potable water facilities.
- Dewatering of the seepage pool at the toe of the seal.
- Adit ventilation and air quality monitoring.
- All other necessary preparatory work to be performed prior to the coring and grouting work.

2.2.2 Core Drilling and Grouting (Bid Item No. 2)

Core drilling and grouting includes but is not limited to the following:

- Installation of seven 4-inch-diameter grout pipes, packers, valves and pressure gauges in the mine seal. Grout pipes will be located and oriented as approximately shown on Figure 5 and as directed by the Engineer's on-site representative. The grout pipe and packer setup shall be suitable for stopping flow through the hole and grouting the hole should a pressurized seam be intercepted. The setup shall allow the introduction into the hole of testing equipment with diameter of 3-3/4 inches.
- Coring a length of approximately eight feet of concrete and rock through each of the seven grout pipes with a 3-7/8-inch-diameter core bit, yielding a 2-11/16-inch-diameter core. Core length shall be measured from the face of the concrete seal.
- Measuring and recording of the water flow issuing from each hole.
- Shutting off the valve at the grout pipe and measuring the pressure buildup in the pipe at 8-hour intervals (during working hours only).
- Providing two full, consecutive calendar days of standby time after all holes are cored to allow nondestructive testing through the holes by the Engineer and other subcontractors, mapping of rock characteristics downstream of the seal, operational testing of the drain pipe shutoff valves, and other tests as Engineer considers necessary. During the standby time, Subcontractor shall continue to operate the fan to maintain the adit ventilated and shall continue to dewater the seepage pool at the toe of the mine seal. Standby days shall be working days (Monday through Friday).
- At the completion of testing by the Engineer and other subcontractors, washing, water pressure testing, and pressure grouting and backfilling the seven core holes in accordance with the attached technical Specifications.
- All ancillary work associated with performing the above activities, including but not limited to providing lighting, air, dewatering, monitoring

air quality, tailgate safety meetings and safety supervision, retracting the air duct and locking up the adit gate during the off-shifts, etc.

2.2.3 Demobilization (Bid Item No.3)

Demobilization includes but is not limited to the following:

- Removal and demobilization of all personnel, equipment, materials and supplies.
- Repair of any damage caused by Subcontractor's operations. Repairs shall be made to the satisfaction of the Engineer and the RWQCB.
- Cleanup of adit, portal and other areas used by Subcontractor to their pre-existing condition.

2.2.4 Core boxes (Bid Item No.4)

This item includes the wooden core boxes actually used to store and transport rock and concrete cores. Core boxes shall be as specified in Attachment II.

2.2.5 Microfine Cement and Prepackaged Nonshrink Hydraulic-Cement Grout (Bid Items no. 5 and 6)

Microfine cement and prepackaged nonshrink hydraulic-cement grout shall be as specified in Attachment II. Measurement for payment for providing microfine cement and hydraulic-cement grout will be made at the end of each shift, and will include all microfine cement and hydraulic-cement grout consumed in mixing grout during that shift as determined by the Engineer. No payment will be made for hydraulic-cement grout or microfine cement used in grout that is wasted or lost due to improper anchorage of grout nipples, or as a result of equipment breakdown. Hydraulic-cement grout or microfine cement used in grout satisfactorily mixed but not injected for reasons beyond the Subcontractor's control will be measured for payment. Payment will be at the respective unit prices specified in the Pricing Schedule, and will include all costs for furnishing, handling, storing, and protecting hydraulic-cement grout and microfine cement until used to produce grout. The unit of measure will be a bag (equivalent to 44 pounds) for microfine cement and a bag (equivalent to 50 lbs) for nonshrink hydraulic-cement grout.

2.2.6 Modifications to Scope of Work (Bid Items No. 7, 8 and 9)

The Engineer reserves the right to alter any elements of the core drilling and grouting program as deemed necessary to best suit the site conditions encountered. Possible modifications include but are not limited to the following:

- Adding or deleting core holes

- Lengthening or shortening holes
- Increasing or reducing the number of calendar days of standby time beyond the two full calendar days included under the "Core Drilling and Grouting" scope item.

Modifications are to be priced using bid items 7 through 9 as follows:

- Add/subtract 8-foot-long core hole: Addition or removal of each 8-foot-long hole to the seven holes included in Item 2. The work includes all activities covered under Item 2 for the added or deleted hole.
- Add/subtract length of core hole: Engineer-directed increase or reduction in the length of drilling and grouting beyond the 8 feet per hole which is included in Item 2. The work includes all activities covered under Item 2 for the added or deleted length of core hole.
- Add/subtract standby time: Engineer-directed increase or reduction in duration of standby time beyond the two full calendar working days which are included in Item 2. The work includes all activities covered under Item 2 for the standby time.

3. PRICING SCHEDULE

Work will be paid at Unit and Lump Sum prices listed in the Pricing Schedule (see attached). It is the responsibility of the Subcontractor to make a thorough investigation of the site conditions and Technical Specifications to determine the scope of work included in the items listed in the Pricing Schedule. The payment of said prices will constitute complete compensation for all work performed under this subcontract, and for all costs of accepting the general risks and liabilities, and shall include but not be limited to, compensation for labor, equipment, materials, services, per diems, supplies and consumables, and overhead and profit to perform the work specified under each item. Work listed in the descriptions of lump sum items in the Scope section is intended to be indicative but not all inclusive.

For items bid on a Unit Price basis, the estimated quantities given on the Pricing Schedule are approximate and are given only as a basis for comparison of bids. The Engineer does not either expressly or by implication warrant that the actual quantities will correspond to the estimated quantities. The Engineer reserves the right to increase or decrease the amount of work performed under unit price items, or to omit such work altogether. No adjustments in Subcontract unit prices will be made, nor will any claim for loss of anticipated profits be allowed on account of any such increase, decrease or omission. Payment for unit price items will be made at the unit prices stated in the Subcontractor's pricing schedule. Unit price items will be measured in accordance with the methods of measurement described above.

Payment will not be made for any of the following:

- Materials wasted or disposed of in a manner that is not called for in the subcontract.

- Materials determined by the Engineer to be unacceptable before or after placement.
- Materials not completely unloaded from the transporting vehicle.
- Core holes drilled beyond the length of the required work as indicated in the subcontract or as directed by the Engineer.
- Materials remaining on hand after completion of the work.
- Loading, hauling, handling, and disposal of rejected materials.
- Standby time due to missing materials or equipment, equipment breakdown, or any other reason except as expressly directed by the Engineer.

4. WORK SCHEDULE

Subcontractor shall complete mobilization within four weeks from receipt of Notice to Proceed (NTP), and shall complete demobilization six weeks from receipt of NTP.

Subcontractor shall provide proposed work schedule with the bid. Provide beginning and end dates for the following activities, assuming a NTP date on or about August 13, 2001:

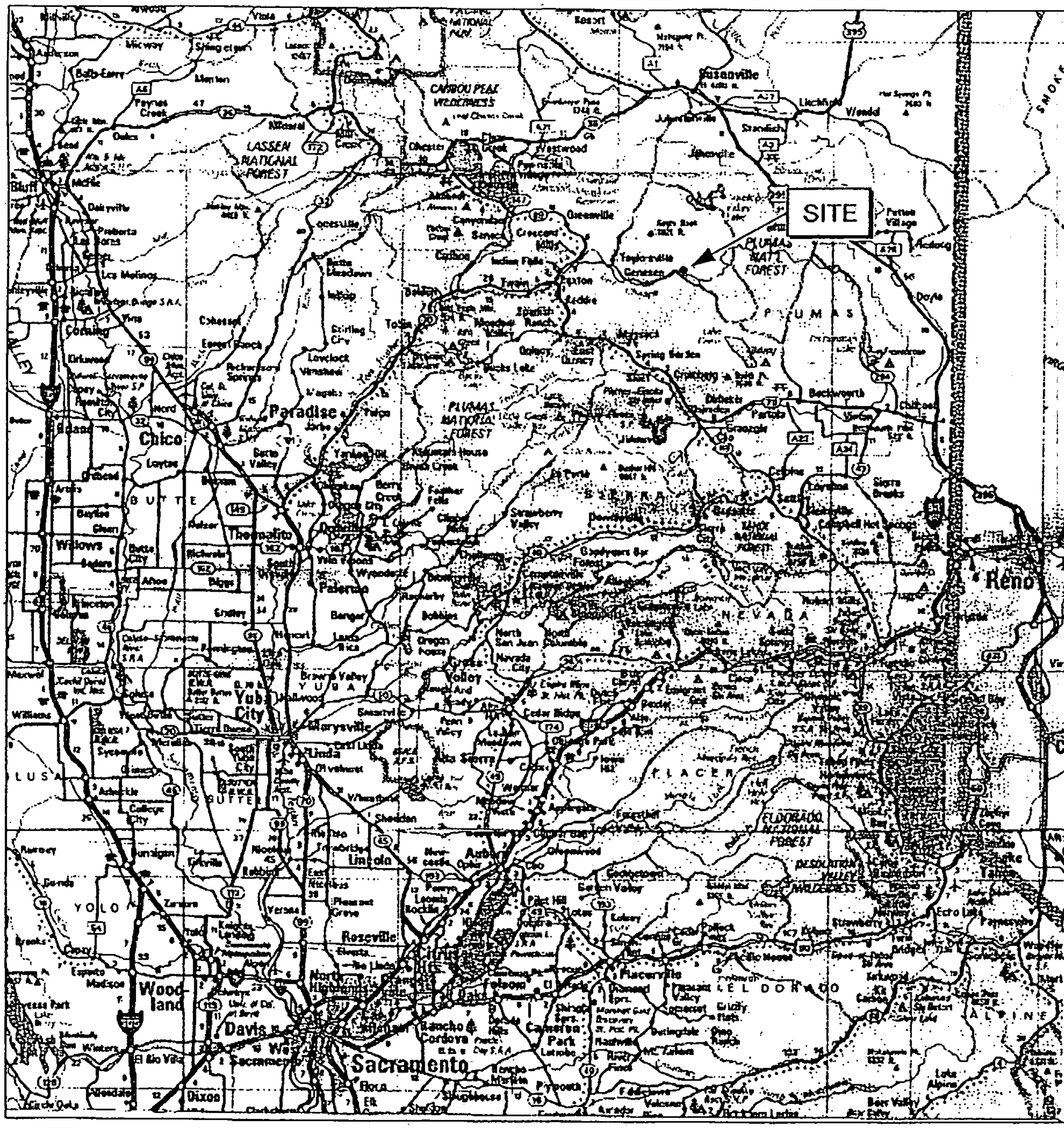
Submittals-	Begin: 8/13/01	End: 8/17/01
Mobilization –	Begin: 8/20/01	End: 8/23/01
Drilling –	Begin: 8/24/01	End: 8/31/01
Standby –	Begin: 9/04/01	End: 9/05/01
Pressure Testing and Grouting –	Begin: 0/06/01	End: 9/10/01
Demobilization -	Begin: 9/11/01	End: 9/12/01

PRICING SCHEDULE

Item #	Item Description	Units	Quanity	Unit Price	Total Price
1	Mobilizaion	Lump Sum	1	15000	15000
2	Core Drilling and Grouting. Includes two full, consecutiveworing days on standby at the completion of drilling	Lump Sum	1	53000	53000
3	Demobilization	Lump Sum	1	6000	6000
4	Core Boxes	Each	7	120	840
5	Microfine Cement	Bag	20	100	2000
6	Prepackaged Hydraulic-Cement (Nonshrink) Grout	Bag	30	100	3000
7	Add/Subtract an 8 foot Core Hole	Each	0	4000	
8	Add/Subtract length of Core Hole	Foot	0	275	
9	Add/Subtract Standby time	Crew-Hour	0	225	
Total					$79,840.00

Sincerely,

Jerry J. Jensen
Secretary/Tresurer



LOCATION OF SITE ABOVE

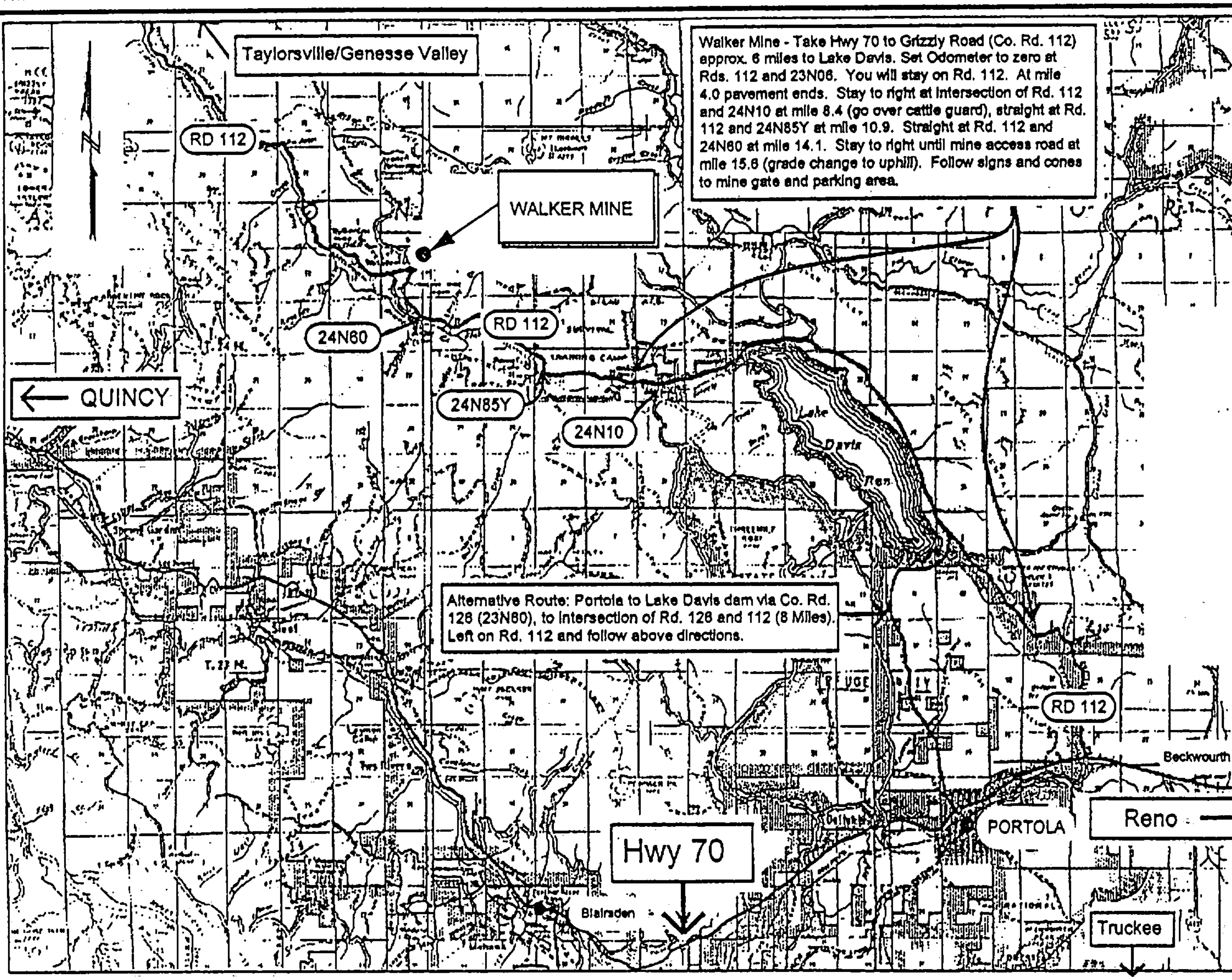
NOTE

Map is taken from California road map dated 1998.

Regional Water Quality Control Board Central Valley Region	Walker Mine Seal Testing & Evaluation Portola, CA	PROJECT LOCATION MAP	
GEI Consultants, Inc.	Project 00387	Nov. 2000	Figure 1

Fig-1.dwg 11-30-00 PYM

Fig-2.dwg 11-30-00 PYM



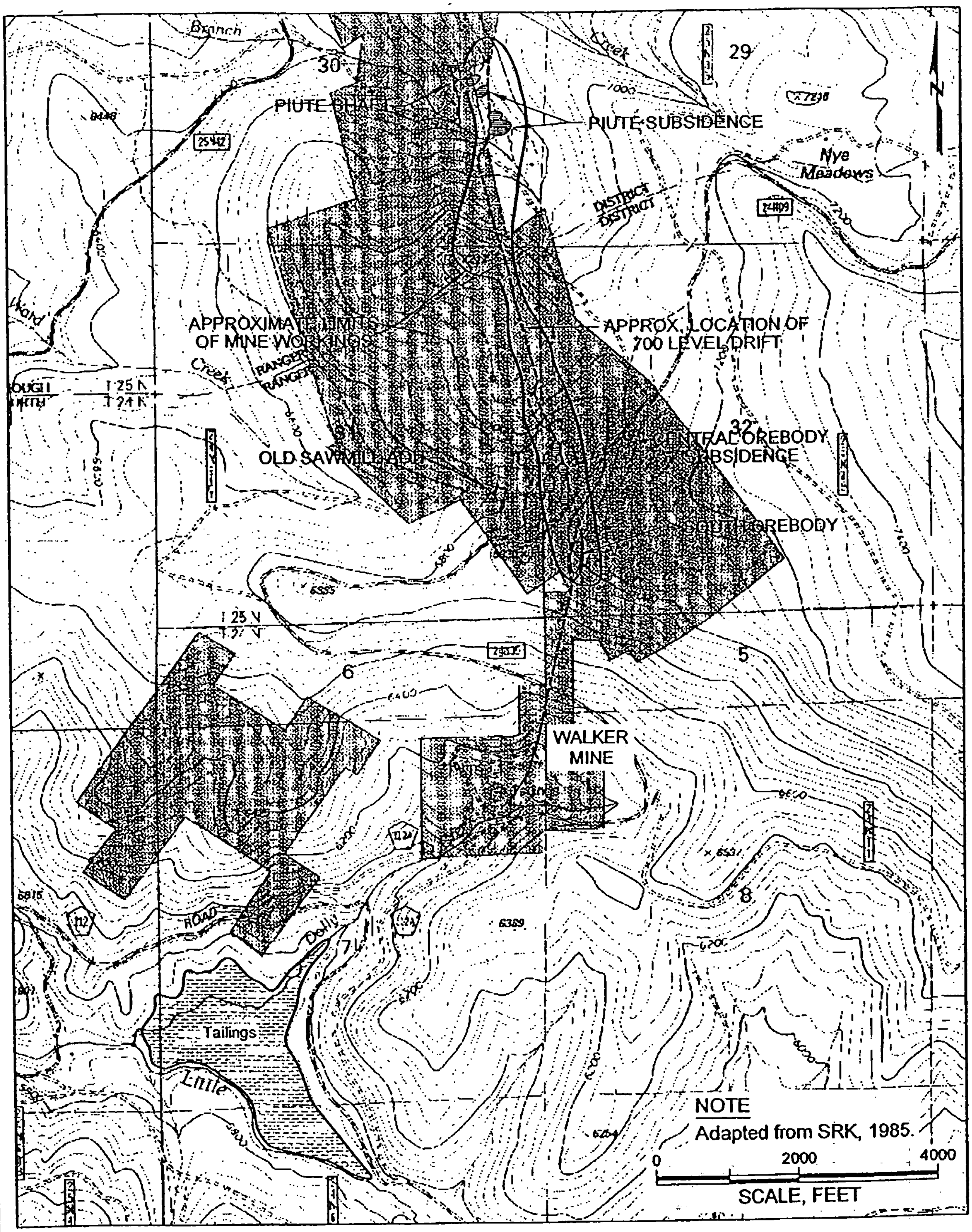
0 2.5 5 10

(Approx.) SCALE, MILES

NOTE

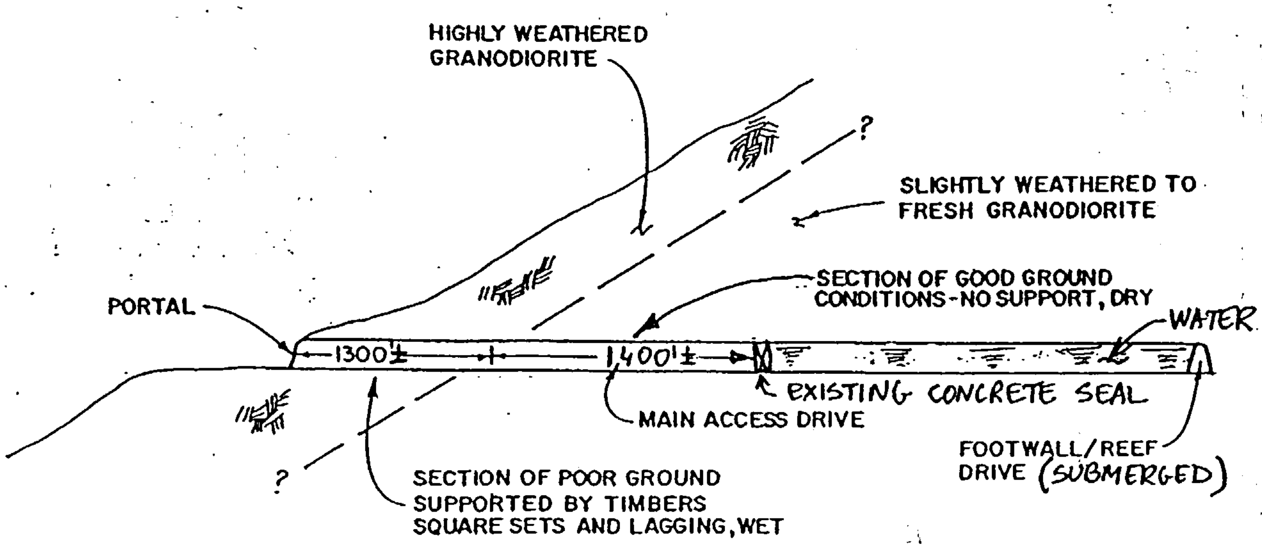
Map is provided by RWQCB.

Regional Water Quality Control Board Central Valley Region	Walker Mine Seal Testing & Evaluation Portola, CA	DIRECTIONS TO SITE	
GEI Consultants, Inc.	Project 00387	Nov. 2000	Figure 2



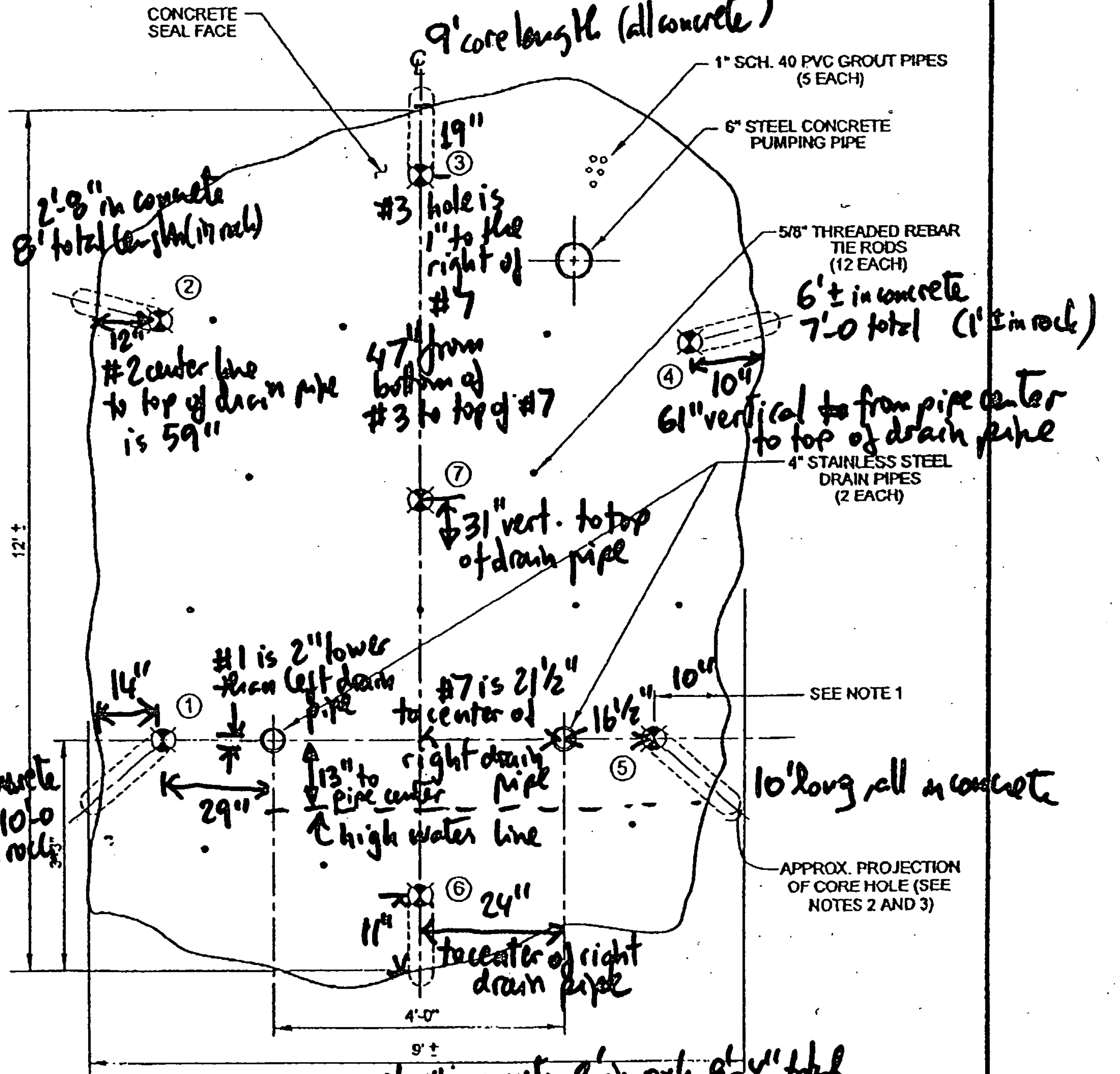
Regional Water Quality Control Board Central Valley Region	Walker Mine Seal Testing & Evaluation Portola, CA	SITE PLAN AND TOPOGRAPHY	
GEI Consultants, Inc.	Project 00387	Nov. 2000	Figure 3

Fig-3.dwg 11-27-00 PYM



(NOT TO SCALE)

PROJECT NO.	PREPARED BY:	FIGURE 4
06901	STEFFEN ROBERTSON & KIRSTEN	
DATE 3/85	Consulting Engineers	MAIN ADIT SECTION
REVISION NO. 0	Modified by GEI Consultants 4/23/01	



NOTES

1. Core hole entry point is about 8 to 12 inches from the rock face. Hole diameter is 4 inches.
2. Hole orientation projected on a vertical plane passing through the entry point:
   - Holes 1, 5 and 6 are oriented at an angle of 2 to 5 degrees downward.
   - Hole 7 is parallel to the axis of the seal.
   - Holes 2, 3 and 4 are oriented at an angle of 2 to 5 degrees upward.
3. Hole orientation projected on a horizontal plane passing through the entry point:
   - Holes 4 and 5 are oriented at an angle of 7 to 10 degrees toward the right (toward the rock sidewall).
   - Holes 1 and 2 are oriented at an angle of 7 to 10 degrees toward the left (toward the rock sidewall).
   - Holes 3, 6 and 7 are parallel to the axis of the seal.

LEGEND

⑥ APPROX. CORE HOLE LOCATION AND NUMBER

0 1 2

SCALE, FEET

Fig-7.dwg 12-04-00 PYM

Regional Water Quality Control Board Central Valley Region	Walker Mine Seal Testing & Evaluation Portola, CA	BOREHOLE LOCATIONS AND ORIENTATIONS
GEI Consultants, Inc.	Project 00387	Nov. 2000 · Figure 5

Note: Handwritten dimensions were measured during the drilling program to reflect actual locations of the coreholes.

Appendix D.2

Technical Specifications

ATTACHMENT II

TECHNICAL SPECIFICATIONS FOR CORING AND GROUTING

FOR

WALKER MINE SEAL TESTING AND EVALUATION PROJECT
PLUMAS COUNTY, CALIFORNIA

TABLE OF CONTENTS

PART 1 – GENERAL

1.1 DESCRIPTION

A. The work consists of furnishing all labor, plant, equipment and materials, and performing all operations, including testing, in connection with core drilling, sampling, washing, and pressure water testing in core holes; making grout connections, furnishing, handling, transporting, storing, mixing and injecting the grout materials; providing care and disposal of drill cuttings, waste water and waste grout; caulking and sealing surface fractures when leaking grout; cleaning of the areas upon completion of the work; and all such other operations as are incidental to the drilling and grouting. It includes furnishing all transportation and services, including fuel, power, water and essential communications, and other operations required for the efficient, effective and safe performance of the drilling and grouting operations.

B. Principal components of the Work include the following:

1. Locating all holes at the direction of the Engineer.

2. Installing grout pipes or nipples with packers, valves, and pressure gauges.

3. Drilling core holes and retrieving concrete and rock cores.

4. Allowing time for nondestructive testing and observation by the Engineer and other subcontractors.

5. Washing and pressure testing the core holes.

6. Proportioning, mixing, pumping and injecting grout into the core holes.

7. Completely backfilling and drypacking all core holes.

8. Protecting the mine seal, drain pipes, valves, monitoring equipment, air duct and other facilities from damage during drilling and grouting operations, and repairing all damaged items to the satisfaction of the Engineer and the RWQCB.

9. Cleaning up the work areas and disposing of all waste products and materials produced by or resulting from the grouting operations in a safe, legal and environmentally-acceptable manner.

C. The Subcontractor shall keep sufficient equipment and crews on the site so that work shall proceed and complete in a timely and orderly manner.

1.2 DEFINITIONS

A. Engineer: The Engineer referred to in these specifications is GEI Consultants, Inc., or the authorized representative thereof. During on site field work, communications with the Engineer shall be through the Engineer's jobsite representative. The Engineer shall have sole authority to make changes in specified procedures, boring locations, and work quantities, and to determine the acceptability of work performed under these specifications.

B. Subcontractor: The Subcontractor referred to in these specifications is the drilling and grouting specialty firm contracted by the Engineer to perform the work described in these specifications and all required incidental work.

C. Lugeon: Permeability unit equivalent to a flow of 1 liter per meter of borehole being tested, per minute, measured at a pressure of 10 atmospheres.

D. Water-Cement Ratio: Ratio by weight of water to cement.

E. Microfine Cement: Finely ground portland or portland/slag cement with an average particle size of 4 microns or less.

1.3 QUALITY ASSURANCE

A. Reference Standards

1. American Society for Testing Materials (ASTM)

- ASTM C94 Standard Specification for Ready-Mixed Concrete
- ASTM C150 Standard Specification for Portland Cement.
- ASTM C940 Expansion and Bleeding of Freshly Mixed Grouts for Preplaced-Aggregate Concrete in the Laboratory
- ASTM C1107 Standard Specification for Packaged Dry, Hydraulic-Cement Grout (Nonshrink)

2. American Petroleum Institute (API) API RP 13B; Standard Procedure for Testing Drilling Fluids, Section 1, Density (Mud Weight) and Section 2, Viscosity and Gel Strength.

3. Corps of Engineers: CRD-C 614 – Methods of Test for Time of Setting of Grout Mixtures.

B. Records: Cooperate with and assist the Engineer in the development of complete and accurate records of the drilling and grouting operations.

C. Subcontractor shall possess a valid California State Contractors License issued by the State Contractors License Board. The license classification shall be appropriate for the work to be performed.

D. Follow applicable ASTM specifications for coring and sample collection, where applicable or otherwise specified by the Engineer.

1.4 SUBMITTALS

A. Injury and Illness Prevention Plan: Seven days prior to mobilization, submit to the Engineer for review a project specific Injury and Illness Prevention Plan, as required by Cal OSHA, to address all of Subcontractor's operations. Subcontractor's Injury and Illness Prevention Plan shall meet the requirements of the Engineer's Health and Safety Plan for the Walker Mine Work.

B. Work Plan: Seven days prior to mobilization, submit to the Engineer for review and approval a Work Plan detailing the following:

1. Proposed drilling methods and equipment, including (a) copies of manufacturer's brochures relating to the equipment; (b) utility requirements and proposed installation; (c) detailed description and sketches for the packer, valve and pressure gauge installation proposed for the head of each core hole; (d) explain how the water flows from the holes and pressures in the holes will be monitored and measured; (e) describe how the angle of the drill holes (from horizontal and vertical axes) will be measured and maintained; (f) describe the procedure that will be used to ensure that the hole alignment is maintained as the drill hole crosses the rock-concrete interface (at the specified angle) and to ensure that good quality core is obtained across the interface for laboratory shear strength testing of the interface.

2. Proposed grouting methods and equipment including (a) specifications or copies of manufacturer's brochures relating to the equipment; (b) explain how the grout will be mixed, maintained in suspension, and placed in the hole; (c) explain how the specified pressures will be maintained; (d) explain how the grout take will be measured; and (e) explain how the holes will be backfilled.

3. Dewatering equipment, lighting, generator and any additional support equipment and requirements to perform the work.

4. The names and addresses of suppliers of the materials to be used, product brochures and Material Safety Data Sheets for all hazardous substances that are to be brought on site, and a list of all materials that are to be provided in bulk.

5. A description of communications system to be provided and used.

6. A detailed schedule for performance of the work, including beginning and end dates for submittal preparation, mobilization, drilling, standby for testing by others, pressure testing and grouting, and demobilization.

7. Procedures and details for the protection of existing piping, valves, fittings and monitoring equipment, during drilling and grouting operations.

C. Reports and Records:

1. At least one week before starting field work, submit to the Engineer calibration certificates for all pressure gauges, flowmeters and pressure transducers.

2. Drillers Core Hole Logs: Upon completion of each core hole, prepare and submit to the Engineer, in a manner satisfactory to him, an accurate Driller's Log of each hole drilled including any water pressure tests performed. Include hole start and finish times, drilling rates, and a nontechnical description of all materials encountered in the drilling, their location in the holes and location of special features, such as seams, open cracks, soft or broken rock or concrete, points where abnormal loss or gain of flush water occurred, and any other items of interest in connection with the purpose for which the drilling is required.

3. Maintain and submit to the Engineer daily logs of grouting operations, including pressures, volumes and grout mix pumped.

PART 2 PRODUCTS

2.1 GENERAL

A. The cement based suspension grout will largely be composed of water, microfine cement and a dispersing agent (superplasticizer).

Superplasticizer shall be added to stabilize the mix and modify the viscosity.

B. Grout for filling the holes shall be prepackaged expansive cement grout.

2.2 MATERIALS

A. Prepackaged expansive cement grout for backfilling the holes shall conform to ASTM Specification C 1107 Standard Specification for Packaged Dry, Hydraulic-Cement Grout (Nonshrink), Grade C.

B. Microfine cement shall be a portland/slag cement or portland cement with a grain size range equivalent to MC-500 microfine cement as supplied by De Neef Construction Chemicals, Waller, TX, or approved equal.

C. Water furnished for use in the drilling and grouting operations shall meet the requirements for water as specified in Paragraph 4.1.3.1 of ASTM C94. The water source must be approved by the Engineer.

D. Dispersing Agent (superplasticizer): Melamine based deflocculator such as NS-200 as supplied by De Neef Construction Chemicals, Waller, TX, or approved equal.

E. Dry-pack: One part Portland cement (type II), 2-1/2 parts minus No. 16 mesh silica sand, and sufficient water that the material can be squeezed into a ball in the hand and will leave the hand damp but not wet.

F. No fast setting chemical grout shall be used at this site.

G. Protect all grouting materials other than water from becoming wet or contaminated. Take special care with microfine cement, which may absorb moisture from the air. Do not use broken or partial bags of portland cement or microfine cement, or bags containing hardened lumps.

2.3 EQUIPMENT

A. Drilling Equipment: The Subcontractor shall determine and submit his drilling method and equipment in the Work Plan. The hole diameter shall be not less than 3-7/8 inches. Coring shall be accomplished in 5-foot advances or less using a 5-foot-long double-tube core barrel with nonrotating inner barrels and diamond bits. Only clean water shall be used as the drilling fluid.

B. Pressure Testing Equipment: The Subcontractor shall furnish equipment necessary for water pressure testing. The following is a minimum list of type of equipment to be furnished by the Subcontractor to perform water pressure testing and pressure grouting:

1. Pumps: non-pulsing positive displacement pump, screw type, similar or equal to a Moyno pump with a capacity of not less than 5 gpm at a pressure of 150 psi.

2. Packers: Pneumatic packers, 3-feet-long, and accessories to operate them including compressed air or nitrogen supply, regulators, gages, and tubing. Provide suitable numbers of packers as necessary to meet working conditions and schedule.

3. Flowmeters: Freshly calibrated flowmeters providing direct readout in cubic feet with smallest reading increment not more than 0.01 cubic foot.

4. Pressure gages: Provide freshly calibrated new pressure gages with capacities of 0-200 psi and increments not greater than 5 psi. Provide sufficient number to cover replacement and recalibration without any delays in work. All pressure gages shall be glycerin filled, plain case with pressures indicated in pounds per square inch as manufactured by Marsh Instrument Company, a unit of General Signal, P.O. Box 1011, Skokie, IL 60076 or approved equal.

C. Cement Grout Equipment: The cement grout plant shall be capable of supplying, mixing, agitating, pumping and delivering the grout to the satisfaction of the Engineer. It shall be maintained in satisfactory operating conditions at all times. The arrangement of the grouting equipment shall be such as to provide a continuous circulation of grout throughout the system and to permit accurate pressure control by operation of a valve on the grout return line, regardless of how small the grout take may be. The equipment and lines shall be prevented from becoming fouled by the constant circulation of grout and by the periodic flushing out of the system with water. The grouting equipment to be furnished by Subcontractor shall be as necessary to perform the work specified herein. As a minimum, the grouting equipment to be furnished shall include the following:

1. Grout pump: Non-pulsing positive displacement pump, screw type, similar or equal to a Moyno pump with capacity of not less than 5 gpm of grout, and capable of generating a pressure of 150

psi at the collar of the grout hole. In no case shall the pump be separated by more than 200 feet of grout line from the header of a hole being grouted.

2. Grout mixer: Mechanical driven, high speed, colloidal type capable of effectively mixing grout having a water-cement ratio ranging from 0.4 to 3.0 measured by weight.

3. Grout storage tanks: mechanically agitated, capable of effectively agitating and holding in suspension all solid mater contained in the grout, having a minimum capacity of 10 cubic feet. The tanks shall be equipped such that all grout entering from the mixer or returning from the hole passes through a U.S. No 100 mesh screen.

4. Valves for grout lines: They shall be the quick-opening hand operated valves capable of withstanding the maximum grouting pressures and capable of accurately controlling pressure and rate of injection.

5. Pressure gages: freshly calibrated new pressure gages with capacity of 0-200 psi and increments not greater than 5 psi. All pressure gages shall be glycerin filled as described above.

6. Grout Packers: Provide pneumatic packers as specified previously for pressure testing.

7. Mechanical Packers shall be available for sealing open grout holes in the event that some grout holes interconnect during the grouting procedure, or water flows from the holes.

2.4 INSTRUMENTATION

A. Measurement Equipment: All measurement equipment shall be suitable for performing accurate and rapid measurement of grout quantities injected and wasted, water for mixing and cement used. Subcontractor shall provide calibrated and clearly marked containers for quick and accurate measurement of the ingredients that are not introduced in full bags in the mixes. Subcontractor shall assume responsibility for accurately producing mixes determined by the Engineer. All measurement systems and equipment will require approval by the Engineer.

PART 3 EXECUTION

3.1 GENERAL REQUIREMENTS

Subcontractor shall meet the following requirements throughout the life of the subcontract:

A. Coordinate all on-site work (including maintenance and repair of equipment) with Engineer at least 72 hours in advance. Subcontractor may work on site only when an authorized representative of the Engineer is present.

B. Allow unimpeded passage through the adit, cooperate with the Engineer and its other subcontractors, and furnish necessary assistance as may reasonably be required during such activities. The Engineer and other subcontractors under the Engineer's direction will enter the adit and perform various activities during the period of execution of the coring and grouting subcontract. Such activities will include but may not be limited to the following:

1. Installation, maintenance and/or testing of valves, drain pipes, and pressure seepage monitoring equipment.

2. Collection and testing of water samples

3. Mapping of rock conditions downstream of the seal

4. Nondestructive testing of concrete seal

C. Locate, supply, and transport all supplies, materials, and equipment necessary for work contained in this subcontract. This may include coring equipment, water, compressed air, electrical power supply, sampling equipment, grouting equipment, instrumentation, pumps, and the means to transport the equipment and materials from the portal to the seal.

D. Provide electrical power, water supply, and fresh air supply throughout the field work. The site is remote and is without electrical power or municipal water supply. The Subcontractor may use the ventilation fan and ducting that is permanently installed in the adit. The ventilation fan requires a portable generator capable of providing 3-phase, 240 volts, with a minimum power of 12 kilowatts.

E. Subcontractor may use the existing railroad tracks in the mine adit for the transport of equipment and supplies; however, rail conditions are uncertain and may need repairs to be operational.

F. Accomplish all work in accordance with applicable industrial safety and Cal OSHA regulations, the Subcontractor's site specific Injury and Illness Prevention Plan, and the Engineer's Health and Safety Plan for the Walker Mine work. Comply with the State Water Resources Control Board's Mine Entry Policy. Throughout the fieldwork conduct daily safety meetings.

G. Minimize the risk of damaging the seal and minimize the release of contaminated water from behind the seal.

H. Avoid disturbance of the pressure measuring instrumentation. The Subcontractor may move the equipment if it interferes with his work; however, the Subcontractor is responsible for returning the equipment to a functional state and to its original location.

I. Improve access to the work site if such improvement is required for the safe performance of Subcontractor's operations.

J. Provide timely removal and legal off-site disposal of all spoils, waste materials, debris, excess grout, and dunnage, so as not to interfere with or delay the progress of the work. At the conclusion of the work, leave the site in the same condition as the site was at the beginning of the work.

K. Provide control of construction water and provide dewatering of seepage water in the area of the mine seal. Seepage flow in the vicinity of the mine seal is estimated to be in the order of 0.2 gallons per minute. The pool next to the mine seal has a volume in the order of 1,000 to 2,000 gallons. The Subcontractor shall discharge the seepage water to the adit floor at a distance of 150 feet or greater from the seal. The Subcontractor shall take measures to prevent this water from returning to the seal area during the work, and shall remove all such measures at the completion of the work.

3.2 PREPARATION

No operations will be permitted to commence until the Subcontractor demonstrates that all required equipment and materials are on site in workable order, clean, tested, calibrated and ready for use and that all personnel understand the assembly and operation of the equipment and their respective responsibilities.

3.3 DRILLING AND GROUTING SEQUENCE

A. The microfine cement grouting operation shall concentrate treatment of the rock mass around the concrete-bedrock contact. Packer depths and initial grout viscosities shall be varied at the Engineer's direction to accommodate existing conditions encountered in individuals holes.

B. Perform drilling and grouting in accordance with the following sequence requirements:

1. All holes shall be drilled before any grouting is done. At the completion of drilling, Subcontractor shall standby until nondestructive testing and related inspections are completed.

2. The crown (top) holes, three approximately, shall be pressure tested and grouted first to the satisfaction of the Engineer.

3. Work may be performed in not more than two holes at the same time.

4. Grout lateral holes, one on each side, of the plug.

5. Drill and grout any additional holes needed for closure. The determination for additional holes, if any, shall be made by the Engineer.

6. For any hole, pressure test the hole in stages as directed by the Engineer immediately in advance of grouting. After completion of pressure testing, grout the hole as specified.

C. Modifications to Grouting Plan: The Engineer reserves the right to alter any elements of the grouting plan as deemed necessary to best suit the site conditions that are encountered. Possible modifications include but are no limited to: modifying the grout mixture formulation, adding or deleting holes, lengthening or shortening holes.

3.4 DRILLING, WASHING AND PRESSURE TESTING

A. Core Hole Head: Prior to coring into the concrete mine seal, a grout pipe, of appropriate inner diameter suitable to accept the drilling and testing tools but not less than 3-7/8 inches in diameter, shall be cast and grouted into the face of the seal. The grout pipe shall be equipped with a valve, pressure gauge and a packer, in a manner so as to enable the immediate control of flows and grouting of the hole in the event that water under pressure is encountered in any fracture or fractures.

B. The holes shall be drilled at the location, in the direction and to the depths shown in the scope of Work or as otherwise directed by the Engineer. All drilling shall use water circulation so as to create and maintain a clean uniform hole. The drilling shall be done in a manner that will obtain the maximum possible recovery of core. Modifications in drilling procedure requested by the Engineer to improve core recovery shall be promptly adopted. The time required to drill each 12 inches of depth shall be measured and provided to the Engineer as part of the drilling core hole logs. At the end of drilling, Subcontractor shall shutoff the hole and measure the existing (background) water pressure in each hole at 8 hour intervals during working hours. This data shall also be provided to the Engineer with the core hole logs.

C. Preservation and Transport of Cores: As each piece of core is withdrawn from the core barrel, it shall be placed in a wooden core box by the Subcontractor in its proper sequence. Each time that core is withdrawn from the core barrel, a wooden separator shall be placed in the core box labeled to indicate the depth from which the core was pulled. Wooden plugs designating the length of lost core shall be placed in the core box at the same depth and labeled, in the event of core loss for any reason. The box shall be labeled with the job designation, subcontractor's name, date sample was obtained, and hole number. The label shall be printed with an indelible marker on the outside and inside of the lid. Each core box shall be constructed of wood and shall be made to contain 10 feet of core. Each box shall have metal hasp, hinges, and a lid. Each box shall contain core samples from only one hole. Subcontractor shall transport all boxed rock and concrete cores to an on-site location outside the adit portal, as designated by the Engineer.

D. Washing Drill Holes: Washing shall be performed before pressure testing using a special washout bit or attachment. All intersected rock seams, fractures, and crevices containing washable material shall be washed with clear water to remove as much material as possible. Washing pressures shall be as approved by the Engineer. The washing procedure will continue to the satisfaction of the Engineer. Washing that is incidental to the drilling operation is not sufficient for this purpose.

E. Pressure Testing: Pressure testing shall be performed in each hole after washing and before grouting begins. The subcontractor shall have pressure gauges and flow meters on standby as a backup. All grout holes shall be tested with clean water under a continuous pressure as approved by the Engineer. The pressure test will generally consist of recording the flow at 1-minute intervals over a 5 minute period performed under a constant pressure not to exceed 150 psi. The pressure will be determined by the Engineer. The Engineer reserves the right to vary the pressure testing procedure at any time. Variation may include changing the length of the tested section to develop a water path pattern, running the test for more than 5 minutes, or increasing and decreasing the pressures.

3.5 GROUTING HOLES

A. Grout mixes will be selected by the Engineer, on the basis of water test data, previous rates of grout take, or inferred geologic conditions. The water-cement ratio of the grout mixes will be specified by the Engineer in terms of weight, rather than volume.

B. Add superplasticizers and other approved additives to the water prior to adding the cementitious materials, unless otherwise recommended by the supplier. Make small batches, so as to minimize waste, unless otherwise authorized by the Engineer. Prevent grout separation at all times.

C. It is anticipated that the water-cement ratio of the grout that will be used will not exceed 3:1 by weight.

D. Perform grouting in accordance with the following general procedures:

1. Immediately after pressure testing, grout the hole to refusal as defined herein.

2. The packer shall be seated at the desired depth. If the packer fails to seat it can be pushed in or withdrawn to adjust and set it in place.

3. The Engineer will determine the grouting pressures. The pressure shall be increased gradually. It is anticipated that grouting pressures will range from 100 psi to 150 psi at the collar but in no event will pressure in excess of 150 psi be required. Grout pressures shall be maintained in the hole by the manipulation of the valve on the return line only. In no case shall the valve on the supply line be throttled to control pressures. All pressure grouting operations shall be performed in the presence of the Engineer.

4. Grout mixtures to be used will be as directed by the Engineer. In general if water pressure tests indicate a low water take, grouting will be started with a low viscosity (lean), microfine cement mix. If the foundation rock permeability is very low (less than 5 Lugeons) the grout shall be injected with an open by-pass, slightly deflated packer and low pressure to fill the hole before high pressures are applied. It is anticipated that the rock has very low permeability.

5. Refusal is achieved when the flow to the grout header pumped under pressure is zero over a period of at least 15 minutes. After refusal is achieved continue to apply pressure for an additional 15

minutes. After the 15 minutes has elapsed, close the stopcock on the injection pipe to maintain the pressure and remove the grouting hose from the grout hole.

6. Backfill all grout holes with expansive (nonshrink) hydraulic-cement grout. Follow Manufacturer's recommendations in mixing and placing this grout. For holes inclined downward, inject the grout through a pipe or hose extended to the bottom of the hole. Gradually withdraw the pipe or hose extended to the bottom of the hole. Repeat this process as many times as may be needed in order that the hole becomes completely filled with hard grout. For holes inclined upward, inject the hydraulic-cement grout through a specially-designed packer set at the mouth of the hole. Extend a pipe or tube through this packer to the end of the hole, to provide for air to be bled off as the grout is injected. Remove the pipe or tube in a manner that does not allow any grout to escape, close off the injection and bleed openings, and leave the packer in place until the grout sets. Dry-pack the void left by removal of the packer. Place the dry-pack material in thin layers and compact it with a hardwood stick and hammer.

7. Flush all grout out of the circulating lines and injection lines, pipes and packers when changing from hydraulic-cement grout to microfine cement grout upon completion of grouting a hole, and/or whenever the grouting of a hole continued longer than two hours. No grout shall be held in the agitator longer than two hours.

3.6 SPECIAL CONDITIONS

A. Grouting requires a continuous injection process until refusal is reached. Crew changes during this process are permitted provided there is sufficient overlap time for transfer of information and activities to ensure complete continuity in the grouting process. If only one crew is available, grouting shall be started early in the day to ensure continuity in the grouting process.

B. Abandoned Holes: If any hole is abandoned for any reason except by written permission of the Engineer before adequate information is obtained by carrying the hole to the required depth, no measurement and no payment will be made for the work done on any such abandoned hole. The Subcontractor shall grout and seal all abandoned holes as specified above.

3.7 CLEANUP AND RESTORATION

A. General: The Subcontractor shall exercise care to preserve the natural landscape and conduct construction operations so as to prevent any unnecessary destruction, scarring, or defacing of the natural surroundings in the vicinity of the work. All trees, native shrubbery, and vegetation shall be preserved and protected from damage. All unnecessary destruction, scarring, damage, or defacing of the landscape resulting from the Subcontractor's operations shall be repaired, replanted, or otherwise corrected as directed by the Engineer and at the Subcontractor's expense.

B. Demobilization: At completion of construction, the Subcontractor shall remove all structures, materials, equipment, and waste of any type from the site and return the construction site to the condition it was found prior to construction.

C. Cleanup: Grout spills shall be minimized and cleanup shall proceed immediately after grouting. Subcontractor shall remove from the adit all waste grout caused by grouting operations and shall dispose off site in a legal manner.

D. Restoration: Any damage to the mine seal or ancillary equipment caused by the Subcontractor shall be repaired by a method approved by the Engineer, at no additional cost to the Engineer.

Appendix E

Appendix E

Field Test Results

E.1 Core Logs

E.2 Photographs of Cores

E.3 Rock Mass Rating (RMR) Measurements

Appendix E.1

Core Logs

BORING LOCATION	Walker Mine - Plumas Co.	DATE START/FINISH	9-9-01
DEPTH DRILLED INTO CONCRETE	8.5'	DRILLED BY	Jensen Drilling Co. / D. McCormick
DIRECTION OF HOLE	Inclined, 5° DOWN from horizontal, 8° WEST (left side)	LOGGED BY	Jose Cercone-WGI TOTAL DEPTH (FT) 10.0
		TOTAL CORE RECOVERY	100%
ELEVATION TOP OF HOLE	See Key Below	CORE DIAMETER	2-3/8 inches

HOLE NO. **BH-1**

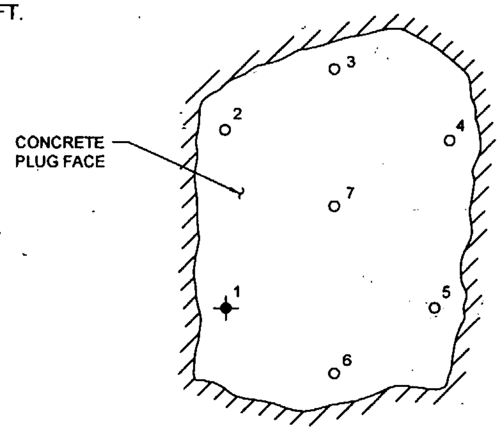
PG. 1 OF 1

RUN NO.	DEPTH FT.	GRAPHIC LOG	CLASSIFICATION OF MATERIALS	% CORE RECOVERY	BOX OR SAMPLE NO.	REMARKS
1	0–4		(0'-8.5') Concrete, 20 to 30% coarse aggregate in a cement/sand matrix, hard, good condition, no visual indication of weathering, reaction rinds, fractures, or segregation. Aggregate is angular, light colored, mostly limestone with sizes 3/4" to 1/8". Less than 1% voids. All fractures are mechanical breaks	100%		Concrete drilling relatively smooth Water circulation 100%
2	4–8			100%		
3	8–10		8.5' Concrete/bedrock contact recovered (8.5'-10') Granite, hard, fresh, moderately fractured clean joints, no oxidation or coating.	100%	☒	BH-1, 9.5-10'

BOTTOM OF HOLE @ 10 FT.

LEGEND

☒ UNCONFINED COMPRESSION STRENGTH AND PULSE VELOCITY



HOLE LOCATION KEY

Washington Washington Group International, Inc.	Walker Mine Seal Testing & Evaluation Portola, California	CONCRETE SEAL CORE DRILLING LOG
GEI Consultants, Inc.	Project 00387	December 2001

BH-1 12-07-01 PYM

BORING LOCATION	Walker Mine - Plumas Co.	DATE START/FINISH	8-29-01 / 8-30-01
DEPTH DRILLED INTO CONCRETE	3.0'	DRILLED BY	Jensen Drilling Co. / D. McCormick
DIRECTION OF HOLE	Inclined, 6° UP from horizontal, 6° WEST (left side)	LOGGED BY	Jose Cercone-WGI TOTAL DEPTH (FT) 8.0
		TOTAL CORE RECOVERY	100%
ELEVATION TOP OF HOLE	See Key Below	CORE DIAMETER	2-3/8 inches

HOLE NO. **BH-2**

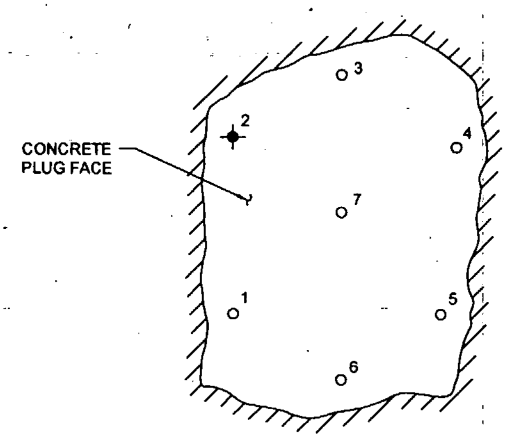
PG. 1 OF 1

RUN NO.	DEPTH FT.	GRAPHIC LOG	CLASSIFICATION OF MATERIALS	% CORE RECOV-ERY	BOX OR SAMPLE NO.	REMARKS
1	0–2		(0'-3.0') Concrete, 20 to 30% coarse aggregate in a cement/sand matrix, hard, good condition, no visual indication of weathering, reaction rinds, fractures, or segregation. Aggregate is mostly angular, light colored, mostly limestone with sizes 3/4" to 1/8". Less than 1% voids. Increase in voids 1-2%	100%		Concrete drilling relatively smooth Water circulation 100%
2	2–5	30°, 10°	Core lost at contact (3.0'-8.0') Granite, leucratic, fine grained, hard, fresh, moderately fractured to dense. Intercepting joints, dipping ~10 to 30°	100%		Granite is very hard, approximately 1 ft/hr drilling rate
3	5–8	10°, 20°, 30°, 30°	10° to 15° joints, clean, tight, 30° joints coated with oxidation.	100%	☒	BH-2, 5.3-6.0'

BOTTOM OF HOLE @ 8 FT.

LEGEND

☒ UNCONFINED COMPRESSION STRENGTH AND PULSE VELOCITY



HOLE LOCATION KEY

Washington Washington Group International, Inc.	Walker Mine Seal Testing & Evaluation Portola, California	CONCRETE SEAL CORE DRILLING LOG
GEI Consultants, Inc.	Project 00387	December 2001

BH-2 12-07-01 PYM

BORING LOCATION	Walker Mine - Plumas Co.	DATE START/FINISH	9-8-01
DEPTH DRILLED INTO CONCRETE	9.5'	DRILLED BY	Jensen Drilling Co. / D. McCormick
DIRECTION OF HOLE	Inclined, 7° UP from horizontal	LOGGED BY	Jose Cercone-WGI TOTAL DEPTH (FT) 9.5
		TOTAL CORE RECOVERY	89%
ELEVATION TOP OF HOLE	See Key Below	CORE DIAMETER	2-3/8 inches

HOLE NO. **BH-3**

PG. 1 OF 1

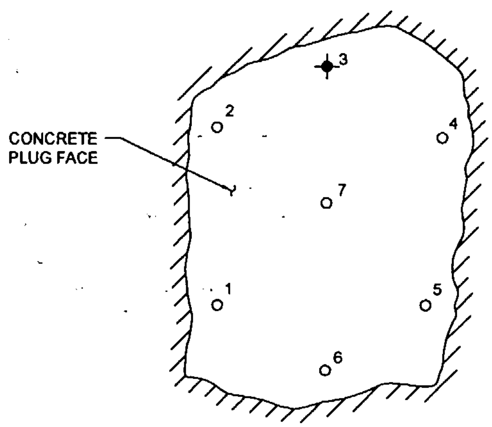
RUN NO.	DEPTH FT.	GRAPHIC LOG	CLASSIFICATION OF MATERIALS	% CORE RECOVERY	BOX OR SAMPLE NO.	REMARKS
1	0–3.5		(0'-9.5') Concrete, 20 to 30% coarse aggregate in a cement/sand matrix, hard, good condition, no visual indication of weathering, reaction rinds, fractures, or segregation. Aggregate is mostly angular, light colored, mostly limestone, with sizes 3/4" to 1/8", less than 1% voids.	100%		BH-3, 0-0.9' Concrete drilling relatively smooth Water circulation 100%
2	3.5–8		All fractures are mechanical breaks	100%		7.5-8 ft: Core erosion due to drilling
3	8–9.5		Some core stayed in the hole	75%		BH-3, 8.4-8.8' BH-3, 8.8-9.5'

BOTTOM OF HOLE @ 9.5 FT.

LEGEND

UNCONFINED COMPRESSION STRENGTH AND PULSE VELOCITY

PETROGRAPHIC EXAMINATION



HOLE LOCATION KEY

Washington
Washington Group International, Inc.

GEI Consultants, Inc.

Walker Mine
Seal Testing & Evaluation
Portola, California

CONCRETE SEAL
CORE DRILLING LOG

Project 00387

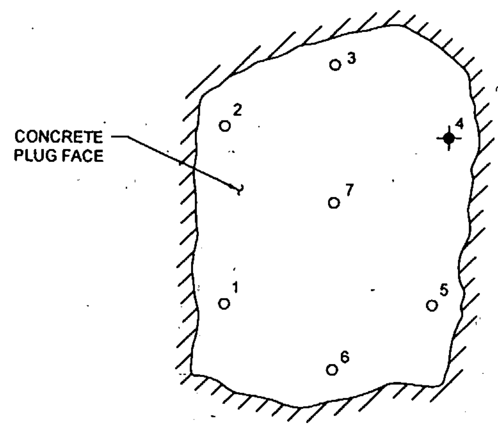
December 2001

BH-3 12-07-01 PYM

BORING LOCATION: Walker Mine - Plumas Co.	DATE START/FINISH: 8-29-01		HOLE NO. BH-4
DEPTH DRILLED INTO CONCRETE: 6.0'	DRILLED BY: Jensen Drilling Co. / D. McCormick		
DIRECTION OF HOLE: Inclined, 5° UP from horizontal, 5° EAST (right side)	LOGGED BY: Jose Cercone-WGI	TOTAL DEPTH (FT): 7.0	
	TOTAL CORE RECOVERY: 100%		
ELEVATION TOP OF HOLE: See Key Below	CORE DIAMETER: 2-3/8 inches		PG. 1 OF 1

RUN NO.	DEPTH FT.	GRAPHIC LOG	CLASSIFICATION OF MATERIALS	% CORE RECOVERY	BOX OR SAMPLE NO.	REMARKS
1	0–4		(0'-6.0') Concrete, 20 to 30% coarse aggregate in a cement/sand matrix, hard, good condition, no visual indication of weathering, reaction rinds, fractures, or segregation. Aggregate is mostly angular, light colored, mostly limestone, with sizes 3/4" to 1/8", less than 1% voids. All fractures are mechanical breaks	100%	-	Concrete drilling relatively smooth Water circulation 100%
2	4–7		(6.0'-7.0') Granite, hard, fresh, moderately fractured, clean joints, no oxidation or coating.	100%		Hole makes a trickle of water (estimated 0.01 to 0.02 gpm)

BOTTOM OF HOLE @ 7.0 FT.



HOLE LOCATION KEY

Washington Washington Group International, Inc.	Walker Mine Seal Testing & Evaluation Portola, California	CONCRETE SEAL CORE DRILLING LOG
GEI Consultants, Inc.	Project 00387	December 2001

BH-4 12-07-01 PYM

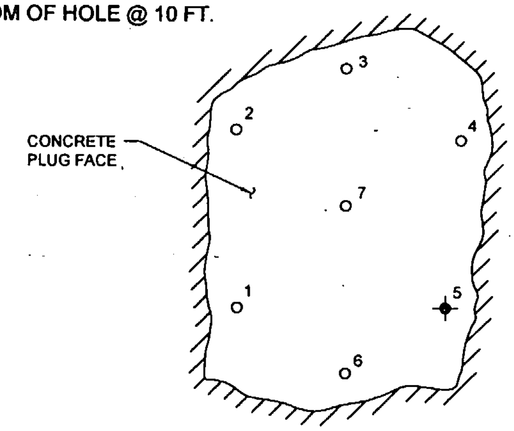
BORING LOCATION	Walker Mine - Plumas Co.	DATE START/FINISH	9-9-01
DEPTH DRILLED INTO CONCRETE	10.0'	DRILLED BY	Jensen Drilling Co. / D. McCormick
DIRECTION OF HOLE	Inclined, 7° DOWN from horizontal, 5° EAST (right side)	LOGGED BY	Jose Cercone-WGI TOTAL DEPTH (FT) 10.0
		TOTAL CORE RECOVERY	100%
ELEVATION TOP OF HOLE	See Key Below	CORE DIAMETER	2-3/8 inches

HOLE NO. **BH-5**

PG. 1 OF 1

RUN NO.	DEPTH FT.	GRAPHIC LOG	CLASSIFICATION OF MATERIALS	% CORE RECOVERY	BOX OR SAMPLE NO.	REMARKS
1	0–4		(0'-10.0') Concrete, 20 to 30% coarse aggregate in a cement/sand matrix, hard, good condition, no visual indication of weathering, reaction rinds, fractures, or segregation. Aggregate is mostly angular, light colored, mostly limestone, with sizes 1/2" to 1/8", less than 1% voids. All fractures are mechanical breaks	100%		Concrete drilling relatively smooth Water circulation 100%
2	4–8			100%		Groundup core at 7'-8"
3	8–10			100%		

BOTTOM OF HOLE @ 10 FT.



HOLE LOCATION KEY

Washington Washington Group International, Inc. GEI Consultants, Inc.	Walker Mine Seal Testing & Evaluation Portola, California	CONCRETE SEAL CORE DRILLING LOG
	Project 00387	December 2001

BH-5 12-07-01 PYM

BORING LOCATION	Walker Mine - Plumas Co.	DATE START/FINISH	9-9-01 / 9-10-01
DEPTH DRILLED INTO CONCRETE	7.5'	DRILLED BY	Jensen Drilling Co. / D. McCormick
DIRECTION OF HOLE	Inclined, 17° DOWN from horizontal	LOGGED BY	Jose Cercone-WGI TOTAL DEPTH (FT) 9.0
		TOTAL CORE RECOVERY	89%
ELEVATION TOP OF HOLE	See Key Below	CORE DIAMETER	2-3/8 inches

HOLE NO. **BH-6**

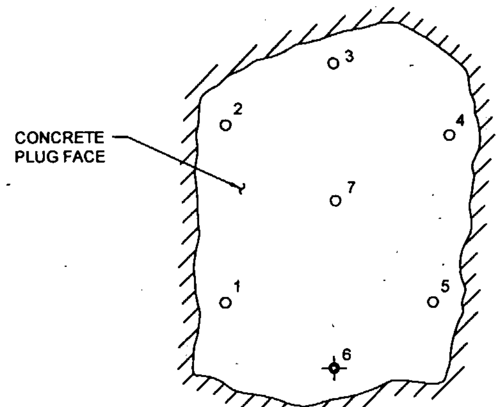
PG. 1 OF 1

RUN NO.	DEPTH FT.	GRAPHIC LOG	CLASSIFICATION OF MATERIALS	% CORE RECOVERY	BOX OR SAMPLE NO.	REMARKS
1	0–4		(0'-6.0') Concrete, 20 to 30% coarse aggregate in a cement/sand matrix, hard, good condition, no visual indication of weathering, reaction rinds, fractures, or segregation. Aggregate is mostly angular, light colored, mostly limestone, with sizes 1/2" to 1/8", less than 1% voids. Fractures are mechanical breaks	100%	BH-6, 0-1.0'	Concrete drilling relatively smooth. Water circulation 100%. Concrete stained from drilling water
2	4–8		(6.0'-7.5') Soft, erodible concrete, segregated, with only small aggregate (lost about 1 foot of core). (7.5'-9.0') Granite, intensely fractured, hard, no sign of oxidation along the joints and fractures.	75%	BH-6, 5.3-5.7'; BH-6, 6.5-6.8'	
3	8–9			100%		

BOTTOM OF HOLE @ 9 FT.

LEGEND

- UNCONFINED COMPRESSION STRENGTH AND PULSE VELOCITY
- PETROGRAPHIC EXAMINATION.



HOLE LOCATION KEY

Washington — Washington Group International, Inc.	Walker Mine Seal Testing & Evaluation Portola, California	CONCRETE SEAL CORE DRILLING LOG
GEI Consultants, Inc.	Project 00387	December 2001

BH-6 12-07-01 PYM

BORING LOCATION: Walker Mine - Plumas Co.	DATE START/FINISH: 8-28-01 / 8-29-01		HOLE NO. **BH-7**
DEPTH DRILLED INTO CONCRETE: 8.0'	DRILLED BY: Jensen Drilling Co. / D. McCormick		
DIRECTION OF HOLE: Inclined, 7° DOWN from horizontal	LOGGED BY: Jose Cercone-WGI	TOTAL DEPTH (FT): 8.0	
ELEVATION TOP OF HOLE: See Key Below	TOTAL CORE RECOVERY: 100%		
	CORE DIAMETER: 2-3/8 inches		PG. 1 OF 1

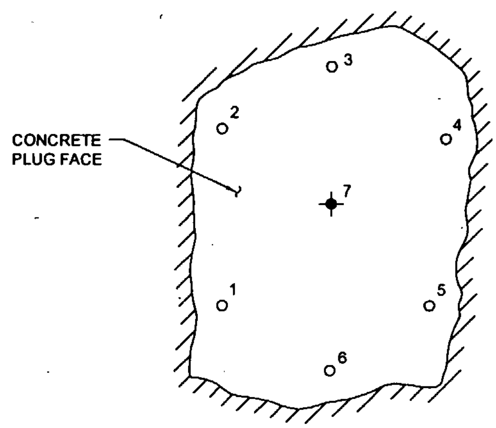
RUN NO.	DEPTH FT.	GRAPHIC LOG	CLASSIFICATION OF MATERIALS	% CORE RECOVERY	BOX OR SAMPLE NO.	REMARKS
1	0–3.0		(0'-6.0') Concrete, 20 to 30% coarse aggregate in a cement/sand matrix, hard, good condition, no visual indication of weathering, reaction rinds, fractures, or segregation. Aggregate is mostly angular, light colored, mostly limestone, with sizes 3/4" to 1/8", less than 1% voids.	100%		Concrete drilling relatively smooth Water circulation 100%
2	3.0–8.0		Fracture w/some soft cement coating - possible contact between two different cement pours, (cold joint). Mechanical fracturing due to drilling operation (typical)	100%	BH-7, 3.1-3.7' (petrographic examination) BH-7, 5.0-6.1' (unconfined compression strength and pulse velocity)	BH-7, 3.1-3.7' Several delays due mostly to equipment/air adjustments BH-7, 5.0-6.1'

BOTTOM OF HOLE @ 8 FT.

LEGEND

UNCONFINED COMPRESSION STRENGTH AND PULSE VELOCITY

PETROGRAPHIC EXAMINATION



HOLE LOCATION KEY

Washington Washington Group International, Inc.	Walker Mine Seal Testing & Evaluation Portola, California	CONCRETE SEAL CORE DRILLING LOG
GEI Consultants, Inc.	Project 00387	December 2001

BH-7 12-07-01 PYM

Appendix E.2

Photographs of Cores

Photo 1 BH-1 core

Photo 2 BH-2 core

Photo 3 BH-3 core

Photo 4 BH-4 core

Photo 5 BH-5 core

Photo 6 BH-6 core

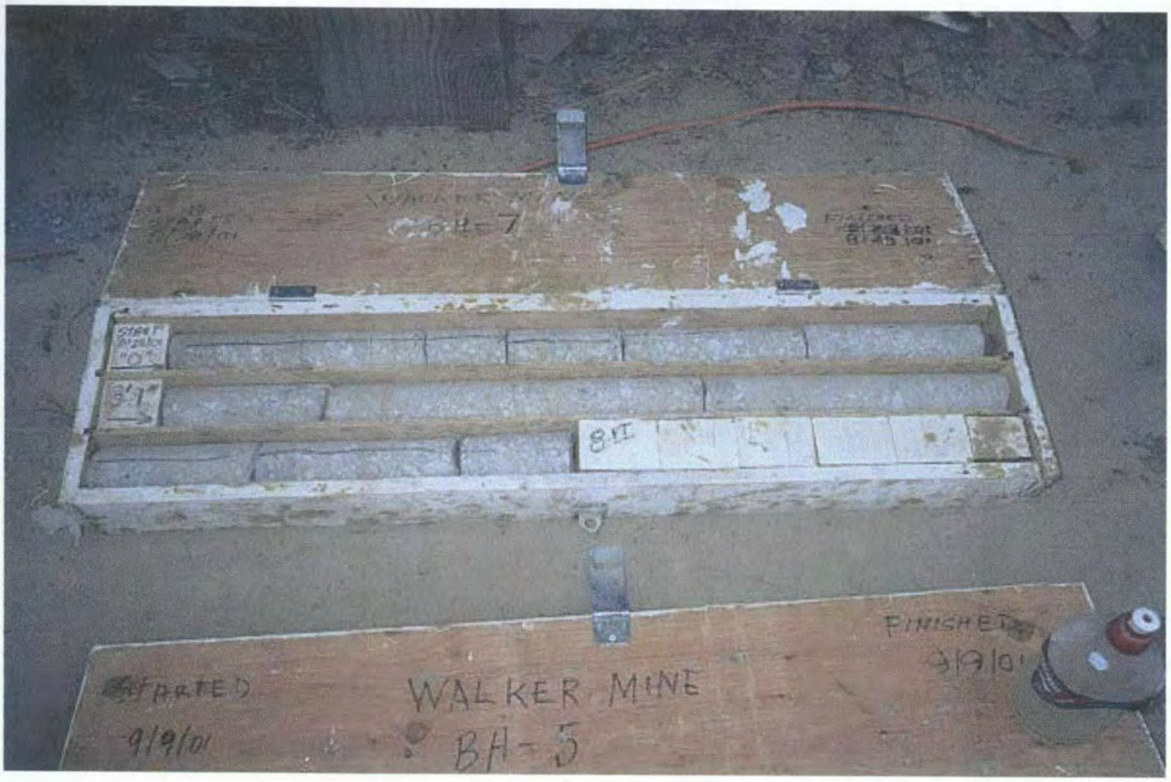


Photo 7 BH-7 core

Appendix E.3

Rock Mass Rating (RMR) Measurements

Table 1 - Summary of Rock Mass Rating Next to Seal
Walker Mine
Portola, CA

Location of Measurement in Tunnel[1]		Classification Parameters[2]						Rock Mass Rating[8] (RMR)
		No. 1 Strength of Intact Rock Material[3] (MPa)	No. 2 Drill Core Quality, RQD[4] (%)	No. 3 Spacing of Joints[5] (mm)	No. 4 Condition of Joints[6]	No. 5 Ground Water[7]	No. 6 Rating Adjustment for Joint Orientation	
Right Wall	Measurement	132	64	660	slightly rough surfaces, separation < 1 mm, unweathered walls	damp	unfavorable[9]	
	Rating	12	13	15	25	10	-10	65
Roof	Measurement	132	82	864	slightly rough surfaces, separation < 1 mm, unweathered walls	damp	unfavorable[9]	
	Rating	12	17	15	25	10	-10	69
Left Wall	Measurement	132	64	772	slightly rough surfaces, separation < 1 mm, unweathered walls	damp	unfavorable[9]	
	Rating	12	13	15	25	10	-10	65

Notes:

1) Right and left sides of tunnel are relative to the direction of tunnel drive, facing the seal. All measurements are within 20 feet of the seal.
2) Parameters are from RMR Table 1, Geomechanics classification of jointed rock mass, in ASTM D 5878 - 95, Standard Guide for Using Rock-Mass Classification Systems for Engineering Purposes (copy attached)
3) Based on the average strength from two uniaxial compression tests performed by CTL, 16,240 and 22,020 psi, or 112 to 152 MPa, average 132 MPa. The unconfined compressive strength estimated from Schmidt Hammer tests confirm this value, see Appendix G.3.
4) Rock Quality Designation (RQD) is calculated as total of intervals greater than 4 inches between joint planes over measurement distance (see Appendix E.3, Table 2 for data).
5) Calculated as the average spacing of the joints within each joint set (see Appendix E.3, Table 3 for data).
6) Joint surfaces are slightly rough, unweathered, and have a separation <1 mm, but are continuous.
7) Inflow is minimal but rock surfaces are damp within 75 feet of concrete seal.
8) RMR range from 61 to 80 is considered "good rock" in ASTM D 5878 - 95.
9) Unfavorable rating is due to orientation of joints relative to tunnel axis, per RMR Table 2 in ASTM D 5878 - 95, drive against dip with dips from 20 to 45 degrees. A few joints strike parallel to tunnel axis mostly with near vertical dips, but range from 45-90 degrees.

Table 2 - Summary of RQD Measurements Used in RMR Classification
Walker Mine
Portola, CA

Location of Measurement In Tunnel[1]	Distance From Seal[2] (ft)	Measurement Interval Length (in)	Measured Lengths >4 inches (in)	RQD[3]	Average RQD[4]
Right Wall	5	72	35	49%	
Right Wall	10	72	41	57%	64%
Right Wall	15	72	62	86%	
Roof	5	60	58	97%	
Roof	10	60	48	80%	82%
Roof	15	60	41	68%	
Left Wall	5	72	49	68%	
Left Wall	10	72	56	78%	64%
Left Wall	15	72	33	46%	

Notes:

1) Right and left sides of tunnel are relative to the direction of tunnel drive, facing the seal.
2) Measurements taken downstream of seal at an orientation parallel to the face of the seal.
3) Rock Quality Designation (RQD) is determined as the total of intervals >4 inches between joint planes over the measurement length.
   RQD = Measured lengths >4 inches divided by measurement interval length, in percent.
4) Average RQD for each location.

Table 3 - Summary of Joint Spacing Measurements Used in RMR Classification
Walker Mine
Portola, CA

Location of Measurement In Tunnel[1]	Joint Set Number	Orientation of Joint Set: Azimuth (deg)	Orientation of Joint Set: Dip (deg)	Length of Joints in Set: Ave. (ft)	Length of Joints in Set: Max. (ft)	Spacing between Joints: Typical (ft)	Spacing between Joints: Ave.[2] (ft)	Spacing between Joints: Ave.[2] (mm)	Spacing between Joints: Description
Right Wall	1	125	43	-	>10.0	2.5			
Right Wall	2	308	81	4.0	6.0	1.0			
Right Wall	3	253	83	-	7.0	>10.0[3]			
Right Wall	4	210	85	2.5	3.5	3.0			
Right Wall	5	44	69	-	3.0	>10.0[3]			
							2.2	660	Wide
Roof	1	125	39	-	>10.0	2.0			
Roof	2	245	56	6.0	8.0	6.0			
Roof	3	335	69	3.0	5.0	0.5			
Roof	4	260	88	-	4.0	>10.0			
Roof	5	233	88	-	5.0	>10.0[4]			
							2.8	864	Wide
Left Wall	1	124	40	-	>10.0	1.6			
Left Wall	2	335	64	4.0	6.0	1.0			
Left Wall	3	245	67	3.0	5.0	>10.0[3]			
Left Wall	4	35	74	-	6.0	5.0			
							2.5	772	Wide

Notes:
1) Right and left sides of tunnel are relative to the direction of tunnel drive, facing the seal. All measurements are within 20 feet of the seal. Orientation of tunnel is N35E.
2) Average spacing of joints for joint sets that have more than 2 joints within a 20-foot distance.
3) Two joints observed within a 20-foot distance.
4) One joint observed within a 20-foot distance.

Standard Guide for Using Rock-Mass Classification Systems for Engineering Purposes[1]

This standard is issued under the fixed designation D 5878; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

1. Scope

1.1 This guide covers the selection of a suitable system of classification of rock mass for specific engineering purposes, such as tunneling and shaft-sinking, excavation of rock chambers, ground support, modification and stabilization of rock slopes, and preparation of foundations and abutments. These classification systems may also be of use in work on rippability of rock, quality of construction materials, and erosion resistance. Although widely used classification systems are treated in this guide, systems not included here may be more appropriate in some situations, and may be added to subsequent editions of this standard.

1.2 The valid, effective use of this guide is contingent upon the prior complete definition of the engineering purposes to be served and on the complete and competent definition of the geology and hydrology of the engineering site. Further, the person or persons using this guide must have had field experience in studying rock-mass behavior. An appropriate reference for geological mapping in the underground is provided by Guide D 4879.

1.3 This guide identifies the essential characteristics of each of the five included classification systems. It does not include detailed guidance for application to all engineering purposes for which a particular system might be validly used. Detailed descriptions of the five systems are presented in STP 984 (**1**),[2] with abundant references to source literature.

1.4 The range of applications of each of the systems has grown since its inception. This guide summarizes the major fields of application up to this time of each of the five classification systems.

1.5 *This standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

D 653 Terminology Relating to Soil, Rock, and Contained Fluids[3]

D 2938 Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens[3]

D 4879 Guide for Geotechnical Mapping of Large Underground Openings in Rock[3]

3. Terminology

3.1 *Definitions:*

3.1.1 *classification, n*—a systematic arrangement or division of materials, products, systems, or services into groups based on similar characteristics such as origin, composition, properties, or use (*Regulations Governing ASTM Technical Committees*).[4]

3.1.2 *rock mass (in situ rock), n*—rock as it occurs in situ, including both the rock material and its structural discontinuities (Modified after Terminology D 653 [ISRM]).

3.1.3 *rock material (intact rock, rock substance, rock element), n*—rock without structural discontinuities; rock on which standardized laboratory property tests are run.

3.1.4 *structural discontinuity (discontinuity), n*—an interruption or abrupt change in a rock's structural properties, such as strength, elasticity, or density, usually occurring across internal surfaces or zones, such as bedding, parting, cracks, joints, faults, or cleavage.

NOTE 1—To some extent, 3.1.1, 3.1.2, and 3.1.4 are scale-related. A rock's microfractures might be structural discontinuities to a petrologist, but to a field geologist the same rock could be considered intact. Similarly, the localized occurrence of jointed rock (rock mass) could be inconsequential in regional analysis.

3.1.5 For the definition of other terms that appear in this guide, refer to STP 984, Guide 4879, and Terminology D 653.

3.2 *Definitions of Terms Specific to This Standard:*

3.2.1 *classification system, n*—a group or hierarchy of classifications used in combination for a designated purpose, such as evaluating or rating a property or other characteristic of a rock mass.

4. Significance and Use

4.1 The classification systems included in this guide and their respective applications are as follows:

4.1.1 *Rock Mass Rating System (RMR) or Geomechanics Classification*—This system has been applied to tunneling, hard-rock mining, coal mining, stability of rock slopes, rock foundations, borability, rippability, dredgability, weatherability, and rock bolting.

4.1.2 *Rock Structure Rating System (RSR)*—This system has been used in tunnel support and excavation and in other ground support work in mining and construction.

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[1] This test method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.12 on Rock Mechanics.

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[2] The boldface numbers given in parentheses refer to a list of references at the end of the text.

[3] *Annual Book of ASTM Standards*, Vol 04.08.

[4] Available from ASTM Headquarters, 100 Barr Harbor Drive, West Conshohocken, PA 19428.

4.1.3 *The Q System or Norwegian Geotechnical Institute System (NGI)*—This system has been applied to work on tunnels and chambers, rippability, excavatability, hydraulic erodibility, and seismic stability of roof-rock.

4.1.4 *The Unified Rock Classification System (URCS)*—This system has been applied to work on foundations, methods of excavation, slope stability, uses of earth materials, blasting characteristics of earth materials, and transmission of ground water.

4.1.5 *The Rock Material Field Classification Procedure (RMFC)*—This system has been used mainly for applications involving shallow excavation, particularly with regard to resistance to erosion, excavatability, construction quality of rock, fluid transmission, and rock-mass stability.

4.2 Other classification systems are described in detail in the general references listed in the appendix.

4.3 Using this guide, the classifier should be able to decide which system appears to be most appropriate for the specified engineering purpose at hand. The next step should be the study of the source literature on the selected classification system and on case histories documenting the application of that system to real-world situations and the degree of success of each such application. Appropriate but by no means exhaustive references for this purpose are provided in the appendix and in STP 984 (1). *The classifier should realize that taking the step of consulting the source literature might lead to abandonment of the initially selected classification system and selection of another system, to be followed again by study of the appropriate source literature.*

5. Bases for Classification

5.1 The parameters used in each classification system follow. In general, the terminology used by the respective author or authors of each system is listed, to facilitate reference to STP 984 (1) or source documents.

5.1.1 *Rock Mass Rating System (RMR) or Geomechanics Classification*

- Uniaxial compressive strength (see Test Method D 2938)
- Rock quality designation (RQD)
- Spacing of discontinuities
- Condition of discontinuities
- Ground water conditions
- Orientation of discontinuities

5.1.2 *Rock Structure Rating System (RSR)*

- Rock type plus rock strength
- Geologic structure
- Spacing of joints
- Orientation of joints
- Weathering of joints
- Ground water inflow

5.1.3 *Q-System or Norwegian Geotechnical Institute System (NGI)*

- Rock quality designation (RQD)
- Number of joint sets
- Joint roughness
- Joint alteration
- Joint water-reduction factor
- Stress-reduction factor

5.1.4 *Unified Rock Classification System (URCS)*

- Degree of weathering
- Uniaxial compressive strength (see Test Method D 2938)
- Discontinuities
- Unit weight

5.1.5 *Rock Material Field Classification Procedure (RMFC)*

- Discrete rock-particle size
- Uniaxial compressive strength (see Test Method D 2938)
- Joint orientation
- Joint-aperture width
- Geologic structure
- Rock-unit thickness
- Seismic velocity
- URCS rating
- Rock quality designation (RQD)
- Mineralogy
- Porosity and voids
- Hydraulic conductivity and transmissivity

5.2 Comparison of parameters among these systems indicates some strong similarities. It is not surprising, therefore, that paired correlations have been established between RMR, RSR, and Q (2). Some of the references in the appendix also present procedures for estimating some in situ engineering properties from one or more of these indexes (2, 3, 4, and 5):

NOTE 2—Reference (2) presents step-by-step procedures for calculating and applying RSR, RMR, and Q values. Applications of all five systems are discussed in STP 984 (1), as is a detailed treatment of RQD.

6. Procedures for Determining Parameters

6.1 The annex of this guide contains tabled and other material for determining the parameters needed to apply each of the classification systems. These materials should be used in conjunction with detailed, instructive references such as STP 984 (1) and Ref (2). The annexed materials are as follows:

6.1.1 *RMR System*

- Classification parameters (five) and their ratings (Sum ratings)
- Rating adjustment for joint orientations (Parameter No. 6) *(RMR = adjusted sum)*
- Effect of discontinuity strike and dip in tunneling
- Adjustments for mining applications
- Input data

6.1.2 *RSR System*

- Schematic of the six parameters
- Rock type plus strength, geologic structure ("A")
- Joint spacing and orientation ("B")
- Weathering of joints and ground water inflow ("C")

$$(RSR = A + B + C)$$

6.1.3 *Q-System:*

- RQD
- Joint set number, J_n
- Joint roughness number, J_r
- Joint alteration number, J_a
- Joint water reduction factor, J_W
- Stress reduction factor SRF

$$(Q = (RQD/J_n) \times (J_r/J_a) \times (J_W/SRF))$$

6.1.4 *URCS*

Degree of weathering (*A–E*)
Estimated strength (*A–E*)
Discontinuities (*A–E*)
Unit weight (*A–E*)
Schematic of notation (*results* = *AAAA* through *EEEE*)

6.1.5 *RMFCP*

Schematic of procedure through performance assessment

Classification (description and definitions),
Rock unit

Classification Elements—Including rock material properties, rock mass properties, and hydrogeologic properties.

Performance Assessment—Performance objectives

Erosion resistance
Excavation Characteristics
Construction Quality
Fluid Transmission
Rock Mass Stability

7. Precision

7.1 Precision statements will be available for some components of some of the classification systems, such as uniaxial compressive strength and rock quality designation.

8. Keywords

8.1 classification; classification system; Q-system (NGI); rock mass, rock mass rating system (RMR), rock material field classification procedure (RMFCP); rock quality designation (RQD); rock structure rating system (RSR); unified rock classification system (URCS)

ANNEX

(Mandatory Information)

A1.1 The materials presented in this Annex for RMR, RSR, URCS, and RMFCP have been extracted from STP 984 **(1)**. The materials for Q (NGI) are from Ref **(4)**.

RMR

TABLE 1—*Geomechanics Classification of jointed rock masses.*

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	PARAMETER		RANGES OF VALUES						
1	Strength of intact roack material	Point-load strength index	> 10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range – uniaxial compressive test is preferred		
		Uniaxial compressive strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5-25 MPa	1-5 MPa	<1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		>2 m	0.6 - 2 m	200 - 600 mm	60 -200 mm	<60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities		Very rough surfaces. Not continuous No seperation Unweathered wall rock.	Slightly rough surfaces. Separation < 1 mm Slightly weathered walls	Slightly rough surfaces. Separation < 1 mm Highly weathered walls	Slickensided surfaces OR Gouge < 5 mm thick OR Separation 1-5 mm. Continuous	Soft gouge > 5 mm thick OR Separation > 5 mm. Continous		
	Rating		30	25	20	10	0		
5	Ground water	Inflow per 10 m tunnel length	None	<10 litres/min	10-25 litres/min	25 - 125 litres/min	> 125		
		Ratio $\frac{\text{joint water pressure}}{\text{major principal stress}}$	OR 0	OR 0.0-0.1	OR 0.1-0.2	OR 0.2-0.5	OR > 0.5		
		General conditions	OR Completely dry	OR Damp	OR Wet	OR Dripping	OR Flowing		
	Rating		15	10	7	4	0		

B. RATING ADJUSTMENT FOR JOINT ORIENTATIONS

Strike and dip orientations of joints		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
Ratings	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 20
Class No.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK MASS CLASSES

Class No	I	II	III	IV	V
Average stand-up time	10 years for 15 m span	6 months for 8 m span	1 week for 5 m span	10 hours for 2.5 m span	30 minutes for 1 m span
Cohesion of the rock mass	> 400 kPa	300 - 400 kPa	200 - 300 kPa	100 - 200 kPa	< 100 kPa
Friction angle of the rock mass	> 45°	35° - 45°	25° - 35°	15° - 25°	< 15°

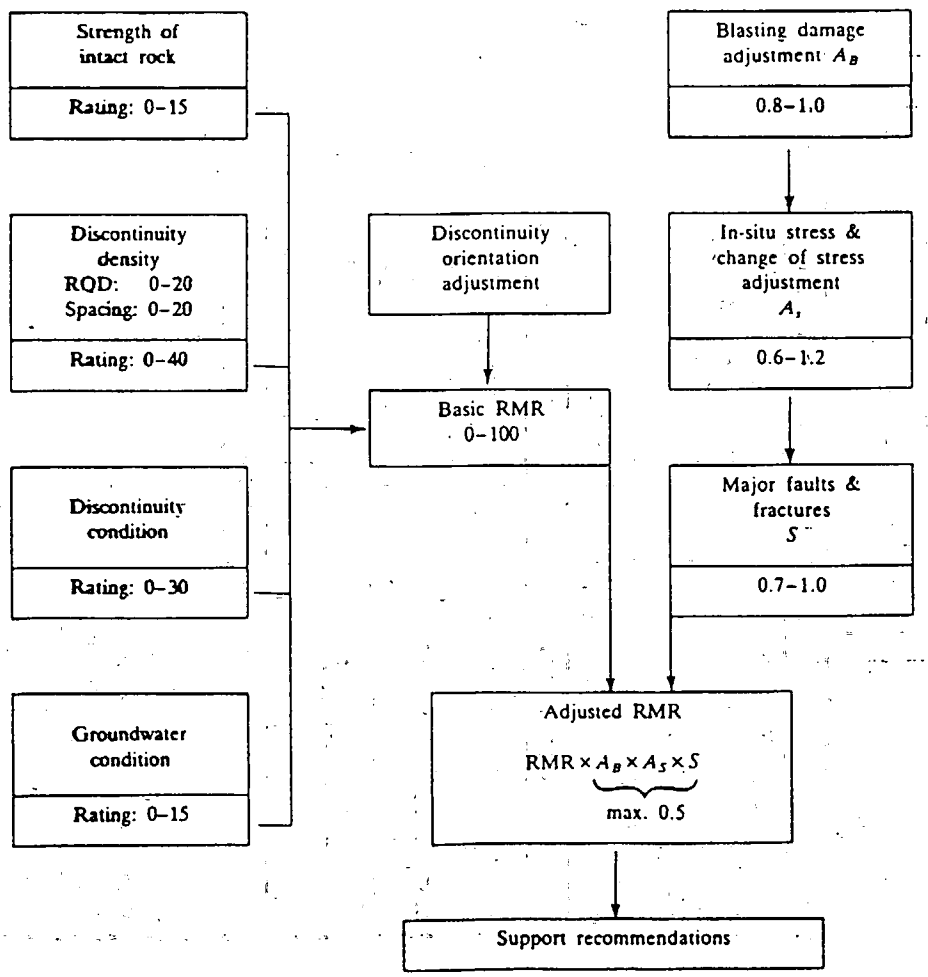
RMR

TABLE 2—*Effect of discontinuity strike and dip orientations in tunneling.*

Strike Perpendicular to Tunnel Axis			
Drive with Dip		Drive against Dip	
Dip 45–90°	Dip 20–45°	Dip 45–90°	Dip 20–45°
Very favorable	Favorable	Fair	Unfavorable

Strike Parallel to Tunnel Axis		Irrespective of Strike
Dip 20–45°	Dip 45–90°	Dip 0–20°
Fair	Very unfavorable	Fair

TABLE 3—*Adjustments to the Geomechanics Classification for mining applications.*



RMR

Input data form for the Geomechanics Classification (RMR System)

Name of project:

Site of survey:

Conducted by:

Date:

STRUCTURAL REGION	ROCK TYPE AND ORIGIN

DRILL CORE QUALITY R.Q.D.*

Excellent quality:	90 - 100%	....
Good quality:	75 - 90%	....
Fair quality:	50 - 75%	....
Poor quality:	25 - 50%	....
Very poor quality:	<25%	....

*R.Q.D. = Rock Quality Designation

WALL ROCK OF DISCONTINUITIES

Unweathered ....

Slightly weathered ....

Moderately weathered ....

Highly weathered ....

Completely weathered ....

Residual soil ....

GROUND WATER

INFLOW per 10 m of tunnel length litres/minute ....

or

WATER PRESSURE kPa ....

or

GENERAL CONDITIONS (completely dry, damp, wet, dripping or flowing under low/medium or high pressure:

....

STRENGTH OF INTACT ROCK MATERIAL

Designation	Uniaxial compressive strength, MPa	OR	Point-load strength index, MPa
Very high:	Over 250		>10
High:	100 - 250		4-10
Medium high:	50 - 100		2-4
Moderate:	25 - 50		1-2
Low:	5 - 25		< 1
Very low:	1 - 5		

SPACING OF DISCONTINUITIES

		Set 1	Set 2	Set 3	Set 4
Very wide:	Over 2 m	......	......	......	......
Wide:	0,6 - 2 m	......	......	......	......
Moderate:	200 - 600 mm	......	......	......	......
Close:	60 - 200 mm	......	......	......	......
Very close:	<60 mm	......	......	......	......

NOTE: These values are obtained from a joint survey and not from borehole logs.

STRIKE AND DIP ORIENTATIONS

Set 1 Strike: .... (average) (from .... to ....) Dip: .... (angle) .... (direction)

Set 2 Strike: .... (from .... to ....) Dip: ....

Set 3 Strike: .... (from .... to ....) Dip: ....

Set 4 Strike: .... (from .... to ....) Dip: ....

CONDITION OF DISCONTINUITIES

		Set 1	Set 2	Set 3	Set 4
PERSISTENCE (CONTINUITY)					
Very low:	<1 m	......	......	......	......
Low:	1 - 3 m	......	......	......	......
Medium:	3 - 10 m	......	......	......	......
High:	10 - 20 m	......	......	......	......
Very high:	> 20 m	......	......	......	......
SEPARATION (APERTURE)					
Very tight joints:	<0,1 mm	......	......	......	......
Tight joints:	0,1 - 0,5 mm	......	......	......	......
Moderately open joints:	0,5 - 2,5 mm	......	......	......	......
Open joints:	2,5 - 10 mm	......	......	......	......
Very wide aperture	> 10 mm	......	......	......	......
ROUGHNESS (state also if surfaces are stepped, undulating or planar)					
Very rough surfaces:		......	......	......	......
Rough surfaces:		......	......	......	......
Slightly rough surfaces:		......	......	......	......
Smooth surfaces:		......	......	......	......
Slickensided surfaces:		......	......	......	......
FILLING (GOUGE)					
Type:		......	......	......	......
Thickness:		......	......	......	......
Uniaxial compressive strength, MPa		......	......	......	......
Seepage:		......	......	......	......

MAJOR FAULTS OR FOLDS

Describe major faults and folds specifying their locality, nature and orientations.

GENERAL REMARKS AND ADDITIONAL DATA

NOTE:

(1) For definitions and methods consult ISRM document: *'Quantitative description of discontinuities in rock masses.'*

RSR

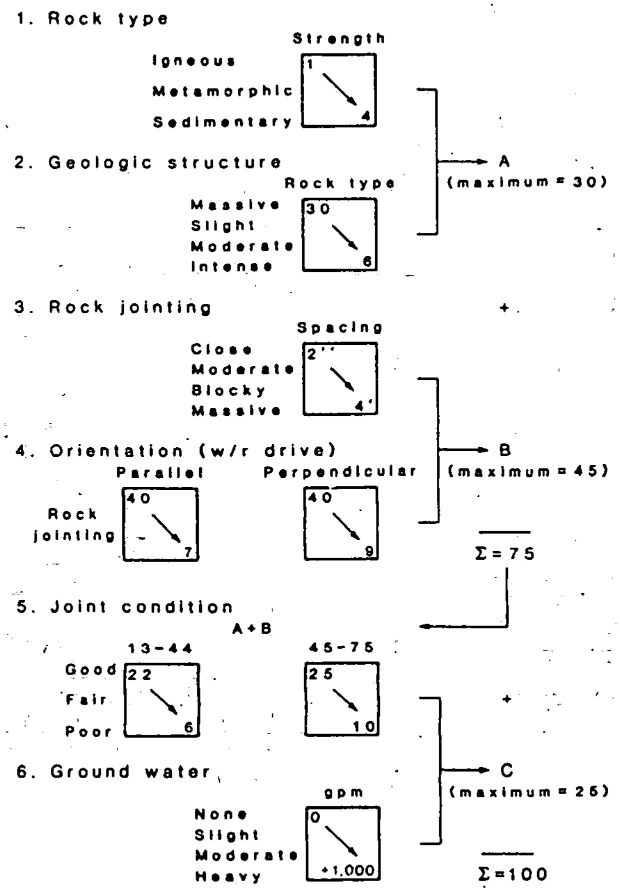


FIG. 1—*Schematic of Rock Structure Rating.*

Parameter A
Rock structure rating
Rock type, strength index and geologic structure
Maximum value 30

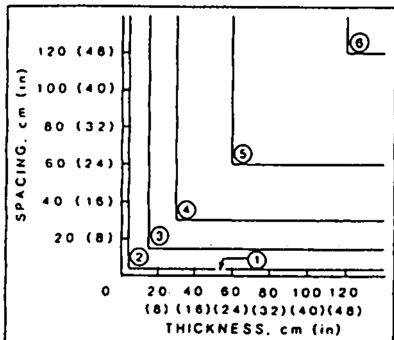
Basic rock type

	Hard	Medium	Soft	Decomp
Igneous	1	2	3	4
Metamorphic	1	2	3	4
Sedimentary	2	3	4	4

	Geological structure			
	Massive	Slightly faulted or folded	Moderately faulted or folded	Intensely faulted or folded
Type 1	30	22	15	9
Type 2	27	20	13	8
Type 3	24	18	12	7
Type 4	19	15	10	6

FIG. 2—*Parameter A.*

RSR



Parameter B
Rock structure rating
Joint pattern and direction of drive

Maximum value 45

	Strike perpendicular to axis					Strike parallel to axis		
	Direction of drive					Direction of drive		
	Both	With dip		Against dip		Both		
	Dip of prominent joints					Dip of prominent joints		
	Flat	Dipping	Vertical	Dipping	Vertical	Flat	Dipping	Vertical
1 Very closely jointed	9	11	13	10	12	9	9	7
2 Closely jointed	13	16	19	15	17	14	14	11
3 Moderately jointed	23	24	28	19	22	23	23	19
4 Moderate to blocky	30	32	36	25	28	30	28	24
5 Blocky to massive	36	38	40	33	35	36	34	28
6 Massive	40	43	45	37	40	40	38	34

Flat: 0-20°; Dipping: 20-50°; Vertical: 50-90°

FIG. 3—*Parameter B*.

Parameter C
Rock structure rating
Ground water and joint condition

Maximum value 25

Anticipated water inflow m^3/min/300m (gpm/1,000 ft)	Sum of parameters A + B					
	13-44			45-75		
	Joint condition					
	Good	Fair	Poor	Good	Fair	Poor
None	22	18	12	25	22	18
Slight <0.75 m^3/min (<200 gpm)	19	15	9	23	19	14
Moderate 0.75-3.8 m^3/min (200-1,000 gpm)	15	11	7	21	16	12
Heavy >3.8 m^3/min (>1,000 gpm)	10	8	6	18	14	10

Joint condition: Good = Tight or cemented; Fair = Slightly weathered or altered; Poor = Severely weathered, altered or open

FIG. 4—*Parameter C*.

Q (NGI)

Ratings for the six Q-system parameters

1. Rock Quality Designation		RQD
A	Very poor	0 - 25
B	Poor	25 - 50
C	Fair	50 - 75
D	Good	75 - 90
E	Excellent	90 - 100

Note: i) Where RQD is reported or measured as ≤ 10 (including 0), a nominal value of 10 is used to evaluate Q.
ii) RQD intervals of 5, *i.e.*, 100, 95, 90, *etc.*, are sufficiently accurate.

2. Joint Set Number		J_n
A	Massive, no or few joints	0.5 - 1.0
B	One joint set	2
C	One joint set plus random joints	3
D	Two joint sets	4
E	Two joint sets plus random joints	6
F	Three joint sets	9
G	Three joint sets plus random joints	12
H	Four or more joint sets, random, heavily jointed, "sugar cube", *etc.*	15
J	Crushed rock, earthlike	20

Note: i) For intersections, use $(3.0 \times J_n)$
ii) For portals, use $2.0 \times J_n$)

3. Joint Roughness Number		J_r
a) Rock-wall contact, and b) rock-wall contact before 10 cm shear		
A	Discontinuous joints	4
B	Rough or irregular, undulating	3
C	Smooth, undulating	2
D	Slickensided, undulating	1.5
E	Rough or irregular, planar	1.5
F	Smooth, planar	1.0
G	Slickensided, planar	0.5
Note: i) Descriptions refer to small scale features and intermediate scale features, in that order.		
c) No rock-wall contact when sheared		
H	Zone containing clay minerals thick enough to prevent rock-wall contact	1.0
J	Sandy, gravelly or crushed zone thick enough to prevent rock-wall contact	1.0

Note: i) Add 1.0 if the mean spacing of the relevant joint set is greater than 3m.
ii) J_r = 0.5 can be used for planar slickensided joints having lineations, provided the lineations are oriented for minimum strength.

4. Joint Alteration Number		ϕ_r approx.	J_a
a) Rock-wall contact (no mineral fillings, only coatings)			
A	Tightly healed, hard, non-softening, impermeable filling, *i.e.*, quartz or epidote		0.75
B	Unaltered joint walls, surface staining only	25-35°	1.0
C	Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock, *etc.*	25-30°	2.0
D	Silty- or sandy-clay coatings, small clay fraction (non-softening)	20-25°	3.0
E	Softening or low friction clay mineral coatings, *i.e.*, kaolinite or mica. Also chlorite, talc, gypsum, graphite, *etc.*, and small quantities of swelling clays.	8-16°	4.0
b) Rock-wall contact before 10 cm shear (thin mineral fillings)			
F	Sandy particles, clay-free disintegrated rock, *etc.*	25-30°	4.0
G	Strongly over-consolidated non-softening clay mineral fillings (continuous, but <5mm thickness)	16-24°	6.0
H	Medium or low over-consolidation, softening, clay mineral fillings (continuous, but <5mm thickness)	12-16°	8.0
J	Swelling-clay fillings, *i.e.*, montmorillonite (continuous, but <5mm thickness). Value of J_a depends on percent of swelling clay-size particles, and access to water, *etc.*	6-12°	8-12
c) No rock-wall contact when sheared (thick mineral fillings)			
KLM	Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition)	6-24°	6, 8, or 8-12
N	Zones or bands of silty- or sandy-clay, small clay fraction (non-softening)		5.0
OPR	Thick, continuous zones or bands of clay (see G, H, J for description of clay condition)	6-24°	10, 13, or 13-20

5. Joint Water Reduction Factor		approx. water pres. (kg/cm²)	J_w
A	Dry excavations or minor inflow, *i.e.*, < 5 l/min locally	< 1	1.0
B	Medium inflow or pressure, occasional outwash of joint fillings	1-2.5	0.66
C	Large inflow or high pressure in competent rock with unfilled joints	2.5-10	0.5
D	Large inflow or high pressure, considerable outwash of joint fillings	2.5-10	0.33
E	Exceptionally high inflow or water pressure at blasting, decaying with time	> 10	0.2-0.1
F	Exceptionally high inflow or water pressure continuing without noticeable decay	> 10	0.1-0.05

Note: i) Factors C to F are crude estimates. Increase J_w if drainage measures are installed.
ii) Special problems caused by ice formation are not considered.

6. Stress Reduction Factor		SRF
a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated		
A	Multiple occurrences of weakness zones containing *clay* or chemically disintegrated rock, very loose surrounding rock (any depth)	10
B	Single weakness zones containing *clay* or chemically disintegrated rock (depth of excavation ≤ 50m)	5
C	Single weakness zones containing *clay* or chemically disintegrated rock (depth of excavation > 50m)	2.5
D	Multiple shear zones in competent rock (*clay-free*), loose surrounding rock (any depth)	7.5
E	Single shear zones in competent rock (*clay-free*) (depth of excavation ≤ 50m)	5.0
F	Single shear zones in competent rock (*clay-free*) (depth of excavation > 50m)	2.5
G	Loose, open joints, heavily jointed or "sugar cube", *etc.* (any depth)	5.0

Note: i) Reduce these values of SRF by 25-50% if the relevant shear zones only influence but do not intersect the excavation.

b) Competent rock, rock stress problems		σ_c/σ_1	σ_θ/σ_c	SRF
H	Low stress, near surface, open joints	> 200	< 0.01	2.5
J	Medium stress, favourable stress condition	200-10	0.01-0.3	1
K	High stress; very tight structure. Usually favourable to stability, may be unfavourable for wall stability.	10-5	0.3-0.4	0.5-2
L	Moderate slabbing after > 1 hour in *massive* rock	5-3	0.5-0.65	5-50
M	Slabbing and rock burst after a few minutes in *massive* rock	3-2	0.65-1	50-200
N	Heavy rock burst (strain-burst) and immediate dynamic deformations in *massive* rock	< 2	> 1	200-400

Note: ii) For strongly anisotropic virgin stress field (if measured): when $5 \le \sigma_1/\sigma_3 \le 10$, reduce σ_c to $0.75\sigma_c$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c to $0.5\sigma_c$, where σ_c = unconfined compression strength, σ_1 and σ_3 are the major and minor principal stresses, and σ_θ = maximum tangential stress (estimated from elastic theory).
iii) Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H).

c) Squeezing rock: plastic flow of incompetent rock under the influence of high rock pressure		σ_θ/σ_c	SRF
O	Mild squeezing rock pressure	1-5	5-10
P	Heavy squeezing rock pressure	> 5	10-20

Note: iv) Cases of squeezing rock may occur for depth $H > 350\ Q^{1/3}$ (Singh *et al.*, 1992). Rock mass compression strength can be estimated from $q = 0.7\ \gamma\ Q^{1/3}$ (MPa) where γ = rock density in kN/m³ (Singh, 1993).

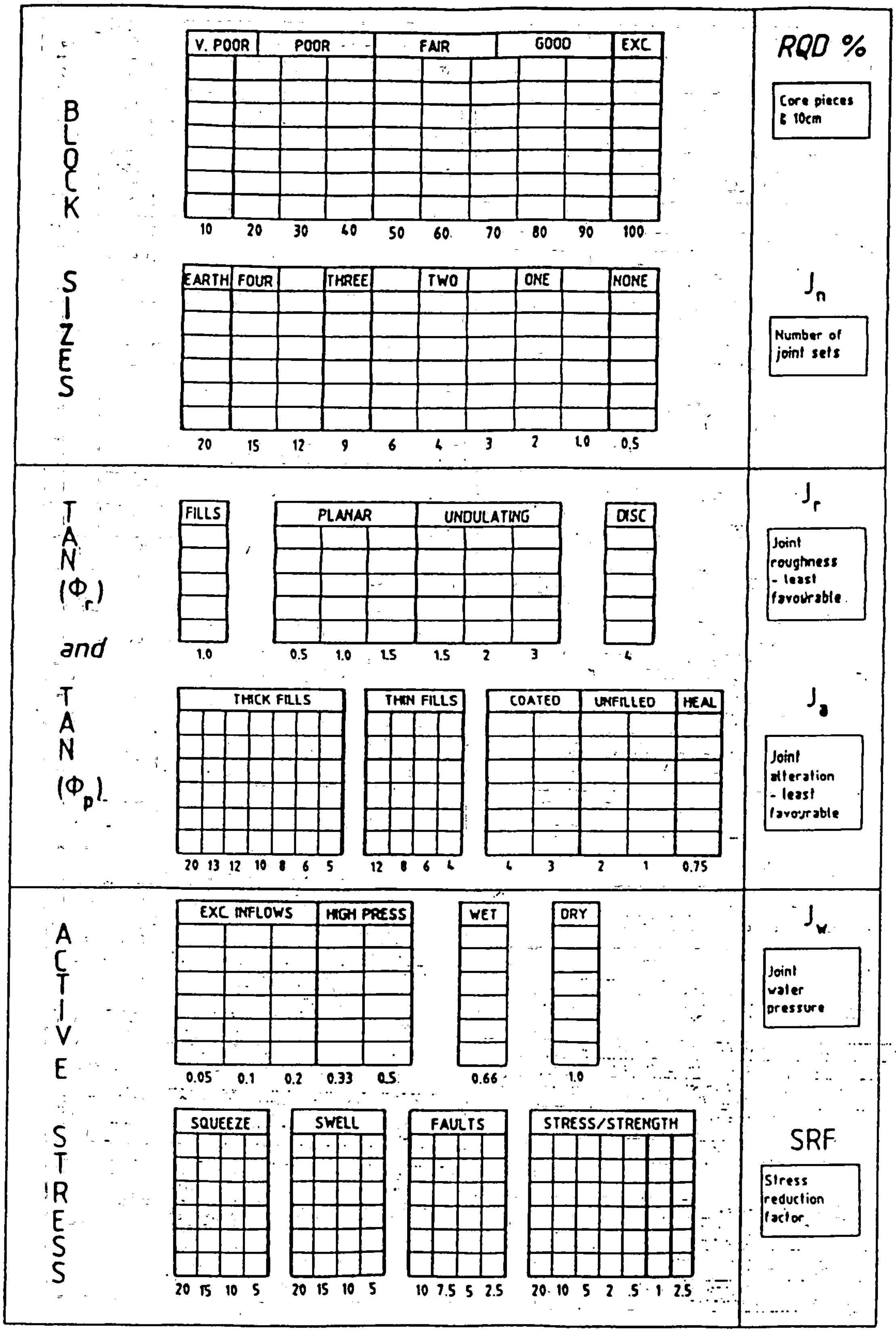
d) Swelling rock: chemical swelling activity depending on presence of water		SRF
R	Mild swelling rock pressure	5-10
S	Heavy swelling rock pressure	10-15

Note: J_r and J_a classification is applied to the joint set or discontinuity that is least favourable for stability both from the point of view of orientation and shear resistance, τ (where $\tau = \sigma_n \tan^{-1}(J_r/J_a)$). Choose the most likely feature to allow failure to initiate.

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

Q (NGI)

Logging chart for assembling Q-parameter statistics



URCS

DEGREE OF WEATHERING

REPRESENTATIVE		ALTERED	WEATHERED			
			>GRAVEL SIZE		<SAND SIZE	
Micro Fresh State (MFS) A	Visually Fresh State (VFS) B	Stained State (STS) C	Partly Decomposed State (PDS) D		Completely Decomposed State (CDS) E	
UNIT WEIGHT RELATIVE ABSORPTION		COMPARE TO FRESH STATE	NON-PLASTIC	PLASTIC	NON-PLASTIC	PLASTIC

ESTIMATED STRENGTH

REACTION TO IMPACT OF 1 LB. BALLPEEN HAMMER				REMOLDING[1]
"Rebounds" (Elastic) (RQ) A	"Pits" (Tensional) (PQ) B	"Dents" (Compression) (DQ) C	"Craters" (Shears) (CQ) D	Moldable (Friable) (MQ) E
>15000 psi[2] >103 MPa	8000-15000 psi[2] 55-103 MPa	3000-8000 psi[2] 21-55 MPa	1000-3000 psi[2] 7-21 MPa	<1000 psi[2] <7 MPa

(1) Strength Estimated by Soil Mechanics Techniques
(2) Approximate Unconfined Compressive Strength

DISCONTINUITIES

VERY LOW PERMEABILITY			MAY TRANSMIT WATER	
Solid (Random Breakage) (SRB) A	Solid (Preferred Breakage) (SPB) B	Solid (Latent Planes Of Separation) (LPS) C	Nonintersecting Open Planes (2-D) D	Intersecting Open Planes (3-D) E
			ATTITUDE	INTERLOCK

UNIT WEIGHT

Greater Than 160 pcf 2.55 g/cc A	150-160 pcf 2.40-2.55 g/cc B	140-150 pcf 2.25-240 g/cc C	130-140 pcf 2.10-2.25 g/cc D	Less Than 130 pcf 2.10 g/cc E

DESIGN NOTATION

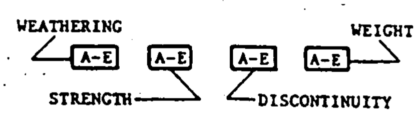
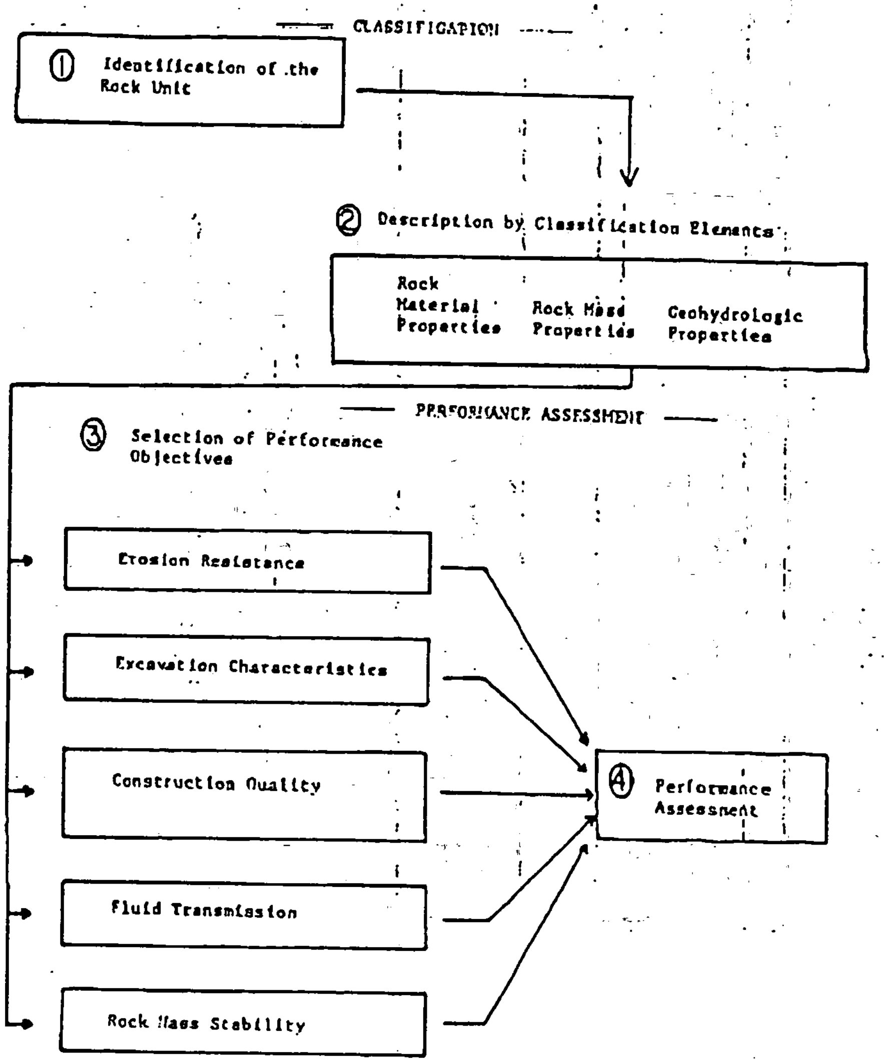


Figure 1. Basic elements of the unified rock classification system.

RMFCP

The following diagram illustrates the procedure:



CLASSIFICATION

ROCK UNIT. A rock unit is an identifiable rock that is consistent in mineral, structural, and hydraulic characteristics. A rock unit can be considered essentially homogeneous for project analysis and for descriptive and mapping purposes. The degree of homogeneity of the rock units at the site of investigation is indicated by assignment of an "outcrop confidence level"

Rock units are delineated by observable and measurable physical features. When a rock unit has been established it can be defined by classification elements and analyzed for performance in relation to selected performance objectives.

CLASSIFICATION ELEMENTS. Classification elements are objective physical properties of the rock unit that define the characteristics of the material. Engineering classification of a rock unit reflects not only the material properties of the rock itself but the structural characteristics of the rock mass in the field, and the interactions between the rock and its system of discontinuities.

(1) Rock Material Properties: The lithologic properties of the rock that can be evaluated in hand specimen (and in many instances, in outcrop) and thus can be subject to meaningful inquiry in the laboratory. They include characteristics such as mineralogic composition, grain size, rock hardness, degree of weathering, unconfined compressive strength, porosity, unit weight, and other index properties.

(2) Rock Mass Properties: The lithologic properties of the rock that must be evaluated on a macroscopic scale in the field. They include description of tectonic features that are too large to be observed directly in their entirety, such as regional structure, karst features, and lineaments. Rock mass properties include features that cannot be sampled for laboratory analysis, such as fractures, joints, and faults, bedding, schistosity, lineations, as well as the lateral and vertical extent of the rock unit.

(3) Geohydrologic Properties: The lithologic properties of the rock that affect the mode of occurrence, location, distribution and flow characteristics of subsurface waters; these properties may include primary and secondary porosity, hydraulic conductivity, transmissivity, and other fluid transmission characteristics.

RMFCP

ROCK CLASSIFICATION PROCESS

The rock classification process involves identifying the rock units at the site of investigation and describing appropriate classification elements. The following outline can be used as a guide in the process. Primary and secondary levels of description are indicated. Additional levels or factors may be added as required for further clarification. Appropriate appendixes are referred to. See Appendix IV for an example of a completed outline.

ROCK UNIT CLASSIFICATION

Project: ______________________
Date: ______________________
Geologist: ______________________

1. ROCK UNIT IDENTIFICATION.

The description of each rock unit should include the location and extent of the unit in outcrop or in stratigraphic section which, in turn, should provide an indication of outcrop confidence level. The rock unit can be identified either by name or alpha-numeric designation.

(a) Designation: (Vishnu schist, Rock Unit L-6, etc.)
(b) Location: (geographic, station, depth, etc.)
(c) Outcrop Confidence Level:

2. CLASSIFICATION ELEMENTS.

(a) Rock Material Properties: To be determined by examination and classification of hand specimens, core sections, drill cuttings, outcroppings, and disturbed samples, using standard geological terminology. Typical elements may include:

Rock formation name: (primary, secondary). See Appendix III
Mineralogy: (principal and accessory minerals, estimate percent; type of cement; note presence of alterable minerals)
Texture and fabric:
Primary porosity: (free-draining or not)
Discrete rock particle size: (See: Definitions)
Rock hardness: (See NEH-8, p. 1-13)
Micro structures: (bedding, foliation, etc.)
Degree of weathering (URCS): See Appendix I
Estimated strength (URCS): See Appendix I
Unit weight (URCS): See Appendix I

c (b) Rock Mass Properties: To be determined by geologic mapping, geophysical survey, remote imagery interpretation, core sample analysis, and geomorphic evaluation. Typical elements may include:

Discontinuities (URCS): See Appendix I
Strike and dip of formation: (show where measured)
Joint analysis: (spacing, orientation, separation, description of wall rock: wavy, rough, smooth, or slickensided)
Joint tightness: (open, cemented, filled, cavernous)
Other structures: (folds, faults, unconformities, rock unit contacts, random fractures, etc.)
Geomorphic features: (karst topography, lava flows, lineaments, etc.)
Voids: (caverns, vugs, sinkholes, lava tubes, etc.: include shape, orientation, type of filling)
Rock quality designation (RQD):
Seismic velocity:
Unified Rock Class: See Appendix I

(c) Geohydrologic Properties: To be determined by pressure testing; water wells, observation wells, drill holes, and/or piezometer data; review of published maps and reports; interpretation of rock material and rock mass properties; dye tests. Typical elements may include:

Primary porosity: (see: Rock Material Properties)
Secondary porosity: (see: Rock Material Properties)
Hydraulic conductivity: See Appendix II
Transmissivity: See Appendix II
Storativity/specific yield:
Soluble rock: (occurrence of limestone, gypsum, or dolomite; also see: Rock Material Properties)
Water table/potentiometric surface: (contour map, dated)
Aquifer type: (confined or unconfined)

RMFCP

PERFORMANCE ASSESSMENT

PERFORMANCE OBJECTIVES. Performance objectives are selected operational elements or conditions that require an assessment of rock material performance. Five performance objectives are considered.

1. Erosion Resistance: Evaluation of the rock to resist erosion in spillways, channels, or other areas where rock material must withstand the stress of flowing water.

2. Excavation Characteristics: Evaluation of rock excavation characteristics, including the type of procedure required (rock, common, etc.) and the fragmentation characteristics and blasting response anticipated.

3. Construction Quality: Analysis of rock quality for riprap, aggregate, embankment fill, foundation, and other construction requirements.

4. Fluid Transmission: Evaluation of rock unit potential for fluid transmission through primary and secondary pores; for investigations concerning reservoir, canal, and dam foundation seepage losses, excavation dewatering, engineering subdrainage for slope stability, point and non-point source pollution, ground water yield for development (water wells, springs, aquifers, and basins), ground water recharge or disposal, and other ground water conditions of concern.

5. Rock Mass Stability: Evaluation of rock mass stability in relation to natural and constructed slopes, adequacy as a foundation material, seismic effects, and other construction requirements.

The performance assessment of rock material is developed through the following process:

1. Classification of the rock unit in terms of the CLASSIFICATION ELEMENTS.

2. Selection of appropriate PERFORMANCE OBJECTIVES based upon project requirements or structure conditions.

3. Identification of the levels of rock capability and limitations using the Performance Assessment Tables 1-5.

4. Further description or amplification of the rock capabilities as required to provide specific performance assessments in support of planning, design, and construction of project elements.

APPENDIX

(Nonmandatory Information)

X1. ADDITIONAL INFORMATION

Afrouz, A. A., *Practical Handbook of Rock Mass Classification Systems and Modes of Ground Failure*, CRC Press, Boca Raton, 1992.

Bell, F. G., *Engineering Properties of Soils and Rocks*, Butterworth-Heinemann, Oxford, 1992.

Bieniawski, Z. T., "Engineering Classification of Jointed Rock Masses", *Transactions of the South African Institution of Civil Engineers*, Vol 15, 1973, pp. 335–344.

Deere, D. U., Hendron, A. J., Jr., Patton, F. D., and Cording, E. J., "Design of Surface and Near-Surface Construction in Rock", in *Failure and Breakage of Rock*, Fairhurst, C., Ed., Society of Mining Engineers of AIME, New York, 1967, pp. 237–302.

Wickham, G. E., Tiedemann, H. R., and Skinner, E. H., "Ground Support Prediction Model, RSR Concept," in *Proceedings*, Second Rapid Excavation and Tunneling Conference, San Francisco, June 1974, Vol I, pp. 691–707.

Williamson, D. A., "Uniform Rock Classification for Geotechnical Engineering Purposes," *Transportation Research Record 783*, National Academy of Sciences, Washington, DC, 1980, pp. 9–14.

REFERENCES

(1) *Rock Classification Systems for Engineering Purposes*, ASTM STP 984, ASTM, 1988.

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

(3) Barton, N., Lien, R., and Lunde, J., "Engineering Classification of Rock Masses for the Design of Tunnel Support," *Rock Mechanics*, Vol 6, No. 4, 1974, pp. 189–236.

(4) Barton, N., and Grimstad, E., "The Q-System Following Twenty Years of Application in NMT Support Selection," *Felsbau*, Vol 12, No. 6, 1994, pp. 428–436.

(5) Bieniawski, Z. T., *Engineering Rock Mass Classifications*, Wiley-Interscience, New York, 1989.

Appendix F

Appendix F

Geochemistry of Acid Mine Drainage

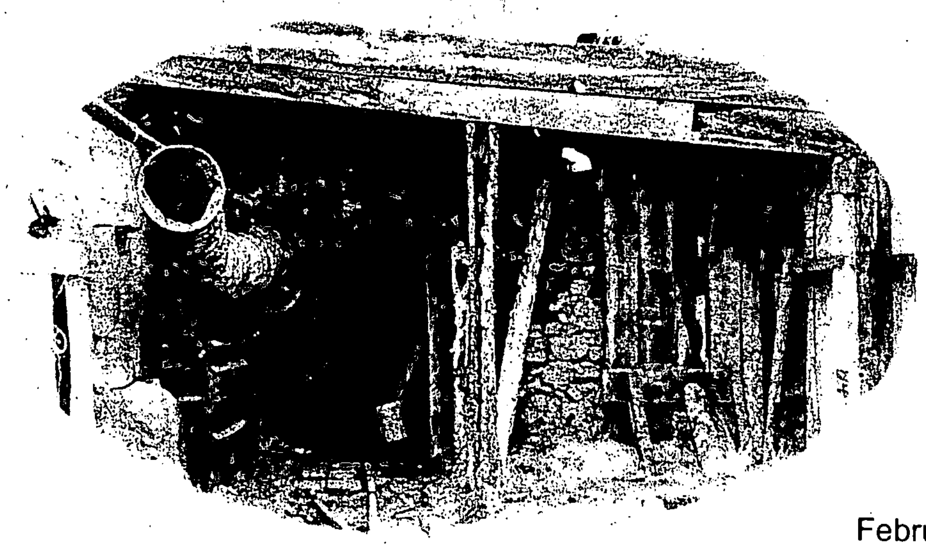
Walker & Associates, Inc.
Geochemistry, Engineering, and Occupational Health

[redacted]

FINAL REPORT

MINE SEAL TESTING AND EVALUATION: GEOCHEMISTRY OF ACID MINE DRAINAGE AND RATE OF DISSOLUTION OF CONCRETE ADIT PLUG

WALKER MINE
PLUMAS COUNTY, CA



February 26, 2002

Submitted to:

GEI Consultants
2201 Broadway
Oakland, CA 94612

[redacted]

2618 J Street, Suite 1
Sacramento, CA 95816
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Walker & Associates, Inc.
Geochemistry, Engineering, and Occupational Health

FINAL REPORT

MINE SEAL TESTING AND EVALUATION: GEOCHEMISTRY OF ACID MINE DRAINAGE AND RATE OF DISSOLUTION OF CONCRETE ADIT PLUG

WALKER MINE
PLUMAS COUNTY, CA

February 26, 2002

SIGNATURE PAGE

Prepared by:

2-26-02

William J. Walker, Ph.D.
Principal Geochemist

Date

Reviewed by:

Lara Pucik 2-26-02

Lara E. Pucik
Senior Environmental Chemist

Date

TABLE OF CONTENTS

LIST OF TABLES

1 INTRODUCTION

One of the primary environmental concerns at the Walker Mine is the durability and physiochemical stability of the concrete adit plug. Due to the corrosive nature of the acid drainage, it is possible that concrete is slowly dissolving in to the surrounding acidic water. Depending on the rate at which the concrete dissolves, the plug may remain durable for many years or may develop holes and weathering fissures leading to structural failure in much less time. In this report, results of the analyses performed and described previously in the Work Plan (April, 2001 GEI) were used to evaluate the extent of possible deterioration of the concrete and steel used to construct the plug.

2 TECHNICAL APPROACH

In this study, a comparison of the chemical similarities and differences between samples was made to determine whether or not significant deterioration is occurring. The Upstream sample was used as a baseline representing background acid mine drainage. The Drainpipe and Seep samples were compared to this baseline and differences in chemistry were attributed to steel corrosion and concrete dissolution, respectively.

To accomplish the comparison, several chemical markers were used to distinguish between pure AMD and AMD containing steel or concrete dissolution products. These markers included calcium, magnesium, sodium, potassium, aluminum, iron, silica, titanium, molybdenum, zinc, copper, and other trace metals.

3 AMD SAMPLING

Acid Mine Drainage was collected from the following four areas:

- Pond

  The standing pond in front of the plug was sampled prior to any other activity in the mine in order to provide an undisturbed sample.

- Seep

  Water seeping from the plug was sampled by collecting a drip stream emanating from the top of the plug.

- Drainpipe

  Water in long-term contact with the drainpipe was collected by sampling the first volume of water to flow out of the pipe upon opening of the valve.

- Upstream

  Water from the main body of AMD behind the plug was sampled by collecting flow from the drainpipe after approximately 25 gallons of water had been allowed to drain from the pipe. This represents 2 pipe pore volumes and means that water flowing through the pipe at the time of collection was sourced from AMD that had not been in long-term contact with the pipe prior to sampling.

Two unpreserved, 1L samples from each area were collected. One sample from each area was placed into a cooler with ice for transport under standard chain of custody protocols to Sequoia Analytical in Sacramento, CA for analysis of:

- pH,
- Total Dissolved Solids (TDS)
- Acidity,
- Alkalinity,
- Aluminum, Calcium, Chromium, Copper, Iron, Potassium, Magnesium, Manganese, Molybdenum, Nickel, Sodium, Lead, Silicon, Titanium, and Zinc.

These constituents were analyzed via the standard USEPA methods included in Appendix E of the Work Plan.

The other split was analyzed in the field for

- pH,
- Oxidation Reduction Potential (ORP),
- Dissolved Oxygen (DO), and
- Iron.

These constituents were all analyzed via portable electrode except for Iron, which was measured using a Hach DR2010 portable spectrophotometer and the Hach Ferrozine colorimetric method for Iron (included in Appendix E of the Work Plan).

4 RESULTS

Below are the results of the geochemical testing of waters emanating from the adit and seeps downgradient from the adit plug.

4.1 Field parameters

Samples were collected in November of 2000 from the locations noted above. Since pH, oxidation reduction potential (ORP), dissolved oxygen (DO) and dissolved total Iron (Fe) are subject to significant change over short periods of time, field equipment was used to measure these parameters directly on site. Other parameters were measured in the samples at an off-site laboratory as well and are discussed later. Results of the field analyses are presented in Table 1.

Based on the data collected in the field, the following observations have been made:

- The samples are all moderately acidic, ranging in pH from only 4.2 to 4.8. While the samples are acidic, the plug dissolution rate may not be too high compared to AMD with much lower pH (e.g. 2 to 3). The pH observed in these waters suggests that the host or wall rock may contain some buffering capacity and therefore is able to neutralize some of acidity generated by the pyrite oxidation. The exact reasons for the slight differences in the pH of the samples is not yet apparent.

- The samples all appear to be fairly well oxygenated, which is also expected in an actively oxidizing ore body. Oxygen is required for sulfide to oxidize to sulfate. The Pond and Seep samples are both in higher oxygen-containing environments because they are directly exposed to air.

- The dissolved oxygen content supports the ORP measurements. Most oxygen is observed in the seep and pond samples directly exposed to air, while the AMD behind the plug is much lower in dissolved oxygen. Here, diffusion of oxygen into the AMD and concurrent consumption by sulfide oxidation limits the dissolved oxygen content.

- Dissolved iron follows the dissolved oxygen and ORP measurements. Samples with low dissolved Fe are the same samples containing higher oxygen contents. High oxygen ensures oxidation of Fe (II) to Fe (III), which then precipitates as solid ferric hydroxide.

- Dissolved Fe in the seep is low (11.6 mg/L) compared to the AMD behind the plug (upstream sample). The same is true of these samples' pH values. This is due to exposure of the leaking AMD in front of the plug to oxygen, which assists in conversion of Fe (II) to Fe (III). Once formed, the Fe (III) rapidly hydrolyzes and precipitates as $Fe(OH)_3$ solid. This reaction series liberates 3 moles of proton for each mole of Fe converted to $Fe(OH)_3$, thereby lowering the pH. This explains the lower pH of seep AMD (4.1) compared to upstream AMD (4.6).

4.2 Chemical composition of AMD and seeps

Results of the off-site analysis of samples collected from the seep and upgradient samples are presented In Table 2.

As noted in Table 2, there are only a few noteworthy differences between the chemistry observed upstream of the plug and the seep emanating around or through the plug. In general, the major chemical and minor chemical constituents are nearly identical. The important points are noted below.

- The pH of the upstream sample is higher than the downstream (seep) sample. As noted earlier, this is due to the drop in total dissolved Fe as Fe (II) oxidizes to Fe (III) and then precipitates as the hydroxide with the generation of additional acidity. This is also why the Fe in the upstream sample is much higher than in the seep sample.

- Silicon in the two samples is identical, suggesting that the silicate portion of the concrete plug matrix is intact. The observed silica concentrations are likely from wall or host rock dissolution upstream of the plug. If the silicate portion of the plug were dissolving, the seep water would likely show a gain in silica content. This is not observed.

- Zinc and copper both increase in the seep water compared to the upstream sample. This may be due to the decrease in pH or possible leaching from the plug.

- Magnesium, calcium, sodium and potassium are all major components of the plug concrete and possibly the mine wall rock. If significant concrete dissolution were occurring, it would likely increase these components as AMD passes through the plug. The observed data show that there is very little difference in chemistry at the two locations. The slight changes noted in the table are well within typical analytical variance and cannot be used to conclusively identify an increase or decrease in concentration.

- Aluminum tends to increase in the seep water compared to the upstream water, suggesting that a gain in Al occurs as AMD seeps from the plug. Aluminum is a significant component of both concrete and wall rock. However, aluminum is usually more resistant than cations such as Mg, Na, K, and Ca to acid attack on concrete surfaces due to its role as a structural element. Therefore, increases in Al due to concrete dissolution should be accompanied by increases in these other cations as well. This is not observed. The increase in Al in the seep may be due only to the decrease in pH, which solubilizes Al in the rock through which the seep runs.

4.3 Plug stability evaluation: Significance of chemistry observations

As described in the chemistry discussion above, there is no apparent difference in most chemical constituents occurring behind the plug in the AMD or in the seep flowing around the contact between the plug and the wall rock. Based on these observations and observations in the field, the following conclusions can be drawn:

- The differences in chemistry between the locations are limited mainly to pH and Fe. The reasons for this have been discussed previously. Field observations on the condition of the concrete plug face have revealed that heavy iron staining has occurred in some locations, which is consistent with the precipitation of Fe (III) upon oxygenation.

- In cases where small differences in chemistry occur, they tend to be within typical analytical variance (<25%) suggesting that most differences may not be significant. Repeated sampling of the different locations is necessary to determine if the differences are consistent and real.

- Dissolution of the concrete would likely result in congruent, measurable increases of Mg, Ca, Na, Si, and Al over time. Presently these consistent increases are not observed in the samples collected.

- Despite the lack of chemical evidence, field observations of the concrete face below the seep water line do show some locations where acid attack is evident. Secondary dissolution products have covered the wall these locations, with thicknesses ranging up to 2 mm. Concrete beneath these alteration products appears to be intact and hard.

- Since the pH of the seep and pond samples is lower than that observed in the AMD on the interior side of the plug, acid attack may be more aggressive on the outside of the plug.

4.4 Estimates of plug dissolution rates

Since the chemistry measured to date has not shown any obvious trends indicating major plug dissolution, the field measurements represent a simple way to provide a conservative estimate of the rate of dissolution of the concrete plug. The features of this analysis are described below.

1. The concrete stability experts noted dissolution products (Crust/coatings) occurring in areas where the concrete was in contact with AMD. The thickness of these coatings was approximately 2mm.

2. Since the plug was completed 13 years ago, the coatings accreted at a rate of 2mm/13 yrs or 0.15mm/yr. In using this estimate, recall that tiny fissures or breaks in the

concrete may allow much more rapid alterations because surface area would be increased and weathering rates are directly proportional to surface area.

3. Normally, rates of dissolution are partial order or parabolic in nature. Rates slow with time. For the sake of conservancy, a linear rate will be assumed. In many wall rock and rock dissolution scenarios, salts and crust build up rapidly, but then slow since the by-products inhibit the reaction by acting as diffusion barriers. Flushing of the salts from the rock may occur if the water table rises or fractures fill and flush openings in rock. This allows the reactions to once again proceed rapidly. Such a scenario may be a seasonal or annual event. Regardless of the mechanism, little is known about the interior face of the plug and hence a conservative approach is best for estimating dissolution rates. Because the plug face is in contact with lower pH water (3.8 compared to 4.6) and has obvious crust and salt build up, the weathering depth (2 mm) represents a good estimate of plug dissolution.

5 SUMMARY AND CONCLUSIONS

Based on chemical analysis of waters from the mine, there is no apparent difference in most chemical constituents occurring behind the plug in the AMD or in the seep flowing around the contact between the plug and the wall rock. From these chemical observations as well as observations in the field, the following conclusions can be drawn:

- The differences in chemistry between the locations are limited mainly to pH and Fe. The reasons for this have been discussed previously. Field observations on the condition of the concrete plug face have revealed that heavy iron staining has occurred in some locations, which is consistent with the precipitation of Fe (III) upon oxygenation.

- In cases where small differences in chemistry occur, they tend to be within typical analytical variance (<25%). This suggests that most differences may not be significant. Repeated sampling of the different locations is necessary to determine if the differences are consistent and real.

- Dissolution of the concrete would likely result in congruent, measurable increases in Mg, Ca, Na, Si, and Al over time. Presently these consistent increases are not observed in the samples collected.

- Despite the lack of chemical evidence, field observations of the concrete face below the seep water line do show some locations where acid attack is evident. Secondary dissolution products have covered the wall these locations, with thicknesses ranging up to 2 mm. Concrete beneath these alteration products appears to be intact and hard.

- Since the pH of the seep and pond samples is lower than that observed in the AMD on the interior side of the plug, acid attack may be more aggressive on the outside of the plug.

- Concrete experts have observed a plug dissolution rate of about 0.15 mm/yr.

- In using the above concrete dissolution rate estimate, recall that tiny fissures or breaks in the concrete may allow much more rapid alterations. This is because surface area under such conditions would be increased and weathering rates are directly proportional to surface area.

TABLES

Table 1. *Results of field analyses conducted November 1, 2000.*

Sample ID	time	pH	ORP [mV]	DO [mg/L]	Fe [mg/L]
Seep	11:15	4.17	350	6.7	11.6
Pond	10:45	4.80	312	7.4	6.3
Drainpipe	11:20	4.53	286	1.7	40.3
Upstream	11:40	4.57	279	1.6	39.9

Table 2. *Results of offsite chemical analyses on samples collected November 1, 2000.*

Analyte		Pond	Drainpipe	Upstream	Seep	Difference (Seep - Upstream)
Acidity	[mg/L]	23.0	91.0	92.0	59.0	-33.0
Total Alkalinity	[mg/L]	<1.00	<1.00	<1.00	<1.00	0
pH	[std units]	4.55	4.51	4.58	3.82	-0.76
Total Dissolved Solids	[mg/L]	355	335	343	234	-109
Aluminum	[mg/L]	2.68	2.27	2.23	4.00	+1.77
Calcium	[mg/L]	61.2	30.1	29.3	24.7	-4.6
Chromium	[mg/L]	<0.00125	<0.00125	<0.00125	<0.00125	0
Copper	[mg/L]	6.41	0.426	1.79	9.06	+7.27
Iron	[mg/L]	3.24	37.4	36.1	7.80	-28.3
Lead	[mg/L]	<0.0500	<0.0500	<0.0500	<0.0500	0
Magnesium	[mg/L]	6.43	7.57	7.34	5.81	-1.53
Manganese	[mg/L]	3.57	5.15	4.97	3.18	-1.79
Molybdenum	[mg/L]	<0.00500	<0.00500	<0.00500	<0.00500	0
Nickel	[mg/L]	0.0101	0.00950	0.00880	0.00825	-0.00055
Potassium	[mg/L]	2.83	2.10	2.05	1.99	-0.06
Zinc	[mg/L]	0.619	0.641	0.587	0.721	+0.134
Sodium	[mg/L]	3.13	2.37	2.30	2.21	-0.09
Silicon	[mg/L]	10.6	13.1	13.0	12.9	-0.10
Titanium	[mg/L]	<0.0100	<0.0100	<0.0100	<0.0100	0

APPENDIX

Sequoia Analytical Data Reports

Sequoia Analytical

819 Striker Avenue, Suite 8
Sacramento, CA 95834
(916) 921-9600
FAX (916) 921-0100
www.sequoialabs.com

December 11 , 2000

Bill Walker
Walker & Associates
2618 J Street, Ste. 1
Sacramento, CA 95816
RE: Walker Mine

Enclosed are the results of analyses for samples received by the laboratory on 11/03/00 15:35. If you have any questions concerning this report, please feel free to contact me.

Sincerely,

Sandra R Hanson

andra R. Hanson
lient Services Representative

A ELAP Certificate Number 2374

Sequoia Analytical

819 Striker Avenue, Suite 8
Sacramento, CA 95834
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FAX (916) 921-0100
www.sequoialabs.com

Walker & Associates
2618 J Street, Ste. 1
Sacramento CA, 95816

Project: Walker Mine
Project Number: N/A
Project Manager: Bill Walker

Reported:
12/11/00 12:08

ANALYTICAL REPORT FOR SAMPLES

Sample ID	Laboratory ID	Matrix	Date Sampled	Date Received
Pond	S011077-01	Water	11/01/00 00:00	11/03/00 15:35
Seep	S011077-02	Water	11/01/00 00:00	11/03/00 15:35
Drainpipe	S011077-03	Water	11/01/00 00:00	11/03/00 15:35
Upstream	S011077-04	Water	11/01/00 00:00	11/03/00 15:35

Sequoia Analytical

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Walker & Associates 2618 J Street, Ste. 1 Sacramento CA, 95816	Project: Walker Mine Project Number: N/A Project Manager: Bill Walker	Reported: 12/11/00 12:08

Total Metals by EPA 200 Series Methods
Sequoia Analytical - Sacramento

Analyte	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method	Notes
Pond (S011077-01) Water Sampled: 11/01/00 00:00 Received: 11/03/00 15:35									
Calcium	61.2	0.200	mg/l	1	0110119	11/08/00	11/10/00	EPA 200.7	
Copper	6.41	0.00500	"	"	"	"	"	"	
Iron	3.24	0.0100	"	"	"	"	"	"	
Potassium	2.83	0.500	"	"	"	"	"	"	
Magnesium	6.43	0.0500	"	"	"	"	"	"	
Manganese	3.57	0.00500	"	"	"	"	"	"	
Sodium	3.13	0.200	"	"	"	"	"	"	
Lead	ND	0.0500	"	"	"	"	"	"	
Zinc	0.619	0.00500	"	"	"	"	"	"	
Seep (S011077-02) Water Sampled: 11/01/00 00:00 Received: 11/03/00 15:35									
Calcium	24.7	0.200	mg/l	1	0110119	11/08/00	11/10/00	EPA 200.7	
Copper	9.06	0.00500	"	"	"	"	"	"	
Iron	7.80	0.0100	"	"	"	"	"	"	
Potassium	1.99	0.500	"	"	"	"	"	"	
Magnesium	5.81	0.0500	"	"	"	"	"	"	
Manganese	3.18	0.00500	"	"	"	"	"	"	
Sodium	2.21	0.200	"	"	"	"	"	"	
Lead	ND	0.0500	"	"	"	"	"	"	
Zinc	0.721	0.00500	"	"	"	"	"	"	
Drainpipe (S011077-03) Water Sampled: 11/01/00 00:00 Received: 11/03/00 15:35									
Calcium	30.1	0.200	mg/l	1	0110119	11/08/00	11/10/00	EPA 200.7	
Copper	0.426	0.00500	"	"	"	"	"	"	
Iron	37.4	0.0100	"	"	"	"	"	"	
Potassium	2.10	0.500	"	"	"	"	"	"	
Magnesium	7.57	0.0500	"	"	"	"	"	"	
Manganese	5.15	0.00500	"	"	"	"	"	"	
Sodium	2.37	0.200	"	"	"	"	"	"	
Lead	ND	0.0500	"	"	"	"	"	"	
Zinc	0.641	0.00500	"	"	"	"	"	"	

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Walker & Associates 2618 J Street, Ste. 1 Sacramento CA, 95816	Project: Walker Mine Project Number: N/A Project Manager: Bill Walker	Reported: 12/11/00 12:08

Total Metals by EPA 200 Series Methods

Sequoia Analytical - Sacramento

Analyte	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method	Notes
Upstream (S011077-04) Water	**Sampled: 11/01/00 00:00**	**Received: 11/03/00 15:35**							
Calcium	**29.3**	0.200	mg/l	1	0110119	11/08/00	11/10/00	EPA 200.7	
Copper	**1.79**	0.00500	"	"	"	"	"	"	
Iron	**36.1**	0.0100	"	"	"	"	"	"	
Potassium	**2.05**	0.500	"	"	"	"	"	"	
Magnesium	**7.34**	0.0500	"	"	"	"	"	"	
Manganese	**4.97**	0.00500	"	"	"	"	"	"	
Sodium	**2.30**	0.200	"	"	"	"	"	"	
Lead	ND	0.0500	"	"	"	"	"	"	
Zinc	**0.587**	0.00500	"	"	"	"	"	"	

The results in this report apply to the samples analyzed in accordance with the chain of custody document. This analytical report must be reproduced in its entirety.

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Walker & Associates 2618 J Street, Ste. 1 Sacramento CA, 95816	Project: Walker Mine Project Number: N/A Project Manager: Bill Walker	Reported: 12/11/00 12:08

Conventional Chemistry Parameters by APHA/EPA Methods

Sequoia Analytical - Sacramento

Analyte	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method	Notes
Pond (S011077-01) Water Sampled: 11/01/00 00:00 Received: 11/03/00 15:35									
Acidity	**23.0**	10.0	mg/l	1	0110176	11/13/00	11/13/00	EPA 305.1	
Total Alkalinity	ND	1.00	"	"	0110071	11/06/00	11/06/00	EPA 310.1	
pH	**4.55**		pH Units	"	0110078	11/03/00	11/03/00	EPA 150.1	
Total Dissolved Solids	**355**	5.00	mg/l	"	0110163	11/08/00	11/10/00	EPA 160.1	
Seep (S011077-02) Water Sampled: 11/01/00 00:00 Received: 11/03/00 15:35									
Acidity	**59.0**	10.0	mg/l	1	0110176	11/13/00	11/13/00	EPA 305.1	
Total Alkalinity	ND	1.00	"	"	0110071	11/06/00	11/06/00	EPA 310.1	
pH	**3.82**		pH Units	"	0110078	11/03/00	11/03/00	EPA 150.1	
Total Dissolved Solids	**234**	5.00	mg/l	"	0110163	11/08/00	11/10/00	EPA 160.1	
Drainpipe (S011077-03) Water Sampled: 11/01/00 00:00 Received: 11/03/00 15:35									
Acidity	**91.0**	10.0	mg/l	1	0110176	11/13/00	11/13/00	EPA 305.1	
Total Alkalinity	ND	1.00	"	"	0110071	11/06/00	11/06/00	EPA 310.1	
pH	**4.51**		pH Units	"	0110078	11/03/00	11/03/00	EPA 150.1	
Total Dissolved Solids	**335**	5.00	mg/l	"	0110163	11/08/00	11/10/00	EPA 160.1	
Upstream (S011077-04) Water Sampled: 11/01/00 00:00 Received: 11/03/00 15:35									
Acidity	**92.0**	10.0	mg/l	1	0110176	11/13/00	11/13/00	EPA 305.1	
Total Alkalinity	ND	1.00	"	"	0110071	11/06/00	11/06/00	EPA 310.1	
pH	**4.58**		pH Units	"	0110078	11/03/00	11/03/00	EPA 150.1	
Total Dissolved Solids	**343**	5.00	mg/l	"	0110163	11/08/00	11/10/00	EPA 160.1	

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Walker & Associates 2618 J Street, Ste. 1 Sacramento CA, 95816	Project: Walker Mine Project Number: N/A Project Manager: Bill Walker	Reported: 12/11/00 12:08

Total Metals by EPA 200 Series Methods
Sequoia Analytical - Walnut Creek

Analyte	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method	Notes
Pond (S011077-01) Water Sampled: 11/01/00 00:00 Received: 11/03/00 15:35									
Silicon	10.6	0.0500	mg/l	1	0K09007	11/09/00	12/01/00	EPA 200.7	
Titanium	ND	0.0100	"	"	"	"	"	"	
Seep (S011077-02) Water Sampled: 11/01/00 00:00 Received: 11/03/00 15:35									
Silicon	12.9	0.0500	mg/l	1	0K09007	11/09/00	12/01/00	EPA 200.7	
Titanium	ND	0.0100	"	"	"	"	"	"	
Drainpipe (S011077-03) Water Sampled: 11/01/00 00:00 Received: 11/03/00 15:35									
Silicon	13.1	0.0500	mg/l	1	0K09007	11/09/00	12/01/00	EPA 200.7	
Titanium	ND	0.0100	"	"	"	"	"	"	
Upstream (S011077-04) Water Sampled: 11/01/00 00:00 Received: 11/03/00 15:35									
Silicon	13.0	0.0500	mg/l	1	0K09007	11/09/00	12/01/00	EPA 200.7	
Titanium	ND	0.0100	"	"	"	"	"	"	

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Total Metals by EPA 200 Series Methods - Quality Control
Sequoia Analytical - Sacramento

Analyte	Result	Reporting Limit	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit	Notes
Batch 0110119 - EPA 3010A										
Blank (0110119-BLK1)				Prepared: 11/08/00 Analyzed: 11/10/00						
Calcium	ND	0.200	mg/l							
Copper	ND	0.00500	"							
Iron	ND	0.0100	"							
Lead	ND	0.0500	"							
Magnesium	ND	0.0500	"							
Manganese	ND	0.00500	"							
Potassium	ND	0.500	"							
Sodium	ND	0.200	"							
Zinc	ND	0.00500	"							
LCS (0110119-BS1)				Prepared: 11/08/00 Analyzed: 11/10/00						
Calcium	1.18	0.200	mg/l	1.25		94.4	80-120			
Copper	0.469	0.00500	"	0.500		93.8	80-120			
Iron	0.482	0.0100	"	0.500		96.4	80-120			
Lead	0.482	0.0500	"	0.500		96.4	80-120			
Magnesium	1.16	0.0500	"	1.25		92.8	80-120			
Manganese	0.473	0.00500	"	0.500		94.6	80-120			
Potassium	4.56	0.500	"	5.00		91.2	80-120			
Sodium	2.37	0.200	"	2.50		94.8	80-120			
Zinc	0.484	0.00500	"	0.500		96.8	80-120			
Matrix Spike (0110119-MS1)	Source: S010524-01			Prepared: 11/08/00 Analyzed: 11/10/00						
Calcium	90.9	0.200	mg/l	1.25	85.3	448	80-120			Q-03
Copper	0.500	0.00500	"	0.500	ND	100	80-120			
Iron	0.526	0.0100	"	0.500	0.0283	99.5	80-120			
Lead	0.492	0.0500	"	0.500	ND	98.4	80-120			
Magnesium	71.9	0.0500	"	1.25	66.9	400	80-120			Q-03
Manganese	0.558	0.00500	"	0.500	0.0645	98.7	80-120			
Potassium	8.61	0.500	"	5.00	3.32	106	80-120			
Sodium	164	0.200	"	2.50	153	440	80-120			Q-03
Zinc	0.506	0.00500	"	0.500	0.00505	100	80-120			

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Walker & Associates 2618 J Street, Ste. 1 Sacramento CA, 95816	Project: Walker Mine Project Number: N/A Project Manager: Bill Walker	Reported: 12/11/00 12:08

Total Metals by EPA 200 Series Methods - Quality Control
Sequoia Analytical - Sacramento

Analyte	Result	Reporting Limit	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit	Notes
Batch 0110119 - EPA 3010A										
Matrix Spike Dup (0110119-MSD1)		**Source: S010524-01**		Prepared: 11/08/00	Analyzed: 11/10/00					
Calcium	87.2	0.200	mg/l	1.25	85.3	152	80-120	4.15	20	Q-03
Copper	0.473	0.00500	"	0.500	ND	94.6	80-120	5.55	20	
Iron	0.496	0.0100	"	0.500	0.0283	93.5	80-120	5.87	20	
Lead	0.475	0.0500	"	0.500	ND	95.0	80-120	3.52	20	
Magnesium	68.7	0.0500	"	1.25	66.9	144	80-120	4.55	20	Q-03
Manganese	0.530	0.00500	"	0.500	0.0645	93.1	80-120	5.15	20	
Potassium	8.31	0.500	"	5.00	3.32	99.8	80-120	3.55	20	
Sodium	156	0.200	"	2.50	153	120	80-120	5.00	20	Q-03
Zinc	0.483	0.00500	"	0.500	0.00505	95.6	80-120	4.65	20	

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Walker & Associates 2618 J Street, Ste. 1 Sacramento CA, 95816	Project: Walker Mine Project Number: N/A Project Manager: Bill Walker	Reported: 12/11/00 12:08

Conventional Chemistry Parameters by APHA/EPA Methods - Quality Control

Sequoia Analytical - Sacramento

Analyte	Result	Reporting Limit	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit	Notes
Batch 0110071 - General Preparation										
Blank (0110071-BLK1)				Prepared & Analyzed: 11/06/00						
Total Alkalinity	ND	1.00	mg/l							
LCS (0110071-BS1)				Prepared & Analyzed: 11/06/00						
Total Alkalinity	22.6	1.00	mg/l	25.0		90.4	80-120			
Matrix Spike (0110071-MS1)	Source: S011016-01			Prepared & Analyzed: 11/06/00						
Total Alkalinity	137	1.00	mg/l	26.5	113	90.6	75-125			
Matrix Spike Dup (0110071-MSD1)	Source: S011016-01			Prepared & Analyzed: 11/06/00						
Total Alkalinity	137	1.00	mg/l	26.5	113	90.6	75-125	0	20	
Batch 0110078 - General Preparation										
Duplicate (0110078-DUP1)	Source: S011077-01			Prepared & Analyzed: 11/03/00						
pH	4.55		pH Units		4.55			0	20	
Batch 0110163 - General Preparation										
Blank (0110163-BLK1)				Prepared: 11/08/00 Analyzed: 11/10/00						
Total Dissolved Solids	ND	5.00	mg/l							
LCS (0110163-BS1)				Prepared: 11/08/00 Analyzed: 11/10/00						
Total Dissolved Solids	476	5.00	mg/l	500		95.2	80-120			
Matrix Spike (0110163-MS1)	Source: S011077-03			Prepared: 11/08/00 Analyzed: 11/10/00						
Total Dissolved Solids	799	5.00	mg/l	500	335	92.8	80-120			

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Walker & Associates 2618 J Street, Ste. 1 Sacramento CA, 95816	Project: Walker Mine Project Number: N/A Project Manager: Bill Walker	Reported: 12/11/00 12:08

Conventional Chemistry Parameters by APHA/EPA Methods - Quality Control
Sequoia Analytical - Sacramento

Analyte	Result	Reporting Limit	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit	Notes
Batch 0110163 - General Preparation										
Matrix Spike Dup (0110163-MSD1)		**Source: S011077-03**			Prepared: 11/08/00 Analyzed: 11/10/00					
Total Dissolved Solids	813	5.00	mg/l	500	335	95.6	80-120	1.74	20	
Batch 0110176 - General Preparation										
Blank (0110176-BLK1)					Prepared & Analyzed: 11/13/00					
Acidity	ND	10.0	mg/l							
Duplicate (0110176-DUP1)		**Source: S011077-01**			Prepared & Analyzed: 11/13/00					
Acidity	27.0	10.0	mg/l		23.0			16.0	20	

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Walker & Associates 2618 J Street, Ste. 1 Sacramento CA, 95816	Project: Walker Mine Project Number: N/A Project Manager: Bill Walker	Reported: 12/11/00 12:08

Total Metals by EPA 200 Series Methods - Quality Control
Sequoia Analytical - Walnut Creek

Analyte	Result	Reporting Limit	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit	Notes
Batch 0K09007 - 200.7										
Blank (0K09007-BLK1)				Prepared: 11/09/00 Analyzed: 12/01/00						
Silicon	ND	0.0500	mg/l							
Titanium	ND	0.0100	"							
LCS (0K09007-BS1)				Prepared: 11/09/00 Analyzed: 12/01/00						
Titanium	0.960	0.0100	mg/l	1.00		96.0	80-120			
LCS (0K09007-BS2)				Prepared: 11/09/00 Analyzed: 12/01/00						
Silicon	8.19	0.0500	mg/l	10.0		81.9	80-120			
LCS Dup (0K09007-BSD1)				Prepared: 11/09/00 Analyzed: 12/01/00						
Titanium	1.02	0.0100	mg/l	1.00		102	80-120	6.06	20	
LCS Dup (0K09007-BSD2)				Prepared: 11/09/00 Analyzed: 12/01/00						
Silicon	8.48	0.0500	mg/l	10.0		84.8	80-120	3.48	20	
Matrix Spike (0K09007-MS1)		Source: W011137-01		Prepared: 11/09/00 Analyzed: 12/01/00						
Titanium	1.08	0.0100	mg/l	1.00	0.0709	101	80-120			
Matrix Spike (0K09007-MS2)		Source: W011137-01		Prepared: 11/09/00 Analyzed: 12/01/00						
Silicon	20.6	0.0500	mg/l	10.0	9.46	111	80-120			
Matrix Spike Dup (0K09007-MSD1)		Source: W011137-01		Prepared: 11/09/00 Analyzed: 12/01/00						
Titanium	1.11	0.0100	mg/l	1.00	0.0709	104	80-120	2.74	20	
Matrix Spike Dup (0K09007-MSD2)		Source: W011137-01		Prepared: 11/09/00 Analyzed: 12/01/00						
Silicon	19.9	0.0500	mg/l	10.0	9.46	104	80-120	3.46	20	

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Walker & Associates 2618 J Street, Ste. 1 Sacramento CA, 95816	Project: Walker Mine Project Number: N/A Project Manager: Bill Walker	Reported: 12/11/00 12:08

Notes and Definitions

Q-03	The RPD and/or percent recovery for this QC spike sample cannot be accurately calculated due to the high concentration of analyte already present in the sample.
DET	Analyte DETECTED
ND	Analyte NOT DETECTED at or above the reporting limit
NR	Not Reported
dry	Sample results reported on a dry weight basis
RPD	Relative Percent Difference

SEQUOIA ANALYTICAL
CHAIN OF CUSTODY

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☐ 1551 Industrial Road • San Carlos, CA 94070 • (650) 232-9600 FAX (650) 232-9612

Company Name: Walker + Associates
Mailing Address: 2618 J Street, Suite 1
City: Sacramento State: CA Zip Code: 95816
Telephone: 916 442 5304 FAX #: 916 442 5313
Report To: Bill Walker Sampler: L. Pucik

Project Name: Walker Mine
Billing Address (if different):
P.O. #:
QC Data: ☐ Level D (Standard) ☐ Level C ☐ Level B ☐ Level A

Turnaround Time: ☒ 10 Working Days ☐ 7 Working Days ☐ 5 Working Days ☐ 3 Working Days ☐ 2 Working Days ☐ 24 Hours ☐ 2 - 8 Hours

☐ Drinking Water ☒ Waste Water ☐ Other

Analyses Requested

Client Sample I.D.	Date/Time Sampled	Matrix Desc.	# of Cont.	Cont. Type	Sequoia's Sample #	pH	Alk/Acidity	TDS	Ca, Mg, Na, K	Si, Ti, Cu, Zn, Pb, Fe, Mn (totals)					Comments
1. Pond	11.1.00 10:45	H_2O	1	unpreserved 1L		X	X	X	X	X	S011077-01				please retain
2. Seep	↓ 11:15	↓	↓	↓		↓	↓	↓	↓	↓	-02				samples... we may
3. Drainpipe	11:20										-03				request further
4. Upstream	11:40	↓	↓	↓		↓	↓	↓	↓	↓	-04				analyses
5.															* METALS
6.															PRESERVED IN
7.															LAB. SLIT
8.															[illegible]
9.															
10.															

Relinquished By: Lisa Pucik	Date: 11-3-00	Time: 15:35	Received By: Monica Gregson Date: 11/3/00 Time: 15:35
Relinquished By:	Date:	Time:	Received By: Date: Time:
Relinquished By:	Date:	Time:	Received By: Date: Time:

Were Samples Received in Good Condition? ☐ Yes ☐ No Samples on Ice? ☐ Yes ☐ No Method of Shipment

Page 1 of 1

Pink - Client
Yellow - Sequoia
White - Sequoia

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November 30 , 2000

Bill Walker
Walker & Associates
2618 J Street, Ste. 1
Sacramento, CA 95816
RE: Walker Mine

Enclosed are the results of analyses for samples received by the laboratory on 11/13/00 14:25. If you have any questions concerning this report, please feel free to contact me..

Sincerely,

Sandra R Hanson

Sandra R. Hanson
Client Services Representative

CA ELAP Certificate Number 2374

Sequoia Analytical

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Walker & Associates 2618 J Street, Ste. 1 Sacramento CA, 95816	Project: Walker Mine Project Number: N/A Project Manager: Bill Walker	Reported: 11/30/00 15:24

ANALYTICAL REPORT FOR SAMPLES

Sample ID	Laboratory ID	Matrix	Date Sampled	Date Received
Pond	S011224-01	Water	11/01/00 00:00	11/13/00 14:25
Seep	S011224-02	Water	11/01/00 00:00	11/13/00 14:25
Drainpipe	S011224-03	Water	11/01/00 00:00	11/13/00 14:25
Upstream	S011224-04	Water	11/01/00 00:00	11/13/00 14:25

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Walker & Associates 2618 J Street, Ste. 1 Sacramento CA, 95816	Project: Walker Mine Project Number: N/A Project Manager: Bill Walker	Reported: 11/30/00 15:24

Total Metals by EPA 200 Series Methods
Sequoia Analytical - Sacramento

Analyte	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method	Notes
Pond (S011224-01) Water	**Sampled: 11/01/00 00:00**	**Received: 11/13/00 14:25**							
Aluminum	2.68	0.0125	mg/l	1	0110360	11/29/00	11/30/00	EPA 200.7	
Chromium	ND	0.00125	"	"	"	"	"	"	
Molybdenum	ND	0.00500	"	"	"	"	"	"	
Nickel	0.0101	0.00500	"	"	"	"	"	"	
Seep (S011224-02) Water	**Sampled: 11/01/00 00:00**	**Received: 11/13/00 14:25**							
Aluminum	4.00	0.0125	mg/l	1	0110360	11/29/00	11/30/00	EPA 200.7	
Chromium	ND	0.00125	"	"	"	"	"	"	
Molybdenum	ND	0.00500	"	"	"	"	"	"	
Nickel	0.00825	0.00500	"	"	"	"	"	"	
Drainpipe (S011224-03) Water	**Sampled: 11/01/00 00:00**	**Received: 11/13/00 14:25**							
Aluminum	2.27	0.0125	mg/l	1	0110360	11/29/00	11/30/00	EPA 200.7	
Chromium	0.00200	0.00125	"	"	"	"	"	"	
Molybdenum	ND	0.00500	"	"	"	"	"	"	
Nickel	0.00950	0.00500	"	"	"	"	"	"	
Upstream (S011224-04) Water	**Sampled: 11/01/00 00:00**	**Received: 11/13/00 14:25**							
Aluminum	2.23	0.0125	mg/l	1	0110360	11/29/00	11/30/00	EPA 200.7	
Chromium	ND	0.00125	"	"	"	"	"	"	
Molybdenum	ND	0.00500	"	"	"	"	"	"	
Nickel	0.00880	0.00500	"	"	"	"	"	"	

The results in this report apply to the samples analyzed in accordance with the chain of custody document. This analytical report must be reproduced in its entirety.

Sequoia Analytical

819 Striker Avenue, Suite 8
Sacramento, CA 95834
(916) 921-9600
FAX (916) 921-0100
www.sequoialabs.com

Walker & Associates 2618 J Street, Ste. 1 Sacramento CA, 95816	Project: Walker Mine Project Number: N/A Project Manager: Bill Walker	Reported: 11/30/00 15:24

Total Metals by EPA 200 Series Methods - Quality Control

Sequoia Analytical - Sacramento

Analyte	Result	Reporting Limit	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit	Notes
Batch 0110360 - EPA 3010A										
Blank (0110360-BLK1)				Prepared: 11/29/00 Analyzed: 11/30/00						
Aluminum	ND	0.0125	mg/l							
Chromium	ND	0.00125	"							
Molybdenum	ND	0.00500	"							
Nickel	ND	0.00500	"							
LCS (0110360-BS1)				Prepared: 11/29/00 Analyzed: 11/30/00						
Aluminum	1.32	0.0125	mg/l	1.25		106	80-120			
Chromium	0.512	0.00125	"	0.500		102	80-120			
Molybdenum	0.524	0.00500	"	0.500		105	80-120			
Nickel	0.504	0.00500	"	0.500		101	80-120			
Matrix Spike (0110360-MS1)	Source: S011222-01			Prepared: 11/29/00 Analyzed: 11/30/00						
Aluminum	1.33	0.0125	mg/l	1.25	0.0154	105	80-120			
Chromium	0.510	0.00125	"	0.500	ND	102	80-120			
Molybdenum	0.524	0.00500	"	0.500	ND	105	80-120			
Nickel	0.503	0.00500	"	0.500	ND	101	80-120			
Matrix Spike Dup (0110360-MSD1)	Source: S011222-01			Prepared: 11/29/00 Analyzed: 11/30/00						
Aluminum	1.30	0.0125	mg/l	1.25	0.0154	103	80-120	2.28	20	
Chromium	0.497	0.00125	"	0.500	ND	99.4	80-120	2.58	20	
Molybdenum	0.511	0.00500	"	0.500	ND	102	80-120	2.51	20	
Nickel	0.489	0.00500	"	0.500	ND	97.8	80-120	2.82	20	

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Walker & Associates 2618 J Street, Ste. 1 Sacramento CA, 95816	Project: Walker Mine Project Number: N/A Project Manager: Bill Walker	Reported: 11/30/00 15:24

Notes and Definitions

DET	Analyte DETECTED
ND	Analyte NOT DETECTED at or above the reporting limit
NR	Not Reported
dry	Sample results reported on a dry weight basis
RPD	Relative Percent Difference

SEQUOIA ANALYTICAL RELOG SHEET

CLIENT: Walker + Assoc RELOG DATE: 11/13/00
PROJECT ID: Walker Mine DATE DUE:
REPORT TO: Bill Walker
DATE REC'D: 11/3/00 T.A.T.: Std TAT

PREVIOUSLY LOGGED SAMPLES

☐ TAT Change status to: ________ Change status as of: Day: ________, Time: ________

☐ CHANGE ANALYSES: + ☐ Add Analyses - ☐ Cancel Analyses

Sequoia Sample #	Client Sample ID	Matrix	Date Sampled	Analyses	Previous Sequoia #
011224-01	Pond	W	11/1/00	Cr, Ni, Mo, Al	S011077-01
-02	Seep				-02
-03	Drainpipe				-03
-04	Upstream				-04

COMMENTS

Authorization (Person/Date/Time): Laura Pucik 11/13/00 @ 1425

Sequoia Project Manager: Sandi Hansen

:QUOIA ANALYTICAL CHAIN OF CUSTODY

☐ 819 Striker Ave ... ite 8 • Sacramento, CA 95834 • (916) 921-9600 FAX (916) 921-...
☐ 404 N. Wiget Lane • Walnut Creek, CA 94598 • (925) 988-9600 FAX (925) 988-9673
☐ 1455 McDowell Blvd. North, Suite D • Petaluma, CA 94954 • (707) 792-1865 FAX (707) 792-0342
☐ 1551 Industrial Road • San Carlos, CA 94070 • (650) 232-9600 FAX (650) 232-9612
... (408) 776-9600 • FAX (408) 782-6308

Company Name: Walker + Associates
Mailing Address: 2618 J Street, Suite 1
City: Sacramento State: CA Zip Code: 95816
Telephone: 916 442 5304 FAX #: 916 442 5313
Report To: Bill Walker Sampler: L. Pucik

Project Name: Walker Mine
Billing Address (if different):
P.O. #:
QC Data: ☐ Level D (Standard) ☐ Level C ☐ Level B — ☐ Level A

Turnaround Time: ☒ 10 Working Days ☐ 7 Working Days ☐ 5 Working Days ☐ 3 Working Days ☐ 2 Working Days ☐ 24 Hours ☐ 2 - 8 Hours

☐ Drinking Water ☒ Waste Water ☐ Other

Analyses Requested

Client Sample I.D.	Date/Time Sampled	Matrix Desc.	# of Cont.	Cont. Type	Sequoia's Sample #	pH	Alk/Acidity	TDS	Ca, Mg, Na, K	Si, Ti, Cu, Zn, Pb, Fe, Mn totals					Comments
1. Pond	11.1.00 10:45	H_2O	1	unpreserved 1L		X	X	X	X	X	S011077-01				please return
2. Seep	↓ 11:15	↓	↓	↓		↓	↓	↓	↓	↓	-02				samples ... we may
3. Drainpipe	11:20										-03				request further
4. Upstream	11:40										-04				analyses
5.															
6.															* METALS
7.															PRESERVED IN
8.															LAB. SLIT
9.															[signature]
10.															

Relinquished By: Lara Pucik	Date: 11.3.00	Time: 15:35	Received By: Monica Gregson 11/3/00	Time: 15:35
Relinquished By:	Date:	Time:	Received By: Date:	Time:
Relinquished By:	Date:	Time:	Received By: Date:	Time:

Were Samples Received in Good Condition? ☐ Yes ☐ No Samples on Ice? ☐ Yes ☐ No Method of Shipment:

Page 1 of 1

White - Sequoia Yellow - Sequoia Pink - Client

Appendix G

Appendix G

Information on Acid Attack of Concrete and Simplified Thermal Study of Concrete Seal

G.1 Review of Concrete Mixture Data Versus Published Data on Acid Attack Of Concrete from the IMM

G.2 Thermal study calculations by G.R. Mass, dated November 8, 2000.

G.3 American Concrete Institute Publication No. 207.2R-95, "Effect of Restraint, Volume Change, and Reinforcement on Cracking of Mass Concrete"

Appendix G.1

Review of Concrete Mixture Data Versus Published Data on Acid Attack Of Concrete from the IMM

Review of Concrete Mixture Data Versus Published Data on Acid Attack Of Concrete from the IMM

The concrete mixture and placement data from the Walker Mine Seal - Final Construction As-Built Report, by SRK dated March 1989, was analyzed and compared with the results of the Iron Mountain Mine (IMM) studies involving field performance of concrete in an AMD environment. A summary of the field studies for IMM is presented in the paper by Connell et al (2000). The purpose of this comparison is to provide a reasonable estimate of the rate of acid attack and depth of affected concrete, at present, and to provide a prediction of expected depth of attack at the end of the 100 year design life of the Walker Mine seal.

Characterization of Concrete in the Seal. The concrete mixture design for the Walker Mine seal was prepared and presented by Engineering Testing Associates, Inc. of Sparks, Nevada in their report dated October 14, 1987. The mixture and sources of materials were as follows:

	Pounds per Cubic Yard
Portland Cement, ASTM C 150, Type II Low Alkali – Calaveras	450
Pozzolan, ASTM C 618, Class N – Lassenite	150
Silica Fume, FORCE 10,000 – W.R. Grace	49.5
Water	292
Fine Aggregate, ASTM C 33 – Teichert Aggregates (Plant MA1)*	1476 (S.S.D.)
Coarse Aggregate. ASTM C 33, Size No. 67 (limestone) - Sierra Aggregates	1546 (S.S.D.)
High-range Water-reducer, ASTM C 494, Type G, Daracem 100 – W.R. Grace	(108 oz.)
Total Weight	3963.5

Design compressive strength, 28 days – 3000 psi
Design slump, after addition of Daracem 100 – 7 inches
Water-to-cementitious materials ratio – 0.45, by wt.

*Original proposed source of fine aggregate was from Spanish Creek but the fineness modulus was too high for concrete to be pumped and the source was changed.

On November 13, 1987 the concrete seal was placed. Prior to the start, a moisture sample of the fine aggregate was taken, and batch weights of the fine aggregate and the water were adjusted for a free moisture content of 5.5 percent. Concrete was pumped approximately 2650 feet from the portal area to the seal location. A total of ten – 8 cubic yard batches (80 cubic yards) were prepared using an onsite batching plant. Each 8 cubic yard batch was truck-mixed and then discharged into a concrete pump for pumping to the seal location. According to the construction reports, approximately 13 cubic yards of concrete were required to fill the 5-inch diameter pump line before concrete began to flow into the placement area. Pumping began at 10:30 a.m. and was completed about 4:30 p.m. Only 6 cubic yards of the ninth batch and none of the tenth batch were discharged into the pump. The estimated volume of concrete actually placed in the roughly 9 foot wide by 12 foot high by 15 foot long seal was 66.2 cubic yards.

The final construction report mentioned that an excess of water was flowing into the placement area when pumping started but that this water was displaced as concrete was placed in the seal. There has been no indication that this condition has affected the quality of the seal or the concrete/rock contact in the invert.

Water content of each batch was varied to maintain as consistent a slump of concrete entering the pump as possible. Except for the partly used ninth batch, the water content of concrete was less than the mixture design water content (292 pcy). Water content and water-to-cementitious materials ratio for the batches was as follows:

Batch No.	Water, pounds per cubic yard	w/cm
1	250	0.39
2	250	0.39
3	260	0.40
4	271	0.42
5	271	0.42
6	271	0.42
7	271	0.42
8	271	0.42
9	307	0.47
Average	269	0.41

During the placement, concrete samples were taken at the pump and at the discharge end of the pump line. The average slump at the pump was 6.6 inches and average slump at the seal was 4.4 inches. Ambient temperature at the seal was a stable 47 F and average concrete temperature at placement was 54 F. At approximately mid-point of the third batch, a set of cylinders was cast at the seal. The cylinders were later tested with the following results:

Compressive Strength	
3 day	1390 psi
7 day	2620 psi
28 day	5550 psi (average of two specimens)

Exposure Conditions.

During the concrete plug (seal) design phase and more recently during these investigations, AMD water samples were tested. It is well recognized that acid waters, when in direct contact with concrete, attack portland cement by dissolving and removing part of the cement constituents, leaving behind a soft, mushy mass. The rate of attack has been reported to depend, in part, on the acidity of the water's pH (Lea, 1971). A summary of the pH test results on the various water samples is as follows:

Summary of pH Test Results

Location	June 1978	RWQCB 06/30/2000	GEI Consultants 11/01/2000(Field)	GEI Consultants 11/03/2000(Laboratory)
Portal(1)	4.1			
Seal seep(2)		4.0	4.17	3.82
Pool(3)		3.7	4.80	4.55
Drainpipe(4)			4.53	4.51
Upstream(5)			4.57	4.58

Notes: (1) Mine drainage water at portal for access adit at 700 level.
(2) Water seeping from the rock/concrete interface at the top of the seal.
(3) Water from the pond at the toe of the seal.
(4) Water obtained from inside the seal drainpipe.
(5) Water obtained from the seal drainpipe after at least two pipe volumes were released.

It can be assumed that AMD water began to pond against the upstream base of the seal while concrete placement was completed. Recent water quality tests would suggest that the pH of this water was about 4.6. The actual time for the AMD reservoir to reach full-height of the seal is not known. Contact grouting of the uppermost crown of the seal was performed on November 23, 1987, ten days after concrete placement.

Since installation of the seal some seepage has developed. The most significant seepage appears to be at the concrete/rock contact in the top and center of the seal. However, staining indicates some seepage along both sidewalls. Water quality samples of the uppermost seepage show an average pH value of about 4.0. In addition to the seepage, a permanent, approximately 18-inch deep pond has formed in the low area at the downstream toe of the seal. The pH of pond water appears to vary from 3.7 to 4.8.

Based on the above, pH values for analysis of the plug can be taken as 4.6 for waters against the upstream face of the seal and 4.0 for seepage. Other than informational value, the effect of the pond at the downstream toe can be ignored since it will have no impact on the long-term performance of seal.

Iron Mountain Mine Studies

In late December of 1998, a series of concrete mixtures were batched and mixed at a local concrete plant in Redding, California. The objective of this work was to investigate the performance of various types of concrete when exposed to AMD water for an extended period of time.

A total of seven different mixtures were prepared in 2 cubic yard batches. After thorough mixing this concrete was tested for fresh concrete properties and a total of twenty – 6-inch diameter by 12-inch high cylindrical concrete specimens were cast. Twelve of these specimens were cured under standard ASTM C 39 conditions and were tested for compressive strength at ages of 7, 28, 56, 90, 180, and 302 days; two specimens were tested at each age. The remaining eight specimens were standard cured for 14 days; coated with curing compound and air-cured for 46 days; and were immersed in water for 48 hours, surface-dried, and weighed. These specimens were then taken to the Iron Mountain Mine area and immersed in acidic waters. Four specimens were immersed in AMD water with a pH of approximately 2.4 and other four specimens were immersed in a small stream of surface drainage with a pH of approximately 3.1. Two of the specimens from each immersion site were removed at 2-month intervals, brushed, surface-dried and weighed. The remaining two specimens at each immersion site were left in the acidic water for a period of 6 months and 8 months, respectively. At the end of each period, the remaining specimens were brushed, surface-dried, and weighed. Immersion testing was started on March 1, 1999 and was completed on October 22, 1999.

The mixture most resembling the concrete mixture used in the Walker Mine seal was identified as SCRR-3. This mixture and the results of testing are as follows:

Materials	Mixture Design Weights (SSD), lbs/yd^3
Portland cement, Type II Low Alkali	451
Pozzolan, fly ash Class F	113
Water	203
Fine aggregate, natural sand	1078
Coarse aggregate, river gravel(1-1/2 in. NMSA)	2131
Water-reducing admixture, Type D	23 ounces
High-range water-reducing admixture, Type F	79 ounces
Air-entraining admixture	11 ounces
Total weight	3976

Water-cementitious materials ratio, by wt.	0.36

Fresh Concrete Test Results

Slump, in.	2.0
Air content, %	4.0
Unit weight, pcf	147.0
Concrete temperature, F	58
Air temperature, F	50

Age, days	Compressive Strength, psi
7	2985
28	3555
56	4240
90	4615
180	5120
302	5525

Immersion Testing, months	~pH 2.4, wt. loss, %	~pH 3.1, wt. loss, %
Brushed Samples		
0	0	0
2	1.53*	0.30*
4	2.82*	0.65*
6	3.85*	0.89*
Un-brushed Sample		
6	3.48	0.52
8	4.15	0.62

*Adjusted to 2 month immersion values equal zero due to lack of complete saturation on initial weight

Based on the immersion test results and the relationship of volume-to-surface area and weight loss-to-initial weight of the cylindrical specimens, the rate of AMD attack in average depth of surface loss can be calculated. The results of this calculation are as follows:

Immersion Testing, months	~pH 2.4 Avg. Surface loss, in.	~pH 3.1 Avg. Surface loss, in.
Brushed Samples (2-month intervals)		
6	0.0471	0.0157
Un-brushed Sample		
6	0.0417	0.0063
8	0.0498	0.0075

It is apparent from the immersion testing results that un-brushed concrete, when exposed to AMD waters, forms a reaction layer that retards the progress of acid attack. This condition was visually observed on the immersion test specimens. Since there is no abrasive action in the movement of AMD water through the Walker Mine seal, the un-brushed condition can be considered more representative of the rate of attack. Accordingly, the 8-month values for rate of attack for un-brushed specimens can be used. A conversion of this value to rate of attack per year is plotted on Figure G.1. Using a semi-logarithmic scale and assuming a straight-line relationship, the curve can be projected to pH values of 4.0 and 4.6 to provide an estimate of the annual rate of attack of concrete in the Walker Mine seal. The corresponding average annual rate of attack is 0.001 in/yr for a pH of 4.0 and 0.00021 for a pH of 4.6.

A comparison can be made between the IMM results, as presented herein, and the visual examination of the Walker Mine seal. From Figure G.1 an average value of 0.001 in/yr, for a pH of 4.0, can be taken as the rate of attack of the seepage water on the downstream face of the seal. This equates to approximately 0.33 mm of surface softening in 13 years of service. Visual observations during inspection of the seal in November of 2000 indicated that the maximum depth of surface softening was on the order of 1 to 2 mm. The difference in these values may be attributed to one or more of the following:

- Exposure of the Walker Mine concrete seal to AMD water at a very early age when attack is much more severe
- Gradual increase in pH values since installation of the seal, wherein attack may have been more severe in early years
- Seasonal variation of pH
- Slightly higher water-cementitious materials ratio of the Walker Mine concrete
- Accuracy of visual estimate of surface softening during inspection
- Accuracy of the predicted rate of surface softening
- Surface softening is assumed to be uniform; however in actuality, as aggregate is exposed the surface area of the paste fraction is reduced

As seen from the IMM studies, the pH of the AMD water has a significant effect on the rate of attack on concrete. The higher pH values of the Walker Mine AMD waters suggests that the rate of attack on the concrete seal will be rather slow as indicated by current performance. At a design life of 100 years, the estimated maximum softening of concrete exposed to AMD waters is on the order of 0.1 inches based on IMM testing. If a value of 1.5 mm in 13 years is used, this softening could reach a depth of 11.5 mm, or less than ½ inch in 100 years. Under either of these conditions, a marked increase in seepage would occur even though structural integrity of the seal would remain intact. Therefore, close monitoring of seepage and seepage rate should continue as an indicator of concrete condition and performance.

Reference

Lea, F.M., 1971, "The Chemistry of Cement and Concrete," 3rd Ed., Chemical Publishing company, Inc.

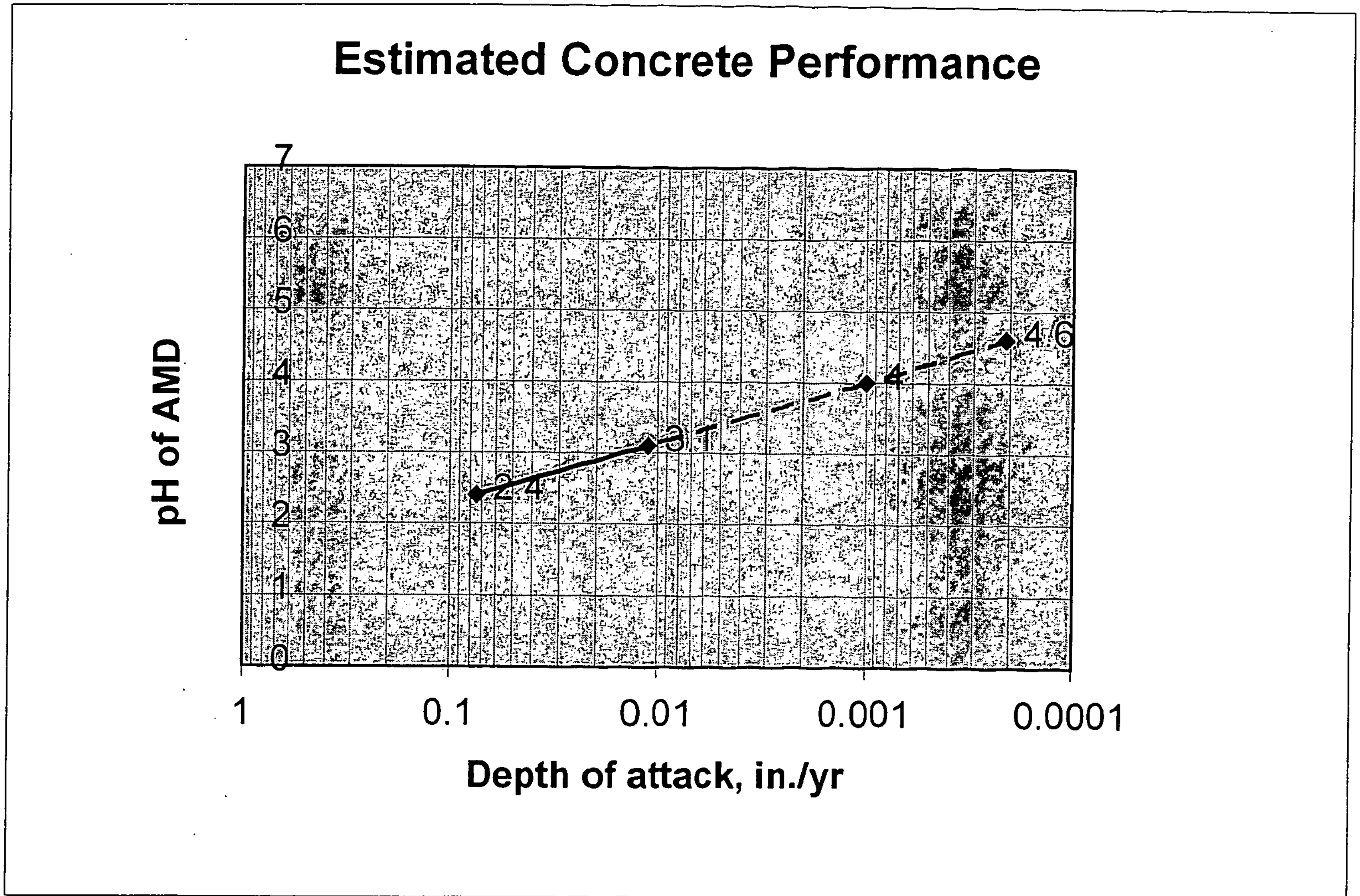
Estimated Concrete Performance
pH of AMD
Depth of attack, in./yr
0
1
2
3
4
5
6
7
1
0.1
0.01
0.001
0.0001
2.4
3.1
4
4.6

Figure G.1

Appendix G.2

Thermal study calculations by G.R. Mass, dated November 8, 2000.

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Littleton, CO 80128
(303) 972-9748
FAX (303) 972-3083
E-mail: grmass@aol.com

Concrete Engineer & Consultant

Memorandum

To: Alberto Pujol

From: Gary R. Mass

CC:

Date: 12/12/00

Re: Walker Mine Thermal Analysis

Transmitted herewith is a copy of my basic thermal analysis and calculations made in accordance with ACI 207.2R as you requested in your e-mail of 12 December 2000.

REGISTERED PROFESSIONAL ENGINEER
GARY R. MASS
No. 18009
Exp. 6/30/2005
CIVIL
STATE OF CALIFORNIA

Gary R. Mass

Gary R. Mass, P.E. Consulting Littleton, CO	SUBJECT Thermal Study Concrete Plug COMPUTED ____ CHECKED ____	PROJECT Walker Mine FILE NO. ____ DATE 11/8/00 PAGE 1 OF 3 PAGES

Ref ACI 207.2R-95

Construction Data:

Concrete Mix

Portland cement, Type II	450 pcy	
Pozzolan, Class N	150 pcy	(23%)
Silica fume	50 pcy	(7.7%)
Total	650 pcy	

Average placement temperature - 54°F

Average annual ambient temperature - 47°F

Concrete volume placed - 66.2 cy

Compressive Strength

3 day	1390 psi
7 day	2620 psi
28 day	5550 psi

Assumed average dimensions of plug

H = 12.0 ft
W = 10.0 ft
T = 15.0 ft

Vol = 66.6 cy

Calculations

$$V = 12 \times 10 \times 15 = 1800\ ft^3$$

$$S = 2(12 \times 10) = 240\ ft^2$$

$$V/S = 1800/240 = 7.5\ ft$$

From Fig. 2.4 concrete temperature peak @ 5 days

From Fig. 2.6 heat dissipated = 33%

Net effective placing temperature = 54 − (54−47)(0.33) = 52°F

Minimum exposure temperature = 47°F

From Fig. 2.5 w/effective placing temp @ 52 = 34°F Rise

From Fig. 2.1 correction for Type II cement peaking @ 5 days

$T_e = 34\left(50/60\right) \cong 28°F$

Correction for mix

Assume silica same as Type II cement

$C_{eq} = 450 + 50 + (150/4) = 538$ pcy

$T_{eff} = 28\left(538/376\right) = 40°F$

Temperature of concrete @ 5 days = 52°F + 40°F = 92°F (Peak)

Temperature strain on cooling to 47°F

Use thermal coefficient expansion/contraction = 5.5×10^{-6} in/in

Strain = $(92-47)(5.5 \times 10^{-6}) = 248 \times 10^{-6}$ in./in.

or total strain in plug

$(15 \times 12)(248 \times 10^{-6}$ in./in.$) = 0.045$ in.

Assume crack width = 0.016

Number of cracks = 0.045/0.016 = 2.8

Gary R. Mass, P.E. Consulting Littleton, CO	SUBJECT Thermal Study Concrete Plug	PROJECT Walker Mine
	COMPUTED ______ CHECKED ______	FILE NO. ______ DATE 11/8/00 PAGE 3 OF 3 PAGES

Thermal Stress

Use equation (4.3)

$f_t = K_R \Delta_c E_c$

Assume 100% base restraint, $K_R = 1.0$

$\Delta_c = (45 \times 5.5 \times 10^{-6}) = 248 \times 10^{-6}$ in./in.

$E_c = 57{,}000\sqrt{5550} = 4.25 \times 10^{6}$ psi

$f_t' = (1.0)(248 \times 10^{-6})(4.25 \times 10^{6}) = 1054$ psi

Allowable tensile strength

$f_t' = 6.7\sqrt{5550} = 499$ psi < 1054 psi

Cracking will occur

Appendix G.3

American Concrete Institute Publication No. 207.2R-95, "Effect of Restraint, Volume Change, and Reinforcement on Cracking of Mass Concrete"

ACI 207.2R-95

Effect of Restraint, Volume Change, and Reinforcement on Cracking of Mass Concrete

Reported by ACI Committee 207

James L. Cope
Chairman

Robert W. Cannon*
Vice Chairman

Edward A. Abdun-Nur
Fred A. Anderson
Howard L. Boggs
Dan A. Bonikowsky
Richard A. Bradshaw, Jr.
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Luis H. Diaz
Timothy P. Dolen
Kenneth D. Hansen
Gary R. Mass*
Alfred T. McCarthy
James E. Oliverson
Robert F. Oury
Jerome M. Raphael[‡]
Ernest K. Schrader
Stephen B. Tatro*
Terry L. West

*Members of the task group who prepared this report.
[†]Chairman of the task group who prepared the report.
[‡]Deceased.

Members of the committee voting on proposed revisions:

John M. Scanlon
Chairman

Howard L. Boggs
Dan A. Bonikowsky
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Robert W. Cannon
Ahmed F. Chraibi
James L. Cope
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Timothy P. Dolen
Michael I. Hammons
Kenneth D. Hansen
Allen J. Hulshizer
Meng K. Lee*
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Robert F. Oury
Ernest K. Schrader
Glenn S. Tarbox
Stephen B. Tatro
Terry L. West

*Chairman, 207.2R Task Group.

This report presents a discussion of the effects of heat generation and volume change on the design and behavior of reinforced mass concrete elements and structures. Particular emphasis is placed on the effects of restraint on cracking and the effects of controlled placing temperatures, concrete strength requirements, and type and fineness of cement on volume change. Formulas are presented for determining the amounts of reinforcing steel needed to control the size and spacing of cracks to specified limits under varying conditions of restraint and volume change.

Keywords: adiabatic conditions; age; cement types; concrete dams; concrete slabs; cooling; cracking (fracturing); crack propagation; crack width and spacing; creep properties; drying shrinkage; foundations; heat of hydration; heat transfer; machine bases; mass concrete; modulus of elasticity; moisture content; placing; portland cement physical properties; portland cements; pozzolans; reinforced concrete; reinforcing steels; restraints; shrinkage; stresses; structural design; temperature; temperature rise (in concrete); tensile strength; thermal expansion; volume change; walls.

CONTENTS

ACI 207.2R-95 supersedes ACI 207.2R-90 and became effective January 1, 1995.

The 1995 revisions consisted of many minor editorial and typographical corrections throughout, as well as some additional explanatory information.

CHAPTER 1—INTRODUCTION

1.1—Scope

This report is primarily concerned with limiting the width of cracks in structural members that occur principally from restraint of thermal contraction. A detailed discussion of the effects of heat generation and volume changes on the design and behavior of mass reinforced concrete elements and structures is presented. It is written primarily to provide guidance for the selection of concrete materials, mix requirements, reinforcement requirements, and construction procedures necessary to control the size and spacing of cracks. Particular emphasis is placed on the effect of restraint to volume change in both preventing and causing cracking and the need for controlling peak concrete temperature. The quality of concrete for resistance to weathering is not emphasized in recommending reduced cements contents; however, it should be understood that the concrete should be sufficiently durable to resist expected service conditions. The report can be applied to any concrete structure with a potential for unacceptable cracking; however, its general application is to massive concrete members 18 in. or more in thickness.

1.2—Definition

Mass concrete is defined in ACI 116R as: "Any volume of concrete with dimensions large enough to require that measures be taken to cope with the generation of heat and attendant volume change to minimize cracking." Reinforced mass concrete in this report refers to concrete in which reinforcement is utilized to limit crack widths that may be caused by external forces or by volume change due to thermal changes, autogenous changes and drying shrinkage.

1.3—Approaches to control of cracking

All concrete elements and structures are subject to volume change in varying degrees, dependent upon the makeup, configuration, and environment of the concrete. Uniform volume change will not produce cracking if the element or structure is relatively free to change volume in all directions. This is rarely the case for massive concrete members since size alone usually causes nonuniform change and there is often sufficient restraint either internally or externally to produce cracking.

The measures used to control cracking depend to a large extent on the economics of the situation and the seriousness of cracking if not controlled. Cracks are objectionable where their size and spacing compromise the appearance, serviceability, function, or strength of the structure.

While cracks should be controlled to the minimum practicable width in all structures, the economics of achieving this goal must be considered. The change in volume can be minimized by such measures as reducing cement content, replacing part of the cement with pozzolans, precooling, postcooling, insulating to control the rate of heat absorbed or lost, and by other temperature control measures outlined in ACI 207.1R and ACI 207.4R. Restraint is modified by joints intended to handle contraction or expansion and also by the rate at which volume change takes place. Construction joints may also be used to reduce the number of uncontrolled cracks that may otherwise be expected. By appropriate consideration of the preceding measures, it is usually possible to control cracking or at least to minimize the crack widths. The subject of crack control in mass concrete is also discussed in Chapter 7 of ACI 224R and in Reference 1. The topic of evaluation and repair of cracks in concrete is covered in detail in ACI 224.1R.

In the design of reinforced concrete structures, cracking is presumed in the proportioning of reinforcement. For this reason, the designer does not normally distinguish between tension cracks due to volume change and those due to flexure. Instead of employing many of the previously recommended measures to control volume change, the designer may choose to add sufficient reinforcement to distribute the cracking so that one large crack is replaced by many smaller cracks of acceptably small widths. The selection of the necessary amount and spacing of reinforcement to accomplish this depends on the extent of the volume change to be expected, the spacing or number of cracks which would occur without the reinforcement, and the ability of reinforcement to distribute cracks.

CHAPTER 1—INTRODUCTION

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The degree to which the designer will either reduce volume changes or use reinforcement for control of cracks in a given structure depends largely on the massiveness of the structure itself and on the magnitude of forces restraining volume change. No clear-cut line can be drawn to establish the extent to which measures should be taken to control the change in volume. Design strength requirements, placing restrictions, and the environment itself are sometimes so severe that it is impractical to prevent cracking by measures to minimize volume change. On the other hand, the designer normally has a wide range of choices when selecting design strengths and structural dimensions.

In many cases, the cost of increased structural dimensions required by the selection of lower strength concrete (within the limits of durability requirements) is more than repaid by the savings in reinforcing steel, reduced placing costs, and the savings in material cost of the concrete itself (see Section 6.5, Example 6.1.).

CHAPTER 2—VOLUME CHANGE

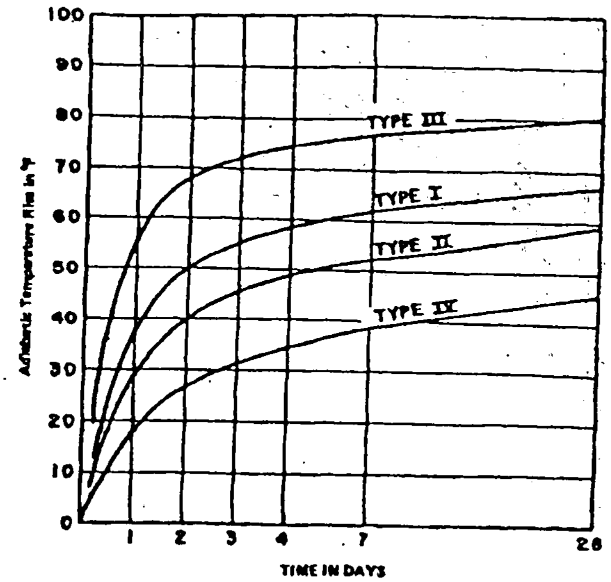
The thermal behavior of mass concrete has been thoroughly discussed in Chapter 5 of ACI 207.1R. This chapter's purpose is to offer some practical guidance in the magnitude of volume change that can be expected in reinforced concrete structures or elements. Such structures utilize cements with higher heat generation, smaller aggregate, more water, and less temperature control than normally used or recommended for mass concrete in dams.

In reinforced concrete elements, the primary concern is with these volume changes resulting from thermal and moisture changes. Other volume changes, which are not considered in this document, are alkali-aggregate expansion, autogenous shrinkage, and changes due to expansive cement. Autogenous shrinkage is the volume change due to the chemical process that occurs during hydration.

The change in temperature to be considered in the design of reinforced concrete elements is the difference between the peak temperature of the concrete attained during early hydration (normally within the first week following placement) and the minimum temperature to which the element will be subjected under service conditions. The initial hydration temperature rise produces little, if any, stress in the concrete. At this early age, the modulus of elasticity of concrete is so small that compressive stresses induced by the rise in temperature are insignificant even in zones of full restraint and, in addition, are relaxed by a high rate of early creep. By assuming a condition of no initial stress, a slightly conservative and realistic analysis results.

2.1—Heat generation

The rate and magnitude of heat generation of the concrete depends on the amount per unit volume of cement and pozzolan (if any), the compound composition and fineness of cement, and on the temperature during hydration of the cement. The hydration temperature is affected in turn by the amount of heat lost or gained as governed by the size of the member and exposure conditions. Thus, it can be seen that the exact temperature of the concrete at any given time depends on many variables.



Cement Type	Fineness ASTM C 115 cm^2/gm	28-Day Heat of Hydration Calories per gm
I	1790	87
II	1890	76
III	2030	105
IV	1910	60

Fig. 2.1—Temperature rise of mass concrete containing 376 lb of various types of cement per cubic yard of concrete

Fig. 2.1 shows curves for adiabatic temperature rise versus time for mass concrete placed at 73 F and containing 376 lb/yd^3 of various types of cement. These curves are typical of cements produced prior to 1960. The same cement types today may vary widely from those because of increased fineness and strengths. Current ASTM specifications only limit the heat of hydration directly of Type IV cements or of Type II cements if the purchaser specifically requests heat-of-hydration tests. Heat-of-hydration tests present a fairly accurate picture of the total heat-generating characteristics of cements at 28 days because of the relative insensitivity with age of the total heat generating capacity of cement at temperatures above 70 F. At early ages, however, cement is highly sensitive to temperature and therefore heat-of-solution tests, which are performed under relatively constant temperatures, do not reflect the early-age adiabatic temperature rise. The use of an isothermal calorimeter for measuring heat of hydration can provide data on the rate of heat output at early ages.[2] More accurate results for a specific cement, mix proportions, aggregate initial placing temperature, and a set of environmental conditions can be determined by adiabatic temperature-rise tests carefully performed in the laboratory under conditions that represent those that will occur in the field.

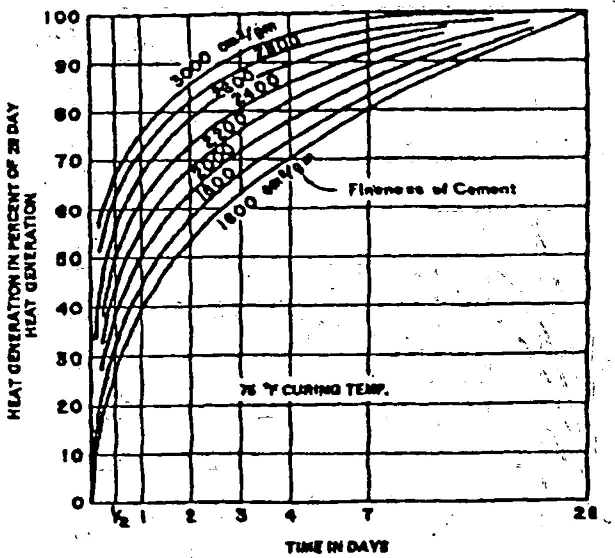


Fig. 2.2—Rate of heat generation as affected by Wagner fineness of cement (ASTM C 115) for cement paste cured at 75 F

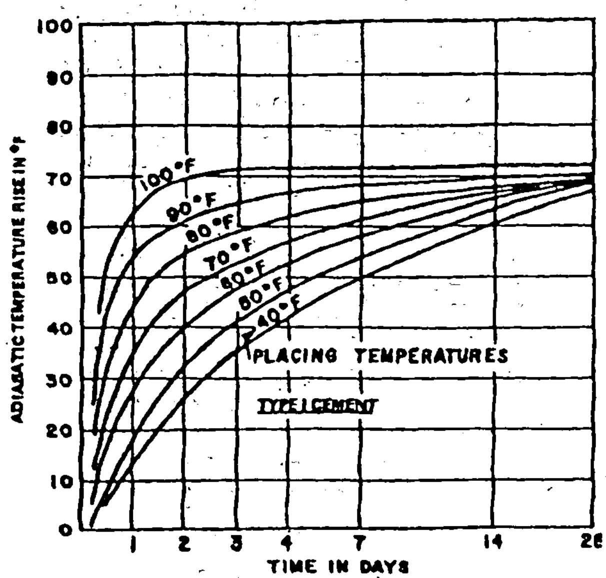


Fig. 2.3—Effect of placing temperature and time on adiabatic temperature rise of mass concrete containing 376 lb/yd³ of Type I cement

The fineness of cement affects the rate of heat generation more than it affects the total heat generation, in much the same fashion as placing temperature. The rate of heat generation as effected by cement fineness and placing temperature is shown in Fig. 2.2 and 2.3, respectively. These two figures are based on extrapolation of data from a study of the heats of hydration of cements by Verbeck and Foster.[3]

There are no maximum limitations on cement fineness in current specifications. By varying both fineness and chemical composition of the various types of cement, it is possible to vary widely the rate and total adiabatic temperature rise of the typical types shown in Fig. 2.1. It is therefore essential that both the fineness and chemical composition of the cement in question be considered in estimating the temperature rise of massive concrete members.

For a given fineness, the chemical composition of cement has a relatively constant effect on the generation of heat beyond the first 24 hr. As shown in Fig. 2.1, the concrete temperature rise for all four cement types is similar between 1 and 28 days. The 28-day adiabatic temperature rise in degrees F may be calculated by

$$H_a = \frac{1.8 h_g \bar{w}_c}{0.22\,(150)\,(27)} \tag{2.1}$$

Where 0.22 in cal/gm-deg C and 150 in lb/ft³ are the specific heat and density, respectively, of the concrete. 1.8 is the conversion factor from Celsius to Fahrenheit, 27 is the conversion factor from yd^3 to ft^3. hg in cal/gm is the 28-day measured heat generation of the cement by heat of hydration as per ASTM C 186, and $\bar{w}_c$ is the weight of cement in lb per yd^3 of concrete. For a concrete mix containing 376 lb of cement per yd^3 of concrete: $H_a = 0.76$ in degrees Fahrenheit.

For low and medium cement contents, the total quantity of heat generated at any age is directly proportional to the quantity of cement in the concrete mix.

However, for high cement-content structural mixtures, the amount of cement may be sufficiently high to increase the very early age heat to a point where the elevated temperature in turn causes a more rapid rate of heat generation. When fly ash or other pozzolans used, the total quantity of heat generated is directly proportional to an equivalent cement content C_{eq}, which is the total quantity of cement plus a percentage to total pozzolan content. The contribution of pozzolans to heat generation as equivalent cement varies with age of concrete, type of pozzolan, the fineness of the pozzolan compared to the cement and pozzolan themselves. It is best determined by testing the combined portions of pozzolan and cement for fineness and heat of hydration and treating the blend in the same fashion as a type of cement.

In general, the relative contribution of the pozzolan to heat generation increases with age of concrete, fineness of pozzolan compared to cement, and with lower heat-generating cements. The early-age heat contribution of fly ash may conservatively be estimated to range between 15 and 35 percent of the heat contribution from same weight of cement. Generally, the low percentages correspond to combined finenesses of fly ash and cement as low as two-thirds to three-fourths that of the cement alone, while the higher percentages correspond to fineness equal to or greater than the cement alone.

The rate of heat generation as affected by initial temperature, member size, and environment is difficult to assess because of the complex variables involved. However, for large concrete members, it is advisable to compute their temperature history, taking into account the measured values of heat generation, concrete placement temperatures, and ambient

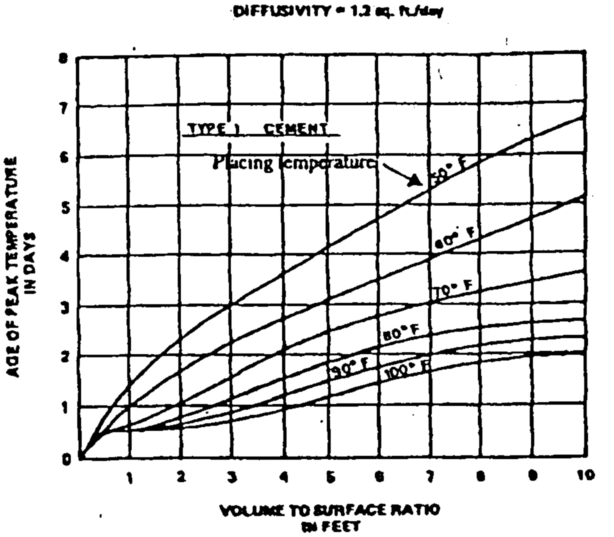


Fig. 2.4—Effect of placing temperature and surface exposure on age at peak temperature for Type I cement in concrete. Air temperature = placing temperature

temperature. The problem may be simplified somewhat if we assume that the placing temperature and ambient air temperature are identical. We can then make a correction for the actual difference, considering the size or volume-to-exposed surface ratio (*V/S*) of the member in question. The *V/S* ratio actually represents the average distance through which heat is dissipated from the concrete.

Usually, peak concrete temperatures for concrete structures may occur at any time during the first week. Fig. 2.4 shows the effect of placing temperature and member *V/S* on the age at which peak concrete temperatures occur for concrete containing Type I cement. Time would be shortened or lengthened for cements of higher or lower heat-generating characteristics.

For comparative purposes, the early-age heat generation of a Type III cement is approximately equivalent to a Type I cement at a 20 F higher placing temperature. In a similar fashion, the heat-generating characteristic of Types II and IV cement correspond closely to that of Type I cement at 10 and 20 F lower placing temperatures, respectively. Fig. 2.4 shows that for *V/S* less than 3 ft, peak temperature will be reached within 1 day under normal placing temperature (80 F or higher).

Fig. 2.5 gives the approximate maximum temperature rise for concrete members containing 4 bags (376 lb) of Type I cement per yd^3 for placing temperatures ranging from 50 to 100 F, assuming ambient air temperatures equal to placing temperatures. Corrections are required for different types and quantities of cementitious materials. A correction for the difference in air and placing temperatures can be made using Fig. 2.6 by estimating the time of peak temperatures from Fig. 2.4. The effect of water-reducing, set-retarding agents on the temperature rise of concrete is usually confined to the first 12 to 16 hr after mixing, during which time these agents

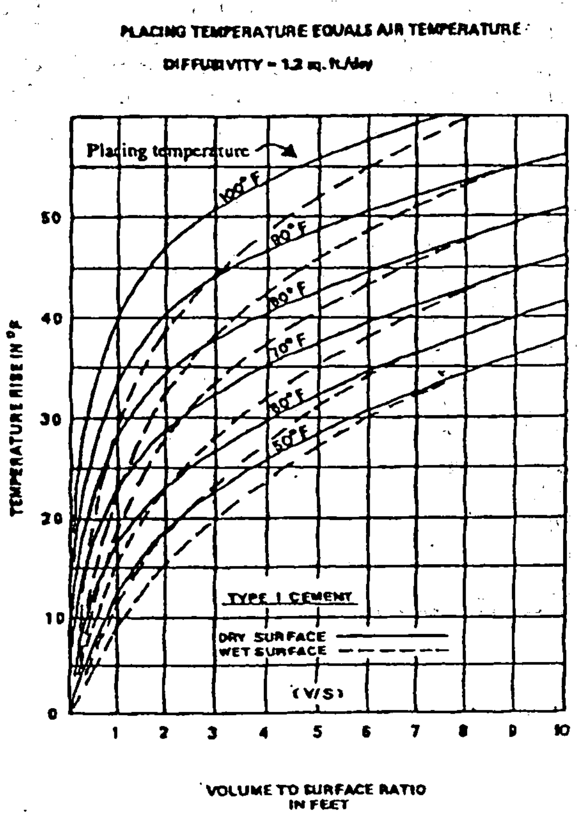


Fig. 2.5—Temperature rise of concrete members containing 376 lbs of cement per cubic yard for different placing temperatures

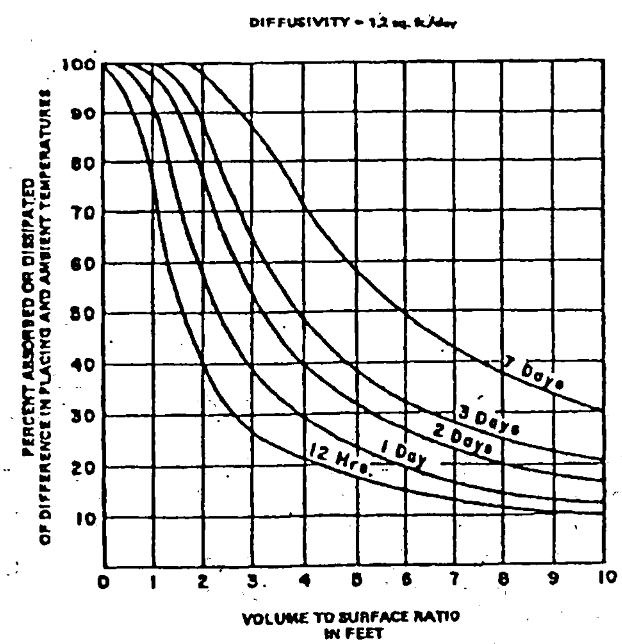


Fig. 2.6—Heat flow between air and concrete for difference between placing temperature and ambient air temperature

have the greatest effect on the chemical reaction. Their presence does not alter appreciably the total heat generated in the concrete after the first 24 hr and no corrections are applied herein for the use of these agents.

A diffusivity of 1.2 ft^2/day has been assumed in the preparation of Fig. 2.4 through 2.6. A concrete of higher or lower diffusivity will, respectively, decrease or increase the volume-to-exposed surface ratio, and can be accounted for by multiplying the actual V/S by 1.2 divided by the actual concrete diffusivity.

2.2—Moisture contents and drying shrinkage

For tensile stress considerations, the volume change resulting from drying shrinkage is similar to volume change from temperature except that the loss of moisture from hardened concrete is extremely slow compared with the loss of heat. Drying shrinkage therefore depends on the length of moisture migration path and often affects the concrete near a surface. When the length of moisture migration or V/S is small, drying shrinkage adds to the stresses induced by external restraint and should be considered in the design of the reinforcement. When the V/S is large, the restraint to drying shrinkage is entirely internal and the result is tension on the surface or an extensive pattern of surface cracks extending only a short distance into the concrete. When surface cracks of this nature do occur, they are small and reinforcement is not particularly effective in altering the size or spacing of these cracks. Reinforcement is also not a solution for surface cracks in fresh concrete which are referred to as plastic cracking (see ACI 116R).

A 24 in. thick slab will lose approximately 30 percent of its evaporable water in 24 months of continuous exposure with both faces exposed to 50 percent relative humidity.[4] If we assume a total drying shrinkage potential at the exposed faces of 300 millionths, then the average drying shrinkage for a 24 in. slab under this exposure would be 90 millionths in 24 months. Concrete is not usually exposed to drying conditions this severe.

Drying shrinkage is affected by the size and type of aggregate used. "In general, concretes low in shrinkage often contain quartz, limestone, dolomite, granite, or feldspar, whereas those high in shrinkage often contain sandstone, slate, basalt, trap rock, or other aggregates which shrink considerably of themselves or have low rigidity to the compressive stresses developed by the shrinkage of paste."[5] In this discussion, an aggregate low in shrinkage qualities is assumed. Drying shrinkage may vary widely from the values used herein depending on many factors which are discussed in more detail in ACI 224R.

2.2.1 *Equivalent temperature change*—In the design of reinforcement for exterior restraint to volume change, it is more convenient to design only for temperature change rather than for temperature and shrinkage volume changes; therefore, it is desirable to express drying shrinkage in terms of equivalent change in concrete temperature T_{DS}. Creep can be expected to reduce significantly the stresses induced by drying shrinkage because of the long period required for full drying shrinkage to develop. We have therefore assumed an equivalent drying shrinkage of 150 millionths and an expansion coefficient of 5×10^{-6} per deg F as a basis in establishing the following formula for equivalent temperature drop. While the rate of drying and heat dissipation differ, their average path lengths (V/S) are the same. There is, however, a limitation on the length of moisture migration path affecting external restraint and its impact on total volume change. This limit has been assumed as 15 in. maximum in determining equivalent temperature change

$$T_{DS} = \left(30 - \frac{2V}{S}\right)\left(\frac{W_u - 125}{100}\right) \qquad (2.2)$$

where

T_{DS} = equivalent temperature change due to drying shrinkage, in deg F
W_u = water content of fresh concrete, lb/yd^3, but not less than 225 lb/yd^3
V = total volume, in.^{3}
S = area of the exposed surface, in.^{2}

2.3—Ambient, placement, and minimum service temperatures

In many structures, the most important temperature considerations are the average air temperatures during and immediately following the placement of concrete, and the minimum average temperature in the concrete that can be expected during the life of the structure. The temperature rise due to hydration may be small, particularly in thin exposed members, regardless of the type or amount of cement used in the mix, if placing and cooling conditions are right. On the other hand, the same member could have a high temperature rise if placed at high temperature in insulated forms.

2.4—Placement temperature

Specifications usually limit the maximum and minimum placing temperatures of concrete. ACI 305R recommends limiting the initial concrete placement temperature to between 75 and 100 F. The temperature of concrete placed during hot weather may exceed the mean daily ambient air temperature by 5 to 10 F unless measures are taken to cool the concrete or the coarse aggregate. Corrections should be made for the difference in air temperature and placing temperature, using Fig. 2.6. For example, if the temperature of the concrete, when placed, is 60 F during the first 24 hr, a concrete section having a V/S of 2 ft would absorb 60 percent of the difference, or 12 F. The maximum placing temperature in summer should be the highest average summer temperature for a given locality, but not more than 100 F.

Minimum concrete temperature recommendations at placing are given in ACI 306R, Table 3.1. These minimums establish the lowest placing temperature to be considered. Placing temperatures for spring and fall can reasonably be considered to be about halfway between the summer and winter placing temperatures.

2.5—Minimum temperature in service

The minimum expected final temperatures of concrete elements are as varied as their prolonged exposure conditions. Primary concern is for the final or operating exposure conditions, since cracks which may form or open during colder construction conditions may be expected to close during operating conditions, provided steel stresses remain in the elastic range during construction conditions. Minimum concrete temperatures can be conservatively taken as the average minimum exposure temperature occurring during a period of approximately 1 week. The mass temperature of earth or rock against concrete walls or slabs forms a heat source, which affects the average temperature of concrete members, depending upon the cooling path or V/S of the concrete. This heat source can be assumed to effect a constant temperature at some point 8 to 10 ft from the exposed concrete face.

The minimum temperature of concrete against earth or rock mass, T_{min}, can be approximated by

$$T_{min} = T_A + \frac{2(T_M - T_A)}{3}\sqrt{\frac{V/S}{96}} \qquad (2.3)$$

where

- T_A = average minimum ambient air temperature over a prolonged exposure period of one week.
- T_M = temperature of earth or rock mass; approximately 40 to 60 F, depending on climate
- V/S = volume to exposed surface ratio, in.

2.6—Heat dissipation and cooling

Means of determining the dissipation of heat from bodies of mass concrete are discussed in ACI 207.1R and can readily be applied to massive reinforced structures. Reinforced elements or structures do not generally require the same degree of accuracy in determining peak temperatures as unreinforced mass concrete. In unreinforced mass concrete, peak temperatures are determined for the purpose of preventing cracking. In reinforced concrete, cracking is presumed to occur and the consequences of overestimating or underestimating the net temperature rise is usually minor compared to the overall volume change consideration. Sufficient accuracy is normally obtained by use of charts or graphs such as Fig. 2.5 to quickly estimate the net temperature rise for concrete members cooling in a constant temperature environment equal to the placing temperature, and by use of Fig. 2.6 to account for the difference in the actual and assumed cooling environment.

Fig. 2.5 gives the maximum temperature rise for concrete containing 376 lb of Type I portland cement per cubic yard of concrete in terms of V/S of the member. V/S actually represents the average distance through which heat is dissipated from the concrete. This distance will always be less than the minimum distance between faces. In determining the V/S consider only the surface area exposed to air or cast against forms. The insulating effect of formwork must be considered in the calculation of volume of the member. Steel forms are poor insulators; without insulation, they offer little resistance to heat dissipation from the concrete. The thickness of wood forms or insulation in the direction of principal heat flow must be considered in terms of their affecting the rate of heat dissipation (see ACI 306R). Each inch of wood has an equivalent insulating value of about 20 in. of concrete but can, for convenience, be assumed equivalent to 2 ft of additional concrete. Any faces farther apart than 20 times the thickness of the member can be ignored as contributing to heat flow. Therefore, for a long retaining wall, the end surfaces are normally ignored.

The V/S can best be determined by multiplying the calculated volume-to-exposed surface ratio of the member, excluding the insulating effect of forms by the ratio of the minimum flow path including forms divided by the minimum flow path excluding forms. For slabs, V/S should not exceed three-fourths of the slab thickness. While multiple lift slabs are not generally classed as reinforced slabs, V/S should not exceed the height of lift if ample time is provided for cooling lifts.

The temperature rise for other types of cement and for mixes containing differing quantities of cement or cement plus pozzolan from 376 lb can be proportioned as per Section 2.1.

Fig. 2.6 accounts for the difference in placing temperatures and ambient air temperatures. The V/S for Fig. 2.6 should be identical to those used with Fig. 2.5. In all previous temperature determinations the placing temperature has been assumed equal to ambient air temperature. This may not be the case if cooling measures have been taken during the hot-weather period or heating measures have been taken during cold weather. When the placing temperature of concrete is lower than the average ambient air temperature, heat will be absorbed by the concrete and only a proportion of the original temperature difference will be effective in lowering the peak temperature of the concrete. When the placing temperature is higher, the opposite effect is obtained. As an example, assume for an ambient air temperature of 75 F that the placing temperature of a 4 ft thick wall 12 ft high is 60 F instead of 75 F. The V/S would be 3.4 ft, assuming 1 in. wooden forms. The age for peak temperature would be 2.3 days from Fig. 2.4. From Fig. 2.6, 50 percent of the heat difference will be absorbed or 7.5 F; therefore, the base temperature or the effective placing temperature for determining temperature rise will be 68 F. In contrast, if no cooling methods are used, the actual placing temperature of the concrete will be 85 F, the age of peak temperature would be 1 day, and the base temperature or effective placing temperature for determining temperature rise will be 81 F.

2.7—Summary and examples

The maximum effective temperature change constitutes the summation of three basic temperature determinations. They are: (1) the difference between effective placing temperature and the temperature of final or operating exposure conditions, (2) the temperature rise of the concrete due to hydration, and (3) the equivalent temperature change to compensate for drying shrinkage. Measures for making these determinations have been previously discussed; therefore, the following example problems employ most of the calculations required in determining the maximum effective temperature change.

Example 2.1—A 2 ft wide retaining wall with rock base and backfill on one side; 20 ft high by 100 ft long placed in two 10-ft lifts, wood forms; summer placing with concrete cooled to 60 F; concrete mix designed for a specified strength of 3000 psi or average strength of 3700 psi at 90 days contains 215 lb of Type II cement (adiabatic curve same as Fig. 2.1), 225 lb of fly ash, and 235 lbs of water per yd^3. The insulating effect of 1 in. thick wood forms on each face would be to effectively increase the thickness by 2(20)/12 = 3.34 ft (assuming 1 in.-thick wood form is equivalent to 20 in. concrete).

1. Determine the V/S

$$V/S = \left[\frac{2\,(10)}{2\,(10)+2}\right]\left(\frac{2+3.34}{2}\right) = 2.43 \text{ ft}$$

2. Determine the difference between effective placing temperature and final exposure temperature:

a. Establish ambient air temperature for summer placement based on locality. Assume 75 F average temperature.

b. Concrete peaks at 2 days from Fig. 2.4. Using Fig. 2.6, the heat absorbed for $V/S = 2.4$ is approximately 60 percent.

c. Net effective placing temperature $T_{pk} = 60 + 0.6(15) = 69$ F.

d. Establish minimum exposure temperature for 1-week duration. Assume 20 F.

e. For final exposure conditions V/S equals approximately 24 in., since heat flow is restricted to one direction by the backfill. For two faces exposed, V/S would equal approximately 12 in.

f. $T_{min} = 20\text{ F} + \frac{2}{3}\,(60\text{-}20)\,\sqrt{24/96} = 33.5$ F, say 34 F.

g. Difference = 69 − 34 = 35 F.

3. Determine the temperature rise:

a. From Fig. 2.5, the temperature rise for Type I cement for dry surface exposure and an effective placing temperature of 69 F and V/S of 2.4 ft = 30 F.

b. From Fig. 2.1, correction for Type II cement peaking at 2 days = T_c = (40/50)(30) = 24 F.

c. Correction for mix. $C_{eq} = 215 + 225/4 = 272$ lb, T_{C+F} = 24 F (272)/(376) = 17.4 F, say 18 F.

d. Temperature of the concrete at the end of 2 days = 69 + 18 = 87 F.

4. Determine the equivalent temperature for drying shrinkage. Since V/S for final exposure conditions is greater than 15 in., no additional temperature considerations are required for external restraint considerations.

5. The maximum effective temperature change $T_E = 35 + 18 = 53$ F.

Example 2.2—Same wall as Example 2.1, except that no cooling measures were taken and the concrete mix contains 470 lb/yd^3 of a Type I cement, having a turbidimeter fineness of 2000 cm^2/gm and 28-day heat of solution of 94 cal/gm.

1\. a. With no cooling measures the placing temperature could be as much as 10 F above the ambient temperature of 75 F or T_p = 85 F.

b. From Fig. 2.4, the concrete peaks at three-fourths of a day for 85 F placing temperature. From Fig. 2.6, 36 percent of the difference in placing and air temperature is dissipated: 0.36 (85-75) = 4 F.

c. Effective placing temperature = 85 − 4 = 81 F.

d. Minimum temperature of the concrete against rock = 34 F.

e. Difference = 81 − 34 = 47 F.

2\. a. The temperature rise from Fig. 2.5 for dry exposure, V/S of 2.4, and T_p of 81 F is 37 F.

b. Correction for fineness and heat of solution of cement.

From Fig. 2.2, the difference in fineness for 2000 versus 1800 at three-fourths of a day (18 hr) = 45/38 = 1.18.

From Eq. (2.1), the temperature difference due to heat of solution: $H_a = 0.76\,(94 - 87) = 5$ F. Note that 87 cal/gm is the 28-day heat of hydration for Type I cement with a fineness of 1790 as shown in Fig. 2.1. From Fig. 2.1, the adiabatic rise for Type I cement at 18 hr = 30 F.

Combining the preceding two corrections, the adiabatic rise of the cement at 18 hr would be 1.18 (30 + 5) = 41 F.

Temperature rise for 376 lb/yd^3 of cement = 41(37)/30 = 51 F.

c. Correction for cement content = 470(51)/376 = 64 F.

3\. No addition for drying shrinkage.

4\. The peak temperature of the concrete at 18 hr: 81 + 64 = 145 F.

5\. The drop in temperature affecting volume change: 145 − 34 = 111 F.

In comparing the preceding two examples, the effect of mix difference and cooling measures combined for a difference in peak temperature of 145 − 87 = 58 F. This constitutes a volume change in Example 2.2 of about twice (.209 percent) that in Example 2.1 for the same wall.

CHAPTER 3—PROPERTIES

3.1—General

This chapter discusses the principal properties of massive concrete that affect the control of cracking and provides guidance to evaluate those properties.

3.2—Strength requirements

The dimensions of normal structural concrete are usually determined by structural requirements utilizing 28-day strength concrete of 3000 psi or more. When these dimensions are based on normal code stress limitations for concrete, the spacing of cracks will be primarily influenced by flexure, and the resultant steel stresses induced by volume change will normally be small in comparison with flexural stresses. Under these conditions, volume control measures do not have the significance that they have when concrete stresses in the elastic range are low and crack spacing is controlled primarily by volume change.

The dimensions of massive reinforced concrete sections are often set by criteria totally unrelated to the strength of concrete. Such criteria often are based on stability requirements where weight rather than strength is of primary importance; on arbitrary requirements for water-tightness per ft of water pressure; on stiffness requirements for the support of large pieces of vibrating machinery where the mass itself is of primary importance; or on shielding requirements, as found in nuclear power plants. Once these dimensions are established they are then investigated using an assumed concrete strength to determine the reinforcement requirements to sustain the imposed loadings. In slabs, the design is almost always controlled by flexure. In walls, the reinforcement requirements are usually controlled by flexure or by minimum requirements as load-bearing partitions. Shear rarely controls except in the case of cantilevered retaining walls or structural frames involving beams and columns.

In flexure, the strength of massive reinforced sections is controlled almost entirely by the reinforcing steel. The effect of concrete strength on structural capacity is dependent on the quantity of reinforcing steel (steel ratio) and the eccentricity of applied loads. If the eccentricity of the loading with respect to member depth *e/d* is greater than 2, Fig. 3.1 shows the relationship of required concrete strength to structural capacity for steel ratios up to 0.005 using 3000 psi as the base for strength comparison. For steel ratios less than 0.005, there is no significant increase in structural capacity with higher strength concretes within the eccentricity limits of the chart. Most massive concrete walls and slabs will fall within the chart limits.

The principal reason for consideration of the effects of lower concrete strengths concerns the early loading of massive sections and the preeminent need in massive concrete to control the heat of hydration of the concrete. If design loading is not to take place until the concrete is 90 or 180 days old, there is no difficulty using pozzolans in designing low-heat-generating concrete of 3000 psi at those ages. Such concrete may, however, have significantly lower early strengths for sustaining construction loadings and could present a practical scheduling problem, requiring more time prior to form stripping and lift joint surface preparation. Normally, the designer investigates only those construction loads which exceed operational live loads and usually applies a lower load factor for these loads because of their temporary nature. From Fig. 3.1 it can readily be seen that for members subject to pure bending ($e/d = \infty$), less than 13 percent loss of capacity will be experienced in loading a member containing 0.5 percent steel when it has a compressive strength of only 1000 psi. Note that while structural capacity is relatively unaffected by the 1000-psi strength, short-term load and creep deflection will be significantly larger than for 3000-psi concrete. This is usually not significant for construction loadings, particularly since members with this low steel ratio have enough excess depth to offset the increase in deflection due to lower modulus of elasticity.

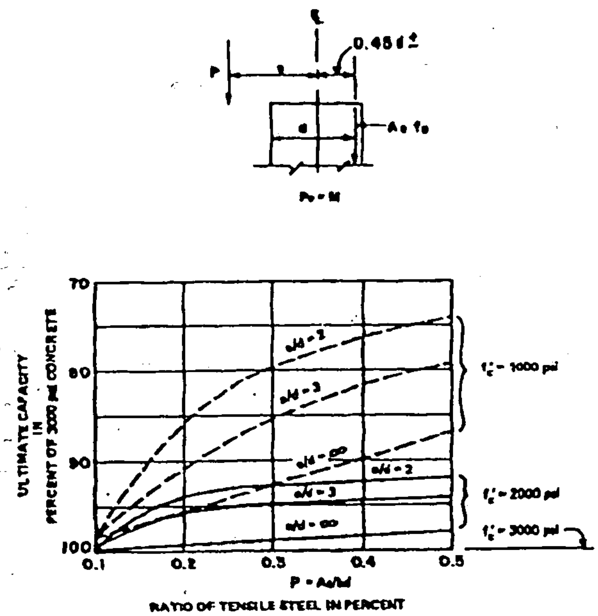


Fig. 3.1—Effect of concrete strength on ultimate capacity; fy = 60,000 psi

Most massive reinforced concrete members subjected to flexural stress will have steel ratios in the range of 0.0015 to 0.002 in the tensile face. Fig. 3.1 shows that in this range, reinforced concrete in flexure is capable of sustaining up to 85 percent of the structural capacity of 3000-psi concrete with concrete strengths as low as 1000 psi. Construction loading rarely controls design. The decrease in load factors normally applied for temporary construction loads will more than account for the 15 percent loss in capacity associated with the lower strength concrete at the time of loading. Therefore, for massive reinforced sections within these limits a simple restriction of limiting imposed flexural loads until the concrete achieves a minimum compressive strength of 1000 psi should be adequate.

From the preceding, it should be obvious that massive reinforced concrete with low reinforcement ratios can tolerate substantially higher percentages of below-strength concrete than can normal structural concrete with high reinforcement ratios. From Fig. 3.1 a minimum strength of 2000 psi results in less than an 8.5 percent loss in ultimate capacity compared with 3000 psi strength.

As previously mentioned, shear strength may control the thickness of a cantilevered retaining wall. The strength of concrete in shear is approximately proportional to $\sqrt{f_c'}$ and, therefore, the loss in shear strength for a given reduction in compressive strength has a greater impact on design than the loss in flexural strength. The design loading for a wall sized on the basis of shear strength is the load of the backfill; rarely will construction schedules allow the lower lifts to attain 90 to 180-day strengths before the backfill must be completed. Since the shear at the base of the wall upon completion of the backfill controls, a design based on 2000 psi will require an approximately 22 percent wider base. For tapered walls, this

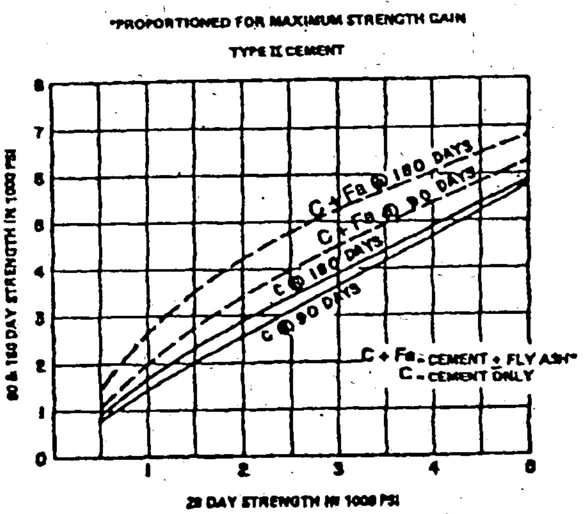


Fig. 3.2—Comparison of 28, 90, and 180-day compressive strength

would mean only an 11 percent increase in total volume. The 22 percent increase in base wall thickness would allow a 30 to 35 percent reduction in flexural reinforcement requirements (using strength design), which would directly offset the cost of the added concrete volume, possibly resulting in a lower overall cost for the wall. By restricting the placing of backfill against any lift until it has obtained a minimum strength of 1000 psi and restricting completion of backfill until the first lift has attained 2000 psi, a reasonable schedule for backfill with respect to concrete construction can be established. A 2000 psi strength requirement at 28 days complies with these types of construction requirements and will provide sufficient strength for durability under most exposure conditions particularly if 90 day strengths exceed 3000 psi.

3.3—Tensile strength

In conventional reinforced concrete design it is assumed that concrete has no tensile strength and a design compressive strength appreciably below average test strength is utilized. Neither approach is acceptable in determining the reinforcing steel requirement for volume-change crack control. The actual tensile strength is one of the most important considerations and should be determined to correspond in time to the critical volume change. Since compressive strength is normally specified, it is desirable to relate tensile and compressive strength.

Tensile strength of the concrete will be affected by the type of aggregates used. A restrained concrete of equal water-cement ratios (w/c) made from crushed coarse aggregate will withstand a larger drop in temperature without cracking than concrete made from rounded coarse aggregate. For a given compressive strength, however, the type of aggregate does not appreciably affect tensile strength. The age at which concrete attains its compressive strength does affect the tensile-compressive strength relationship such that the older the concrete, the larger the tensile strength for a given compressive strength.

The most commonly used test to determine the tensile strength of concrete is the splitting tensile test. This test tends to force the failure to occur within a narrow band of the specimen rather than occurring in the weakest section. If the failure does not occur away from the center section, the calculations will indicate a higher than actual strength. The tensile strength for normal weight concrete is usually taken as $6.7\sqrt{f_c'}$ and drying has little effect on the relationship.

Direct tensile tests made by attaching steel base plates with epoxy resins indicate approximately 25 percent lower strengths. Such tests are significantly affected by drying.[6]

If the concrete surface has been subjected to drying, a somewhat lower tensile strength than $6.7\sqrt{f_c'}$ should be used to predict cracks initiating at the surface. Where drying shrinkage has relatively little influence on section cracking, a tensile strength of $6\sqrt{f_c'}$ appears reasonable. The design tensile strength of concrete has a direct relationship to the calculated amount of reinforcing needed to restrict the size of cracks. Under these conditions, a minimum tensile strength of $4\sqrt{f_c'}$ is recommended where drying shrinkage may be considered significant.

In the preceding expressions it is more appropriate to use the probable compressive strength at critical cracking rather than the specified strength. For normal structural concrete it is therefore recommended that at least 700 psi be added to the specified strength in the design of concrete mixes. For massive reinforced sections (as described in Section 3.2) it is recommended that mixes be designed for the specified strength. The strength of concrete that controls the critical volume change for proportioning crack-control reinforcement may occur either during the first 7 days following placement or after a period of 3 to 6 months, depending primarily upon peak temperatures. If the cracking potential occurring upon initial cooling exceeds the cracking potential occurring during the seasonal temperature drop, the critical volume change will occur during the first week.

When the critical volume change is seasonal, some allowance should be made for the strength gain beyond 28 days at the time of cracking, particularly where fly ash is utilized. The strength gain from 28 days to 90 and 180 days of age as a percentage of the 28-day strength varies with the 28-day strength, depending on the cement and the proportions of fly ash or other pozzolans used. For concrete mixes properly proportioned for maximum strength gain, Fig. 3.2 gives a typical comparison for mixes with and without fly ash that use Type II cement.

When the critical volume change occurs during the first week, it is probably prudent to use 7-day standard-cured strengths in proportioning crack-control reinforcement. The 7-day strength of concrete normally ranges from 60 to 70 percent of 28-day strengths for standard cured specimens of Types II and I cements, respectively. Slightly lower strengths may be encountered when fly ash or other pozzolans are utilized. In-place strengths will vary depending on section mass and curing temperatures.

3.4—Modulus of elasticity

Unless more accurate determinations are made, the elastic modulus in tension and compression for hardened concrete may be assumed equal to $w_c^{1.5}$ $33\sqrt{f_c'}$ (in psi) which for normal weight concrete $57{,}000\sqrt{f_c'}$. It also should be based on probable strength as discussed in Section 3.3. The modulus of elasticity in mass concrete can depart significantly from these values, and should be based on actual test results whenever possible.

3.5—Creep

Creep is related to a number of factors, including elastic modulus at the time of loading, age, and length of time under load. Although creep plays a large part in relieving thermally induced stresses in massive concrete, it plays a lesser role in thinner concrete sections where temperature changes occur over a relatively short time period. Its primary effect as noted in Section 2.2, is the relief of drying shrinkage stresses in small elements. In general, when maximum temperature changes occur over a relatively short time period, creep can only slightly modify temperature stresses.

3.6—Thermal properties of concrete

The thermal properties of concrete are coefficient of expansion, conductivity, specific heat, and diffusivity.

The relationship of diffusivity, conductivity, and specific heat is defined by

$$h^2 = \frac{K}{C_h \cdot w_c} \qquad (3.1)$$

where

h^2 = diffusivity, ft²/hr
K = conductivity, Btu/ft·hr·F
C_h = specific heat, Btu/lb·F
w_c = weight of concrete, lb/ft³

These thermal properties have a significant effect on the change in concrete volume that may be expected and should be determined in the laboratory using job materials in advance of design, if possible. ACI 207.1R and ACI 207.4R discuss these properties in detail and present a broad range of measured values.

Where laboratory tests are not available, it is recommended that the thermal coefficient of expansion C_T be assumed as 5×10^{-6} in./in./F for calcareous aggregate, 6×10^{-6} in./in./F for silicious aggregate concrete, and 7×10^{-6} in./in./F for quartzite aggregate.

CHAPTER 4—RESTRAINT

4.1—General

To restrain an action is to check, suppress, curb, limit, or restrict its occurrence to some degree. The degree of restrain, K_R, is the ratio of actual stress resulting from volume change to the stress which would result if completely restrained. Numerically, the strain is equal to the product of the degree of restraint existing at the point in question and the change in unit length which would occur if the concrete were not restrained.

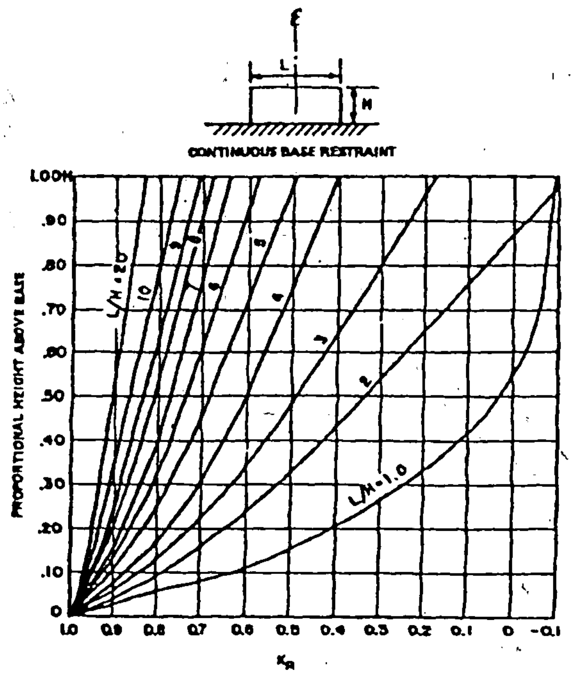


Fig. 4.1—Degree of tensile restraint at center section

All concrete elements are restrained to some degree by volume because there is always some restraint provided either by the supporting elements or by different parts of the element itself. Restrained volume change can induce tensile, compressive, or flexural stresses in the elements, depending on the type of restraint and whether the change in volume is an increase or decrease. We are normally not concerned with restraint conditions that induce compressive stresses in concrete because of the ability of concrete to withstand compression. We are primarily concerned with restraint conditions which induce tensile stresses in concrete which can lead to cracking.

In the following discussion, the types of restraint to be considered are external restraint (continuous and discontinuous) and internal restraint. Both types are interrelated and usually exist to some degree in all concrete elements.

4.2—Continuous external restraint

Continuous restraint exists along the contact surface of concrete and any material against which the concrete has been cast. The degree of restraint depends primarily on the relative dimensions, strength, and modulus of elasticity of the concrete and restraining material.

4.2.1 *Stress distribution*—By definition, the stress at any point in an uncracked concrete member is proportional to the strain in the concrete. The horizontal stress in a member continuously restrained at its base and subject to an otherwise uniform horizontal length change varies from point to point in accordance with the variation in degree of restraint throughout the member. The distribution of restraint varies with the length-to-height ratio (L/H) of the member. The case of concrete placed without time lapses for lifts is shown graphically in Fig. 4.1, which was derived from test data re-

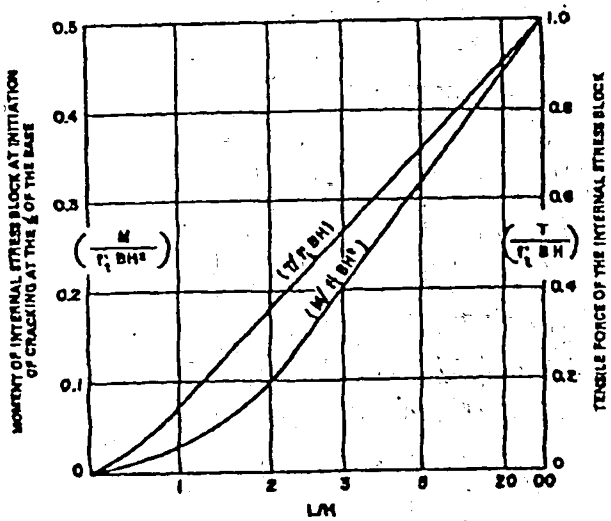


Fig. 4.2—Internal forces at initiation of cracks at restrained base

ported in 1940 by Carlson and Reading.[4.7]

For L/H equal to or greater than 2.5, restraint K_R at any point at a height h above the base may be approximated by

$$K_R = [(L/H-2)/(L/H+1)]^{h/H} \qquad (4.1)$$

For L/H less than 2.5, restraint K_R at any point may be approximated by

$$K_R = [(L/H-1)/(L/H+10)]^{h/H} \qquad (4.2)$$

Using the degree of restraint K_R, from Fig. 4.1 or calculated from Eq. (4.1) or (4.2), the tensile stress at any point on the centerline due to a decrease in length can be calculated from

$$f_t = K_R \Delta_c E_c \qquad (4.3)$$

where

K_R = degree of restraint expressed as a ratio with 1.0 = 100 percent
Δ_c = contraction if there were no restraint
E_c = sustained modulus of elasticity of the concrete at the time when Δ_c occurred and for the duration involved

The stresses in concrete due to restraint decrease in direct proportion to the decrease in stiffness of the restraining foundation material. The multiplier to be used in determining K_R from Fig. 4.1 is given by

$$\text{Multiplier} = \frac{1}{1 + \dfrac{A_g E_c}{A_F E_F}}$$

where

A_g = gross area of concrete cross section
A_F = area of foundation or other element restraining shortening of element, generally taken as a plane surface at contact
E_F = modulus of elasticity of foundation or restraining element

For mass concrete on rock, the maximum effective restraining mass area A_F can be assumed at $2.5A_g$ and the values of the multipliers are then shown in the following table.

Multipliers for foundation rigidity

$\frac{E_F}{E_c}$	Multipliers
∞	1.0
2	0.83
1	0.71
0.5	0.56
0.2	0.33
0.1	0.20

4.2.2 *Cracking pattern*—When stress in the concrete due to restrained volume change reaches the tensile strength of the concrete, a crack will form. If a concrete member is subject to a uniform reduction in volume but is restrained at its base or at an edge, cracking will initiate at the base or restrained edge where the restraint is greatest and progress upward or outward until a point is reached where the stress is insufficient to continue the crack. After initial cracking, the tension caused by restraint in the region of the crack is transferred to the uncracked portion of the member, thereby increasing the tensile stresses above the crack. For L/H greater than about 2.5, Fig. 4.1 indicates that if there is enough tensile stress to initiate a crack, it should propagate to the full block height because of the stress-raising feature just mentioned. It has also been found from many tests that once begun, a crack will extend with less tensile stress than required to initiate it (see ACI 224R).

From the preceding discussion, unreinforced walls or slabs, fully restrained at their base and subject to sufficient volume change to produce full-section cracking, will ultimately attain full-section cracks spaced in the neighborhood of 1.0 to 2.0 times the height of the block. As each crack forms, the propagation of that crack to the full height of the block will cause a redistribution of base restraint such that each portion of the wall or slab will act as an individual section between cracks. Using Eq. (4.3) and K_R values from Fig. 4.1 or Eq. (4.1) or (4.2) to determine the stress distribution at the base centerline, the existing restraining force and moment at initiation of cracking can be determined from the internal stress block for various L/H, and is shown in Fig. 4.2. Since cracks do not immediately propagate to the full block height throughout the member, a driving force of continuing volume change must be present.

A propagating crack will increase the tensile stress at every section above the crack as it propagates. Throughout the section the stress increase is the same proportion as the proportional increase in stress that occurred at the present crack position in propagating the crack from its previous position. From Fig. 4.3, the maximum restraining force in the stress block, corresponding to maximum base shear, occurs with the volume reduction producing initial cracking. The maximum moment of the internal stress block, corresponding to maximum base restraint, does not occur until the crack propagates to a height of 0.2 to 0.3 times the height of section. At that point, the crack is free to propagate to its full height without a further reduction in volume. From Fig. 4.3 the maximum base restraint at the centerline of a block having an L/H of 2.5 is approximately $0.2f_t'BH^2$. This may be assumed as the minimum base restraint capable of producing full-block cracking. The corresponding spacing of full-block cracking in unreinforced concrete would therefore be approximately 1.25 H.

Prior to cracking, the stress in the reinforcement of nonflexural members subjected to shrinkage depends primarily on the differences in coefficients of expansion between steel and concrete. Where the coefficients are equal, the reinforcement becomes stressed as crack propagation reaches the steel. The tensile force of the cracked portion of the concrete is thus transferred to the steel without significantly affecting base restraint. The moment of the steel stressed throughout the height of the crack adds directly to the restraining moment of the internal stress block at the centerline between cracks. When the combined internal stress moment and steel stress moment equals $0.2f_t'BH^2$ then the combined restraint is sufficient to produce full block height cracking at the centerline between cracks.

For L/H values less than 2, Fig. 4.1 indicates negative restraint at the top. For decreasing volume, this would mean induced compression at the top. Therefore, full-section cracking is not likely to occur.

At any section, the summation of crack widths and extension of concrete must balance the change in concrete volume due to shrinkage. To control the width of cracks it is thus necessary to control their spacing, since extensibility of concrete is limited. If the change in volume requires a minimum crack spacing less than $2H$, then reinforcement must be added to assure this spacing. From these postulations, if the required spacing is L' then the restraining moment of the reinforcing steel at the existing crack spacing of $2L'$ would be $0.2f_t'BH^2$ minus the restraining moment of Fig. 4.2 for $L/H = 2L'/H$.

A linear approximation of this difference can be determined by

$$M_{RH} = 0.2f_t'BH^2\left(1 - \frac{L'}{2H}\right) \qquad (4.4)$$

where

M_{RH} = restraint moment required of reinforcing steel for full-height cracking
f_t' = tensile strength of concrete
H = height of block

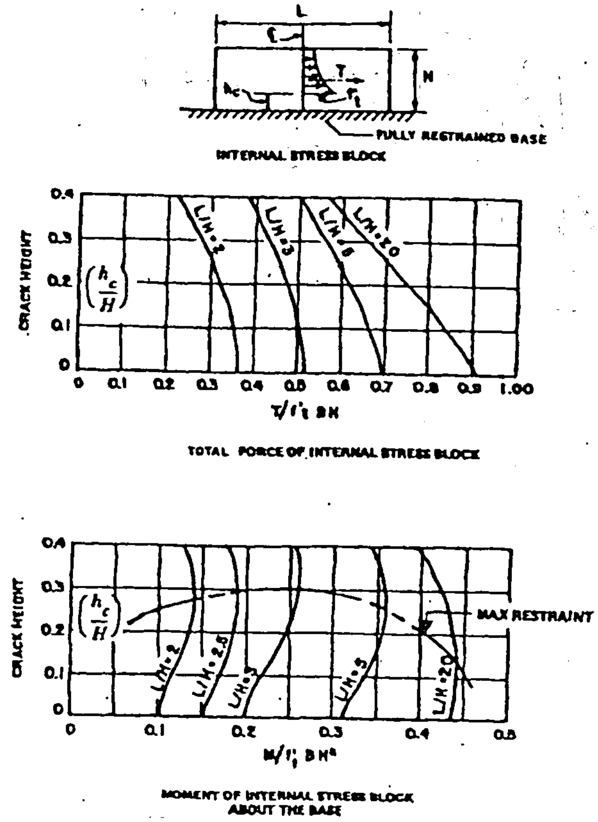


Fig. 4.3—Effect of crack propagation on internal forces

B = width of block

4.3—Discontinuous external or end restraint

When the contact surface of the concrete element under restraint and the supporting element is discontinuous, restraint to volume change remains concentrated at fixed locations. This is typical of all concrete elements spanning between supports. It is also typical for the central portions of members supported on materials of low tensile strength or of lower shear strength than concrete, which require substantial frictional drag at the ends to develop restraint.

4.3.1 *Stress distribution of members spanning between supports*—A member that is not vertically supported throughout its length is subject to flexural stress as well as stress due to length change. When a decrease in volume or length occurs in conjunction with flexural members spanning between supports, additional rotation of the cross sections must occur. If the supports themselves are also flexural members, a deflection will occur at the top of the supports and this deflection will induce moments at the ends of the member undergoing volume change. These flexural stresses will be in addition to the tensile stresses induced by the shear in the deflected supports (see Fig. 4.4). The end moments thus induced will increase tensile stresses in the bottom face and decrease tensile stresses in the top face of the member undergoing volume change. The magnitude of induced stress depends on the relative stiffnesses of the concrete element under restraint and the supporting members and may be de-

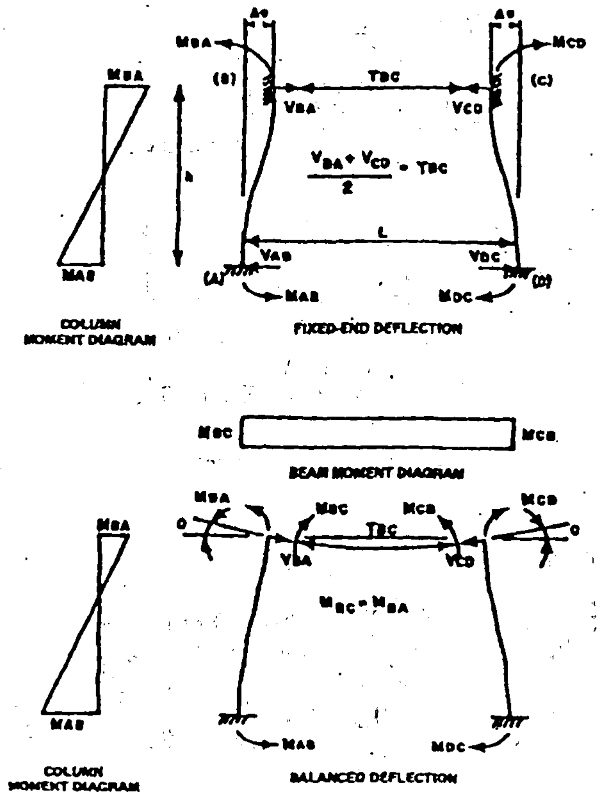


Fig. 4.4—Flexure of a simple frame induced by beam shortening

termined when the degree of restraint K_R has been determined for the support system. For members spanning two supports, the degree of restraint can be approximated by

$$K_R = \frac{1}{1 + \frac{A_B h^3}{4 L I_c}} \qquad (4.5)$$

where L and A_B = the length and area, respectively, of the member undergoing volume change, and I_c and h = the average moment of inertia and height respectively of the two supporting end members.

The change in bottom face steel stress for members spanning flexural supports can be approximated by

$$\Delta f_s = \frac{K_R C_T T_E E_S}{2pnj}\left[\frac{h}{d}\left(\frac{K_f}{K_f + K_c}\right) + 4pnj\right] \qquad (4.6)$$

where

C_T = linear thermal coefficient as defined in Section 3.6
T_E = design temperature change including shrinkage effects
E_S = elastic modulus of steel
K_f = stiffness of beam or floor system undergoing volume change
K_c = average stiffness of vertical restraining elements subject to deflection by volume change

For complicated frames and members spanning continuously over more than two supports, the stress induced in the member from the change in volume should be determined by a frame analysis considering the effects of sideway, member elongations under direct load, and shear deflections of the support members.

If the supporting members are very stiff relative to the member undergoing volume change, the deflection at the top of the supporting members will be essentially a shear deflection and no end moments will be induced in the member. Under these conditions the change in steel stress throughout the member will simply be

$$\Delta f_s' = 2K_R C_T T_E E_S \qquad (4.7)$$

A temperature gradient through a wall or slab with ends fixed or restrained against rotation will induce bending stresses throughout the member. When the restraint to rotation is sufficient to crack the member, cracking will be uniformly spaced throughout. Rotational stiffness is dependent on the moment of inertia of the cracked section. The ratio of the moments of inertia of cracked to uncracked sections in pure bending is $6jk^2$. Using this, the fixed-end moment for a cracked section would be

$$\text{FEM} = (T_1 - T_2)\, C_T E_c b d^2 \left(\frac{jk^2}{2}\right) \qquad (4.8)$$

where $T_1 - T_2$ is the temperature difference across the member, and C_T = the expansion coefficient of the concrete.

4.3.2 *Stress distribution of vertically supported members*—The distribution of stresses due to volume change in members subject to a discontinuous shear restraint at the base, but vertically supported throughout its length, is dependent on the L/H of the member, which for all practical purposes is the same as Fig. 4.1 where L is the distance between points of effective shear transfer at the base. As the L/H approaches infinity, the distribution of stress approaches uniformity over the cross sectional area at any appreciable distance from the support.

For slabs placed on the subgrade material of little or no tensile strength and lower shear strength than the slab concrete, the distance between points of effective shear transfer depends on the frictional drag of the slab ends. A decrease in slab volume will curl the ends of the slab upward. Cracking will initiate at approximately the center of the base when the full depth of the member has a parabolic tensile stress distribution (see Fig. 4.5) with the stress at the base equal to the tensile strength of the concrete. The cracking moment for this internal stress distribution will be $f_t' BH^2/10$. (Fig. 4.6 shows internal restraint.) The balancing external restraining moment depends entirely on the weight of the concrete and

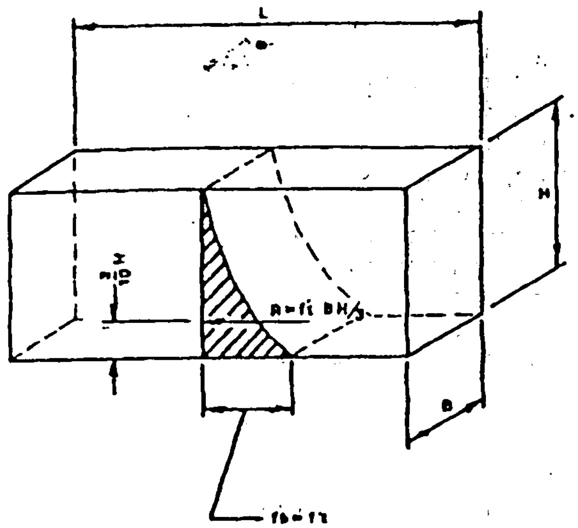


Fig. 4.5—Internal stress distribution of slabs on low-strength subgrade

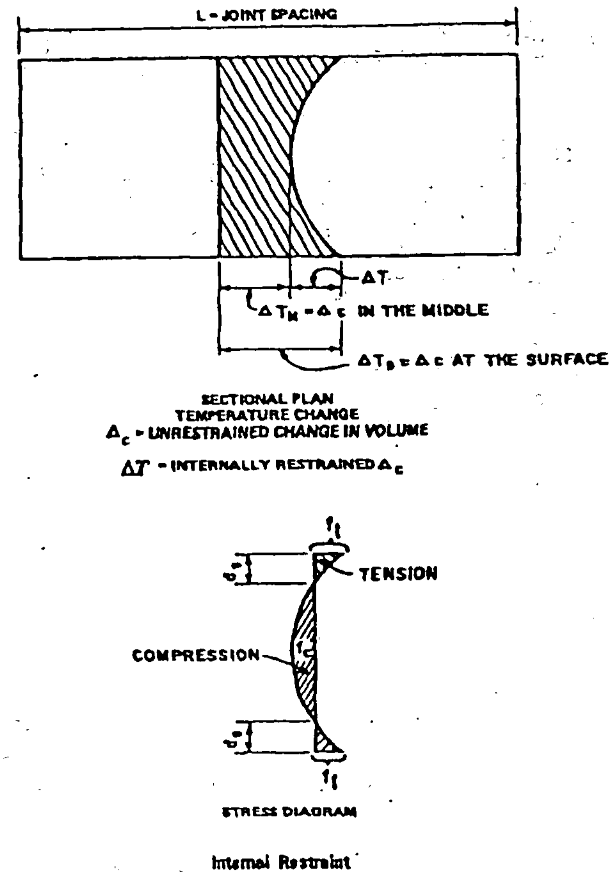


Fig. 4.6—Internal restraint

the distribution of the base pressure. Assuming a parabolic base pressure distribution over two-thirds of the curling slab base, as shown in Fig. 4.7, the restraining moment will equal $0.075\, w_c BHL^2$, or

$$\frac{f_t' BH^2}{10} = 0.075\, w_c BHL^2$$

For $f_t' = 300$ psi $w_c = 144$ lb/ft^3, and $L = 20\sqrt{H}$ (for L and H in ft).

When the overall slab length exceeds $20\sqrt{H}$, the distribution of stress in the central portion of the slab will approximately equal that of continuously restrained base having an L/H of $(L - 20\sqrt{H})/H$. When the spacing of cracks must be less than $20\sqrt{H}$, reinforcement must be provided. When the ratio of $(L - 20\sqrt{H})/H$ is less than 2, a minimum tensile force of $f_t'BH/3$ must be provided by the reinforcing steel to provide multiple cracks between the end sections. If the ratio of $(L - 20\sqrt{H})/H$ is greater than 2.5 the reinforcement must be capable of developing the full drag force of the end sections. This would be the full tensile force T of Fig. 4.2 for L/H corresponding to $(L - 20\sqrt{H})/H$. Thus the reinforcement requirements are

$$A_s = \frac{T}{f_s} \geq \frac{f_t' BH}{3 f_s} \tag{4.9}$$

where f_t' = tensile strength of concrete and f_s = allowable steel stress.

4.3.3 *Cracking pattern of vertically supported members*—When the stress of a member subject to discontinuous restraint or restrained at its ends exceeds the tensile strength of the concrete, a single crack will form between the points of restraint. Any additional cracking of the member must be

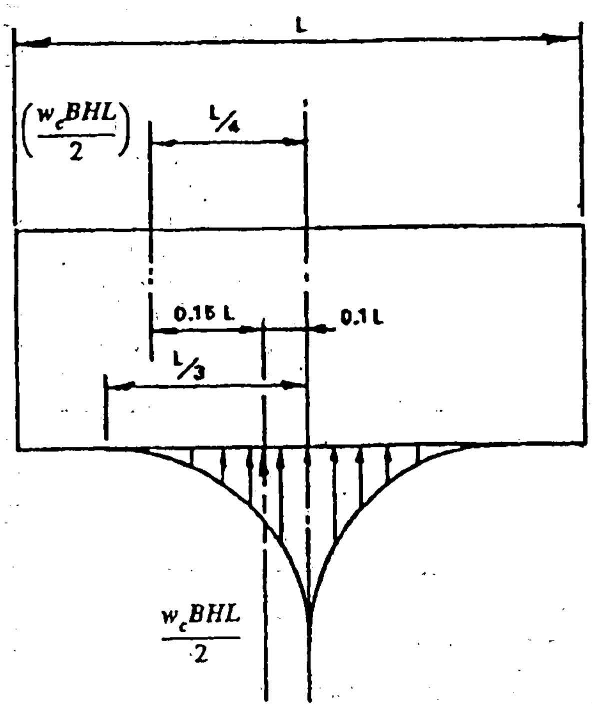


Fig. 4.7—Pressure distribution and restraining moments of curling slab

provided by enough reinforcing steel at a controlled stress level to equal the total restraint force induced at the member ends.

4.4—Internal restraint

Internal restraint exists in members with nonuniform volume change on a cross section. This occurs, for example, within walls, slabs, or masses with interior temperatures greater than surface temperatures or with differential drying shrinkage from outside to inside. It also occurs in slabs projecting through the walls of buildings with cold outside edges and warm interiors and in walls with the base or lower portions covered and the upper portions exposed to air.

Internal restraint depends on the differential volume change within a member. Its effects add algebraically to the effects of external restraint, except that their summation will never exceed the effects of 100 percent external restraint. Therefore, where high external restraint conditions exist the effects of internal restraint may be negligible.

4.4.1 *Stress distribution and cracking*—Internal restraint is similar to continuous edge restraint, except that the effective restraining plane is the plane of zero stress in the internal stress block and is dependent on the actual temperature gradient in the concrete (see Fig. 4.6). For section stability, the summation of tensile stress induced by the temperature or moisture gradient on a cross section must be balanced by an equal compressive force. This balance line locates the depth d_s of the internal stress block. If the depth of the tensile stress block d_s is large in comparison to the spacing of joints L, then the stress induced by volume change will not be significant. As an example, if the annual temperature range at the surface is four times the range in concrete, then a 100 ft thick dam would have a 15 ft deep tensile stress block using the distribution shown in Fig. 5.3.5 of ACI 207.1R. If we assume a 50 ft spacing of joints, the L/d_s ratio would be 3.3 and the degree of restraint at the surface would be 25 percent using Fig. 4.1 of this report and L/d_s as L/H. In contrast, from the same chart the daily cycle shows a penetration of only 2 to 2.5 ft. Using 2 ft as d_s, the degree of restraint at the surface would be approximately 85 percent and assuming a concrete tensile strength of 300 psi, a concrete modulus of 3×10^6 psi and a coefficient of thermal expansion of 5×10^{-6} in./in./F, cracking would occur at the face with a 24 F drop in surface temperature. For equal stress the annual temperature variation would have to be 82 F. Cracking from the daily temperature cycle is not usually significant in dams and large masses, particularly in moderate climates, because of the limited penetration or significance of such cracks. The 24 F drop in mean daily temperature corresponds to normal winter temperature fluctuations for moderate climates. See Chapter 5 of ACI 207.1R for a more complete discussion of surface cracking.

Temperatures on the opposite faces of a wall or slab may not be equal because of a difference in exposure conditions. The variation of temperatures through the slab or wall may be assumed to be parabolic or exponential.

Temperature distribution of this sort will curl the slab or wall if unrestrained, or induce bending stresses along the member if its ends are restrained as previously discussed in Section 4.3.1.

The plane of zero stress of the tensile stress block for projecting portions of concrete walls or slabs may be determined by a heat-flow analysis or by trial as just described. The proportion of cold volume to total volume is larger for members of this type than for dams or other large concrete masses. The penetration of the daily temperature cycle may therefore be assumed somewhat more than the 2 to 2.5 ft penetration previously mentioned for dams. Restraint at the free edge may also be determined for these cases from Fig. 4.1 by setting the depth of the tensile stress block d_s as a fixed plane 3 ft inside the exterior surface.

CHAPTER 5—CRACK WIDTHS

5.1—General

Reinforcement is utilized to restrict the size of cracks that would otherwise occur. Large-sized, randomly spaced cracks are objectionable and may indicate that the reinforcement transverse to the crack has yielded. This may be cause for concern, depending on the structure in question and the primary purpose of the reinforcement. Surface-crack widths are important from an esthetic viewpoint, are easy to measure, and are the subject of most limitations. While the width of a crack at the surface may initially be larger than the crack width at the reinforcement, the difference may be expected to decrease with time.

For water-retention elements, very narrow, just-visible cracks (0.002 in.) will probably leak, at least initially; however, nonmoving cracks up to 0.005 in. may heal in the presence of excess moisture and therefore would not be expected to leak continually. Any leakage may be expected to stain the exposed concrete face or create problems with surface coatings.

Most thermal cracks transverse to reinforcement do not appear to have significant impact on corrosion. (ACI 224R, ACI 224.1R).[8]

Fiber reinforcement is of some benefit in controlling cracks but may not be cost effective.

5.1.1 *Controlled cracking*—It has been common practice for many years to use expansion and contraction joints to reduce the size and number of uncontrolled cracks. In sidewalk and pavement construction, formed grooves have also been used to create planes of weakness and thereby induce cracking to coincide with the straight lines of the grooves. This concept has been expanded in the United Kingdom as a method of controlling cracks in massive walls and slabs. The British install plastic or metal bond breakers to induce cracks at specific locations. The British research indicates that a cross-sectional reduction of as little as 10 percent has proved successful in experiments, but 20 percent is recommended to assure full section cracking in practice.[9] The depth of surface grooves is obviously limited by any continuous reinforcement; therefore, some form of void must be cast into massive sections to achieve the needed section reduction. These voids can be formed with plastic pipes or deflatable duct tubes. Alternately, the reduction may be accomplished by using proprietary crack-inducing water barriers that have been designed to act as both bond breakers and water stops. The principal advantage of a crack-control system is that cracking can essentially be hidden by the formed grooves. Also, the crack size (width) loses its significance when there is a water barrier and the reinforcement crossing the crack is principally minimum steel that is not required for structural integrity.

5.2—Limitations

It is desirable to limit the width of cracks in massive structures to the minimum practical size, in keeping with the function of the structure. Reinforced mass concrete structures are generally designed in accordance with ACI 318. The crack-control provisions of ACI 318 develop reasonable details of reinforcement, in terms of bar size and spacing, for general conditions of flexure. The Commentary to the ACI Building Code says that the code limitations are based on crack widths of 0.016 in. for interior exposure and 0.013 in. for exterior exposure. The permissible crack widths versus exposure conditions in Table 4.1 of ACI 224R represent a historical viewpoint of "tolerable crack width." While they may not represent a current consensus, they do offer guidance to what has been considered acceptable. ACI 350R establishes minimum percentages of shrinkage and temperature reinforcement for sanitary engineering structures based on the spacing of construction joints from 20 to 60 ft. In addition, it restricts the working stress and z-value of Eq. (10-4) of ACI 318, based on the thickness of cover and type of exposure. For an 18 in. thick member with 2.5 in. cover, exposed to liquids, the crack width corresponding to the ACI 318 Commentary would be 0.011 in. for flexure and 0.009 in. for direct tension.

Limiting crack width by utilization of reinforcement becomes increasingly difficult as member size increases. The most effective means to control thermal cracking in any member is to restrict its peak hydration temperatures. This becomes increasingly important with increasing member size. For massive structures, the amount of reinforcement required to restrict crack width to less than 0.009 in. becomes impractical when any of the accepted formulas to predict crack width are used. Cracks of this width will allow some leakage; however, leakage will be minimum and controllable.

5.3—Calculations

A number of crack-width equations are proposed in the literature. ACI 318 adopts an expression based on one developed in a statistical study by Gergely and Lutz[10] reported in ACI SP-20.

$$w = 0.076 \sqrt[3]{d_c A}\, \beta f_s 10^{-3} \tag{5.1}$$

where

w = maximum crack width at surface, in.
d_c = cover to center of bar, in.
A = average effective concrete area around a reinforcing bar ($2d_c$ x spacing), in.^{2}
β = distance form neutral axis to the tensile face divided by distance from neutral axis to steel
f_s = calculated steel stress, ksi

In the preceding formula, the β-ratio is taken as 1 for massive sections.

The maximum crack width for tension members is generally accepted as larger than the just-given expression for flexure. ACI 224R suggests the following to estimate maximum tensile crack width

$$w = 0.10 f_s \sqrt[3]{d_c A}\, 10^{-3} \tag{5.2}$$

The preceding expressions for maximum crack width for flexure and tension are based on applied loads without consideration for volume change. Any restraint of volume change will increase directly the actual crack width over that estimated by these formulas. Thus, any procedure which makes a reasonable estimation of expected volume change in its analysis will improve predictability. When the expected change in volume has been accounted for, Committee 207 believes the application of the Gergely and Lutz expression for crack width provides sufficient limitations in determining crack reinforcement without additional conservatism. Committee 207 has therefore chosen this expression to apply its procedures. The designer is always at liberty to chose a more conservative expression.

CHAPTER 6—APPLICATION

6.1—General

Determination of restraint, volume change, appropriate concrete properties, and crack widths have been discussed. They will now be combined for calculation of steel areas. Exterior loads that induce tensile stress in the concrete in addition to those induced by volume change must also be accounted for in steel area calculations.

6.2—Volume change plus flexure

For both normal structural and massive members, the change in stress f_s induced by a decrease in volume of flexural members (discussed in Section 4.3.1) should be added directly to the service-load stress, and crack width should be checked as per Sections 5.2 and 5.3.

For normal structural members, ACI 318 can be followed. This requires a value of z, a quantity limiting distribution of flexural reinforcement

$$z = f_s \sqrt[3]{d_c A} \tag{6.1}$$

where

f_s = calculated stress in reinforcement,

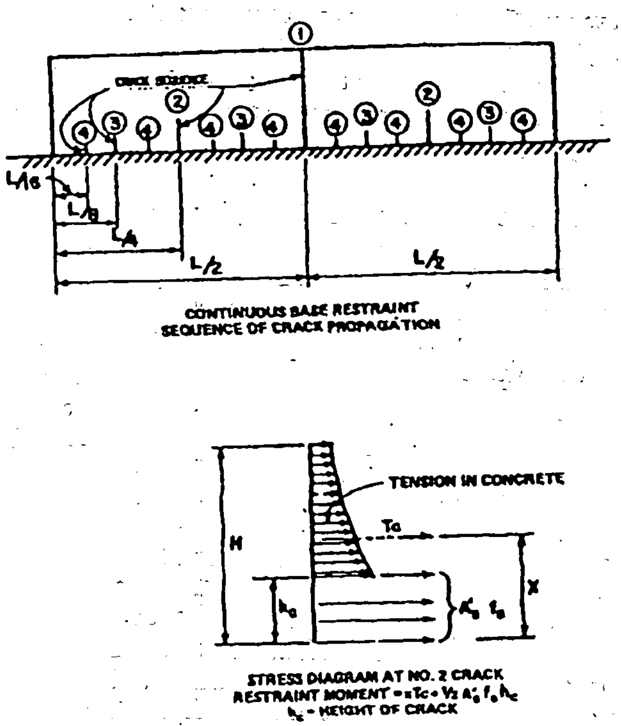


Fig. 6.1—Sequence of crack propagation and distribution of stress at No. 2 crack

d_c = thickness of concrete cover measured from the concrete surface at which cracks are being considered to the center of the nearest reinforcing bar

A = effective tension area of concrete surrounding a group of reinforcing bars and having the same centroid as that of reinforcement, divided by the number of bars

to be checked in lieu of crack width (notation as in ACI 318). The value of z should be limited to 175 for normal interior exposure, and 145 for normal exterior exposure.

For reinforced mass concrete, the combined stress should be limited by crack width based on Chapter 5. In addition, the minimum ratio of tensile-steel reinforcement for massive concrete members in flexure should be based on steel stress not to exceed $0.9 f_y$, where f_y is the specified yield stress of steel in ksi.

6.3—Volume change without flexure

The spacing of cracks is largely dependent on the conditions of restraint when a decrease in volume occurs in a member not subject to flexure. Stress in the reinforcing steel can be determined using the Gergely-Lutz crack width formula with a β of 1.0 by assuming a bar cover and spacing and calculating the stress in reinforcement f_s from

$$f_s = \frac{w \times 10^3}{0.076 \sqrt[3]{d_c A}} \text{ (in ksi)} \tag{6.2}$$

where w is the permissible crack width.

6.3.1 *Continuous external restraint*—Members subject to continuous restraint at their bases or on one or more edges will crack under continuing volume change as described in Section 4.2.2. Cracks are not uniform and will vary in width throughout the height of the member.

Fig. 6.1 shows the sequence of cracking for a member subject to uniform volume change and continuous base restraint. As each new crack forms at approximately the midpoint of the uncracked portions of the base, the previously formed cracks will extend vertically. The maximum width of each crack will occur at vertical locations just above the top of the previously formed cracks. Below this point there are two more times the number of cracks to balance volume change. The concrete at the top of the partially extended crack is assumed stressed to f_t'. Therefore the summation of crack widths on any horizontal plane must approximately equal the total volume change ($K_R L C_T T_E$) minus concrete extensibility Lf_t'/E_c.

The extensibility of concrete is affected significantly by creep; therefore, the time required for a given volume change to occur will directly affect the temperature drop T_E, producing cracking.

Hognestad[11] found that for the normal range of service-load stress for high-strength reinforcement, which is between 30 and 40 ksi, a mean value of the ratio of maximum crack width to average crack width was 1.5. If N is the number of cracks and w is the maximum crack width then the $N \cdot w/1.5$ will be the summation of crack widths in a given length and

$$\frac{N \cdot w}{1.5} = 12L(K_R C_T T_E - f_t'/E_c) \tag{6.3}$$

for L in ft. If the average crack spacing equals L', then $NL' = L$ and

$$L' = \frac{w}{18(K_R C_T T_E - f_t'/E_c)} \tag{6.4}$$

For most structures, the hydration heat effects are dissipated during the first week after placement. At this age, the extensibility or tensile strain capacity of the concrete is generally less than 100 microstrains and the effective temperature drop would constitute only hydration heat. For hot-weather placements, the maximum temperature drop will not occur until the concrete is 3 to 6 months old. At this age, creep and tensile strain capacity may be improved to provide more crack resistance. The age of critical volume change will be the age which requires the minimum average crack spacing L' from Eq. (6.4). For most parts of the United States, the critical volume change will occur for summer placement. A value for tensile strain capacity f_t'/E_c of 0.0001 for early-age cracking and 0.00015 for seasonal cracking is recommended.

It is necessary to calculate the required average crack spacing to determine the required restraining moment to be supplied by the reinforcing steel. Cracking throughout a member may or may not extend the full height of the member, depending on the *L/H* relationship (see Fig. 6.1). When cracks extend for just a portion of the height, only the reinforcing steel below the top of the crack is effective in contributing to the internal restraint moment. (From Fig. 6.1, the internal restraint moment between full-block cracks = $T_c x + A_s' f_s h_c/2$.) Even when some cracks do extend the full height, others extend only part way, so that the same situation applies between full-height cracks. For this reason, reinforcement is more effectively distributed if the wall is examined at several locations above the base to determine the average crack spacing required at each location corresponding to the degree of restraint K_R at each distance h from the base. The additional restraining moment $(A_s' f_s h_c)/2$ required of the reinforcing steel between the point h and the restrained base to produce the required crack spacing L' at h can be conservatively determined by substituting h for H in Eq. (4.4)

$$M_{Rh} = 0.20 f_t' Bh^2\left(1 - \frac{L'}{2h}\right) \tag{6.5}$$

The degree of restraint K_R to be used in the calculation of L' at h can be calculated as indicated in Section 4.2.1 or can be read directly from Fig. 4.1 as the proportional height above the base (*h/H*) corresponding to the actual *L/H* curves. It is conservative and usually convenient to assume the distance h as the free edge distance H and read K_R in Fig. 4.1 at the free edge using *L/h* as *L/H.*

In determining the volume change reinforcement required in each face of walls with continuous base restraint, calculations at lift intervals or at some arbitrary intervals above the base should be made as follows

$$A_b = 0.4\frac{f_t' Bh}{f_s N_H}\left(1 - \frac{L'}{2h}\right) \tag{6.6}$$

where

h = interval distance above the base being considered

N_H = total number of bars in the h distance above the base

A_b = area of bars required in each face of the wall

$A_s'h/N_H$ = A_b

As the distance h from the base increases, steel requirements will first increase and then decrease. Maximum steel requirements depend on base length, effective temperature drop and coefficient of thermal expansion. Fig. 6.2 gives the point of maximum steel requirements in terms of base length and design temperature for a coefficient of thermal expansion of 5 x 10^{-6} in./in./F. The same curve can be used for other expansion coefficients by using another design temperature equal to $C_T T_E/5 \times 10^{-6}$. Fig. 6.2 also provides the point h above which only minimum steel is required. Recommendations for minimum steel requirements are given in Section 6.4. Only minimum steel is required where $L¢$ is greater than $2h$. Fig. 6.3, 6.4, and 6.5 give the maximum steel requirements in terms of crack width, effective temperature drop, and base length for concrete walls having a C_T = 5 x 10^{-6}/F. These figures can be used to proportion steel requirements in place of the multiple calculations described above with only slightly higher total steel quantities being required.

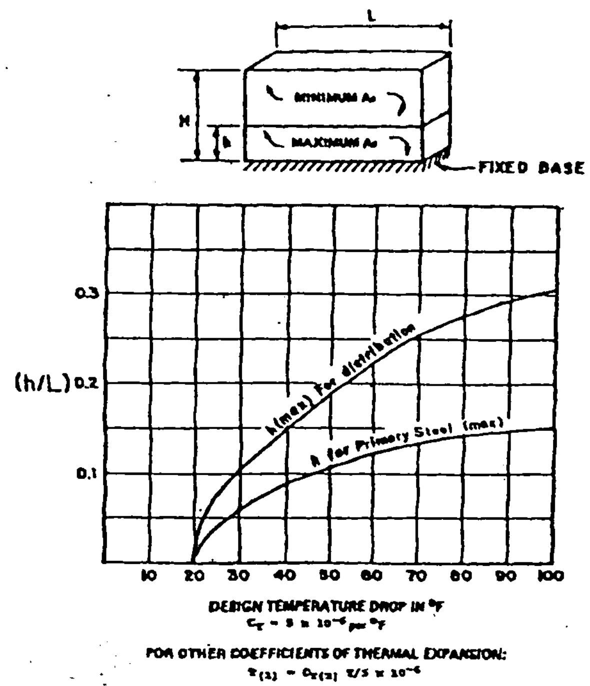


Fig. 6.2—*Wall height requiring maximum temperature and shrinkage reinforcement as a ratio of base length*

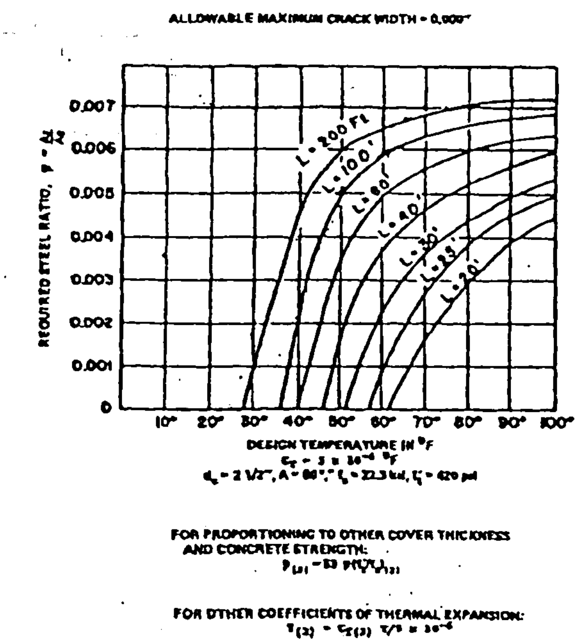


Fig. 6.3—*Maximum temperature and shrinkage reinforcement for walls with fixed bases*

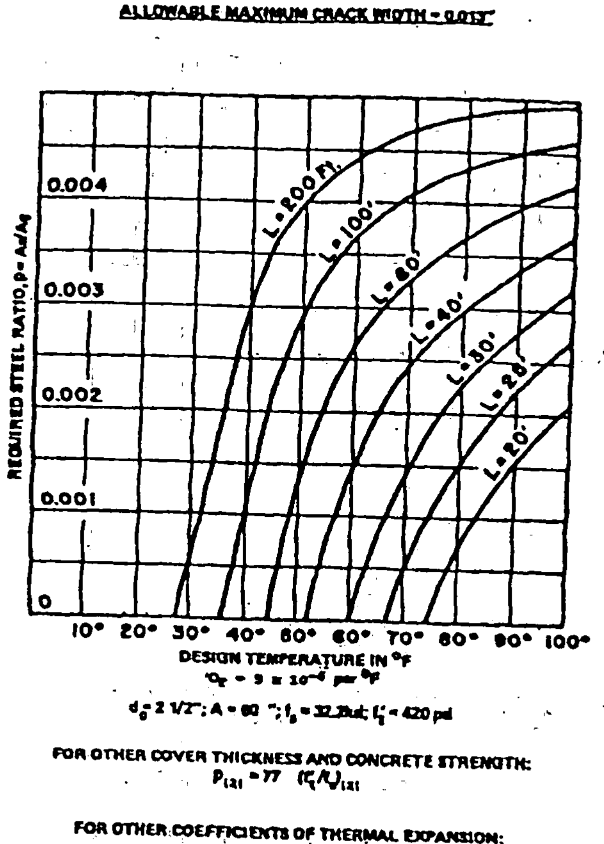


Fig. 6.4—Maximum temperature and shrinkage reinforcement for walls with fixed bases

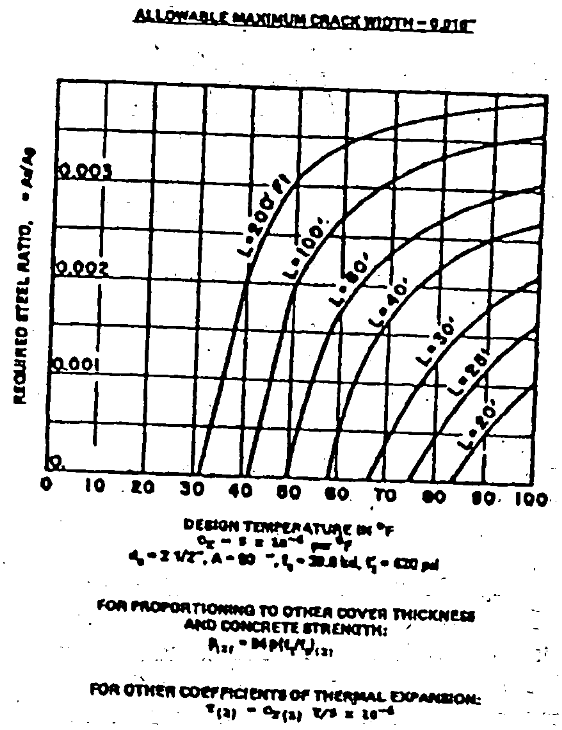


Fig. 6.5—Maximum temperature and shrinkage reinforcement for walls with fixed bases

The maximum height h over which these steel quantities are required can be determined from Fig. 6.2. Above h, only minimum steel is required. Requirements for concrete properties and cover distances other than noted can be proportioned as shown.

For slabs with continuous base restraint or walls with one side continuously restrained

$$A_b = 0.20\frac{f_t'}{f_s}\left(1-\frac{L'}{2H}\right)\frac{BH}{N_B\left(\frac{H-d_c}{H}\right)} \tag{6.7}$$

where N_B = total number of bars in the free face of the slab or wall.

In the case of relatively thick slabs, the amount of reinforcement required in the top face of the slab may be reduced by including the effect of the reinforcement in the sides. For this

$$A_b = 0.20\frac{f_t'}{f_s}\left(1-\frac{L'}{2H}\right)\frac{BH}{N_B\left(\frac{H-d_c}{H}\right)+\frac{N_H}{2}} \tag{6.8}$$

Only minimum steel is required where L' is greater than $2H$ (see Section 6.4).

In applying Eq. (6.7) and (6.8) to relatively large masses, the amount of reinforcement required will make it quite obvious that additional measures to control volume change should be used to control crack widths. Reinforcement is not practical in controlling the crack widths of very large externally restrained masses, and for these structures the principles of mass concrete construction described in ACI 207.1R must be followed to control cracking. The preceding formulas for crack spacing, however, can be utilized to establish a somewhat higher allowable temperature drop than normally used for mass concrete by acknowledging an acceptable crack. This can be seen in the design temperatures corresponding to zero steel requirements for the lengths of wall shown in Fig. 6.3 through 6.5.

Design temperatures in unreinforced sections should be kept approximately 10 F less than indicated for zero steel requirements because of the apparent sensitivity of crack widths to temperature in the cracking temperature range. Table 6.3.1 is based on this criteria.

When the expected temperature drop for the planned contraction joint spacing exceeds the design temperature limits

Table 6.3.1—Design temperature limits for unreinforced concrete walls (for limiting cracks to 0.009 in.)

Contraction joint spacing, ft	Coefficient of thermal expansion x 10^{-6}			
	4	5	6	7
100	30 F	24 F	20 F	17F
60	37 F	30 F	25 F	21 F
40	44 F	35 F	29 F	25 F
20	62 F	50 F	42 F	36 F

Section 6.2. If not, determine the steel requirements as per Section 6.3.2.

c. For members subject to internal restraint, provide reinforcement as per Section 6.3.3 if the required average crack spacing is less than twice the depth of the tensile stress block.

The following example problems illustrate this design procedure.

Example 6.1—Basement wall of power plant 30 ft high by 200 ft long is to be designed to retain backfill as a cantilevered wall for construction conditions. The wall is subject to ground water for its full height, with base slab on rock. It will be placed in 80 F ambient temperatures. Minimum final or operating air temperature will be 50 F. Assume the wall tapers from its maximum thickness at the base to 18 in. at the top. Maximum thickness at the base is controlled by shear and is 40 in. for 3000 psi concrete and 48 in. for 2000 psi concrete. Design for limited leakage by limiting crack width to 0.009 in. and determine required wall thickness and reinforcement for the following conditions:

a. Design for 3000 psi (3700 psi average strength) at 28 days and use the 470 lb/yd^3 mix of Example 2.2.

b. Same as (a) except contraction joints spaced 67 ± ft apart.

c. Design for 2000 psi at 28 days using mix of Example 2.1, no contraction joints and concrete cooled to 60 F placing temperature.

6.1(a)

Step 1.1—Volume-to-surface ratio (assume 10-ft lifts and wooden forms). Average thickness for first two lifts = 33 in. = 2.75 ft.

Wooden forms = 1.67 ft. of concrete

$$V/S = \left[\frac{2.75 + 2\,(1.67)}{2\,(10) + 2.75}\right]10 = 2.68 \text{ ft}$$

Step 1.2—Following Example 2.2 in Section 2.7, the effective placing temperature for 80 F concrete without cooling measures would be approximately 84 F.

Step 1.3—The minimum temperature T[inf/min] of concrete against earth, using Eq. (2.3), is 54 F.

Step 1.4—The temperature rise following Example 2.2 is 68 F.

Step 1.5—The design temperature equals 84 + 68 – 54 = 98 F.

Note: Seasonal temperature controls, since [5 (98) – 150] > [5 (68) – 100], as discussed in Section 6.3.1.

Step 2—Restraint (Fig. 4.1).

Step 3—Physical properties from Fig. 3.2; f_c' at 6 months = 4500 psi, tensile strength $f_c' = 6\sqrt{4500} = 402$ psi; and tensile strain capacity = 150 x 10^{-6} in./in., assume C_T = 5 x 10^{-6} in./in./F.

Step 4—Limiting crack width = 0.009 in.

Step 5(b)—f_s = 22 ksi for 2-1/2-in. cover and 12-in. spacing of bars from Eq. (6.2). Using Fig. 6.2 and 6.3, maximum temperature and shrinkage reinforcement is required for full height of wall for average thickness of 33 in.

Examples:—Eq. (6.4) at h = 5 ft

$$L' = \frac{0.009}{18\,(0.95 \times 5 \times 98 - 150) \times 10^{-6}} = 1.58 \text{ ft}$$

Eq. (6.6) at 5 ft

$$A_b = \frac{0.4\,(370)}{22,000}\left(\frac{38 \times 5 \times 12}{10}\right)\left(1 - \frac{1.58}{10}\right) = 1.40 \text{ in.}^2$$

From Fig. 6.3, ρ = 0.007 and $\rho_2 = 53\rho$ (402/22,000), or ρ_2 = 0.0068, since tensile strength is 402 psi and not 420 psi,∴ $A_b = 0.0068B$ x 12/2 = 0.0408 B and at h = 5 ft, A_b = 0.0408 x 38 = 1.55 in.^{2}

h, ft	Average B, in.	K_R (L = 200 ft) Fig. 4.1	L' 6.4*	$L'/2h$	A_b 6.6*	With adjusted ρ A_b Fig. 6.3	Reinforcement
5	38	0.95	1.58	0.16	1.40	1.55	#10 at 10
10	34	0.86	1.84	0.09	1.35	1.39	
15	31	0.79	2.11	0.07	1.26	1.26	#9 at 10
20	27	0.74	2.35	0.06	1.11	1.10	
25	23	0.71	2.53	0.05	0.96	0.94	#9 at 12
30	20	0.66	2.88	0.05	0.83	0.82	#8 at 12

* Formula:

Concrete = 537 yd^3 @ $70 .... $37,600
Temperature reinforcement = 27 tons @ $800 21,600
Cost (excluding forms) $59,200

6.1(b)

Everything same as (a) except L = 67 ft. From Fig. 6.2; maximum steel required only for the first 20 ft.

h, ft	Average B, in.	K_R (L = 200 ft) Fig. 4.1	L' 6.4*	$L'/2h$	A_b 6.6*	With adjusted ρ A_b Fig. 6.3	Reinforcement
5	38	0.79	2.11	0.21	1.31*	1.44	#10 at 12
10	34	-0.61	3.35	0.17	1.24*	1.28	
15	31	0.45	7.1	0.24	1.04*	1.17	#9 at 12
20	27	0.31	200	>1	0.24†	Minimum steel	#5 at 12
25	23	0.18	200	>1	0.21†		#4 at 12
30	20	0.07	200	>1	0.18†		

* Formula.

† A_b/min. = A_g x 0.0015/N_b (see Section 6.4).

Temperature reinforcement = 14.49 tons @ $80 .... $11,590

Note savings in reinforcing steel of $10,000 to be weighed against the cost of two joints and added construction time.

6.1(c)—For 2000 psi (2500 psi average strength) concrete, f_t at 6 months (using Fig. 3.2 and $C + F_a$) = $6\sqrt{4800}$ or 416 psi.

Steps 1.1-1.5—V/S for the first two lifts = 2.81.

For a 60 F placing temperature, the concrete peaks at 2 days from Fig. 2.4.

Approximately 12 F is absorbed using Fig. 2.6.

The temperature rise would be 19 F using Fig. 2.5. and accounting for cement type and quantity.

The design temperature equals 60 + 12 + 19 – 54 = 37 F.

From Fig. 6.3, ρ_2 = 53 (f_t/f_s) = 53 (0.003) x (416/22,000) = 0.0030

$A_b = \rho_2 B$ x 12/2 = 0.018 B

Steps 2-4—Assume same as (a).

Step 5(b)—From Fig. 6.2 and 6.3 maximum steel ratio equals 0.003 for first 25 ft of wall.

h, ft	Average B, in.	K_R (L = 200 ft) Fig. 4.1	L' 6.4*	$L'/2h$	A_b (minimum steel)	With adjusted p A_b Fig. 6.3	Reinforcement
5	46	0.93	39 ft	1+	0.41	0.83	#6 at 12
10	41	0.86	55 ft	1+	0.37	0.74	
15	36	0.79	200 ft	1+	0.32	0.65	#5 at 12
20	31	0.73			0.28	0.56	
25	26	0.67			0.23	0.47	#4 at 12
30	23	0.61			0.21	0.41	

* Concrete = 612 yd³ @ $70 .... $42,500
Temperature reinforcement = 6.5 tons @ $800 5,200
48,000
Savings in stress steel 3 tons for 8 in. additional depth .... -2,400
Net cost (excluding forms) $45,600
Cost of cooling concrete assumed equal to savings in cement costs. Note Example c is $13,600 less than Example a for the same design requirements.

Example 6.2—Culvert roof 36 in. thick supporting 20 ft of fill, spanning 20 ft between 4 ft thick walls, 20 ft high by 100 ft long resting on a rock base, placed in 80 F ambient air, minimum final air temperature 20 F, no cooling of concrete, mix same as Example 2.1, stress steel #9 at 10 stressed to 24,000 psi in bottom face.

Step 1.1—The volume-to-surface ratio

$$V/S = \left[\frac{3\,(20)}{2\,(20+3)}\right]\left(\frac{3+2}{3}\right) = 2.2 \pm \text{ft}$$

Step 1.2—Effective placing temperature = 90 – 0.6(10) = 84 F (Using Fig. 2.6).

Step 1.3—Final temperature is

$$20 + \frac{2\,(60-20)}{3}\sqrt{\frac{36}{96}} = 36\,\text{F}$$

Step 1.4—From Fig. 2.5, the temperature rise for a wet surface condition = 34 F.

For the same concrete placed at 69 F, the temperature rise for Example 2.1 was 30 F.

Considering adjustments for cement type and proportions, the actual rise for Example 2.1 was 18 F.

Therefore, the actual rise = 34(18)/30 = 20 F.

Step 1.5—Design temperature = 84 + 20 – 36 = 68 F.

Step 2—Restraint for end supports [Eq. (4.5)]

$$\frac{A_B h^3}{4 L_b I_c} = \frac{(1)\,(3)\,(20)^3}{4\,(20)\,(1)\,(4)^3/12} = 56 \pm$$

$$\therefore K_R = \frac{1}{1+56} = 0.0175$$

Step 3—f_t' = 405 psi $C_T = 5 \times 10^{-6}$ in./in./F

Step 4—Assume w = 0.013 in., d_c = 2 in.∴ A_{dc} = 125 for #9 bars at 10 in. o.c.

$$\therefore f_s = \frac{0.013 \times 10^3}{0.076\,\sqrt[3]{125}} = 34.3\text{ ksi allowable}$$

Step 5—Steel requirements [Eq. (4.6)]

$$p = \frac{1.20}{12\,(33)} = 0.003,\ n = 9,\ j = 0.94,\ h = 20\text{ ft}$$

$$K_f = (3)^3/20 = 1.35,\ K_c = (4)^3/20 = 3.2$$

$$\Delta f_s = \frac{0.0175\left(5 \times 10^{-6}\right)(68)\left(29 \times 10^6\right)}{2\,(0.003)\,9\,(0.94)}$$

$$\left[\frac{20}{3}\left(\frac{1.35}{1.35+3.2}\right) + 4\,(0.003)\,9\,(0.94)\right] = 7000\text{ psi}$$

Note: This is less than allowable of 34,300 psi; therefore no additional steel is required for volume change in the stress direction.

6.2(a)

For the roof slab of Example 6.2 find the temperature steel parallel to the wall. Assume 3-½ in. cover to center of temperature steel or f_s = 26,000 psi for bar at 12 in. spacing.

Note: Since the temperature rise of the slab is only 20 F the wall does not offer enough restraint to crack the slab therefore design the slab as an extension of the wall with a design temperature drop of 68 F.

Step 2—Restraint at 5 ft from the wall for L/h = 100/25 = 4, K_f = 0.40.

Step 5—From Eq. (6.4) Note: $K_R C_T T_E$ of 0.4(5)(68) is less than f_t'/E of 150.∴ No cracking will occur and only minimum steel is required

$$A_b = 0.0015\,(12)\,(32.5)/2 = 0.29\text{ in.}^2/\text{ft}$$

Reinforcement = #5 at 12 in. each face.

Example 6.3—A 6 ft thick power-plant base slab supporting widely spaced walls. Construction joints but no contraction or expansion joints. Assumed placed in 75 F average ambient air temperature with final unheated interior of 50 F. Slab is designed for operating uplift conditions requiring #11 bars at 12 in. o.c. stressed to 24,000 psi.

a. Assume same concrete mix and conditions as Example 2.2.

b. Assume same concrete mix and conditions as Example 2.1.

6.3(a)

Step 1—The maximum *V/S* for a slab shall be 75 percent of the slab thickness. See paragraph 2.6. Therefore, *V/S* = 0.75 (6 ft) = 4.5 ft maximum.

a. Effective placing temperature using Fig. 2.6 and temperature peak of 1.5 days from Fig. 2.4.

$$T_{PE} = 85\text{ F} - 0.03(10) = 82\text{ F} \pm$$

b. Temperature rise using Fig. 2.5 for wet surface conditions.

For *V/S* = 4.5 at T_{PK} = 82 F; temperature rise = 41 F.

From Example 2.2: Adjustments for cement type = 41/30 = 1.38; adjustments for cement content 470/376 = 1.25.

∴ Net temperature rise = 1.38(1.25)(41) = 71 F.

c. Final temperature using Eq. (2.3)

$$T_F = 50 + (2/3\,(60-50))\sqrt{54/96} = 55\text{ F}$$

d. Design temperature drop

$$82 + 71 - 55 = 98 \text{ F}$$

Step 2—Restraint (Fig. 4.1)—Without contraction or expansion joints the length is unspecified therefore assume L/H is greater than 20 or $K_R = 0.9$ maximum.

Step 3—Physical properties, $f_t' = 6\sqrt{4600} = 405$ psi.

Step 4—Limiting crack width = 0.013 in. For bars at 12 in. o.c. and cover of $2\frac{1}{2}$ in. the allowable steel stress from Eq. (6.2) is 32,200 psi.

Step 5—Steel requirements

$$L' = \frac{(0.013)}{18[0.9(5)(98) - 150]10^{-6}} = 2.5 \text{ ft} \quad \text{(Eq. 6.4)}$$

$$A_b = \frac{0.20(405)}{32,000}\left(1 - \frac{2.5}{12}\right)\frac{12(72)}{1} \quad \text{(Eq. 6.7)}$$

$$= 1.73 \text{ in.}$$

Check:

Δf_s for flexure (Eq. 4.7)

$\Delta f_s = 2(0.9)(5 \times 10^{-6})(98)(29 \times 10^6) = 25,600$ psi

$\Sigma f_s = 24,000 + 25,600 = 49,600$

Since combined stress is greater than the allowable, additional steel is needed, however, maximum steel requirements will be less than 1.56 + 1.73 = 3.29 in.^{2}/ft or #11 at 6 in. o.c. Assume final bar spacing of 7 in. o.c. for an allowable steel stress of 38,500 psi.

$$A_s = 1.56\left(\frac{24}{38.5 - 25.6}\right) = 2.90 \text{ in.}^2/\text{ft} \; \#11 \; @ \; 6 \therefore \text{OK}$$

6.3(b)

For Example b the design temperature would be 34 F and $\Delta f_s = 8900$ psi so that combined stress equals 32,900 psi which exceeds allowable of 32,200 psi by less than 3 percent; therefore, no additional steel is needed for temperature.

CHAPTER 7—REFERENCES

7.1—Recommended references

The documents of the various standards-producing organizations referred to in this document are listed below with their serial designations.

American Concrete Institute

116R	Cement and Concrete Terminology—SP-19(85)
207.1R	Mass Concrete
207.4R	Cooling and Insulating Systems for Mass Concrete
223-83	Standard Practice for the Use of Shrinkage-Compensating Concrete
224.1R	Causes, Evaluation, and Repair of Cracks in Concrete Structures
305R	Hot Weather Concreting
306R	Cold Weather Concreting
318	Building Code Requirements for Reinforced Concrete
350R	Environmental Engineering Concrete Structures

ASTM

C 496	Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens
C 186	Standard Test Method for Heat of Hydration of Hydraulic Cement

7.2—Cited references

1. Carlson, Roy W.; Houghton, Donald L.; and Polivka, Milos, "Causes and Control of Cracking in Unreinforced Mass Concrete," ACI JOURNAL, *Proceedings* V. 76, No. 7, July 1979, pp. 821-837.

2. Milestone, N. B., and Rogers, D. E., "Use of an Isothermal Calorimeter for Determining Heats of Hydration at Early Ages," *World Cement Technology* (London), V. 12, No. 8, Oct. 1981, pp. 374-380.

3. Verbeck, George J., and Foster, Cecil W., "Long-Time Study of Cement Performance in Concrete. Chapter 6—The Heats of Hydration of the Cements," *Proceedings*, ASTM, V. 50, 1950, pp. 1235-1262.

4. Carlson, Roy W., "Drying Shrinkage of Large Concrete Members," ACI JOURNAL, *Proceedings* V. 33, No. 3, Jan.-Feb. 1937, pp. 327-336.

5. Troxell, George Earl, and Davis, Harmer E., *Composition and Properties of Concrete*, MacGraw-Hill Book Co., New York, 1956, p. 236.

6. Raphael, Jerome M., "Tensile Strength of Concrete," ACI JOURNAL, *Proceedings* V. 81, No. 2, Mar.-Apr. 1984, pp. 158-165.

7. "Control of Cracking in Mass Concrete Structures," *Engineering Monograph* No. 34, U.S. Bureau of Reclamation, Denver, 1965.

8. Darwin, David; Manning, David G.; Hognestad, Eivind; Beeby, Andrew W.; Rice, Paul F.; and Ghowrwal, Abdul, "Debate: Crack Width, Cover, and Corrosion," *Concrete International: Design & Construction*, V. 7, No. 5, May 1985, pp. 20-35.

9. Turton, C. D., "Practical Means of Control of Early Thermal Cracking in Reinforced Concrete Walls," Paper presented at the ACI Fall Convention, New Orleans, 1977.

10. Gergely, Peter, and Lutz, LeRoy A., "Maximum Crack Width in Reinforced Concrete Flexural Members," *Causes, Mechanism, and Control of Cracking in Concrete*, SP-20, American Concrete Institute, Detroit, 1968, pp. 87-117.

7.3—Additional references

1. Hognestad, Eivind, "High Strength Bars As Concrete Reinforcement, Part 2. Control of Flexural Cracking," *Journal*, PCA Research and Development Laboratories, V. 4, No. 1, Jan. 1962, pp. 46-63. Also, *Development Department Bulletin* D53, Portland Cement Association.

2. Concrete Manual, 8th Edition, U.S. Bureau of Reclamation, Denver, 1981, p. 17.

3. Tuthill, Lewis H., and Adams, Robert F., "Cracking Controlled in Massive, Reinforced Structural Concrete by

Application of Mass Concrete Practices," ACI JOURNAL, *Proceedings* V. 69, No. 8, Aug. 1972, pp. 481-491.

4. Houghton, D. L., "Determining Tensile Strain Capacity of Mass Concrete," ACI JOURNAL, *Proceedings* V. 73, No. 12, Dec. 1976, pp. 691-700.

APPENDIX

Notation

A = effective tension area of concrete surrounding a group of reinforcing bars and having the same centroid as that reinforcement, divided by the number of bars

A_B = area of a member subject to volume change

Ab = area of reinforcing bar

A_F = area of foundation or other element restraining shortening of element

A_g = gross area of concrete cross section

A_s = area of steel for a given width

A_s' = area of steel per ft of length for a given width

B, b = width of cross section

C = weight of portland cement per yd^3 of concrete, lb

C_{eq} = weight of portland cement plus a percentage of the weight of pozzolan per yd^3 of concrete, lb

C_h = specific heat, Btu/lb · F

C_T = linear thermal coefficient, 5×10^{-6} per F for limestone aggregate, 6×10^{-6} per F for siliceous river gravel aggregate

d = depth of member from compressive face to the centroid of the reinforcement

d_c = thickness of concrete cover measured from the concrete surface at which cracks are being considered to the center of the nearest reinforcing bar

d_s = assumed depth of tensile stress block for internal restraint considerations

e = eccentricity of a load with respect to the centroid of the section

E_c = modulus of elasticity of concrete

E_F = modulus of elasticity of foundation or restraining element

E_s = modulus of elasticity of steel

Fa = weight of fly ash per yd^2 of concrete, lb

f_c' = specified compressive strength of concrete, psi

f_s = calculated stress in reinforcement, psi

f_t = tensile stress, psi

f_t' = tensile strength of concrete, psi

f_y = design yield stress of steel

H = perpendicular distance from restrained edge to free edge. Where a slab is subject to edge restraint on two opposite edges, H is one-half the distance between edges. For slab on grade, H is the slab thickness in feet

H_a = adiabatic temperature rise of the concrete

h = height of vertical restraining element, column or wall, above fixed base or elemental height of a wall

h^2 = diffusivity in ft^2 per hour

h_c = elemental height of crack above base

h_g = 28 day heat generation of the cement by heat of hydration, cal/gm

I_c = moment of inertia of gross concrete section subjected to flexure by the restraining forces

j = ratio of distance between centroid of compression and centroid of tension to the depth d of a flexural member. $j = 1 - k/3$

K = conductivity, Btu/ft/hr/F

K_c = stiffness of vertical restraining element subjected to flexure by the restraining forces

K_f = stiffness of floor system being tensioned by restraint

K_R = degree of restraint. Ratio of actual stress resulting from volume change to the stress which would result if completely restrained. In most calculations, it is convenient to use the ratio of the difference in free length change and actual length change to the free length change

k = ratio of depth of compressive area to the depth d of flexural member using the straight line theory of stress distribution

L = distance between contraction or expansion joints in the direction of restraint or overall length of a member undergoing volume change

L' = calculated average distance between cracks

N = number of cracks

N_B = number of reinforcing bars in the free (unrestrained) face of a slab or wall

N_H = number of reinforcing bars spaced along the H face or faces perpendicular to the plane of restraint

n = ratio of modulus of elasticity of steel to that of concrete

p = area of steel divided by the appropriate area of concrete

M_{RH} = restraining moment to be supplied by the stress reinforcing steel for full height cracking

M_{Rh} = same as preceding for partial height

S = surface area of a concrete member exposed to air

T = tensile force, lb

T_A = average minimum ambient air temperature over a prolonged exposure period of 1 week

T_c = temperature generated by the total quantity of cementitious materials if all were portland cement

T_{C+F} = temperature generated by the mixture of portland cement and pozzolan

T_E = effective temperature change in members including an equivalent temperature change to compensate for drying shrinkage

T_{DS} = equivalent temperature drop to be used in lieu of drying shrinkage

T_M = temperature of earth or rock mass

T_{min} = minimum temperature of concrete against earth or rock mass, F

T_p = placing temperature of the fresh concrete

T_{PK} = effective placing temperature after accounting for heat gained from or lost to the air, F

T_1 = high temperature in a temperature gradient
T_2 = low temperature in a temperature gradient
V = volume of a concrete member
$\overline{W}_u$ = the water content of the fresh concrete lb per yd^3
$\overline{W}_c$ = weight of cement per cu yd of concrete, lb
w = maximum surface crack width, in.
w_c = weight of concrete, lb/ft^3 Section 4.3.2
x = distance between resultant tension force and the compression face, in.
z = quantity limiting distribution of flexural reinforcement, psi, see ACI 318
β = ratio of the distance from the neutral axis to the tension face of a flexural member to the distance from the neutral axis to the tension steel. Where flexure is not involved, $R = 1$
β = ratio of distance from neutral axis to the tensile face to the distance from neutral axis to steel
Δ_c = contraction of the concrete, in./in.

Metric conversions

1 in.	=	25.4 mm
1 ft	=	0.3048 m
1 in.^{2}	=	645.1 mm^2
1 ft^2	=	0.0929 m^2
1 in^3	=	16.39 x 10^{3}mm^3
1 ft^3	=	0.0283 m^3
1 yd^3	=	0.7646 m^3
1 lb	=	0.4536 kg
1 lb/in.^{2} (psi)	=	6895 Pa
1 kip/in.^{2} (ksi)	=	6.895 MPa
1 lb/ft^2	=	47.88 Pa
1 lb/ft^3 (pcf)	=	16.02 kg/m^3
1 lb/yd^3	=	0.5933 kg/·m^3
1 Btu/lb·F	=	4.87 J/(kg·K)
1 Btu/lb·hr·F	=	1.731 W/m·K
1 in./in./F	=	1.8 mm/mm/C

Temperature

t_C = $(t_F - 32)/1.8$

Difference in temperature

t_C = $t_F/1.8$

This report was submitted to letter ballot of the committee and approved in accordance with ACI balloting procedures.