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Alberto Pujol
GEI Consultants, Inc., Oakland, California

Rob Busby
Regional Water Quality Control Board, Central Valley Region, Sacramento, California

Steve Rosenbaum
Regional Water Quality Control Board, Central Valley Region, Sacramento, California

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Alberto Pujol
GEI Consultants, Inc.
Oakland, California (USA)

Rob Busby
Regional Water Quality Control Board,
Central Valley Region
Sacramento, California (USA)

Steve Rosenbaum
Regional Water Quality Control Board,
Central Valley Region
Sacramento, California (USA)

ABSTRACT

The integrity of a mine plug impounding more than 30 million gallons of acidic mine drainage water (AMD) was investigated using a combination of technologies. A two-phase investigation program was adopted which allowed the full depth of the mine plug to be explored without release of detrimental AMD. The condition of the concrete, support rock, rock/concrete interface, and drainage pipes and valves was evaluated. Phase 1 included (1) a review of plug design documents and construction data, (2) review of data from other mines on acid attack of concrete, (3) detailed visual inspections, (4) use of nondestructive testing techniques to assess the condition of the concrete, and (5) geochemical testing of seepage and drain pipe waters. Phase 2 explored uncertainties identified during Phase 1 and included (1) coring into the concrete plug and adjacent rock, (2) cross-hole sonic logging, (3) laboratory testing of concrete and rock samples, (4) operational testing of valves, and (5) measurements of the thickness of pressurized piping components. While several minor defects were detected, none were significant enough to affect the mine plug's performance. The investigation confirmed the integrity of the mine plug after 13 years of operation.

BACKGROUND

The Walker Mine is an inactive underground copper mine located in the northern Sierra Nevada, in Plumas County, California. Mining operations took place between 1916 and 1941. Underground workings were developed between about elevation (El.) 5,400 feet and El. 7,000. The "700 Level Main Access Adit" is located at about El. 6,200 and is the lowest point at which the underground workings reach the surface. This adit intersected the orebody at a distance of about 3,000 feet from the portal (SRK, 1985).

Since 1919, acidic and metal-laden drainage water (Acid Mine Drainage or AMD) issuing from the adit portal has affected the downgradient streams (Dolly Creek and Little Grizzly Creek). After the mine was abandoned in 1941, the discharge from the mine was reported to have totally eliminated aquatic life in Dolly Creek and Little Grizzly Creek for a distance of approximately ten miles downstream of the mine site (SRK, 1985).

MINE PLUG DESIGN AND CONSTRUCTION

In November of 1987, the California Regional Water Quality Control Board (RWQCB) constructed a concrete mine plug, or seal, in the Main Access Adit to stop AMD discharges. The seal is located at a distance of about 2,700 feet from the portal and is seated in granodiorite. Its cross section is about 9 feet wide by 12 feet high and its length is 15 feet (Figure 1). Two 4-inch-diameter stainless steel pipes controlled by rotary control valves are embedded in the seal to allow draining of the impounded water. 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 monitored by the RWQCB.

The seal was designed for a design life of at least 100 years. 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. Determination of the length of the plug was based on hydraulic gradient and rock shear strength considerations. An allowable shear strength of 182 pounds per square inch (psi) was estimated for the rock mass adjacent to the rock-concrete interface. In addition, a maximum hydraulic gradient of 40 psi per foot of length of plug was set based on experience, a criterion which controlled the design length of the seal (SRK, 1985).

A view of the seal shortly after its construction in 1987 is shown on Photo 1. Prior to placement of concrete, soil debris was excavated from the tunnel invert, loose rock pieces were scaled, and the rock surface was pressure-washed. Joints and fractures exposed in the rock surface were tight and were not considered to be 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. The asperities provide a good interlocking surface for the rock-concrete contact.

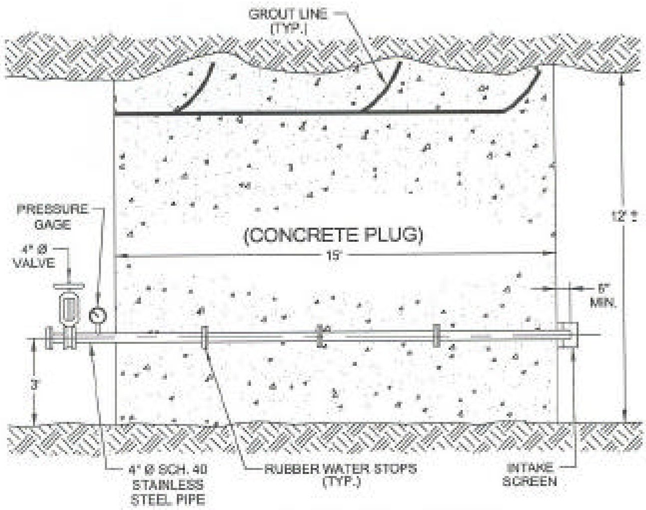

Fig. 1 Section through mine plug (SRK, 1985)

The concrete mixture used for the seal had a design 28-day compressive strength of 3,000 psi. Each cubic yard of concrete contained 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 by pumping from the mine portal. The total volume placed was approximately 66.2 cubic yards. Concrete test cylinders made from one sample taken during the placement averaged a 28-day compressive strength of 5,500 psi.

WATER QUALITY PARAMETERS BEFORE AND AFTER PLUG CONSTRUCTION

Typical water quality parameters for portal water and unimpacted nearby streams before construction of the seal are shown in Table 1 below. A seasonal variation was reported 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 milligrams per liter (mg/l) (SRK, 1985).

Photo 1. Seal after construction (December 1987).

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 Table 1. A comparison of the values in 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/l, the alkalinity increased from 0 to 56 mg/l, the copper concentration decreased from 29 to 0.1 mg/l, and the zinc concentration decreased from 0.9 to 0.02 mg/l.

The chemistry of the water impounded behind 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 Table 1.

Data on the discharge rate of portal water has been obtained by the RWQCB. Measurements taken from 1957 to construction of the seal in 1987 give an average discharge flow rate of about 200 gpm and a peak of about 1,100 gpm. Measurements taken after the construction of the seal give an average portal discharge flow rate of 6 gpm and a peak discharge of about 18 gpm from shallow groundwater inflow unaffected by mine drainage.

Table 1. Water Quality Parameters

Parameter	Unit	Prior to Mine Seal		After Seal Construction		
		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: ND = Not Detected, NA = Not Analyzed
1. Source: SRK, 1985; Table 2
2. Median values from water samples collected at the portal and tested semiannually from 1996 through 2000.
3. Samples collected just downstream of the seal in June 2000. The seal is located 2,700 feet from the portal.

Installation of the seal has been a success. Discharge of AMD from the adit has 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. 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 30 million gallons of AMD.

OBJECTIVES OF INTEGRITY TESTING

The RWQCB operations and maintenance plan for the Walker Mine site requires integrity testing of the mine seal. 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. Consequently, an investigation was undertaken in October 2000 to assess the following specific issues:

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,

Using a combination of technologies, a comprehensive two-phase approach was adopted which allowed the full depth of the mine plug to be explored without release of detrimental AMD. Phase 1 included (1) a review of plug design documents and construction data, (2) extrapolation of data from other mines on acid attack of concrete, (3) detailed visual inspections, (4) use of nondestructive testing techniques to assess the condition of the concrete, and (5) geochemical testing of seepage and drain pipe waters. Phase 2 explored uncertainties identified during Phase 1 and included coring into the concrete plug and adjacent rock, additional nondestructive testing, and operational testing of valves.

PHASE 1 INVESTIGATIONS OF MINE PLUG INTEGRITY

Seepage Observations

The adit and mine seal were inspected during an initial site visit on October 31 through November 2, 2000. 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. 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.

The hydraulic pressure of the impounded water measured at the seal 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, water seeps through the concrete-rock interface and 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, drips down the face of the seal, and accumulates in a pool on the floor at the downstream toe of the seal. Large iron hydroxide deposits of a dark reddish color have formed over the seal face. There was no evidence of seepage flow occurring through the concrete itself. Also, there appeared to be minimal leakage through the rock beyond the seal.

The total seepage flow rate was measured to be about 0.15 gallons per minute (gpm) by using a hand pump and a timer.

The design studies had estimated an average hydraulic conductivity of 10^{-5} centimeters per second (cm/sec) or less for the rock penetrated by the mine (SRK, 1985). Based on back-calculations of flow, it was 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 design documents.

More important than absolute flow amounts is the trend of seepage flows versus time. A review of historical correspondence yielded several qualitative observations of seepage made at different hydraulic heads. Comparison with the measured seepage rate suggests that leakage is not increasing over time and may have decreased somewhat. However, it has not completely sealed itself.

Rock Condition

Rock exposed along the adit consists mainly of coarse-grained, light gray granodiorite. Structures observed along the adit include joints, cleavage, and sheared zones. Beyond the initial timber-supported zone, the rock is generally sound, and there is no installed support. 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.

Generally the rock is slightly to moderately jointed 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 one to three feet apart, and are rough, tight and clean or with a thin filling of clay.

Detailed observations and measurements of the rock condition and quality were made in the 20-foot-long reach of adit immediately downstream of the seal to document the current conditions and develop a Rock Mass Rating (RMR) in accordance with ASTM D 5878 (ASTM, 1996). Six classification parameters are used to develop the RMR: 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. The estimated RMR range was 65 to 69, indicating "good" quality rock.

Condition of Concrete Surface

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 (Mass, 2000). No surface erosion was observed. The concrete appeared dense and well consolidated. No cracking of any nature was observed on the seal face. The entire face of the seal was moist. Surface deposits appear to be associated with areas of seepage and seepage flow (Photo 2). Efflorescence, whitish deposits of calcium carbonate, was isolated to two small areas at 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.

Photo 2. Seal on November 1, 2000. Red paint marks are impact points for nondestructive testing.

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

Twelve 5/8-inch-diameter threaded form ties and the two 4-inch-diameter stainless steel drain pipes penetrate the full thickness of the seal. 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.

A horizontal water line on the concrete face, as evidenced by iron staining, was observed approximately 18 inches above the floor of the adit (Photo 3). This water line was generated by the presence of the ponded seepage against, and immediately downstream of the seal. The ponded water was removed for access and inspection.

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 of those areas 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 center of the calcium carbonate deposit on the lower left side of the seal (Photo 2) was wet from active seepage flow. It was found that the thickness of the deposit ranged from 1 to 2 millimeters (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 area above and below the water line that had been created by the ponded seepage were observed. 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 (Photo 3).

The area immediately below the protruding steel pipe that had been used for pumping concrete into the seal (Photo 2) 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.

It was 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. The concrete that is not exposed to direct contact with the acid water can be expected to be sound and hard.

If a value of 2 mm in 13 years is extrapolated linearly over a design life of 100 years, the softening of concrete directly exposed to AMD could reach a depth of 15 mm, or slightly more than ½ inch over the design life.

Nondestructive Testing of Concrete

The nondestructive impulse response (IR), impedance log and impact echo (IE) testing techniques were used to assess the condition of the concrete within the seal (CTL, 2000). For IR testing, a receiving sensor (geophone) was coupled to the face of the seal. 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-foot 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 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.

Photo 3. Closeup of bottom left quadrant showing water line of seepage pool (Nov. 2,2000)

Like the IR testing, the IE test uses stress waves to detect flaws within concrete structures. However, the frequency range used in the IE test is considerably higher, since 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. Despite using three different size impactors, the energy generated was insufficient to penetrate the entire length of seal.

The nondestructive test data indicated 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 Photo No. 1 for the location of the tremie pipe). This zone, approximately 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. This zone appeared to be likely shallow in thickness.

Typical values for stress wave velocity in 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 (fps). The thickness of the seal calculated assuming a stress wave velocity in the concrete of 13,125 fps 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 fps, indicating good quality concrete in the body of the seal with no major 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 fps, which indicates 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.

Condition of Drain Pipes

The drain piping consists of two independent 4-inch diameter stainless steel pipes that extend through the concrete seal. The as-built drawing (Figure 1) 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.

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. The 4-inch shutoff valves are precision flow control valves designed for highly erosive service and have a minimum shutoff rating of 285 psi. The valves are installed with 1/16-inch thick, raised-face flange gaskets that appear in good condition. The exteriors of the valves are not corroded and are in good condition. 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.

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.

While the valves were not leaking and appeared to be in good condition, it was learned that they 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.

AMD Testing

AMD samples were collected from the standing pond in front of the seal and from the drip stream emanating from the top of the seal. In addition, 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, and 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. This volume equals two pipe volumes; it was therefore assumed 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.

The samples were tested for pH, iron (Fe), total dissolved solids, alkalinity, and major metals. The samples were all moderately acidic with pH ranging from 4.2 to 4.8, indicating 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. The mildly acidic pH observed in these waters suggests that the host or wall rock contains some buffering capacity and is able to neutralize some of the pyrite oxidation-generated acidity.

No apparent difference was noted in most chemical constituents occurring in the AMD behind the seal versus 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 tended to be within typical analytical variance (<25%) suggesting that most differences may not be significant. The pH of the AMD behind the seal was higher than that of the downstream (seep) sample, while the Fe in the upstream sample was much higher than in the seep sample. In contact with air the Fe (II) oxidizes to Fe (III) and then precipitates as a hydroxide, with the generation of additional acidity and the corresponding decrease in pH. Heavy iron staining has occurred on the concrete face and drain pipes, consistent with the precipitation of Fe (III) upon oxygenation.

It also should 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.

Review of Iron Mountain Mine Data on Acid Attack of Concrete

In 1998, as part of the mine reclamation studies for the Iron Mountain Mine in California, the performance of various types of concrete exposed to AMD water for an extended period of time was investigated (Connell et al, 2000). Order-of-magnitude estimates of the rate of acid attack and expected depth of attack at the Walker Mine seal were compared with the results of the Iron Mountain Mine (IMM) studies involving field performance of concrete in an AMD environment. Of seven different mixtures tested at the Iron Mountain Mine, the mixture most resembling the concrete mixture used in the Walker Mine seal was identified. The rate of AMD attack for this mixture, calculated as an average depth of surface loss, is plotted on Figure 2. Extrapolation of the curve predicts approximately 0.4 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 one to two mm. The difference in these values may be attributed to differences in water chemistry, concrete composition, and exposure factors or to data inaccuracies.

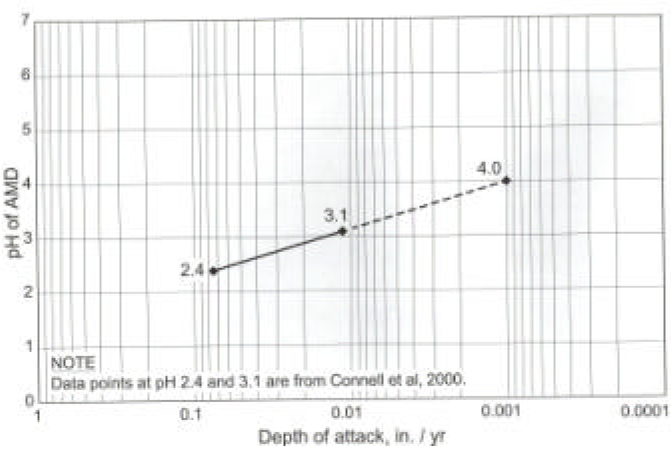

Fig. 2 Estimated rate of AMD attack of concrete based on Iron Mountain Mine studies.

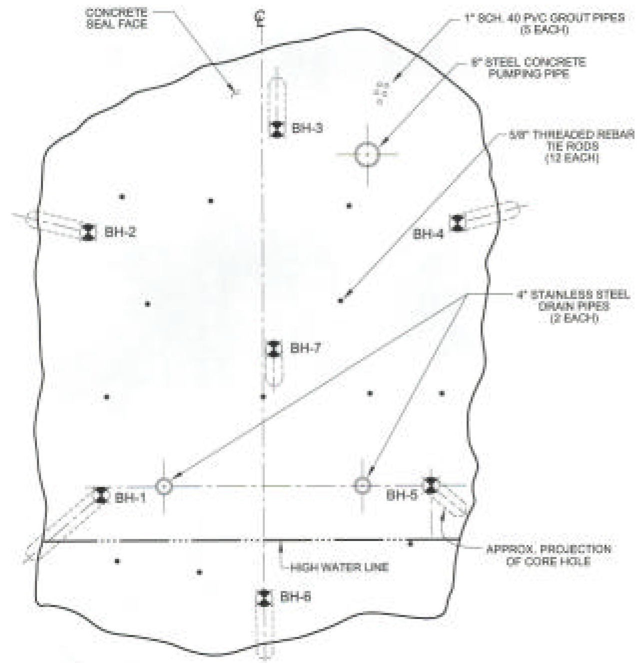

Fig. 3 Approximate borehole locations and orientations.

PHASE 2 INVESTIGATIONS OF PLUG INTEGRITY

The Phase 2 investigations explored uncertainties identified during Phase 1 and included (1) limited coring into the concrete plug and adjacent rock, (2) cross-hole sonic logging, (3) laboratory testing of concrete and rock samples, (4) ultrasonic thickness testing of exposed mechanical components, and (5) operational testing of drain valves.

Drilling and Testing of Concrete and Rock

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 further nondestructive testing of the seal and the rock-concrete interface. The locations of the core holes are shown on Figure 3. Six of the core holes, BH-1 through BH-6, were located along the perimeter and 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.

To align each core hole, 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 (Photos 4 and 5). 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 (Photo 5). Water was intercepted in only one drill hole, BH-4, although water build-up was not enough to register on the pressure gauge. The flow from BH-4 was 0.01 to 0.02 gpm.

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 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 pre-existing crack. The results of the packer tests are presented in Table 2 below.

Upon completion of drilling and nondestructive testing, the core holes that intersected the rock-concrete interface were pressure-grouted. A thin grout using microfine cement was injected to attempt to seal hairline cracks along the rock-concrete interface. Grout takes were minimal. 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.

Table 2. Water Pressure Testing of Boreholes

Hole	Zone Tested* (feet)	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
(**) cfm: cubic feet per minute

Both drilling conditions and examination of the concrete samples confirmed that the concrete in the seal is generally dense, sound, and hard. The coarse aggregate consists of angular 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 was a mechanical break or a pre-existing fracture was inconclusive. 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, again suggesting 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, only one sample was obtained of concrete that was adhered to rock (from hole BH-1). Elsewhere, breaks occurred between the concrete and the rock.

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.

Photo 4. Coring of hole BH-1 (September 9, 2001).

<u>Cross-Hole Sonic Testing</u>

In this method (ACI, 1998), 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. The probes are sealed units, and the holes are filled with water to provide coupling between the probes and the concrete. A recording unit measures the time taken for the ultrasonic pulse to pass through the concrete between the probes. 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 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 (CTL, 2001).

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 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.

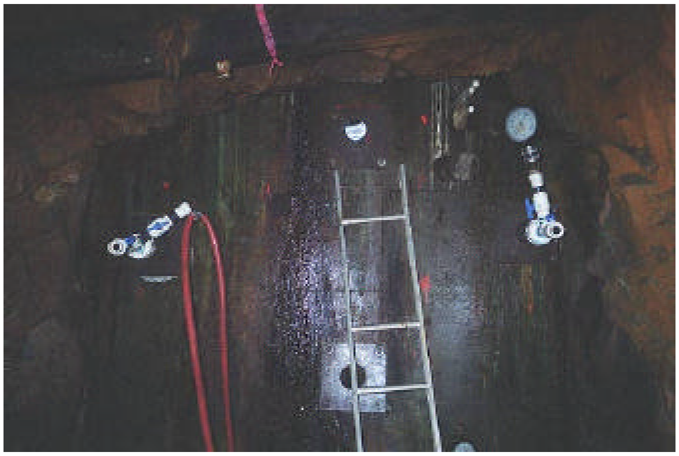

Photo 5. Core boles BH-2, -3, -4 and –7 in the upper half of the seal. Holes BH-2 and BH-4 are equipped with shutoff valves (September 10, 2001).

For the test to be successfully performed, the core holes must be full of water. Holes BH-1, 5, 6 and 7 were inclined slightly downward making it possible to retain water in these holes during testing. Holes BH-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.

Twelve cross-hole sonic logging profiles were obtained. Measurements of equivalent pulse velocities gave between 12,000 and 12,500 fps. All test profiles showed continuous concrete between holes, with no breaks in arrival signals at any point. 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.

Laboratory Testing Of Concrete And Rock Samples

The concrete and rock cores were visually examined, and samples were selected for laboratory testing. Samples for testing were selected from holes BH-3, BH-6 and BH-7 for the following reasons:

- BH-3 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.
- BH-7, the hole drilled through the center of the adit, was selected as a potential indicator of average concrete characteristics. Samples included the diagonal crack discussed above.

Five compression strength tests on concrete core samples yielded 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, 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, indicative of reasonably dense, well-consolidated concrete. The laboratory ultrasonic pulse velocity of these cores ranged from 13,211 to 14,889 fps and averaged 13,884 fps, values expected for well-consolidated concrete of this type.

Petrographic examinations of three core samples obtained from holes BH-3, BH-6, and BH-7 were made to ascertain evidence of acid attack. The core samples from holes 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 the bottom of hole BH-6 was poor, exhibiting soft paste and no coarse aggregate, probably due to segregation.

Similar tests were performed on two cores of granitic rock. The measured compressive strengths were 16,240 and 22,020 psi and 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.

Schmidt Hammer Tests

Schmidt hammer tests were performed on the face of the seal and adjacent rock to obtain an index value of unconfined compressive strength and to develop a baseline to use in future monitoring events. The index measurements of the unconfined compressive strength of the rock and joint surfaces along the walls and roof of the adit near the seal 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 the rock core samples.

Index strength measurements of the concrete surface of the seal yielded an average index value of 2,600 psi. 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.

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. This technique was used to measure the wall thickness of solid steel in pipes and valves, therefore providing an indication of internal corrosion. Testing was performed at 63 points spaced along the exposed 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 uses the measured time of travel of the ultrasonic waves and wave velocities for the applicable metal (stainless steel or carbon steel) to provide a direct readout of thickness for each test.

The results of the ultrasonic thickness testing of pipes and valves confirmed that the thickness of the pipe walls agrees with the design documents and has not been significantly reduced by corrosion. Variances between the nominal wall thickness and the ultrasonic thickness measurements were relatively small and within the permitted manufacturing tolerance. The main value of this data is to establish a baseline that can be used in future monitoring events to assess trends.

Operational Testing Of 4-Inch Shutoff Valves

An operational test of the shutoff valves was conducted to check valve operability. As a precaution, 4-inch backup valves were installed downstream of the existing shutoff valves prior to the operating test.

All valves in the drain pipe system were exercised and were found to be functional. The actuators of the shutoff valves were not frozen and operated smoothly. The valves gave tight shutoff upon closing. During the testing activities, the interior of the valves was observed from the downstream side (the nonpressure side, see Photo 6). Although this side of the valve contained AMD prior to the inspection, there was no discernible corrosion of the visible interior stainless steel components of the valves.

Photo 6. Interior of 4-inch control valve (June 13, 2001).

DISCUSSION AND CONCLUSIONS

Based on the findings from visual observations, nondestructive tests, and intrusive explorations, it was concluded 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 the explorations, including a zone of segregated, weak, concrete at the bottom of the seal; 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; and a possible crack or weakened plane. However, all three defects were judged to be localized features of little significance to the strength of the seal.

The seepage rate was estimated 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. The seepage rate is expected to be directly proportional to the hydraulic head. More important than absolute flow amounts is the trend of flows versus time. A review of historical qualitative observations suggests that, so far, the hydraulic conductivity of the seal/rock system has not been increasing over time.

The concrete surfaces exposed to the acid mine water have been affected to some extent. The acid attack 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 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.

There is no reason to believe that the 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. In addition, the results of nondestructive testing indicate that the thickness of sound concrete in the seal is approximately equal to the 15-foot seal design thickness, suggesting that the thickness of soft, deteriorated concrete is small. A review of data on acid attack on concrete obtained 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.

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.

The results of the nondestructive testing program 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, 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.

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 could be due to drilling and sampling techniques. It is also possible that the concrete and the rock are not well bonded throughout. However, 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.

While no evidence of acid attack was observed in the rock-concrete interface where it was traversed by the core holes, a small amount of seepage is occurring along the concrete/rock contact at the roof and sidewalls of the adit, suggesting 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 openings 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. Hairline cracks may become healed through the leaching and redeposition of calcium carbonate. If acid attack 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. Monitoring of long-term trends in the seepage rate through the seal would provide advance warning for gradual deterioration of concrete along seepage paths. The occurrence of seepage and the seepage rate are thus key indicators of concrete condition and performance.

In summary, the condition of the 15-foot-long concrete seal and surrounding rock mass is judged adequate to resist the design water pressure head of 570 feet. While several minor defects were detected, none were significant enough to affect the mine plug's performance. The investigation confirmed the integrity of the mine plug after 13 years of operation.

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