

WALKER MINE SEAL TESTING AND EVALUATION  
PLUMAS COUNTY, CA

SITE VISIT REPORT  
AND  
SAMPLING AND ANALYSIS WORK PLAN

SIGNATURE PAGE

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*April 12, 2001*

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# 1. Background and History

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## 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 globe valves; these apparently have not been opened since the seal was constructed. A sampling port with a pressure transducer is mounted on one of the drain pipes. The transducer is connected to a datalogger which is monitored by the RWQCB. A view of the seal shortly after its construction in 1987 is shown as Photo No. 14 (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. The water level behind the seal varies seasonally, peaking after the spring snowmelt. At its peak, the hydraulic head on the seal exceeds 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 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. 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 1988: 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 ¾-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 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. It 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) Water quality data for two samples taken from downstream of the seal in June 2000: One of them (labeled "seep") was taken from seepage water emerging from the rock/concrete interface along the crown. The other (labeled "pool") was taken from a pool of water at the toe of the seal. Selected parameters from these tests are shown in Table 1 below. Typical water quality parameters reported in 1985 by SRK for portal water and unimpacted nearby streams are also shown in Table 1. 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).
- (7) 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.

**TABLE 1  
WATER QUALITY PARAMETERS**

Parameter	Unit	Portal <sup>1</sup>	Streams <sup>1</sup>	Seep <sup>2</sup>	Pool <sup>2</sup>
Ca	mg/l	24.5	5.8	29	33
Na	mg/l	2.7	2.8	2.6	3.1
K	mg/l	1.6	0.7	2.4	2.8
Mg	mg/l	6.4	2.2	6.8	6.8
SO <sub>4</sub>	mg/l	146	5	200	200
HCO <sub>3</sub>	mg/l	0	23	ND	ND
PH	units	4.1	7.6	4.0	3.7
NO <sub>3</sub>	mg/l	4.5	0.7	ND	ND
NH <sub>3</sub>	mg/l	0.01	0.01	NA	NA
Cl	mg/l	1	ND	ND	ND
Cu	mg/l	29	0.03	14	12
Zn	mg/l	0.93	0.01	0.76	0.76
Fe	mg/l	1.0	0.15	3.3	1.2

- Notes:
1. Typical water quality parameters reported by SRK (SRK, 1985; Table 2) before the seal was constructed.
  2. 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**

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### **2.1 Purpose**

The purpose of the Walker Mine seal testing and evaluation program is to assess the integrity of the seal. Specifically, the following characteristics of the mine seal are to be assessed:

- 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.
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.
3. Upon approval of the work plan by the RWQCB, conduct the additional testing and evaluation activities. Prepare a Seal Testing and Evaluation Report summarizing the data and findings from the testing activities, data interpretation, conclusions and recommendations.

This document includes both the Site Visit Report and the Sampling and Analysis Work Plan. The findings and conclusions of the site visit are presented in Section 3. The investigations proposed for additional characterization of the seal are delineated in Section 4. Data and additional information are included in appendices.

## 3. Site Visit Report

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### 3.1 General

The site visit took place 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, geochemistry specialist
- Patrick Morrison, SDV-ACCI, graphics and project support

The general sequence of activities occurred as follows:

- Preparatory activities occurred on October 31<sup>st</sup>. A generator was brought to the mine site and the adit was ventilated. Morris, Pujol, Kollerer, and later Morrison, entered the mine. 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. At the end of the day Morrison left the site.
- The main visit took place on November 1<sup>st</sup>. The generator was again hooked up and the mine ventilated. 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. Pucik left the site. A meeting was held over dinner to discuss the day's observations. During the meeting, a preliminary outline of the Site Visit Report and Sampling and Analysis Work Plan was reviewed.

- A smaller group including Mass, Fox, Cercone, Pujol and Kollerer returned to the mine site on November 2<sup>nd</sup> to wrap up the site visit and demobilize. The adit was ventilated prior to entering the mine. 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 wheel barrow were stored near the entrance to the adit, by the battery packs. The ventilation duct was retracted into the adit and the mine was locked up. We left the site 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. No problems were encountered with air quality. 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 Observed Seepage Conditions**

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.

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 daylight 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 oxide deposits of a dark reddish color have formed over the seal face and were sampled during our visit. 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  $\frac{3}{4}$ -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- $\frac{1}{4}$  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- $\frac{1}{8}$  inches. When we arrived at 9:55 am on November 2, the water level was at 7- $\frac{1}{8}$  inches and we estimate that we pumped about 150 gallons to draw down the water level to about 2 - $\frac{1}{8}$  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 Observed Rock Condition

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.

With the exception of poor ground conditions in the first 1,300 feet of the adit, the adit appears to be sound and stable. 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, 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 Observed Concrete Condition

The exposed face of the concrete seal was closely examined by Gary Mass and Mike Knarr. 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 oxide and calcium carbonate from seepage water, the original formed surface of the seal was plainly visible and in good condition (See Photo No. 1). 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. Efflorescence is generally associated with the leaching of  $\text{Ca(OH)}_2$  from the cement due to movement of water through, or adjacent to the concrete. When seepage water exits the concrete and reacts with air,  $\text{CaCO}_3$  forms and is deposited on the concrete surface. The existing deposits of  $\text{CaCO}_3$  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. 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. 5, 6 and 7). 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 that would 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. 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. Furthermore, 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 treaded 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 water line (See Photos No. 2 and 3).

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. 4).

Based on these observations, it can be concluded that concrete 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. However, concrete along the concrete/rock contact where seepage is occurring has probably been affected in a similar manner as that observed on the face. 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. We believe that concrete that is not exposed to direct contact with the acid water can be expected to be sound and hard.

As mentioned earlier, iron staining and iron deposits have developed on the exposed face of the seal. The most severe accumulation of iron deposits is down the face on the right-of-center side of the seal. 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.

### **3.4.3 Potential Presence of Transverse Thermal Cracks**

A simplified thermal study using ACI 207.2R-95, Effect of Restraint, Volume Change, and Reinforcement on Cracking of Mass Concrete, was made to analytically investigate the potential for transverse cracking in the concrete seal during the curing period. Parameters used in this study were taken from the final construction report prepared by SRK. Details of the methodology used are presented in Appendix B, including the above-mentioned ACI publication and the calculations performed.

The results of this study indicate a potential that transverse cracks might be present within the concrete. However, there is no visible evidence of cracking or structural distress.

Furthermore, the nondestructive testing described in Section 3.5 did not detect transverse cracking; on the contrary, the test results indicate that the concrete is of good quality and without discontinuities. Therefore, it is felt that, if they exist, these cracks are most likely well distributed and do not affect the structural integrity of the seal. Additional investigations such as limited coring and sonic logging of the concrete will be considered to confirm this assessment.

### **3.5 Nondestructive Testing of Concrete**

The concrete seal was tested by Construction Technology Laboratories, Inc. (CTL) using the nondestructive impulse response, impedance log and impact echo testing techniques. A detailed report by CTL describing the test methods and results is included in Appendix C. In summary, the report 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 pipe in the upper right quadrant of the seal (see Figure 4 for the location of the tremie pipe). This zone, with approximate dimensions of 4 feet high by 2 feet wide, was probably caused by "blind spots" developing during the concrete placement.

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 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. 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.

It is suggested that the zone of poor concrete consolidation in the upper right quadrant of the plug be inspected by cross hole sonic logging and coring to a limited depth during the second phase of the testing and evaluation program.

## 3.6 Observed Arrangement and Condition of Drain Pipes

### 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. 8 through 13). 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 assemblies are included as Figures 5 (plan) and 6 (elevation).

### 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 D. 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, globe (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 downstream. The plug can operate through a 90-degree rotation and, unlike most control valves, the 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 is our understanding that the two 4-inch valves have been in the closed position, under upstream pressure, and have not been operated since they were installed 13 years ago.

**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 appear 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.

**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 show no corrosion and appear 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 is a 5-inch diameter, glass-faced gauge with a range from 1 to 300 psig 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 psig manufactured by Druck, Inc. 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 appears to be 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 are in good condition and, although coated with metal oxide deposits, show 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. A 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 are in good condition and show no corrosion. The carbon steel tie bolts and nuts are in fair condition and show noticeable corrosion, especially where the carbon steel is in contact with stainless steel. The threaded rod wall anchors are in good to fair condition. These rods are covered with oxide deposits, but show only mild surface corrosion.

**Auxiliary Piping:** Most of the 3/4-inch and smaller piping is stainless steel. The thickness (Schedule) of this piping is not known, although it is likely to be Schedule 40. 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 not magnetic and silver in color; the carbon steel piping was strongly magnetic and dull gray in color. One carbon steel pipe segment is the first segment on the piping branch for the pressure transmitter, just upstream of the isolating ball valve. The other two carbon steel pipes are connected to the blind flanges, one on each flange. (See Figure 6).

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 good to fair condition with some corrosion at the threaded connections at each end. The entire surfaces of the carbon steel pipes downstream of the blind flanges are covered with a thin layer of corrosion.

Product information for piping and fittings is included in Appendix D.

### 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 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
- 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

### 3.7 AMD Sampling

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 mine 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 for analysis of:

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

These constituents will be analyzed via the standard USEPA methods included in Appendix E. The other 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 (included in Appendix E).

A solid sample of presumed iron oxide buildup deposited by the seep on the concrete seal face was also collected by scraping the material off the face. After examination underneath a microscope, this sample may be analyzed for various elemental and mineralogical constituents.

Results from Sequoia Analytical Laboratory will be included in the Seal Testing and Evaluation Report. 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	ORP	DO	Fe
		[Units]	[mV]	[mg/L]	[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

Based on the limited data collected in the field, the following observations have been 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 reason for the slight differences in the pH of the samples is not yet apparent.
- 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 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 iron follows the dissolved oxygen and ORP measurements. Samples with low dissolved Fe are the samples containing the higher oxygen contents. High oxygen ensures oxidation of Fe (II) to Fe (III) which then precipitates as solid ferric hydroxide.

### **3.8 Discussion and Conclusions of 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. 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 measured 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  $7 \times 10^{-6}$  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 that are consistent with those 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 13 years since the seal was constructed.

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

### 3.8.2 Concrete Seal

Based on our observations and the findings of the nondestructive tests, we conclude that, in general, the concrete forming the seal is in good condition. Acid water attack on the submerged and wet portions of the downstream face is limited to the outer one-tenth of an inch. The concrete underlying this thin, deteriorated, layer is hard and sound. 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. The nondestructive test results, although approximate by nature, also indicate that the thickness of sound concrete in the seal agrees closely with the design thickness. Additionally, both visual observation and the nondestructive test techniques indicate that, in general, the concrete/rock contact is good.

Additional comments and issues for consideration include the following:

- A zone of poor concrete consolidation was identified by nondestructive testing in the upper right quadrant of the seal (see Figure D.2 in Appendix B). This zone is believed to be associated with poor consolidation under and around the elbow of the original concrete "tremie pipe" and is probably relatively shallow in thickness. It is suggested that a limited intrusive exploration be considered to evaluate this zone further.
- 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 is recommended as a means to provide advance warning for gradual deterioration of concrete along seepage paths. Concrete that is not exposed to direct contact with the acidic water can be expected to remain sound and hard.
- 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.
- Coring through the interface of the rock and concrete at the roof of the adit near the main seepage locations should be considered to directly examine the condition of the interface.
- A simplified thermal study was made to analytically investigate the potential for thermal transverse cracking in the concrete seal during curing. 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 program. It is believed that, if they exist, these cracks are most likely well distributed and do not alter the structural integrity of the seal.

### 3.8.3 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. The exterior of the carbon steel components exhibits a thin layer of corrosion, but this is not a cause for immediate concern at this time. However, over the long term, the carbon steel 3/4 inch pipe segment in the pressure transmitter line needs to be given particular attention because it is more susceptible to corrosion and is in a pressurized line upstream of the control valve. This piping should be protected from further corrosion by wrapping with polyethylene tape. Eventually, when there is an opportunity (at a time of low hydraulic head or if the mine is drained), this pipe segment should be replaced with stainless steel pipe.

**Valves:** The 4-inch shutoff valves in the seal drain lines, as well as most of the 3/4-inch ball valves, have 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 may also have become frozen. Thus, we know that these valves are not leaking, but we do not know if they still are operational or if they will give tight shutoff once they are exercised.

In considering how to proceed, two alternative courses of action present themselves. The first alternative is to do nothing. If at some point in the future the mine has to be drained, the valves would be operated at that time, with the understanding that the valves may not open at all, in which case the drain lines would have to be hot-tapped. More likely, the valves may become damaged as they are opened so that they will then not close or will leak when they are closed. If in that future event the valves are damaged, they would have to be replaced at that time.

A second alternative is to take appropriate measures now to verify that the valves are operational and, hereafter, 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. If the valves are to be considered fully operational, they need to be opened and closed to test whether they still rotate correctly and shut completely. However, the possibility exists that, if 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. Two options for conducting an operational test on the valves are apparent:

- Option #1: Using the existing piping arrangement, shut the downstream ball valves on the 3/4-inch lines tapped into the blind flange. Then test the operation of the 4-inch valves by opening and reclosing them. After the test, open the downstream ball valves to verify

that there is no leakage past the closed 4-inch shutoff valve. If the 4-inch valve is damaged and cannot be closed, then in effect abandon the 4-inch valve in place and use the two 3/4-inch ball valves as the shutoff valves for the drain line.

- Option #2: Modify the existing piping by installing a new 4-inch backup valve downstream of each existing 4-inch shutoff valve, and reinstall the blind flanges. Close the new backup valves and test the operation of the existing 4-inch valves. Then open the new backup valves and downstream 3/4-inch ball valves to verify that there is no leakage past the existing 4-inch valves after the testing. If an existing 4-inch valve is damaged, abandon it in place and use the new 4-inch backup valve as the drain shutoff valve.

There are some considerations before a testing option is selected. One is that the 3/4-inch ball valves have probably been in the mine without regular maintenance for as long as the 4-inch shutoff valves. Therefore, before testing as outlined in Option #1 could proceed, the ball valves would need to be removed from the mine and tested to verify that they are in good condition or replaced with new valves. In addition, all carbon steel piping downstream of the blind flanges would need to be replaced with stainless steel, since this piping would become pressurized if a shutoff valve could not be closed after the test. If this occurs, additional 3/4-inch piping, valves, and end caps would be required in order to exercise and maintain the ball valves as the shutoff valves. In addition, mine flow would be restricted to the 3/4-inch lines, at least until the mine was fully drained and the piping could be modified; this could create future operational limitations.

Option # 2 requires the selection, purchase, and installation of two new 4-inch backup valves. Once installed, each of the drain line valves could be exercised regularly, and the arrangement would maintain the full capacity of the 4-inch drains. In the future, if a different arrangement is required (such as connection to a water treatment facility), the blind flanges could be removed and other 4-inch piping connected to the drain lines. The lowest cost option for backup valves that provide tight shutoff would be flanged, ANSI Class 150, stainless steel ball valves.

After coordination and discussion of alternatives with the RWQCB, it is our understanding that the RWQCB desires to check the operating condition of the 4-inch shutoff valves. We recommend adoption of Option 2, since it would not reduce drain capacity.

**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. Wood cribbing under the pipes may be sufficient for this purpose, although it would require periodic replacement. Adjustable metal pipe supports could also be used.

#### 3.8.4 AMD Chemistry

Samples of AMD were obtained from upstream and downstream of the seal for laboratory testing and geochemistry evaluations. The water behind the seal remains mildly acidic, with a pH slightly over 4, suggesting 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. Sampling and Analysis Work Plan

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### 4.1 Purpose

The purpose of the Sampling and Analysis Work Plan is to describe the scope and outline the procedures for additional seal testing and evaluation activities. The additional testing and evaluation activities are recommended based on the observations and findings from the site visit, initial phase of nondestructive tests, and AMD sampling which are described in Section 3.

### 4.2 Scope

The proposed scope of testing and evaluation activities is summarized as follows:

- (1) **Drilling, Sampling and Grouting:** Holes will be 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 will be pressure-grouted after the testing is complete.
- (2) **Nondestructive Testing of Concrete Seal:** The condition of the concrete will be evaluated by performing cross-hole sonic logging of the concrete mass.
- (3) **Laboratory Testing of Concrete and Rock Samples:** Core samples of concrete will be tested to assess the pulse wave velocity and compressive strength of the concrete. Petrographic examinations of concrete cores along the concrete-rock interface will be made to ascertain the evidence of acid attack and quantify the magnitude and depth of alteration. Lastly, the compressive strength of the support rock will also be measured.
- (4) **Review of Concrete Mixture Data versus Published Data on Acid Attack of Concrete from the Iron Mountain Mine (IMM) near Redding, California:** The composition of the concrete in the seal will be reviewed and compared to test concrete mixes for which detailed acid attack data have been developed at the IMM. The IMM data will be correlated to the Walker Mine conditions, and conclusions will be drawn as to the estimated effects of long-term AMD attack on the concrete.
- (5) **Nondestructive Testing of Exposed Mechanical Components:** The ultrasonic thickness measurement technique will be used to measure the wall thickness of solid steel in pipes and valves, therefore providing an estimate of internal corrosion.

- (6) Operational Testing of 4-inch Shutoff Valves: The 4-inch shutoff valves will be operated to verify that they still can open and close. This activity will include procuring and installing 4-inch valves downstream of the existing shutoff valves in case the existing valves are damaged during the operational test.
- (7) Geochemical Evaluation of AMD Chemistry: The analytical chemistry data for AMD samples taken from upstream and downstream of the seal will be evaluated for indications of concrete dissolution and steel corrosion.
- (8) Review and Evaluation of Geotechnical and Structural Design: The geotechnical and structural design of the seal will be reviewed. The concrete and rock strength data from laboratory testing will be used to verify that the safety factor of the seal remains at the level assumed in the design.

These investigations are described in more detail below.

#### 4.2.1 Drilling, Sampling and Grouting

Holes will be 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 will be used for the sonic logging described in Subsection 4.2.2 below. The cores will also serve to examine the condition of the interface between the concrete seal and the rock walls, as well as to take samples of the concrete and rock immediately adjacent to the rock-concrete interface. A minimum core barrel size of 3-7/8 inches external diameter will be used.

Drilling will be performed through grout pipes cast into the face of the seal and equipped with pipe packers and valves. This will allow the control of water loss through the core holes if a pressurized joint or crack should be intercepted. The drilling contractor will be required to submit the design of the grout pipe for approval by GEI and the RWQCB. A preliminary scope of work and technical specifications for the drilling and grouting subcontract are included in Appendix F.

Seven continuous cores will be taken starting at the downstream face of the plug, with their locations as approximately shown on Figure 7. The core holes will be angled away from the axis of the seal, to ensure that the rock-concrete interface is cut within the middle third of the length. This is in order to obtain maximum information on concrete and rock conditions while at the same time minimizing the risk of intercepting a pressurized seam. One 8-foot-long core hole will be through the center of the seal and parallel to its axis. It is estimated that an approximate 56-foot total core length will be taken. The drilling operation and the cores taken from these holes will be observed and logged.

The drilling procedures will be recorded and the water color noted. The difference in flow rate between the drill water injected during coring and the water returning from the drill hole will be measured. During the drilling, sudden changes in drilling advancement rates that may indicate voids or changes in material properties will be recorded and evaluated. The depths where such changes occur will be compared with the features of the core. Any anomalies such as poorly consolidated concrete, cracking, and voiding identified in the concrete cores will be correlated with the results of the NDT program. The interface conditions will be examined in detail. All core hole observations will be documented.

Upon completion of drilling and nondestructive testing, the flow rate from the core holes will be measured and the water sampled for additional chemical analyses if considered appropriate. The valve at the grout pipe will be closed and the water pressure in the hole will be measured. The core holes will then be pressure-grouted; a thin grout using microfine cement will be injected to attempt to seal hairline cracks along the rock-concrete interface. Once grouting refusal is achieved, a thick zero-bleed grout will be used to backfill the holes, and the grout pipes will be removed. The grout take and pressure required to seal the core holes will be measured and recorded. The volume of grout that penetrated voids or cracks intercepted by the core holes will be estimated.

#### **4.2.2 Nondestructive Testing of Concrete Seal**

Following the coring of the seal but before grouting, cross-hole sonic logging between the core holes will be carried out. The method is described in Appendix G. Perimeter and diagonal sonic profiles will be obtained, and any anomalies such as voiding, poor compaction and cracking of the concrete between these profiles will be located and sized. The results will be presented as plots of equivalent pulse velocity of the concrete for each sonic log profile.

In addition, nondestructive Schmidt hammer tests will be done on the downstream face of the concrete and on the rock surface immediately downstream of the seal to estimate the unconfined compressive strength of the rock and concrete and to correlate these readings with the values obtained from laboratory testing of core samples.

#### **4.2.3 Laboratory Testing of Concrete and Rock Samples**

As indicated above, the cores will be visually examined and logged. The concrete-rock interface will be examined in detail. Evidence of chemical attack will be recorded. Concrete and rock cores will be tested for the following:

- Unconfined compressive strength of concrete (ASTM C42) and rock (D2938) to confirm strength values assumed in the design.
- Petrographic examination of concrete (ASTM C856) and rock (ASTM C295) to assess the condition of the concrete and rock and evaluate deterioration from the AMD attack.

- Pulse velocity through concrete (ASTM C597) and rock (ASTM D2845) to correlate compressive wave velocities with the values assumed for NDT evaluations.

It is anticipated that at least two tests of each type listed above would be performed on each material (rock and concrete). The ASTM procedures listed above are included in Appendix H for reference.

#### **4.2.4 Review of Concrete Mixture Data Versus Published Data on Acid Attack of Concrete from the IMM**

As part of the mine reclamation studies for the Iron Mountain Mine, Mass, Washington, and GEI conducted a long-term field test of concrete performance for concrete in contact with AMD. Seven different concrete mixtures were immersed for eight months in two different AMD streams with pH values averaging 2.5 and 3.5 respectively. The concrete weight loss and compressive strength were measured over the 8-month test period. The test results were used to estimate the long-term rate of acid attack and durability of concrete structures that will be in contact with or submerged in AMD. A summary of this work is presented in Connell et al (2000). For the Walker Mine seal, available design and construction records pertinent to the mix design and quality of the concrete in the seal will be reviewed; this information is contained in the Final Construction Report (SRK, 1989). The mix composition of the seal concrete will be compared to the mixtures tested at Iron Mountain. Using the test results of the mixture that most closely resembles the Walker seal concrete, an estimate will be made of the long-term rate of acid attack that the Walker seal concrete is likely to be experiencing. This estimate will be compared to the physical observations made during the site visit.

#### **4.2.5 Nondestructive Testing of Exposed Mechanical Components**

The internal condition of the exposed mechanical components (piping and valves) will be assessed by thickness testing using ultrasonic testing equipment. Ultrasonic waves are 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 is calibrated by inputting wave velocities for the applicable metal (stainless steel or carbon steel). A direct readout of thickness is obtained from the test. This is then compared to the design thickness. Laminations, dips, or pitting from internal erosion should result in a reduced thickness reading.

#### **4.2.6 Operational Testing of 4-inch Shutoff Valves**

Prior to performing an operational test of the 4-inch shutoff valves, 4-inch backup valves will be installed downstream of the existing 4-inch shutoff valves. Stainless steel ball valves are recommended for the application as providing a reasonable balance between performance and cost (see Appendix I for catalog information of the proposed valves). It is anticipated that a purchase order will be issued to a valve maintenance contractor for the installation of

the new valves, operational testing and onsite maintenance of the existing valves. The work is expected to consist of the following activities:

- Remove the blind flange from the end of each drain pipe
- Install a backup valve on each drain pipe
- Re-install the blind flange against the downstream end of the backup valve
- Close the backup valves
- Open and close the existing shutoff valves. If the handwheel actuator does not work, remove it and decide whether to repair it on site or send it to the shop for repair.
- After the actuators have been maintained and the shutoff valves have been opened and closed several times, and once they are closed, open the 3/4-inch ball valves and open the backup valves. Observe whether the 4-inch shutoff valves close leak tight or whether they leak. Document the performance of all valves.
- Close the 4-inch backup valves
- Close the 3/4-inch ball valves
- Install chains with padlocks to impede unauthorized operation of the shutoff valves.

#### 4.2.7 AMD Geochemistry

Results of the ongoing chemical analyses described in Section 3.7 will be used to evaluate the extent of possible deterioration of the concrete and steel in the seal. The chemical similarities and differences between samples will be assessed for indications of whether or not significant deterioration is occurring. The Upstream sample will be assumed to be a baseline representing background AMD. The Drain pipe and Seep samples will be compared to this baseline. Differences in chemistry will be evaluated and potentially attributed to steel corrosion and concrete dissolution, respectively.

To accomplish the comparison, several chemical markers will be used that should distinguish between baseline AMD and AMD containing steel or concrete dissolution products. These include calcium, magnesium, sodium, potassium, aluminum, iron, silica, molybdenum, zinc, copper, and other trace metals. If possible, dissolution rates for some of these constituents will be estimated, leading to order-of-magnitude estimates of the mass of concrete and steel dissolved per year.

#### 4.2.8 Review and Evaluation of Geotechnical and Structural Design

The geotechnical and structural review of the concrete seal will consider the original design studies as well as the testing and performance data developed during this investigation including the following:

- Observations during the on-site examination of the concrete seal, including the configuration and position of the seal in the tunnel, and the appearance and soundness of the seal concrete

- Results of nondestructive testing of the seal
- Observations from the examination of cores and core holes
- Results of laboratory strength testing of concrete and rock
- Observations of the rock immediately downstream of the seal and along the interface with the seal, including rock configuration, joint spacing and direction, and rock integrity
- Observations and test results regarding the integrity and strength of the rock-concrete interface.
- Geochemical evaluations of analytical data for the seepage water.

During the core drilling operation, the rock joints exposed immediately downstream of the plug will be mapped in detail to estimate the rock mass rating (RMR) and to establish a detailed baseline, thus documenting the current conditions for use in future monitoring events. The geometry of the rock surface in the adit immediately downstream of the seal will be correlated to the lengths of concrete and rock measured in the core holes to estimate the rock surface geometry at the interface with the seal. The observations and measurements will be correlated with the conditions disclosed by the 1987 video of the rock surface at the location of the seal before it was constructed.

The design parameters used for the original design of the concrete seal will be reviewed with regard to the data and observations outlined above, to confirm that these design parameters are still suitable. The purposes of the review are as follows:

- Evaluate whether the rock-concrete interface shear strength is consistent with the value used in the original design.
- Evaluate whether the water pressures measured in the core holes are consistent with the assumptions used in the original design.
- Evaluate whether the strength of the concrete is consistent with the value used in the original design.

### **4.3 Seal Testing and Evaluation Report**

A Seal Testing and Evaluation Report will be prepared once the additional investigations and analyses are concluded. The report will contain a description of the site conditions, a summary of pertinent seal design, construction and performance information, a description of the testing and evaluation activities conducted under this project, the data and findings obtained from these activities, data interpretation, conclusions and recommendations. The conclusions will address the existing and anticipated structural strength and stability of the seal, and in particular, the seal's characteristics listed in Section 2.1. The recommendations will address proposed seal improvements as well as suggested testing and analysis activities for the next 10-year inspection.

## 5. Limitations

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This report presents observations made and conclusions drawn from (1) a visual inspection 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 testing of the concrete in the seal. 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 inspection, 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 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 nondestructive 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.

## 6. Disclosure Statement

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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 first of a series of written reports to be prepared under the agreement. Additional reports to be prepared include a Health and Safety Plan Addendum, Monthly Status Reports, Subcontract Documents for various activities, and a Seal Testing and Evaluation Report.

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

<u>Subcontractor</u>	<u>Subcontract Amount</u>
Washington Group International, Inc.	\$ 57,100
Construction Technology Laboratories, Inc.	43,040
Walker and Associates, Inc.	15,000
Gary Mass	10,000
SDV-ACCI	10,500
Drilling and Grouting Subcontractor (to be determined)	60,000 (estimated)
Valve Purchase, Testing, Installation and Maintenance (to be determined)	10,000 (estimated)

## 7. References

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

American Concrete Institute, "Effect of Restraint, Volume Change, and Reinforcement on Cracking of Mass Concrete," ACI 207.2R-95.

Connell, A., Wanket, D., Mass, G.R., Pujol, A., and Christiansen, A., "Resistance of Concrete to Acidic Water," Proceedings of the AML 2000 Conference, Steamboat Springs, Colorado, September 2000.

Regional Water Quality Control Board, Videotape entitled "Walker Mine Seal Project", November 13, 1987.

State Water Resources Control Board, "Walker Mine Safety Plan," January 30, 1998.

Steffen Robertson and Kirsten, "Walker Mine Project Draft Final Feasibility and Design Report Contract No. 4-051-150-0," September 1985.

Steffen Robertson and Kirsten, "Walker Mine, Plumas County; Case No. 355 – Final Construction As-Built Report," March 7, 1989.

Steffen Robertson and Kirsten, Drawing No. 06901-01 Rev. B, "As-Built Walker Mine Plug Plan, Profile and Details," dated March 1989.

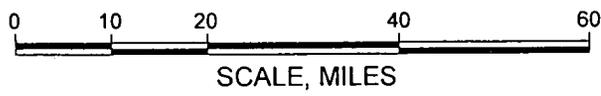
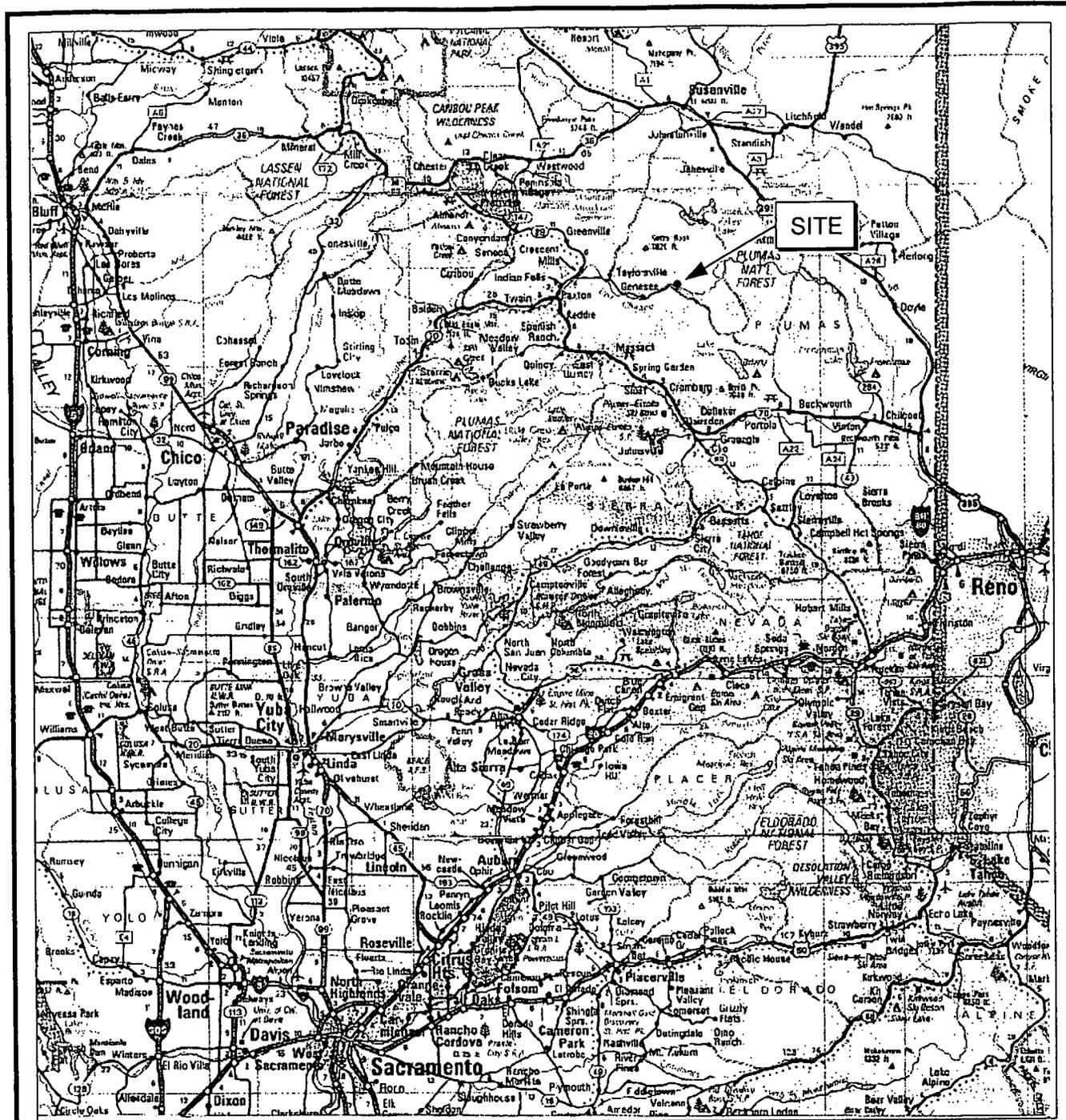
Steffen Robertson and Kirsten, "Walker Mine, Plumas County; Case No. 355, Attachment to Final Construction As-Built Report," September 1989.

Steffen Robertson and Kirsten, "Walker Mine – Site Inspection Report," December 1989.

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

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# FIGURES



**NOTE**

Map is taken from California road map dated 1998.

LOCATION OF SITE ABOVE

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



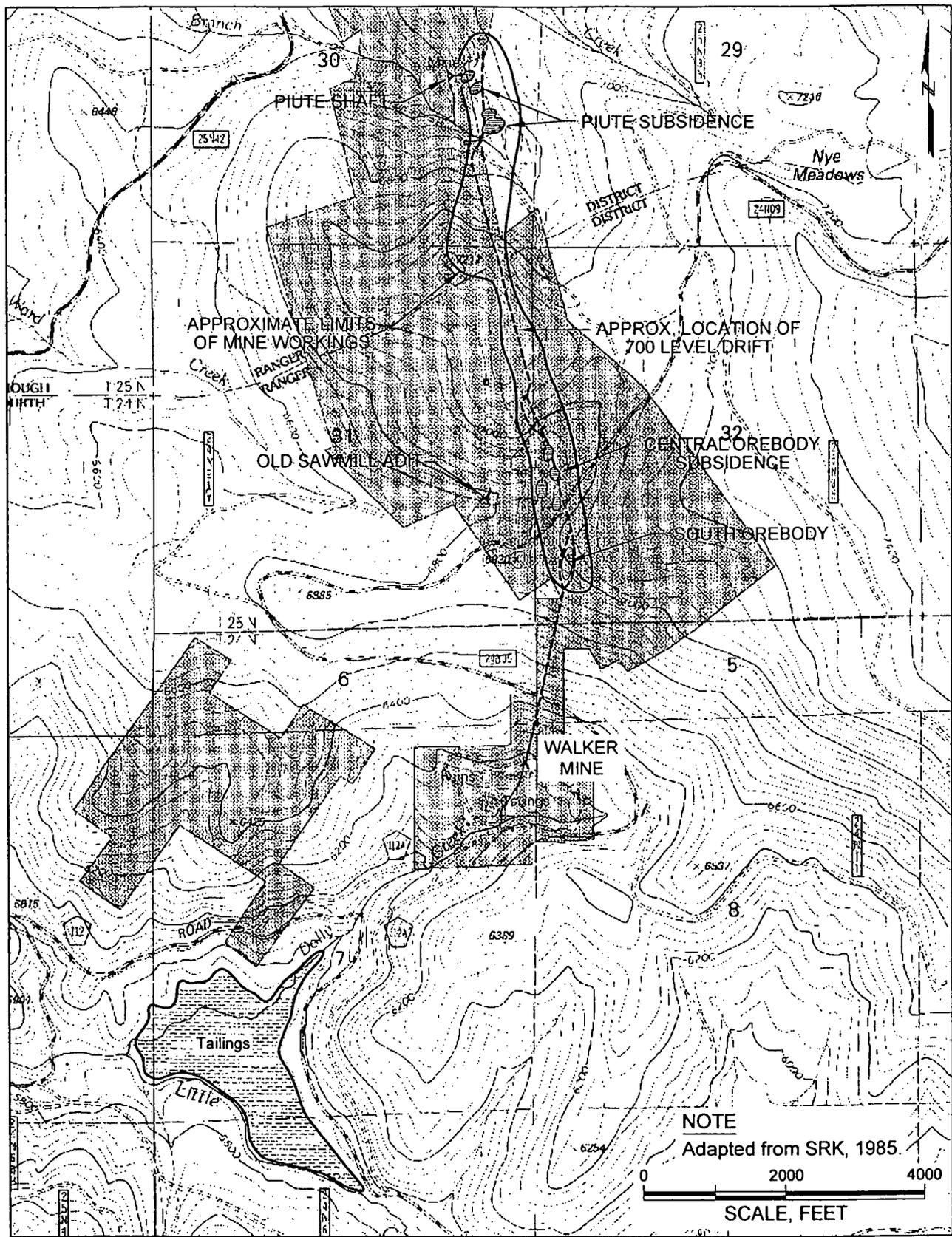


Fig-3.dwg 11-27-00 PYM

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

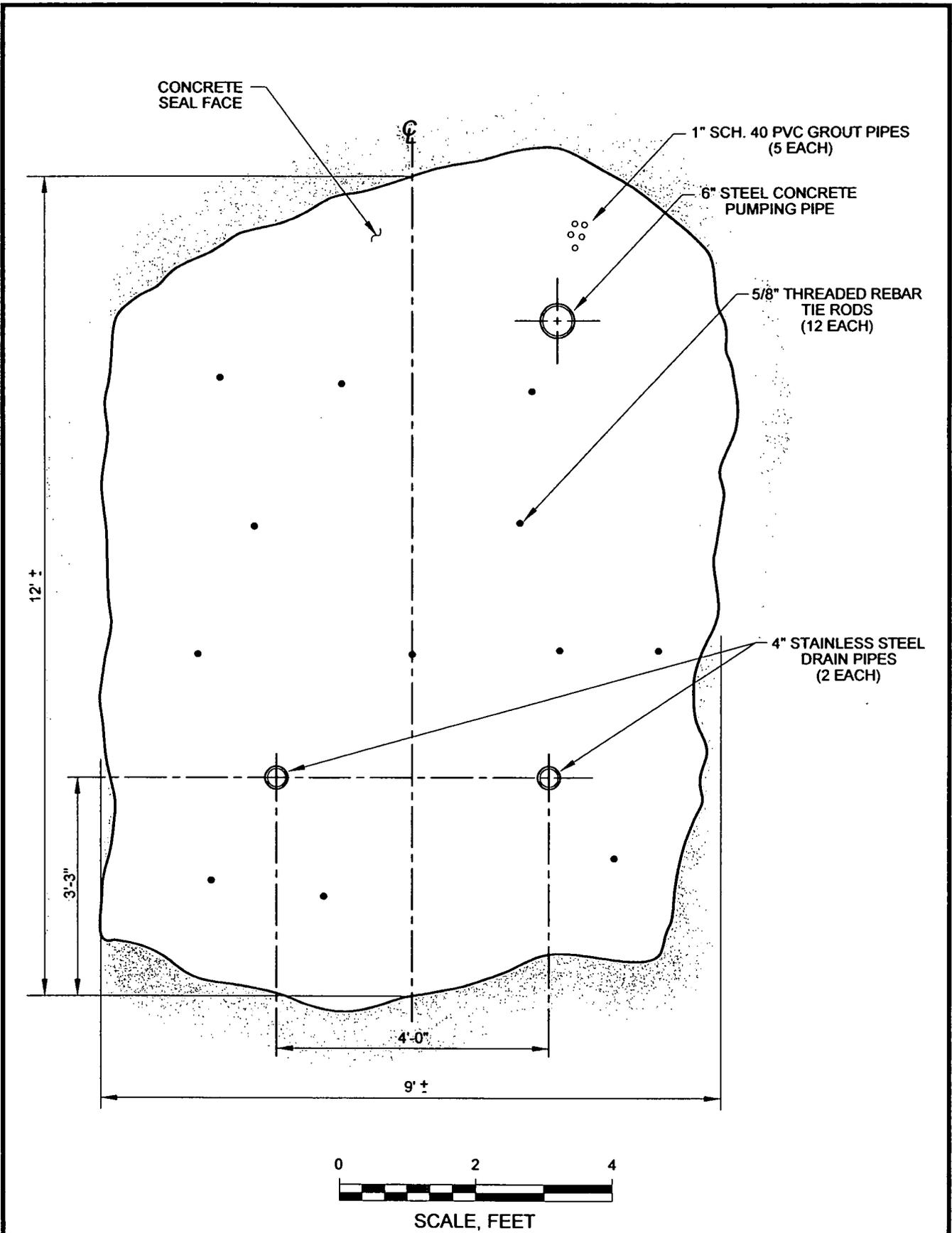
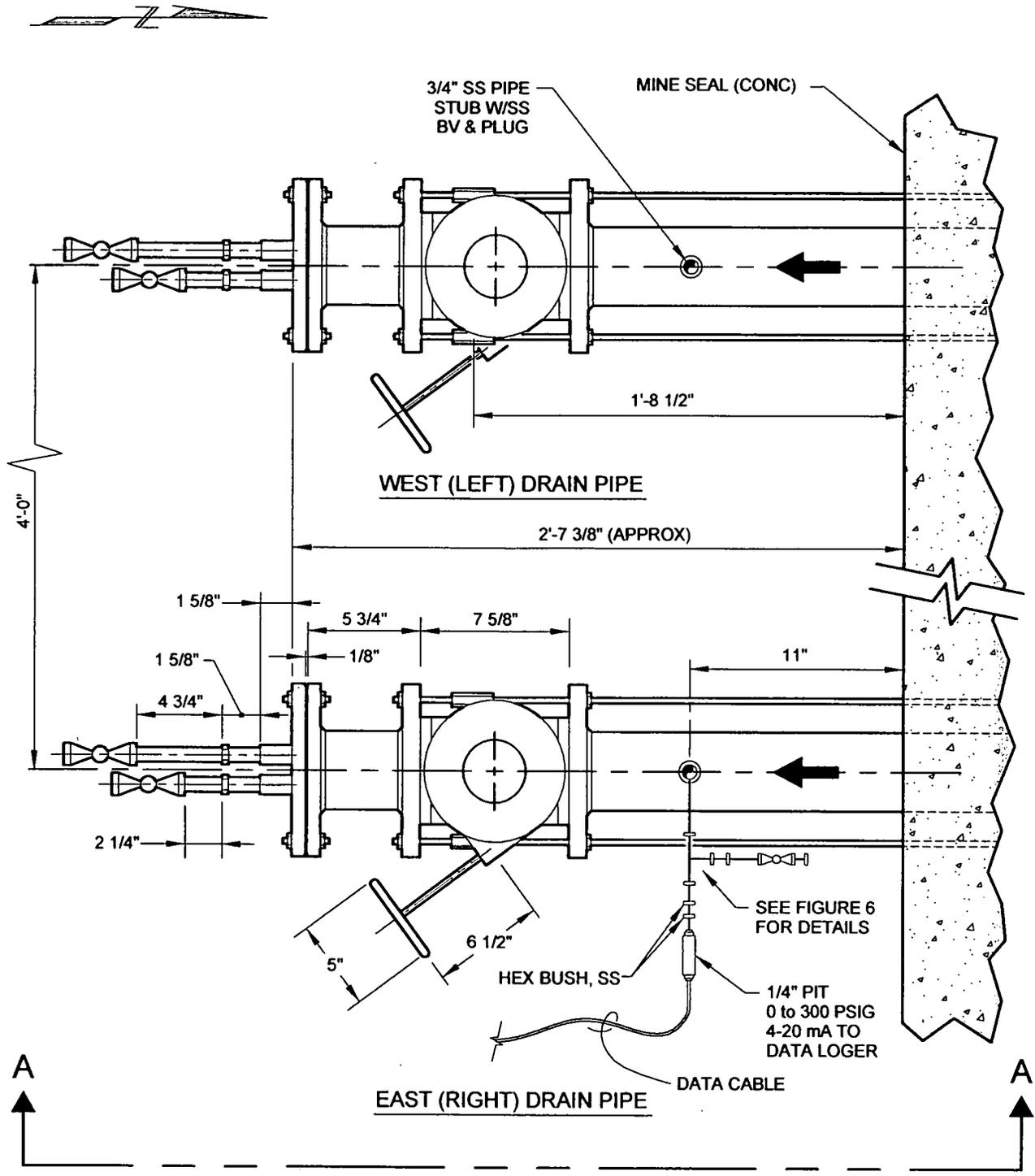


Fig-4.dwg 11-13-00 PYM

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	Nov. 2000	Figure 4



**NOTES**

1. SEE FIGURE 6 FOR LEGEND OF ABBREVIATIONS.

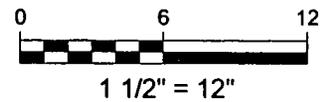
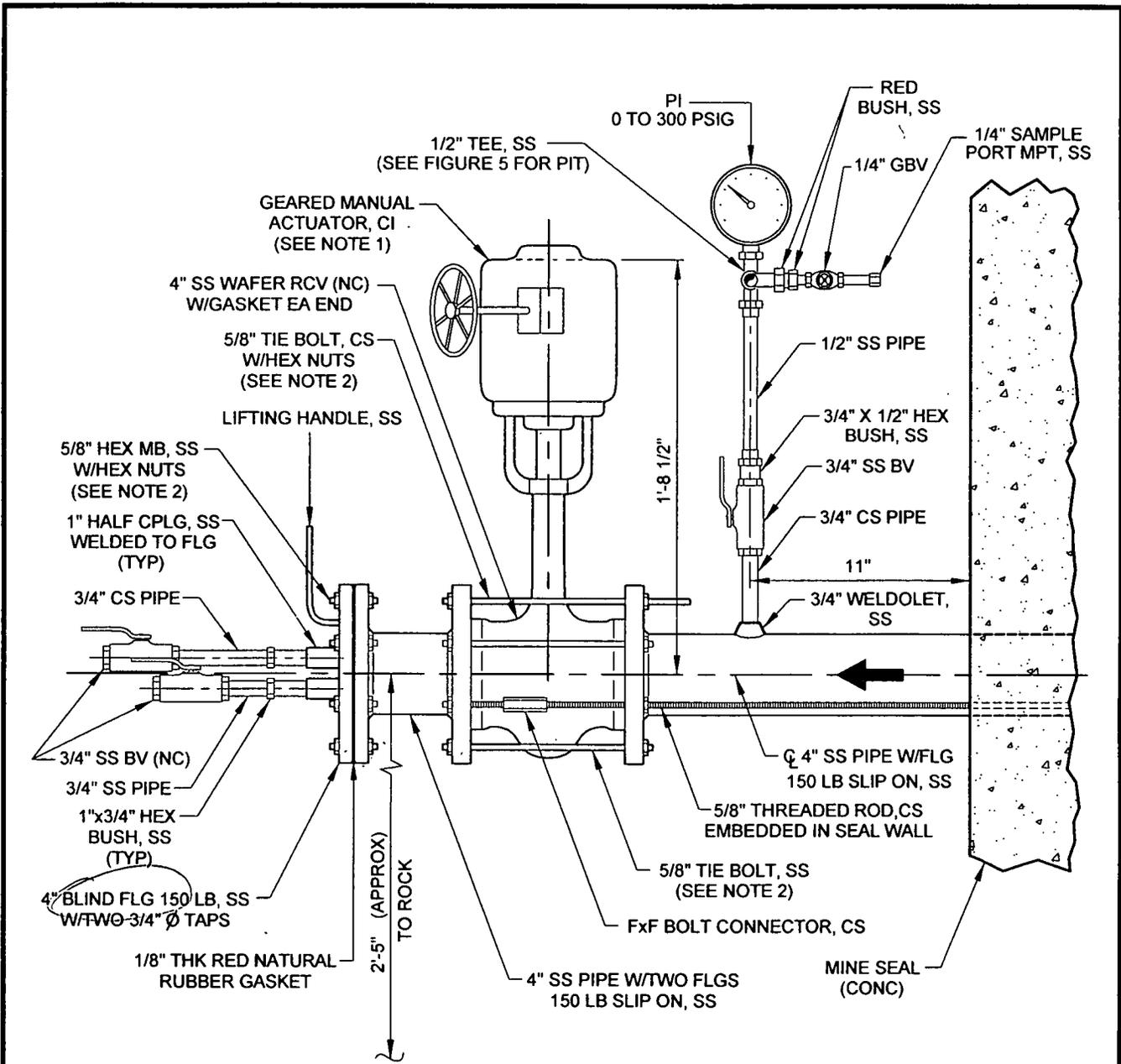


Fig-5.dwg 11-15-00 PYM

Regional Water Quality Control Board, Central Valley Region	Walker Mine Seal Testing & Evaluation Portola, CA	4-INCH DRAIN PIPES PLAN	
Washington Group International, Inc.		Nov. 2000	Figure 5
 <b>GEI Consultants, Inc.</b>	Project 00387		



**LEGEND**

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

**NOTES**

1. ACTUATOR HANDWHEEL LOCKED IN CLOSED POSITION WITH CHAIN AND PADLOCK.
2. SOME FLANGE FASTENERS ARE CARBON STEEL.

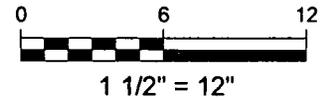
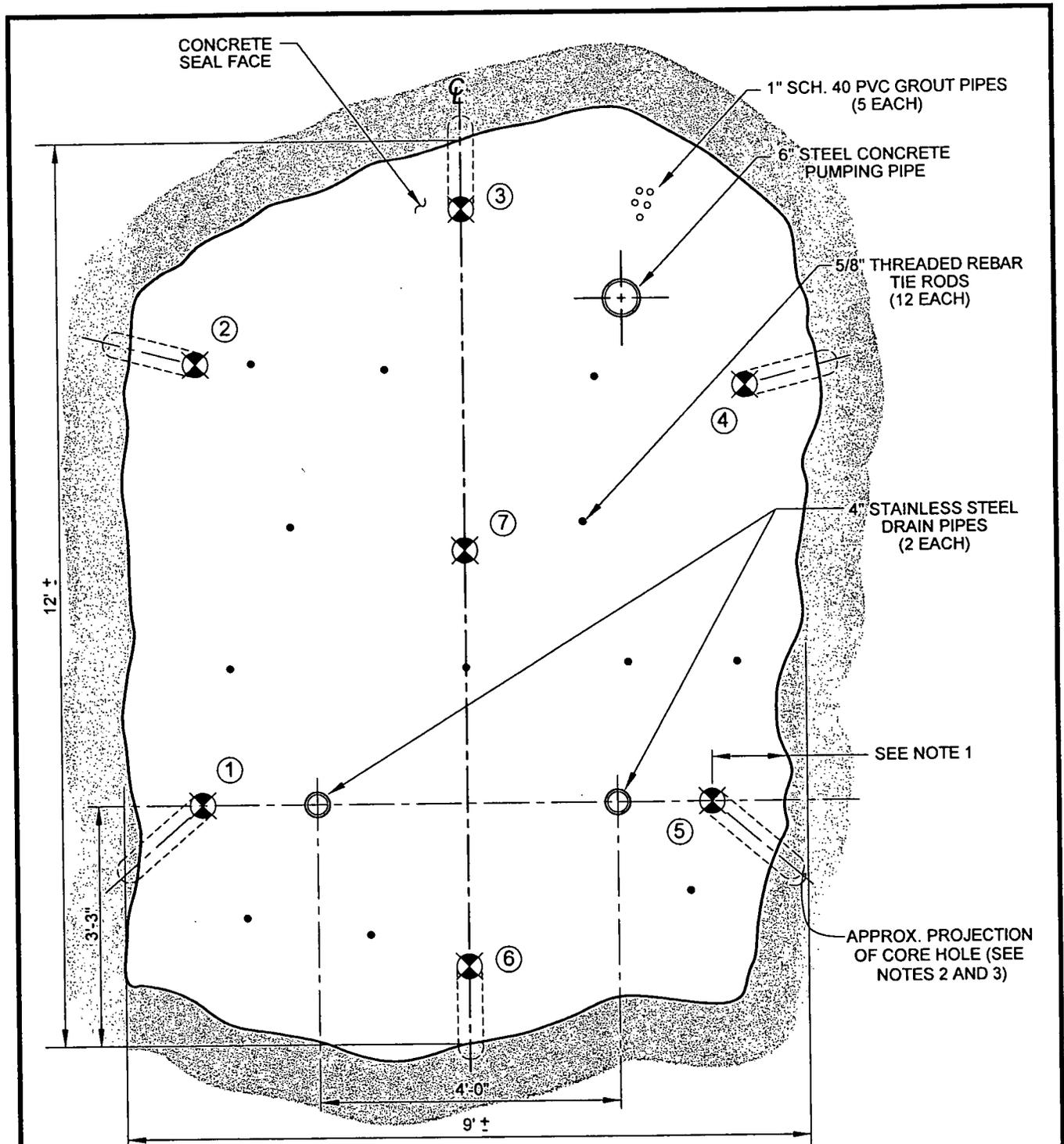


Fig-6.dwg 11-15-00 PYM

Regional Water Quality Control Board, Central Valley Region	Walker Mine Seal Testing & Evaluation Portola, CA	EAST (RIGHT) DRAIN PIPE ELEVATION A-A	
Washington Group International, Inc.	Project 00387	Nov. 2000	Figure 6
GEI Consultants, Inc.			



**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



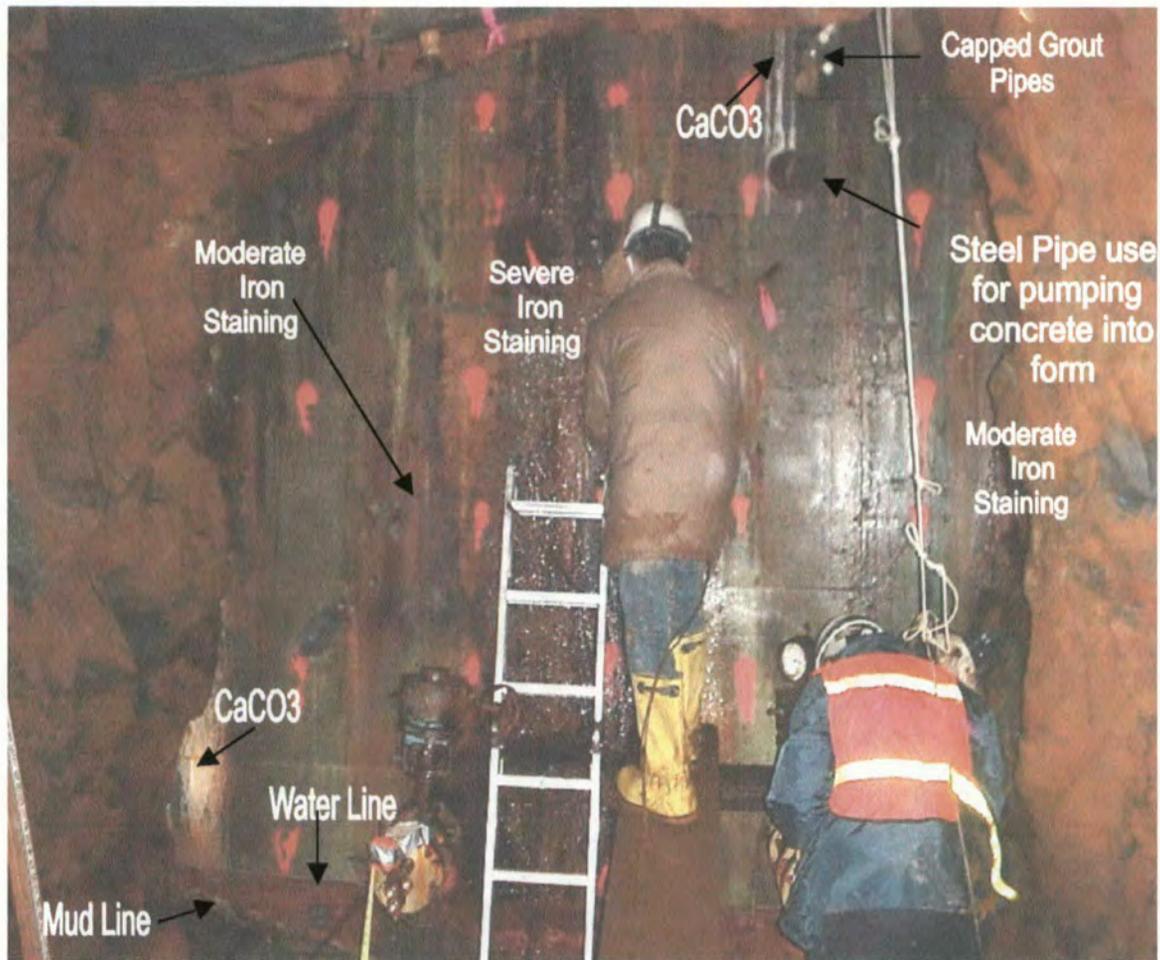
SCALE, FEET

Fig-7.dwg 12-04-00 PYM

Regional Water Quality Control Board Central Valley Region	Walker Mine Seal Testing & Evaluation Portola, CA	SUGGESTED BOREHOLE LOCATIONS AND ORIENTATIONS	
GEI Consultants, Inc.	Project 00387	Nov. 2000	Figure 7

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# PHOTOGRAPHS



**Photo No. 1 Walker Mine Concrete Seal. Red paint marks are impact points for nondestructive testing.**



**Photo No. 2 Bottom left quadrant of the seal.**



**Photo No. 3 Closeup of bottom left quadrant showing the water line of the seepage pool.**



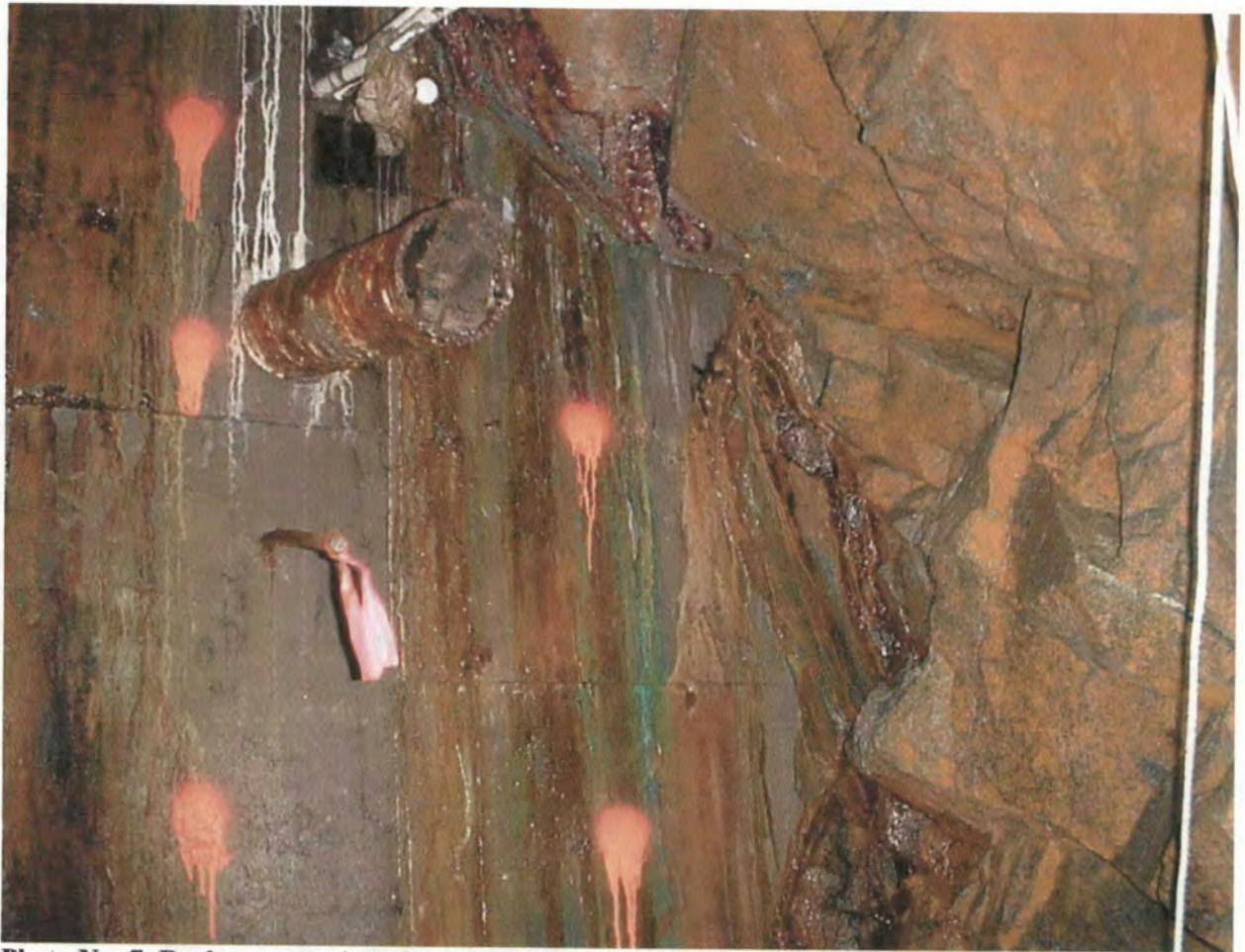
**Photo No. 4 Upper right quadrant showing the original concrete pumping pipe stub.**



**Photo No. 5** Upper part of seal and adit roof



**Photo No. 6 Closeup of rock-concrete interface at adit roof.**



**Photo No. 7 Rock-concrete interface along right side of seal**



**Photo No. 8 Seal face and chained-up valves on November 2. Seepage pool next to seal is practically dewatered.**



**Photo No. 9 Left drain pipe and valve with measuring tape attached.**



**Photo No. 10 Left drain pipe on November 2.**



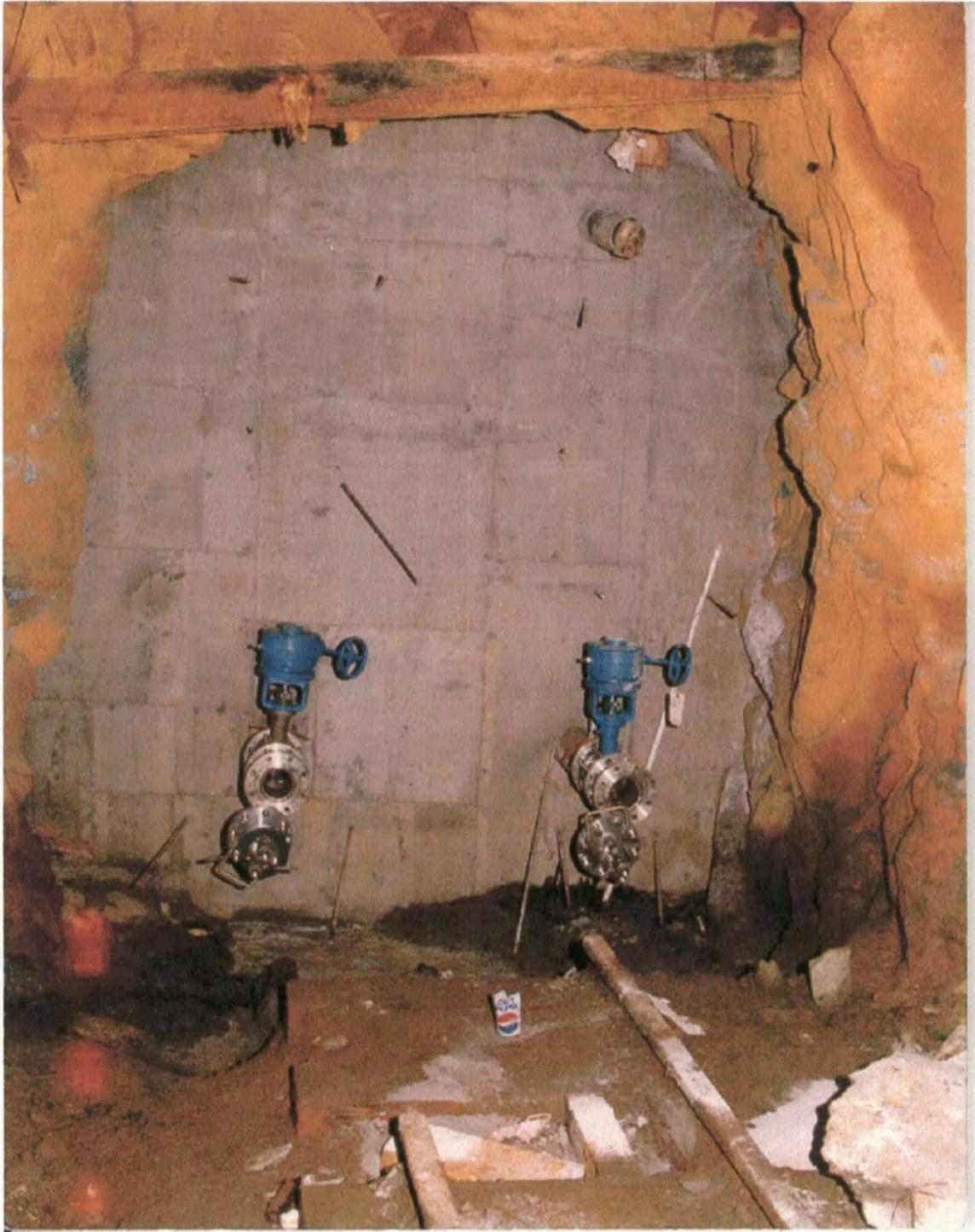
**Photo No. 11 Right drain pipe and valve**



**Photo No. 12 Right drain pipe on November 2.**



**Photo No. 13 Right drain pipe from top showing pressure gauge, sampling valve, pressure transducer and control valve operator.**



**Photo No. 14 Seal after construction (December 1987).**



# Appendix A

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**SRK's As-Built Drawing for Walker Mine Seal**





# Appendix B

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## Information Related to Simplified Thermal Study of Concrete Seal

- American Concrete Institute Publication No. 207.2R-95, "Effect of Restraint, Volume Change, and Reinforcement on Cracking of Mass Concrete"
- Thermal study calculations by G.R. Mass, dated November 8, 2000.

**ACI 207.2R-95**

# Effect of Restraint, Volume Change, and Reinforcement on Cracking of Mass Concrete

Reported by ACI Committee 207

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Terry L. West

<sup>\*</sup>Members of the task group who prepared this report.  
<sup>†</sup>Chairman of the task group who prepared the report.  
<sup>‡</sup>Deceased.

Members of the committee voting on proposed revisions:

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<sup>\*</sup>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

ACI Committee Reports, Guides, Standard Practices, and Commentaries are intended for guidance in designing, planning, executing, or inspecting construction and in preparing specifications. Reference to these documents shall not be made in the Project Documents. If items found in these documents are desired to be part of the Project Documents, they should be phrased in mandatory language and incorporated in the Project Documents.

rise (in concrete); tensile strength; thermal expansion; volume change; walls.

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- 2.2—Moisture contents and drying shrinkage
- 2.3—Ambient, placement, and minimum service temperatures
- 2.4—Placement temperature
- 2.5—Minimum temperature in service
- 2.6—Heat dissipation and cooling

ACI 207.2R-95 supersedes ACI 207.2R-90 and became effective January 1, 1995.  
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The 1995 revisions consisted of many minor editorial and typographical corrections throughout, as well as some additional explanatory information.  
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## 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.

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

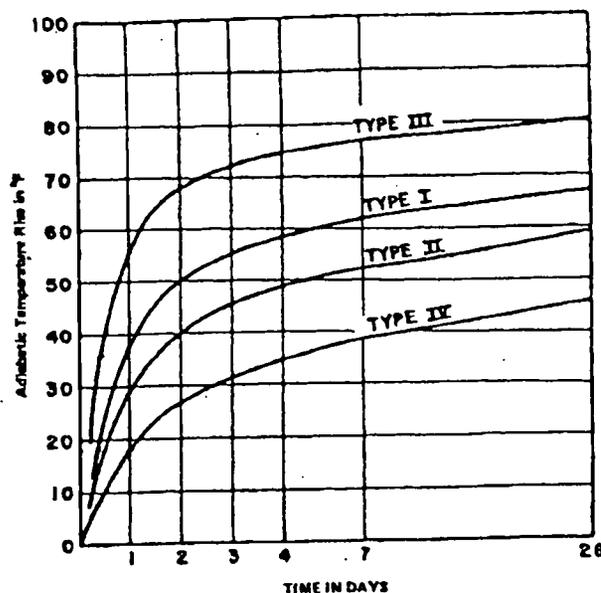
The thermal behavior of mass concrete has been thoroughly discussed in Chapter 5 of ACI 207.1R. This chapter's purpose is to offer some practical guidance in the magnitude of volume change that can be expected in reinforced concrete structures or elements. Such structures utilize cements with higher 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 de-



Cement Type	Fineness ASTM C 115 cm <sup>2</sup> /gm	28-Day Heat of Hydration Calories per gm
I	1780	67
II	1890	76
III	2030	106
IV	1910	60

Fig. 2.1—Temperature rise of mass concrete containing 376 lb of various types of cement per cubic yard of concrete

pends on many variables.

Fig. 2.1 shows curves for adiabatic temperature rise versus time for mass concrete placed at 73 F and containing 376 lb/yd<sup>3</sup> 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.<sup>2</sup> 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.

207.2R-4

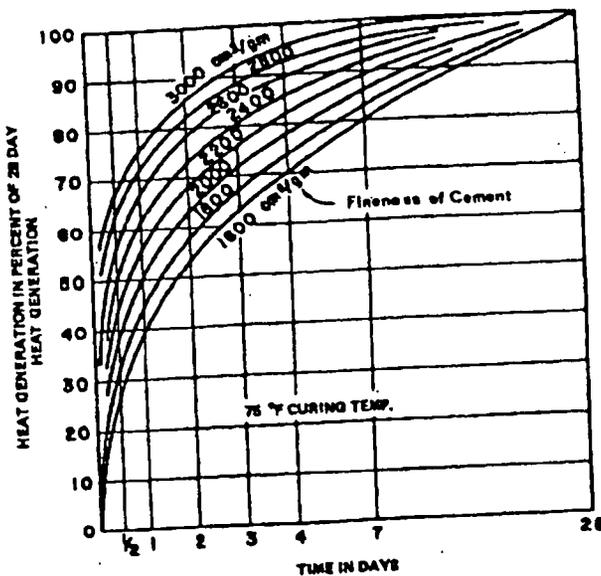


Fig. 2.2—Rate of heat generation as affected by Wagner fineness of cement (ASTM C 115) for cement paste cured at 75 F

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.<sup>3</sup>

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

$$H_a = \frac{1.8h_g \bar{w}_c}{0.22(150)(27)} \quad (2.1)$$

Where 0.22 in cal/gm-deg C and 150 in lb/ft<sup>3</sup> 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<sup>3</sup> to ft<sup>3</sup>.  $h_g$  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<sup>3</sup> of concrete. For a concrete mix containing 376 lb of cement per yd<sup>3</sup> of concrete:  $H_a = 0.76$  in degrees Fahrenheit.

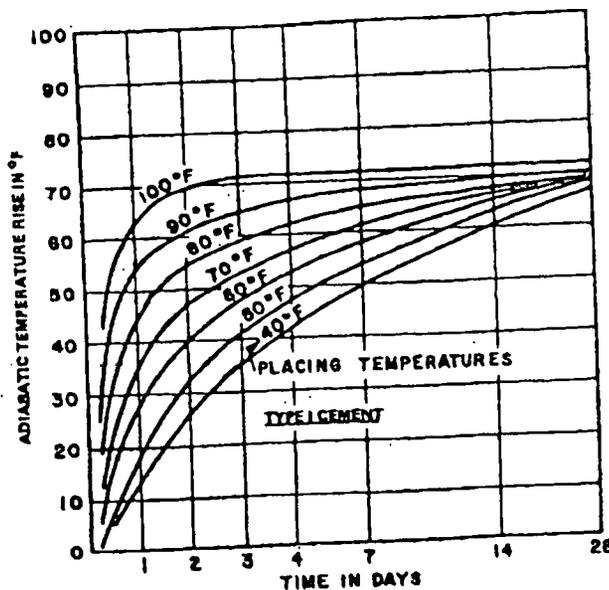


Fig. 2.3—Effect of placing temperature and time on adiabatic temperature rise of mass concrete containing 376 lb/yd<sup>3</sup> of Type I cement

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

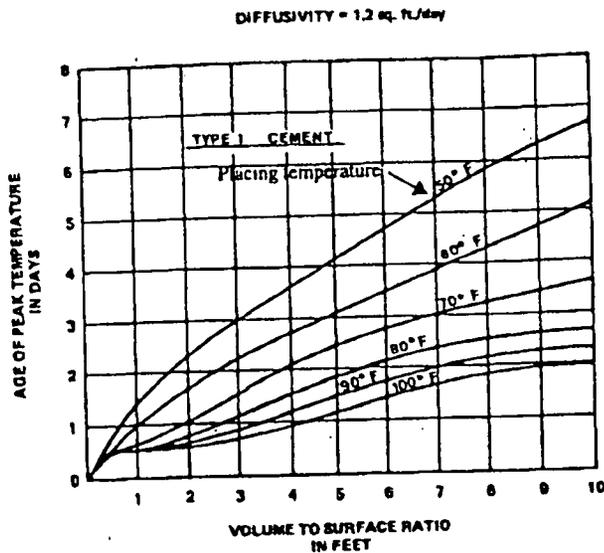


Fig. 2.4—Effect of placing temperature and surface exposure on age at peak temperature for Type I cement in concrete. Air temperature = placing temperature

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

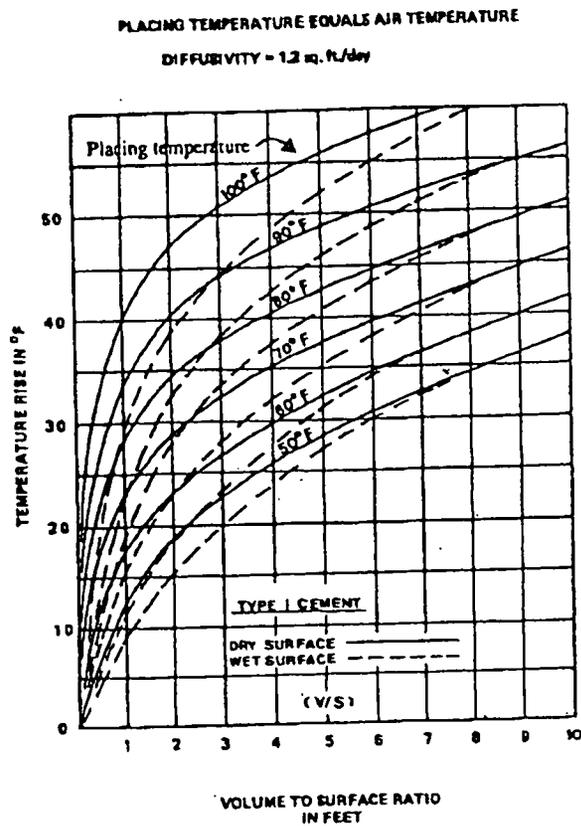


Fig. 2.5—Temperature rise of concrete members containing 376 lbs of cement per cubic yard for different placing temperatures

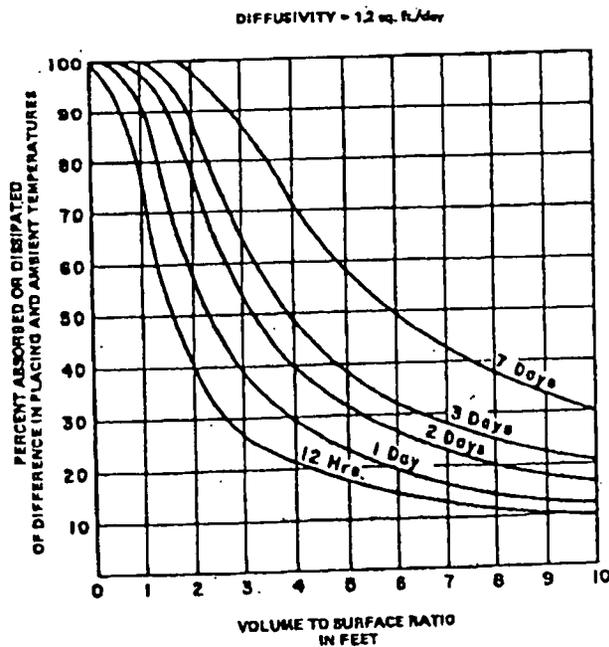


Fig. 2.6—Heat flow between air and concrete for difference between placing temperature and ambient air temperature

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<sup>2</sup>/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.<sup>4</sup> 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."<sup>5</sup> 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  $5 \times 10^{-6}$  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

$$T_{DS} = \left(30 - \frac{2V}{S}\right) \left(\frac{W_u - 125}{100}\right) \quad (2.2)$$

where

$T_{DS}$  = equivalent temperature change due to drying shrinkage, in deg F

$W_u$  = water content of fresh concrete, lb/yd<sup>3</sup>, but not less than 225 lb/yd<sup>3</sup>

$V$  = total volume, in.<sup>3</sup>

$S$  = area of the exposed surface, in.<sup>2</sup>

## 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 condi-

tions, 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

$$T_{min} = T_A + \frac{2(T_M - T_A)}{3} \sqrt{\frac{V/S}{96}} \quad (2.3)$$

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  $\text{yd}^3$ . 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:

- Establish ambient air temperature for summer placement based on locality. Assume 75 F average temperature.
- 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.
- Net effective placing temperature  $T_{pk} = 60 + 0.6(15) = 69$  F.
- Establish minimum exposure temperature for 1-week duration. Assume 20 F.
- 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.
- $T_{min} = 20 \text{ F} + \frac{1}{2}(60 - 20) \sqrt{24/96} = 33.5$  F, say 34 F.
- Difference =  $69 - 34 = 35$  F.

3. Determine the temperature rise:

- 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.
- From Fig. 2.1, correction for Type II cement peaking at 2 days =  $T_c = (40/50)(30) = 24$  F.
- Correction for mix.  $C_{eq} = 215 + 225/4 = 272$  lb,  $T_C + F = 24 \text{ F} (272)/(376) = 17.4$  F, say 18 F.
- 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  $\text{lb}/\text{yd}^3$  of a Type I cement, having a turbidimeter fineness of 2000  $\text{cm}^2/\text{gm}$  and 28-day heat of solution of 94  $\text{cal}/\text{gm}$ .

- 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.
  - 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.
  - Effective placing temperature =  $85 - 4 = 81$  F.
  - Minimum temperature of the concrete against rock = 34 F.
  - 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  $\text{cal}/\text{gm}$  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  $\text{lb}/\text{yd}^3$  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.

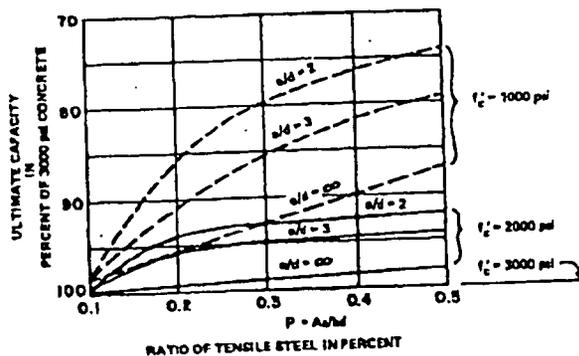
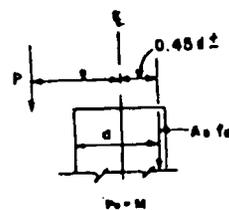


Fig. 3.1—Effect of concrete strength on ultimate capacity;  $f_y = 60,000$  psi

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

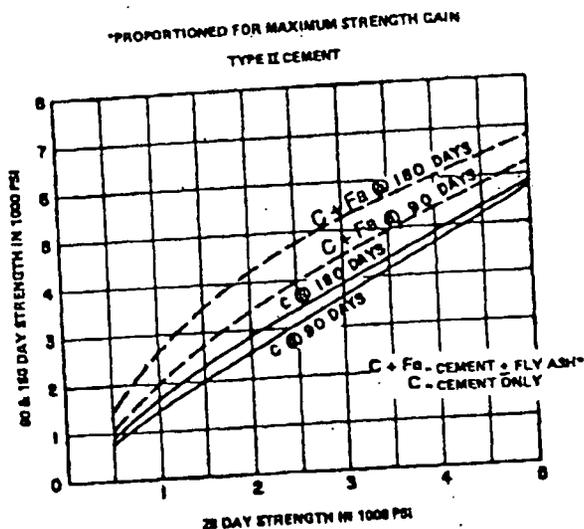


Fig. 3.2—Comparison of 28, 90, and 180-day compressive strength

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 section of the specimen rather than occurring in the weakest 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.<sup>6</sup>

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

### CRACKING OF MASSIVE CONCRETE

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

$$h^2 = \frac{K}{C_h \cdot w_c} \quad (3.1)$$

where

- $h^2$  = diffusivity, ft<sup>2</sup>/hr
- $K$  = conductivity, Btu/ft-hr-F
- $C_h$  = specific heat, Btu/lb-F
- $w_c$  = weight of concrete, lb/ft<sup>3</sup>

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  $5 \times 10^{-6}$  in./in./F for calcareous aggregate,  $6 \times 10^{-6}$  in./in./F for silicious aggregate concrete, and  $7 \times 10^{-6}$  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 restraint,  $K_R$ , 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.

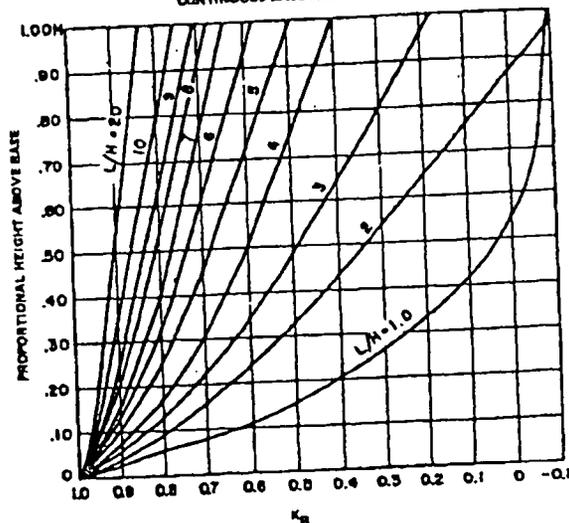
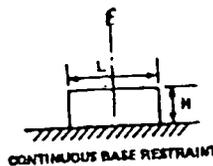


Fig. 4.1—Degree of tensile restraint at center section

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-

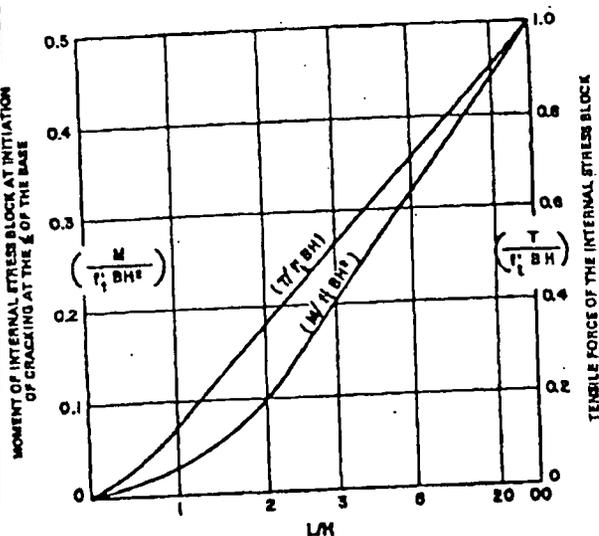


Fig. 4.2—Internal forces at initiation of cracks at restrained base

ported in 1940 by Carlson and Reading.<sup>4.7</sup>  
 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

$$K_R = [(L/H - 2) / (L/H + 1)]^{h/H} \quad (4.1)$$

For  $L/H$  less than 2.5, restraint  $K_R$  at any point may be approximated by

$$K_R = [(L/H - 1) / (L/H + 10)]^{h/H} \quad (4.2)$$

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

$$f_t = K_R \Delta_c E_c \quad (4.3)$$

where

$K_R$  = degree of restraint expressed as a ratio with 1.0 = 100 percent

$\Delta_c$  = contraction if there were no restraint

$E_c$  = sustained modulus of elasticity of the concrete at the time when  $\Delta_c$  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

$$\text{Multiplier} = \frac{1}{1 + \frac{A_g E_c}{A_F E_F}}$$

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

CRACKING OF MASSIVE CONCRETE

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, 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  $0.2f'_cBH^2$ . 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 non-flexural 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  $0.2f'_cBH^2$  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  $0.2f'_cBH^2$  minus the restraining moment of Fig. 4.2 for  $L/H = 2 L'/H$ .

A linear approximation of this difference can be determined by

$$M_{RH} = 0.2f'_cBH^2 \left( 1 - \frac{L'}{2H} \right) \quad (4.4)$$

where

- $M_{RH}$  = restraint moment required of reinforcing steel for full-height cracking
- $f'_c$  = tensile strength of concrete
- $H$  = height of block

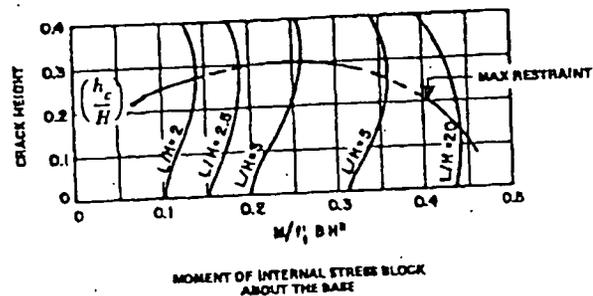
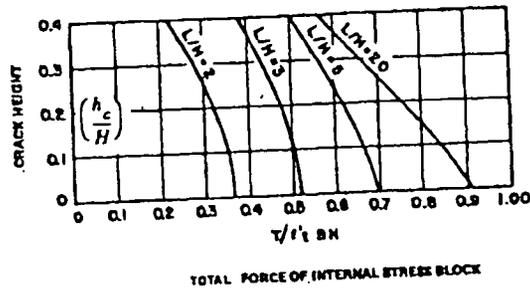
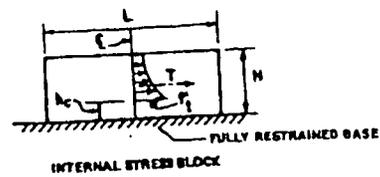


Fig. 4.3—Effect of crack propagation on internal forces

$B$  = 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-

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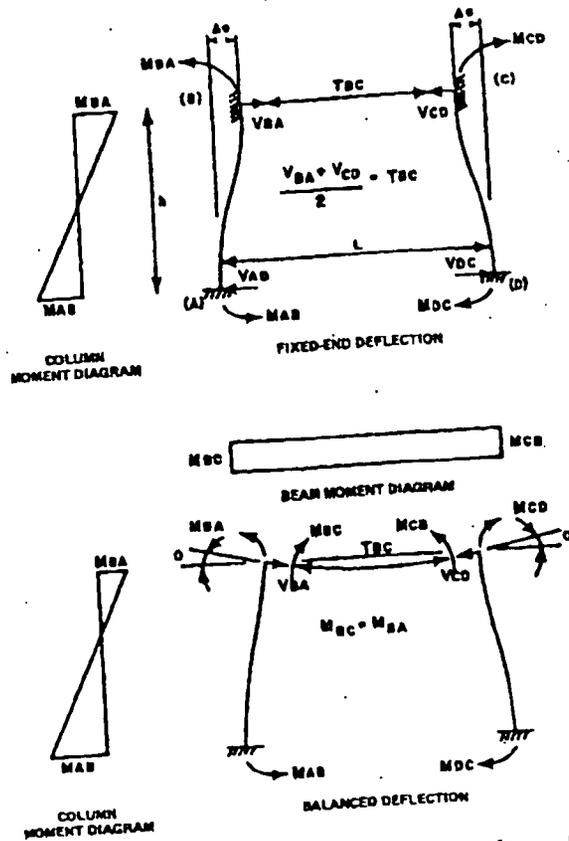


Fig. 4.4—Flexure of a simple frame induced by beam shortening

terminated 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

$$K_R = \frac{1}{1 + \frac{A_B h^3}{4L I_c}} \quad (4.5)$$

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

$$\Delta f_s = \frac{K_R C_T T_E E_S}{2pnj} \left[ h \left( \frac{K_f}{K_f + K_c} \right) + 4pnj \right] \quad (4.6)$$

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

$$\Delta f_s = 2K_R C_T T_E E_S \quad (4.7)$$

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

$$FEM = (T_1 - T_2) C_T E_c b d^2 \left( \frac{jk^2}{2} \right) \quad (4.8)$$

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  $f_t' B H^2 / 10$ . (Fig. 4.6 shows internal restraint.) The balancing external restraining moment depends entirely on the weight of the concrete and

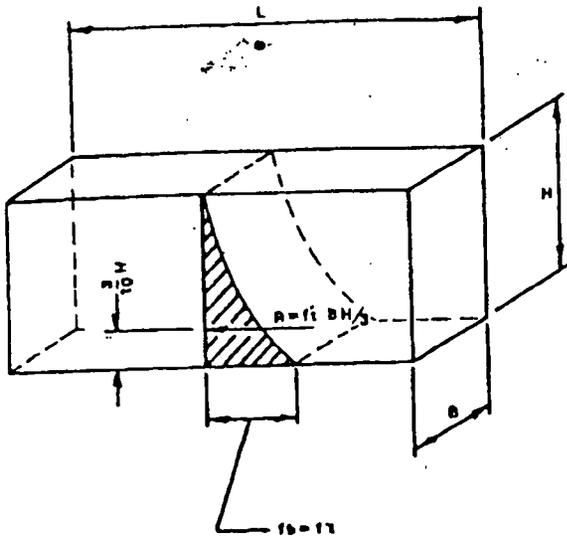


Fig. 4.5—Internal stress distribution of slabs on low-strength subgrade

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<sup>3</sup>, 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

$$A_s = \frac{T}{f_s} \geq \frac{f_t' BH}{3f_s} \quad (4.9)$$

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

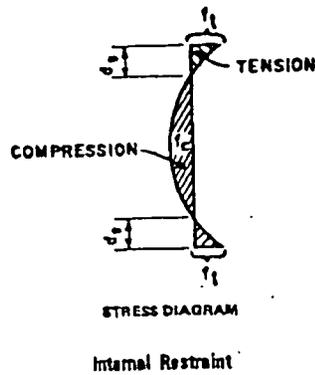
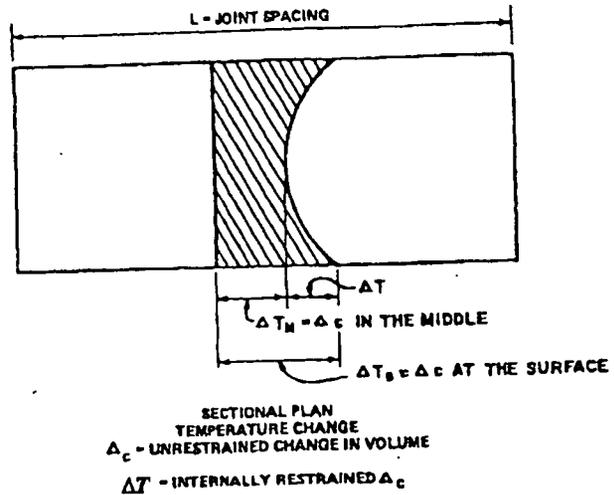


Fig. 4.6—Internal restraint

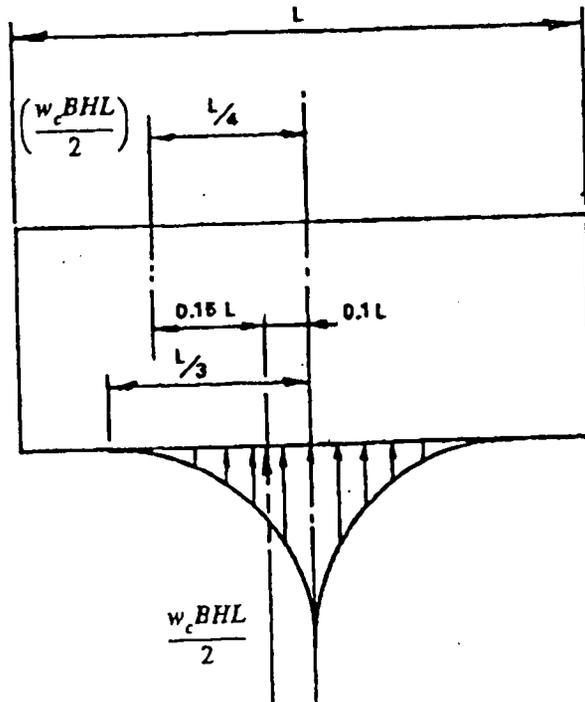


Fig. 4.7—Pressure distribution and restraining moments of curling slab

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  $3 \times 10^6$  psi and a coefficient of thermal expansion of  $5 \times 10^{-6}$  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).<sup>8</sup>

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.<sup>9</sup> 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 us-

## CRACKING OF MASSIVE CONCRETE

ing 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<sup>10</sup> reported in ACI SP-20.

$$w = 0.076 \sqrt[3]{d_c A} \beta f_s 10^{-3} \quad (5.1)$$

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 ( $2d_c \times$  spacing), in.<sup>2</sup>  
 $B$  = distance from 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  $B$ -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

$$w = 0.10 f_s \sqrt[3]{d_c A} 10^{-3} \quad (5.2)$$

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 choose 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

$$z = f_s \sqrt[3]{d_c A} \quad (6.1)$$

where

$f_s$  = calculated stress in reinforcement

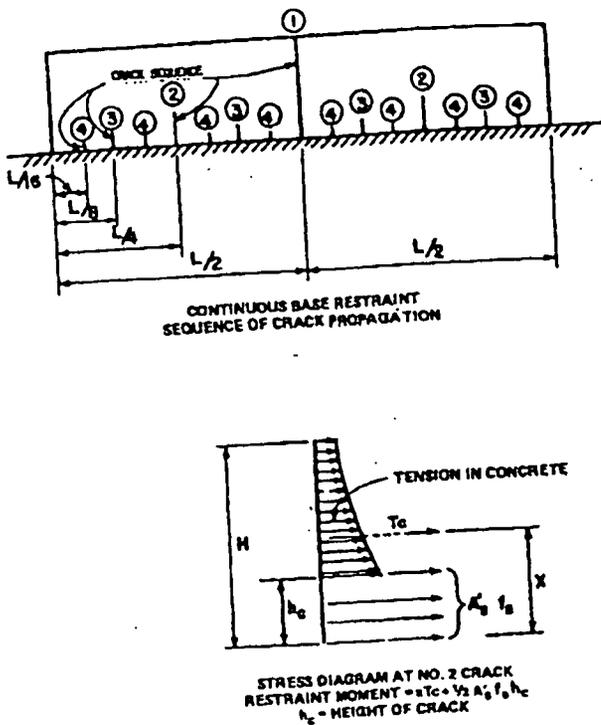


Fig. 6.1—Sequence of crack propagation and distribution of stress at No. 2 crack

- $d_c$  = 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  $\beta$  of 1.0 by assuming a bar cover and spacing and calculating the stress in reinforcement  $f_s$  from

$$f_s = \frac{w \times 10^3}{0.076 \sqrt[3]{d_c A}} \quad (\text{in ksi}) \quad (6.2)$$

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  $L f'_t / E_c$ .

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<sup>11</sup> 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

$$\frac{N \cdot w}{1.5} = 12L (K_R C_T T_E - f'_t / E_c) \quad (6.3)$$

for  $L$  in ft. If the average crack spacing equals  $L'$ , then  $NL' = L$  and

$$L' = \frac{w}{18 (K_R C_T T_E - f'_t / E_c)} \quad (6.4)$$

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)

$$M_{Rh} = 0.20 f_i' B h^2 \left( 1 - \frac{L'}{2h} \right) \quad (6.5)$$

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

$$A_b = 0.4 \frac{f_i' B h}{f_s N_H} \left( 1 - \frac{L'}{2h} \right) \quad (6.6)$$

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  $5 \times 10^{-6}$  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

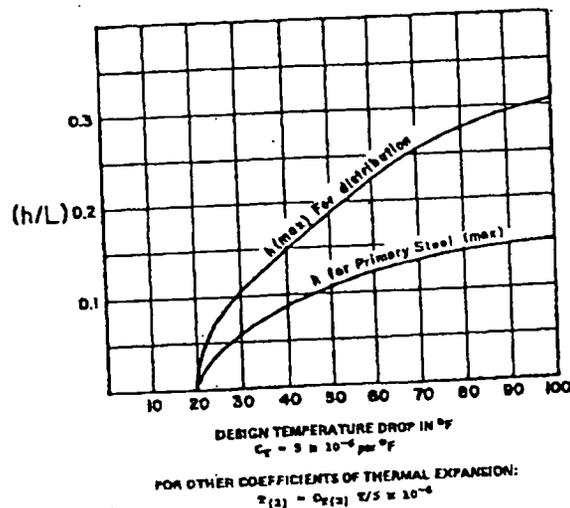
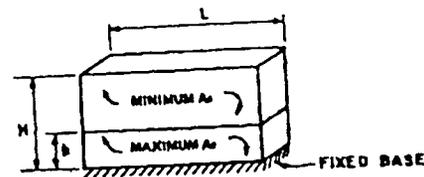


Fig. 6.2—Wall height requiring maximum temperature and shrinkage reinforcement as a ratio of base length

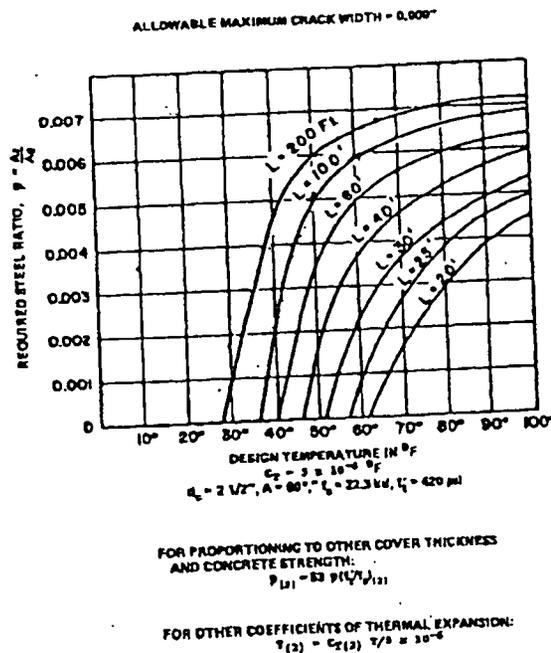
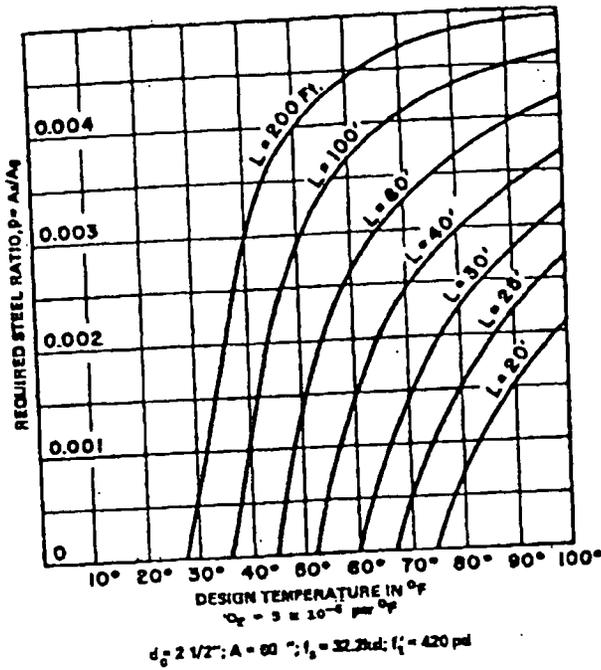


Fig. 6.3—Maximum temperature and shrinkage reinforcement for walls with fixed bases

Section 6.4. Only minimum steel is required where  $L \leq 2h$ . 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  $C_T = 5 \times 10^{-6}/F$ . 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.

ALLOWABLE MAXIMUM CRACK WIDTH = 0.017"

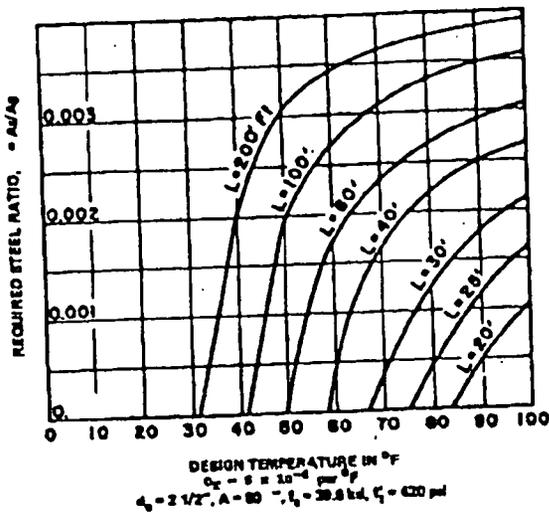


FOR OTHER COVER THICKNESS AND CONCRETE STRENGTH:  
 $P_{(2)} = 77 (f'_c/A_g)_{(2)}$

FOR OTHER COEFFICIENTS OF THERMAL EXPANSION:  
 $\epsilon_{(2)} = \epsilon_{(1)} \frac{2}{5} \times 10^{-5}$

Fig. 6.4—Maximum temperature and shrinkage reinforcement for walls with fixed bases

ALLOWABLE MAXIMUM CRACK WIDTH = 0.016"



FOR PROPORTIONING TO OTHER COVER THICKNESS AND CONCRETE STRENGTH:  
 $P_{(2)} = 84 (f'_c/A_g)_{(2)}$

FOR OTHER COEFFICIENTS OF THERMAL EXPANSION:  
 $\epsilon_{(2)} = \epsilon_{(1)} \frac{2}{5} \times 10^{-5}$

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

$$A_b = 0.20 \frac{f'_c}{f_s} \left(1 - \frac{L'}{2H}\right) \frac{BH}{N_B \left(\frac{H-d_c}{H}\right)} \quad (6.7)$$

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

$$A_b = 0.20 \frac{f'_c}{f_s} \left(1 - \frac{L'}{2H}\right) \frac{BH}{N_B \left(\frac{H-d_c}{H}\right) + \frac{N_H}{2}} \quad (6.8)$$

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 $\times 10^{-6}$			
	4	5	6	7
100	30 F	24 F	20 F	17 F
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

of Table 6.3.1, or when a larger spacing of contraction joints is desired, the utilization of crack control measures discussed in Section 5.1.1 in conjunction with these limits may be used to control the width of cracks in between contraction joints.

**6.3.2 Discontinuous external or end restraint**—Cracking will occur when the stress induced in the concrete by volume change exceeds the tensile strength of concrete as described in Section 4.3.3. When more than one crack is required to control crack widths, the total force in the reinforcing steel must equal the total restraint force induced at ends of the member. For members with continuous base support, this will require a minimum steel ratio of

$$p(\min) = f'_1 / f_s \quad (6.9)$$

More than one crack will be required when the permissible crack width is less than the volume change  $LC_T T_E$ .

For members spanning between supports, the change in stress of the bottom face reinforcement due to thermal cracking may be determined from Eq. (4.6) of Section 4.3.1.

When the volume change is less than the permissible crack width, no steel is required for volume change except as may be required as minimum (see Section 6.4).

**6.3.3 Internal restraint**—For relatively large masses, the spacing of surface cracks will be controlled by internal restraint as described in Section 4.4. These cracks, independent of external restraint conditions, are not deep enough to require more than nominal amounts of reinforcing near the surface to control crack widths. In the example given in Section 4.4, the surface tensile stress due to daily temperature fluctuations was more than the surface stress due to the annual change in temperature. The depth of tensile stress block for the daily temperature fluctuations was less than 2.5 ft in the example. If this is assumed as the maximum depth of the critical restraint plane for internal restraint, then a maximum surface crack spacing in large masses of approximately 5 ft can be expected. If  $LC_T T_E$ , using the maximum normal daily temperature fluctuation for  $T_E$ , is less than  $w/12L'$ , for  $L'$  in ft, then no surface reinforcement is required (note  $L'$  should not be taken as more than 5 ft). If  $C_T T_E > w/12L'$ , then the minimum steel requirements of Section 6.4 should probably be utilized.

When internal restraint results from exposure of projecting elements from warm interiors, such as slabs projecting through exterior walls or walls projecting out of the ground, determine the depth of the tensile stress block and restraint factor as outlined in Section 4.4.1. If the required average crack spacing is less than twice the depth of the tensile stress block determine the size of bars to be distributed throughout the tensile stress block by

$$A_b = \frac{1f'_1 B d_s}{3f_s N_H} \quad (6.10)$$

where  $N_H$  = the total number of bars distributed throughout  $d_s$ .

**6.4—Recommendation for minimum reinforcement**

The minimum requirements of ACI 318 should apply to all superstructure slabs and beams. The minimum total quantity of temperature and shrinkage reinforcement recommended for walls, slabs, and footings less than 48 in. thick, which have been investigated for crack control by the measures outlined herein, is 0.0015 times the cross sectional area  $A_g$  of the member. When shrinkage-compensating cement is used, the recommendation of ACI 223 for design of reinforcement should be followed. Not less than one-half nor more than two-thirds of the total quantity of reinforcement should be placed in any one face. For crack control the maximum bar spacing should be limited to 12 in. on center. For members more than 48 in. thick the minimum temperature and shrinkage requirements in each face should be limited by depth of cover  $d_c$  and bar spacing such that

$$A_s' = \frac{f'_1 A}{f_s} \text{ or } \frac{A}{100} \text{ (as a limit for } f'_1/f_s) \quad (6.11)$$

The minimum bar size and spacing for members of this size should not be less than #6 bars at 12 in. on center.

No minimum temperature and shrinkage reinforcement is required for members 6 ft or more in thickness that are constructed by the principles and practices of ACI 207.1R to control the cracking of mass concrete provided the environmental conditions are such that cracking from internal restraint as discussed in Section 6.3.3 is not significant.

**6.5—Design procedure**

The basic procedure for problem solving is:

1. Determine the maximum effective temperature as outlined in Section 2.6.
2. Determine the restraint characteristics of the element or structure as outlined in Chapter 4.
3. Determine the physical properties of the concrete; tensile strength, elastic modulus, and coefficient of thermal expansion as outlined in Chapter 3.
4. Determine the allowable maximum crack width from Section 5.2 or by some other established criteria.
5. Determine the area of steel required to maintain cracking at the acceptable level.

a. For members subject to continuous external edge restraint determine the required average crack spacing for the height of slab or height intervals of 5 to 10 ft above the base of walls as per Section 6.3.1. Where the required crack spacing is less than the spacing of joints, provide reinforcement as per Section 6.3.2. In lieu of this the reinforcement in walls may be proportioned using Fig. 6.2 through 6.5. When the element or member is of sufficient size to require more than 1.5 in.<sup>2</sup>/ft of temperature reinforcement or when its cost exceeds one-third the cost of concrete (excluding formwork), additional measures to control volume change should be applied as recommended in ACI 207.1R.

b. For discontinuous external or end restraint, reinforcement will be required if  $w \leq LC_T T_E$ . If the member is subject to flexure, determine the change in steel stress as per

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<sup>3</sup> 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 (in F/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  $f'_t = 6 \sqrt{4500} = 402$  psi; and tensile strain capacity =  $150 \times 10^{-6}$  in./in., assume  $C_T = 5 \times 10^{-6}$  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,  $\rho = 0.007$  and  $\rho_2 = 53\rho$  (402/22,000), or  $\rho_2 = 0.0068$ , since tensile strength is 402 psi and not 420 psi.  $A_b = 0.0068B \times 12/2 = 0.0408B$  and at  $h = 5$  ft,  $A_b = 0.0408 \times 38 = 1.55 \text{ in.}^2$

h, ft	Average B, in.	$K_R$ (L = 200 ft) Fig. 4.1	$L'$ 6.4*	$L'/2h$	$A_b$ 6.6*	With adjusted $\rho$ $A_b$ 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<sup>3</sup> @ \$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 $\rho$ $A_b$ 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.  
\*  $A_b/\text{min.} = A_b \times 0.0015/N_b$  (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,  $\rho_2 = 53(f/f'_t) = 53(0.003) \times (416/22,000) = 0.0030$

$$A_b = \rho_2 B \times 12/2 = 0.018 B$$

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.	$K_R (L=200 \text{ ft})$ Fig. 4.1	$L'$ 6.4°	$L'/2h$	$A_b$ (minimum steel)	With adjusted $\rho$ $A_b$ 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 yd<sup>3</sup> @ \$70 ..... \$42,500  
 Temperature reinforcement = 6.5 tons @ \$800 ..... 5,200  
 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)]

$$\frac{A_b h^3}{4L_b J_c} = \frac{(1)(3)(20)^3}{4(20)(1)(4)^3/12} = 56 \pm$$

$$\therefore K_R = \frac{1}{1+56} = 0.0175$$

Step 3— $f'_i = 405 \text{ psi}$   $C_T = 5 \times 10^{-6} \text{ in./in./F}$

Step 4—Assume  $w = 0.013 \text{ in.}$ ,  $d_c = 2 \text{ in.}$   $A_{dc} = 125$  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)]

$$\rho = \frac{1.20}{12(33)} = 0.003, n = 9, j = 0.94, h = 20 \text{ ft}$$

$$K_f = (3)^3/20 = 1.35, K_c = (4)^3/20 = 3.2$$

$$\Delta f_s = \frac{0.0175(5 \times 10^{-6})(68)(29 \times 10^6)}{2(0.003)9(0.94)}$$

$$\left[ \frac{20}{3} \left( \frac{1.35}{1.35+3.2} \right) + 4(0.003)9(0.94) \right] = 7000 \text{ psi}$$

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 3-1/4 in. cover to center of temperature steel or  $f_s = 26,000 \text{ psi}$  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  $f'_i/E$  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,  $V/S = 0.75(6 \text{ ft}) = 4.5 \text{ ft}$  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  $T_{PK} = 82 \text{ F}$ ; temperature rise = 41 F. From Example 2.2; Adjustments for cement type = 41/30 = 1.38; adjustments for cement content 470/376 = 1.25.

$\therefore$  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 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:

$\Delta f_s$  for flexure (Eq. 4.7)

$$\Delta f_s = 2(0.9) (5 \times 10^{-6}) (98) (29 \times 10^6) = 25,600 \text{ psi}$$

$$\Sigma f_s = 24,000 + 25,600 = 49,600$$

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.<sup>2</sup>/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.

$$A_s = 1.56 \left( \frac{24}{38.5 - 25.6} \right) = 2.90 \text{ in.}^2/\text{ft} \#11 @ 6 \text{ in. OK}$$

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,  $5 \times 10^{-6}$  per F for limestone aggregate,  $6 \times 10^{-6}$  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

MANUAL OF CONCRETE PRACTICE

2R-28

- $T_1$  = high temperature in a temperature gradient
- $T_2$  = low temperature in a temperature gradient
- $V$  = volume of a concrete member
- $w$  = the water content of the fresh concrete lb per  $yd^3$
- $W_c$  = weight of cement per cu yd of concrete, lb
- $w_c$  = 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
- $\beta$  = 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,  $R = 1$
- $\beta$  = ratio of distance from neutral axis to the tensile face to the distance from neutral axis to steel
- $\Delta_c$  = contraction of the concrete, in./in.

Metric conversions

- 1 in. = 25.4 mm
- 1 ft = 0.3048 m

- 1 in.<sup>2</sup> = 645.1 mm<sup>2</sup>
- 1 ft<sup>2</sup> = 0.0929 m<sup>2</sup>
- 1 in<sup>3</sup> = 16.39 x 10<sup>3</sup> mm<sup>3</sup>
- 1 ft<sup>3</sup> = 0.0283 m<sup>3</sup>
- 1 yd<sup>3</sup> = 0.7646 m<sup>3</sup>
- 1 lb = 0.4536 kg
- 1 lb/in.<sup>2</sup> (psi) = 6895 Pa
- 1 kip/in.<sup>2</sup> (ksi) = 6.895 MPa
- 1 lb/ft<sup>2</sup> = 47.88 Pa
- 1 lb/ft<sup>3</sup> (pcf) = 16.02 kg/m<sup>3</sup>
- 1 lb/yd<sup>3</sup> = 0.5933 kg/m<sup>3</sup>
- 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 =  $(t_F - 32)/1.8$
- Difference in temperature =  $t_F/1.8$

This report was submitted to letter ballot of the committee and approved in accordance with ACI balloting procedures.

RECEIVED  
DEC 15 2000

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**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.

Ref. ACI 209.2R-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 -  $54^{\circ}\text{F}$

Average annual ambient temperature -  $47^{\circ}\text{F}$

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

$$H = 12.0 \text{ ft}$$

$$W = 10.0 \text{ ft}$$

$$T = 15.0 \text{ ft}$$

$$\text{Vol} = 66.6 \text{ cy}$$

Calculations

$$V = 12 \times 10 \times 15 = 1800 \text{ ft}^3$$

$$S = 2(12 \times 10) = 240 \text{ ft}^2$$

$$V/S = 1800/240 = 7.5 \text{ ft}$$

From Fig. 2.4 concrete temperature peak @ 5 days

From Fig. 2.6 heat dissipated = 33%

$$\text{Net effective placing temperature} = 54 - (54 - 47)(0.33) = 52^\circ\text{F}$$

Minimum exposure temperature =  $47^\circ\text{F}$

From Fig. 2.5 w/ effective placing temp @ 52 =  $34^\circ\text{F}$  Rise

From Fig. 2.1 correction for Type II cement peaking @ 5 days

$$T_c = 34 \left( \frac{50}{60} \right) \approx 28^\circ\text{F}$$

Correction for mix

Assume silica same as Type II cement

$$C_{eq} = 450 + 50 + (150/4) = 538 \text{ pcy}$$

$$T_{CF} = 28 \left( \frac{538}{376} \right) = 40^\circ\text{F}$$

Temperature of concrete @ 5 days =  $52^\circ\text{F} + 40^\circ\text{F} = 92^\circ\text{F}$  (Peak)

Temperature strain on cooling to  $47^\circ\text{F}$

Use thermal coefficient expansion/contraction =  $5.5 \times 10^{-6}$  in./in.

$$\text{Strain} = (92 - 47) (5.5 \times 10^{-6}) = 248 \times 10^{-6} \text{ in./in.}$$

or total strain in plug

$$(15 \times 12) (248 \times 10^{-6} \text{ in./in.}) = 0.045 \text{ in.}$$

Assume crack width = 0.016

$$\text{Number of cracks} = 0.045 / 0.016 = 2.8$$

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 3 OF 3 PAGES

## Thermal Stress

Use equation (4.3)

$$f_t = K_R \Delta_c E_c$$

Assume 100% base restraint,  $K_R = 1.0$

$$\Delta_c = (45 \times 5.5 \times 10^{-6}) = 248 \times 10^{-6} \text{ in./in.}$$

$$E_c = 57,000 \sqrt{5550} = 4.25 \times 10^6 \text{ psi}$$

$$f_t = (1.0)(248 \times 10^{-6})(4.25 \times 10^6) = 1054 \text{ psi}$$

Allowable tensile strength

$$f_t' = 6.7 \sqrt{5550} = 499 \text{ psi} < 1054 \text{ psi}$$

Cracking will occur



## **Appendix C**

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**CTL Report entitled "Evaluation of Concrete Mine Seal Using Nondestructive Testing"**



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.

November 28, 2000

Mr. Alberto Pujol

Page 3

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,  $\beta$  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, $\beta$ (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  $\beta$  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  $\beta$  (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.



November 28, 2000

Mr. Alberto Pujol

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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.**

Allen G. Davis, Ph.D., P.E.  
Senior Principal Engineer

Malcolm K. Lim, P.E.  
Project Manager

Attachments

## 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).

**IR Testing Equipment**

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.

**Method Description**

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.

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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<sup>2</sup> (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
<b>PRINCIPLES &amp; CONDITIONS</b>	<ul style="list-style-type: none"> <li>- Measurement of the propagation time of longitudinal stress waves.</li> <li>- 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.</li> <li>- Analysis is in the time domain.</li> </ul>	<ul style="list-style-type: none"> <li>- Measurement of the dynamic response of the shaft in the frequency domain</li> <li>- The shaft is struck with a hammer equipped with a load cell and the velocity or acceleration response of the shaft head is recorded.</li> <li>- Analysis is in the frequency domain.</li> </ul>	<ul style="list-style-type: none"> <li>Measurement of the response in both time and frequency.</li> <li>- The equipment and test procedure is as for the Mobility test.</li> <li>- Analysis is in both time and frequency domains.</li> </ul>
<b>ADVANTAGES</b>	<ul style="list-style-type: none"> <li>- No pre-placed tubes.</li> <li>- Portable equipment.</li> <li>- Rapid.</li> </ul>	<ul style="list-style-type: none"> <li>- No pre-placed tubes.</li> <li>- Stiffness measurement.</li> <li>- Portable equipment.</li> <li>- Rapid.</li> </ul>	<ul style="list-style-type: none"> <li>- As for Mobility test, plus:</li> <li>- Effective shape of shaft derived from analysis.</li> </ul>
<b>DISADVANTAGES</b>	<ul style="list-style-type: none"> <li>- Confuses necking and bulging.</li> <li>- Does not measure diameter.</li> <li>- Unable to determine defects in shafts &gt; 100 ft or with <math>l/d &gt; 30/1</math>.</li> </ul>	<ul style="list-style-type: none"> <li>- Results interpretation is delicate.</li> <li>- Limitations on geometry of pile to be tested, as for sonic echo.</li> </ul>	<ul style="list-style-type: none"> <li>- Requires very good test data for accurate analysis.</li> <li>- Full analysis can not yet be completed on site at time of test.</li> </ul>

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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:

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

$d$  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:

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

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)

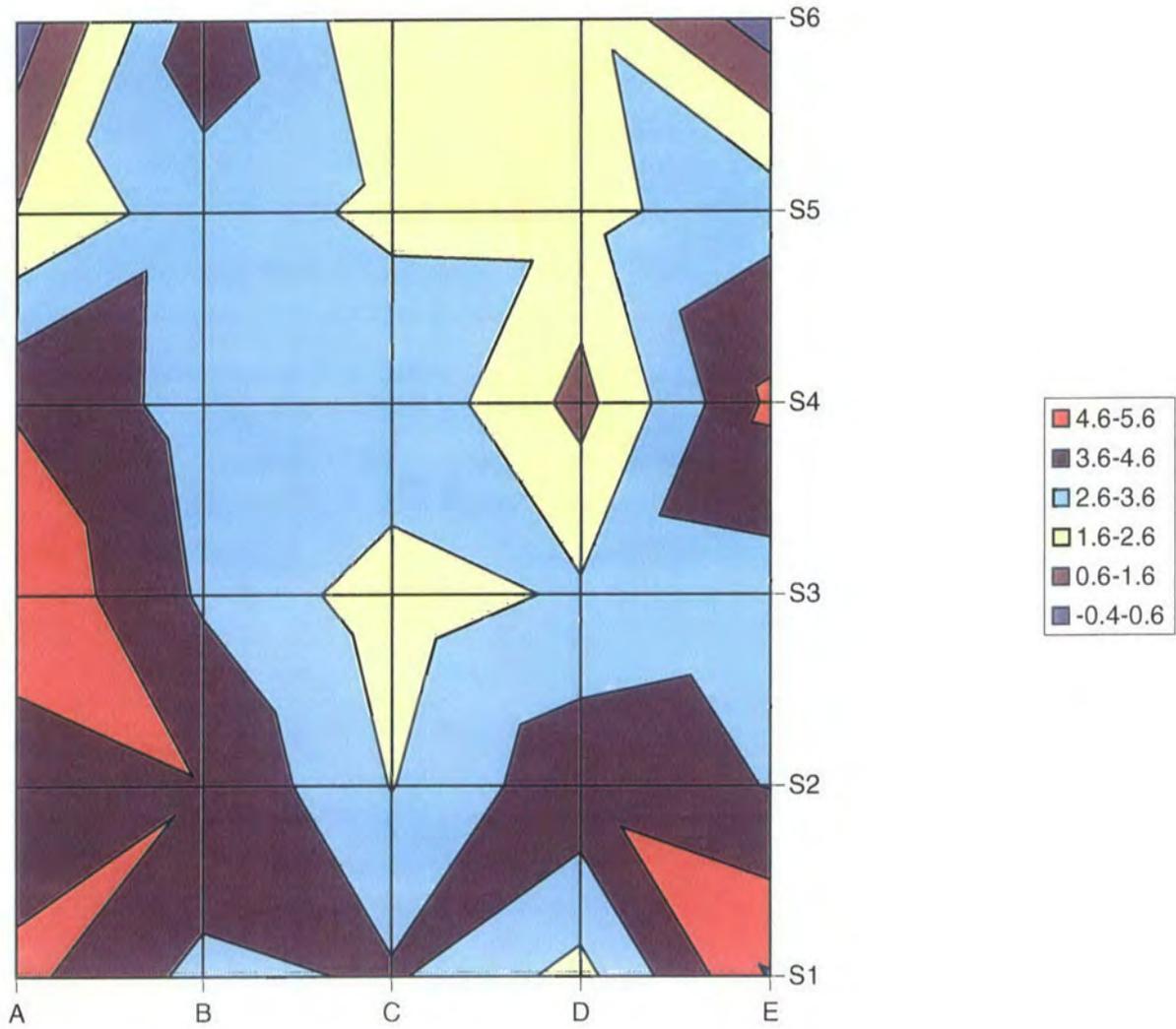
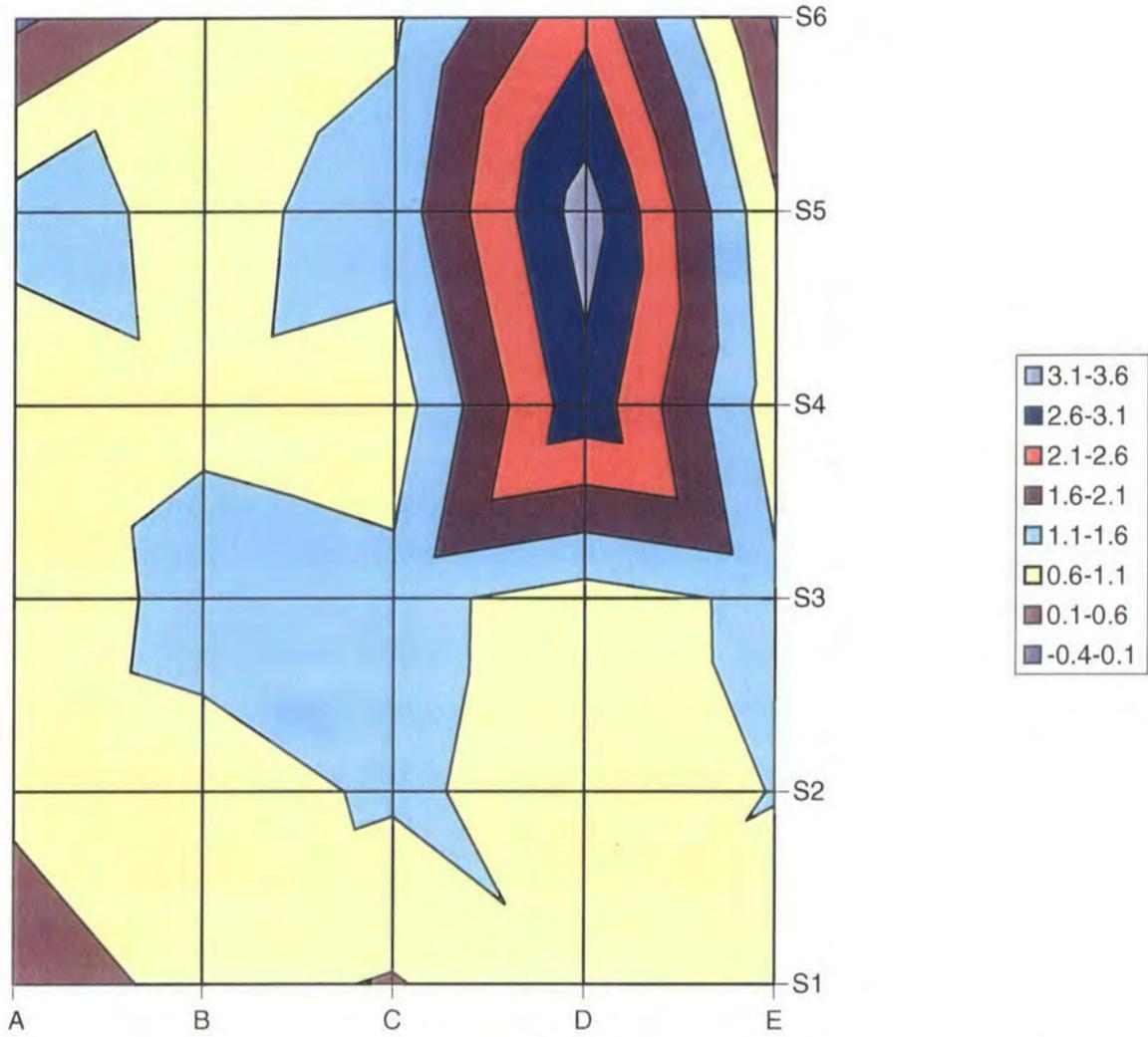


Figure D.2 Walker Mine Plug  
Average Mobility



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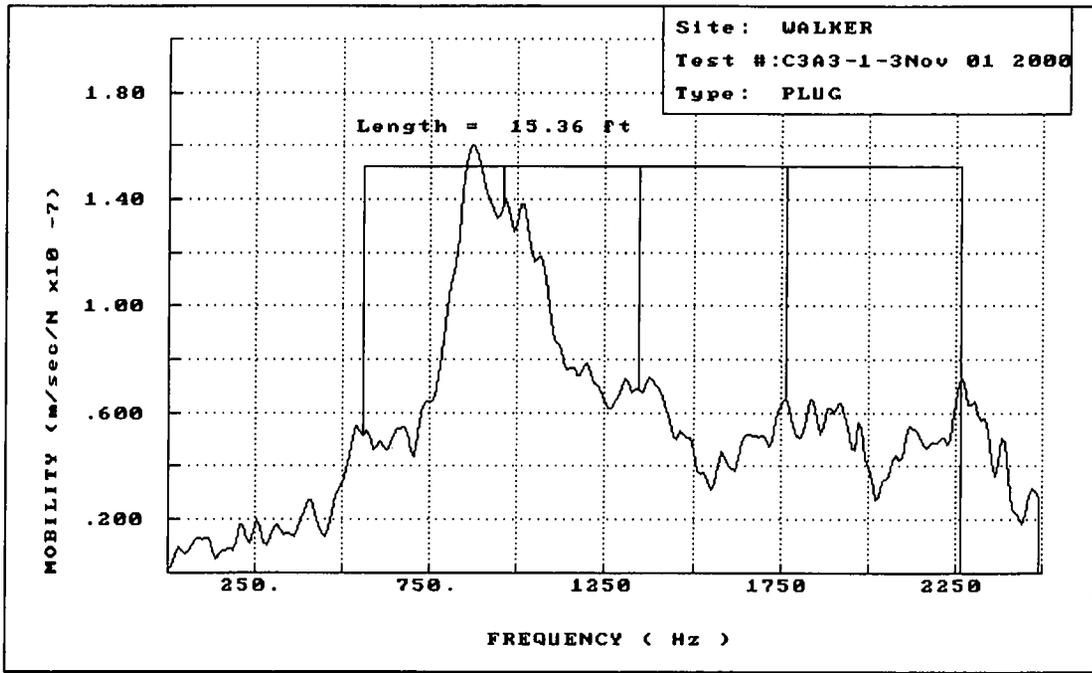


Figure D.3 IR test response – Plug Depth Measurement

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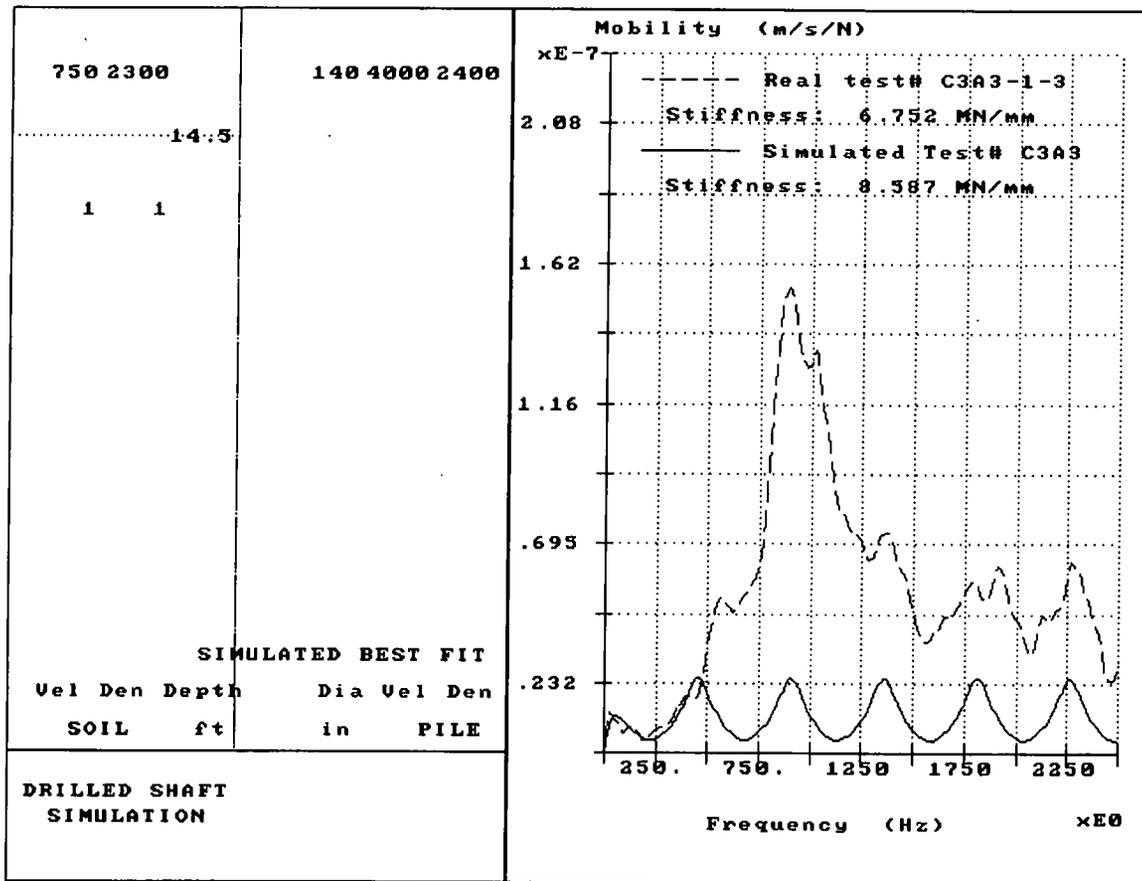


Figure D.4 Simulation Plot of IR Response

Plate 1: View of Plug Face





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## Appendix D

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### Product Information for Valves, Piping and Fittings

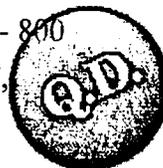
- Rotary Control Valves (Leslie Controls)
- Manual Gear Actuators (DeZurik)
- Stainless Steel Information (DeZurik, Enduro Stainless Steel, Allegheny Ludlum Steel Corp.)
- Piping Information (Crane)
- Fittings (Flowline Corp., Grinnell)
- Pressure Transducer (Druck)

# Rotary Control Valves

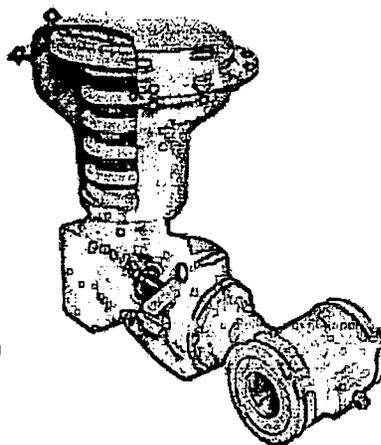
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## *Class GKM Rotary Control Valves*

- 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 torsional 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



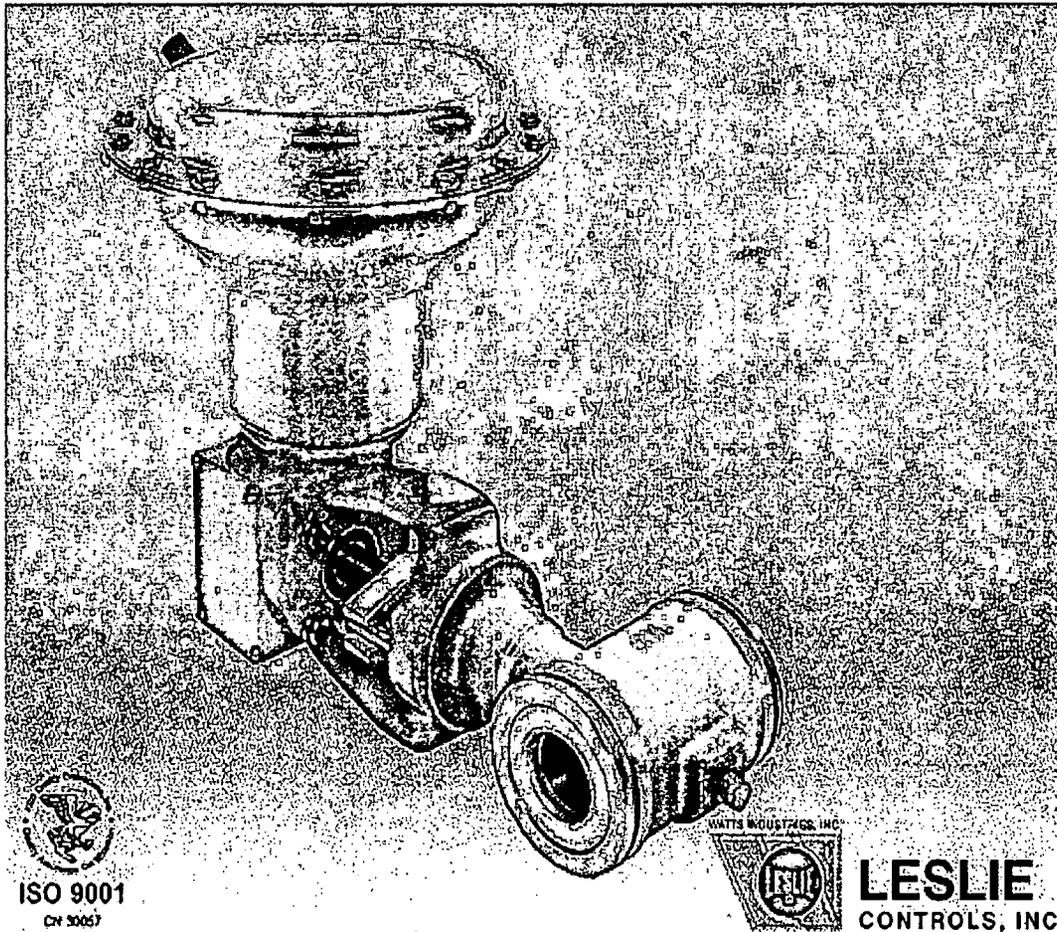
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PRODUCT DATA BULLETIN

# K-MAX

## ROTARY CONTROL VALVE

### ANSI 150-600 CLASS



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<a href="#">xxxx(Flow to Open)</a>	<a href="#">xxxxFull Port, Fail Closed</a>
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Page 2

## 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**

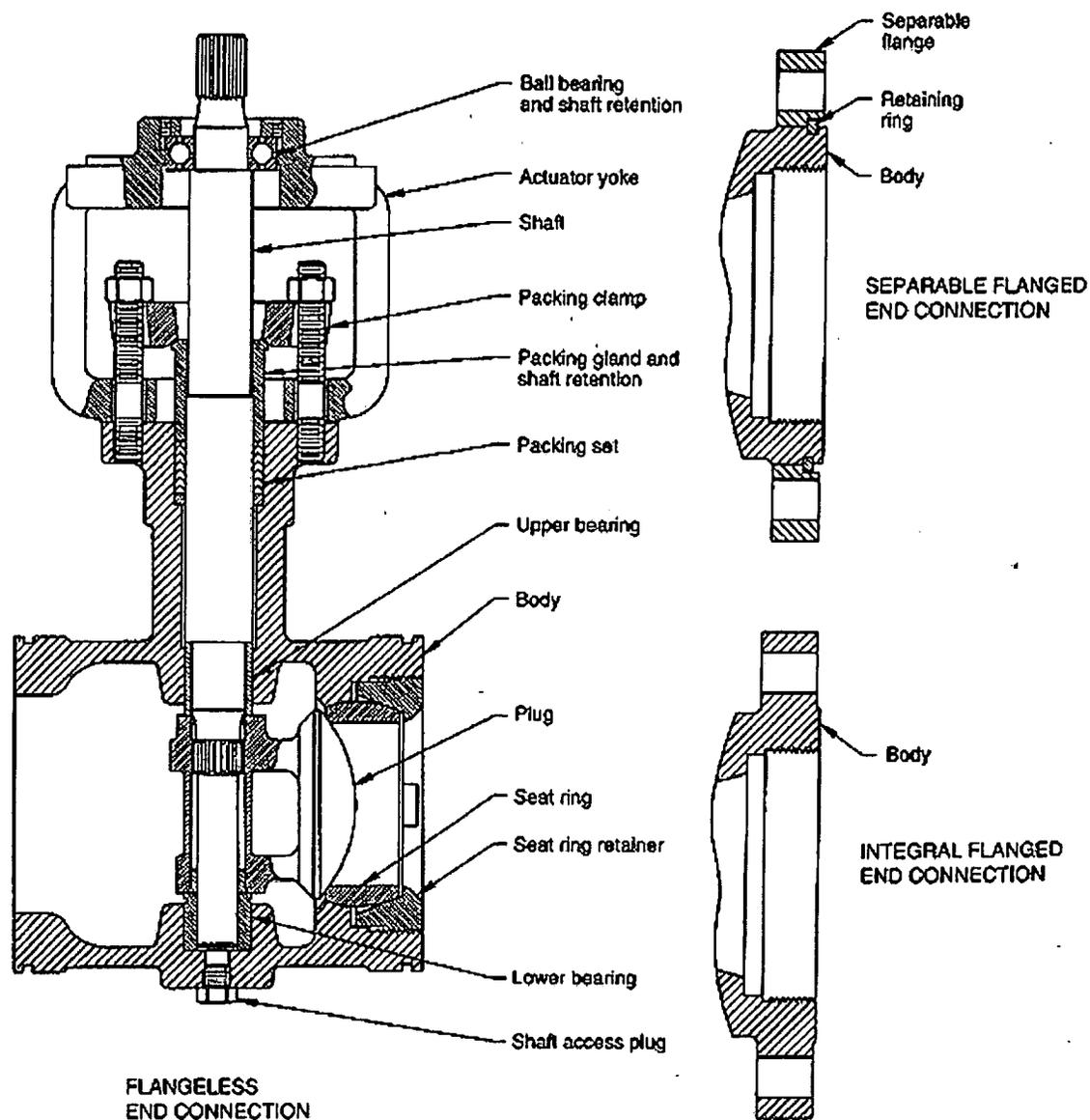
As an option for slurry applications, bearing seals are available to provide protection from abrasive particles entering the bearing area.

### **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.

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.



## 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

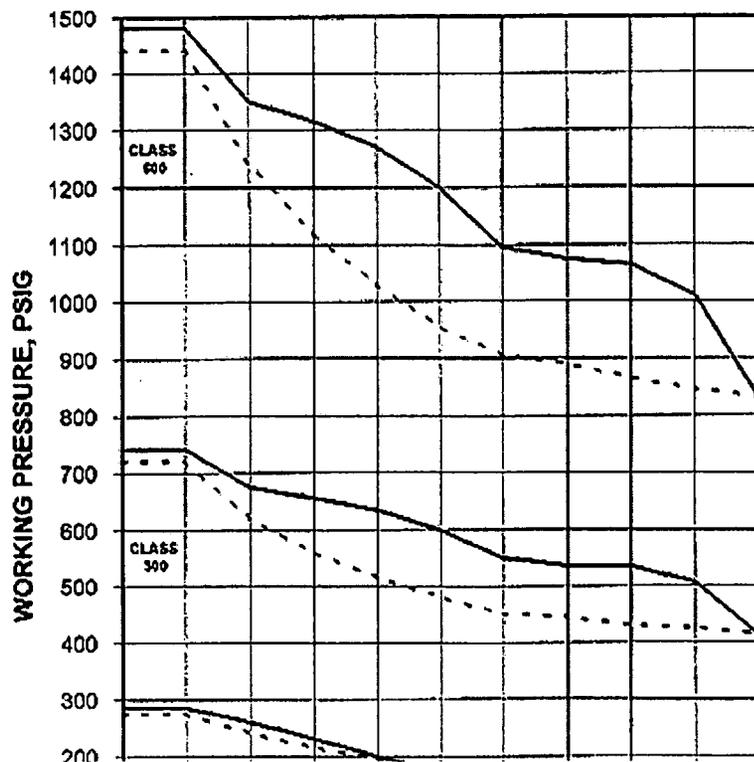
317 stainless steel, ASTM A743 grade CG8M

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.

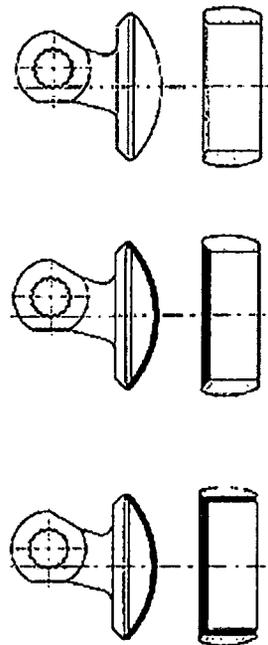
**Partial Alloy 6**

Alloy 6 on seat ring and plug seating surfaces.

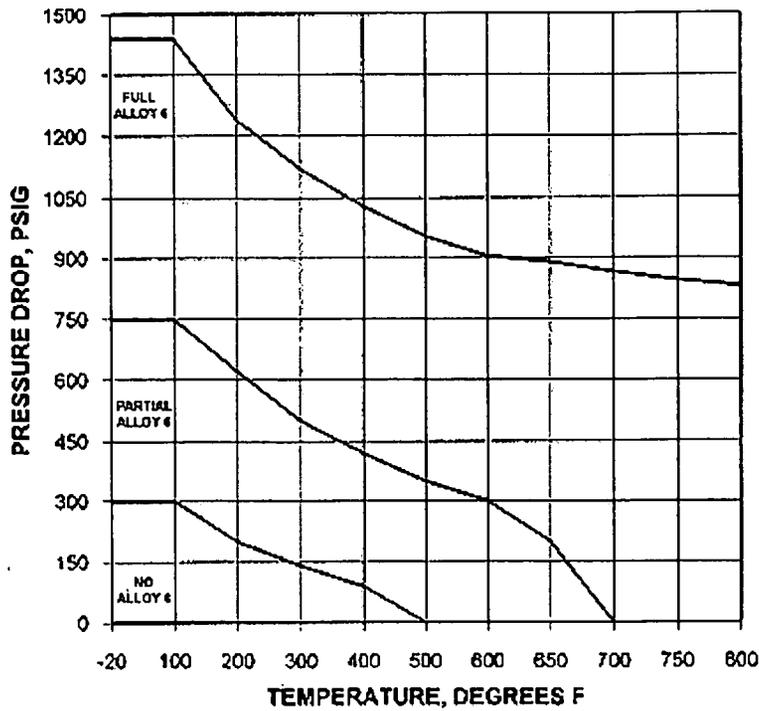
**Full Alloy 6**

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

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### ALLOY 6 RECOMMENDED USEAGE (ANSI CLASS 600 RATING)



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Trim Material- Alloy 6 Code	Body Material	Plug Material	Seat Ring Material		Shaft Material	Bearing Material
			size 2" - 8" full trim valves	size 1" - 1.5" full trim valves and size 1" - 8" reduced trim valves		
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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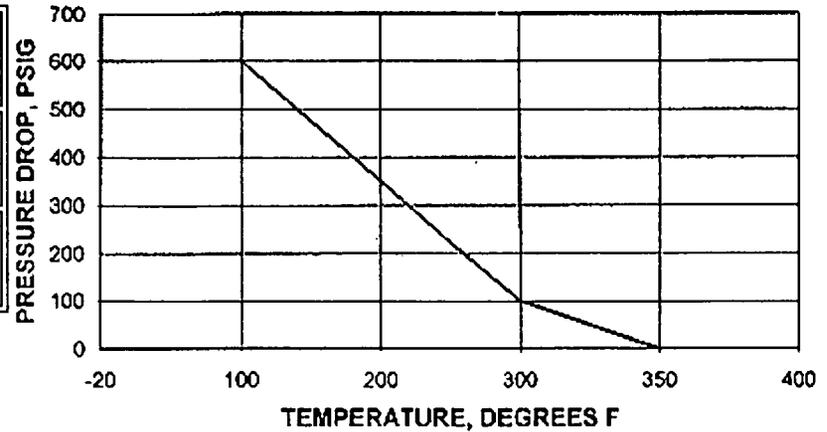
## K-Max Valve Technical Specifications

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### Seat Seal

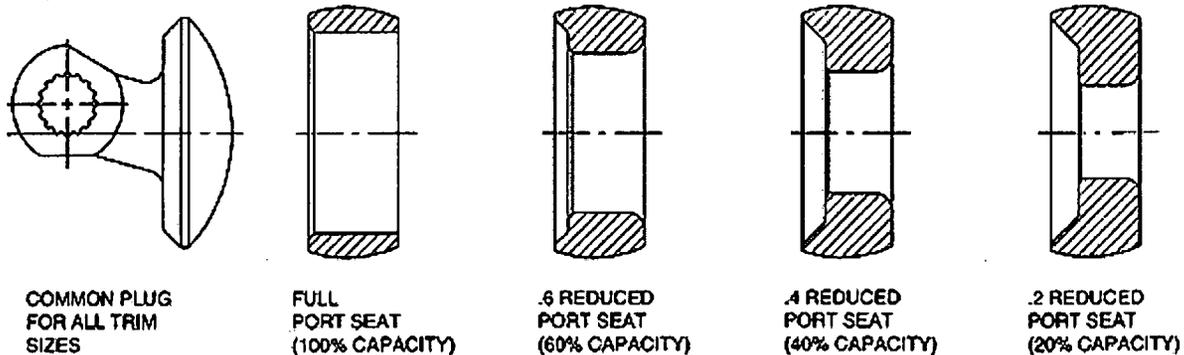
Metal to metal seat (standard)	- ANSI class IV
Metal to metal seat (optional)	- ANSI class V
PTFE soft seat	- ANSI class VI

### 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.

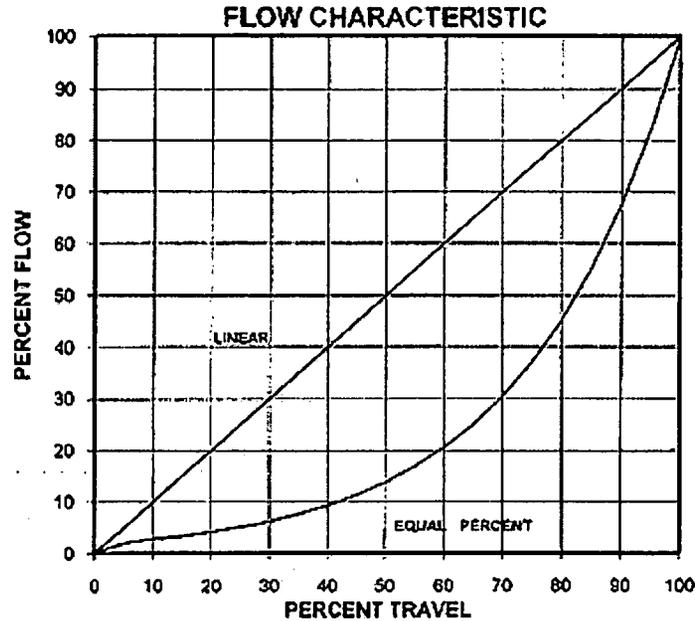


### Options

- Fluoroelastomer bearing 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,  $\Delta P_c$ , multiply Kc by the quantity  $P_1 - P_v$ , where  $P_1$  = upstream pressure (PSIA), and  $P_v$  = vapor pressure (PSIA).  $\Delta P_c = K_c (P_1 - P_v)$ .

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## K-Max Valve Flow Capacity Specifications

### Flow Coefficients (Cv), Linear Characteristic

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Flow to Open	Valve Size	Trim Size	Percent Travel (60° Rotation)									
			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	Valve Size	Trim Size	Percent Travel (60° Rotation)									
			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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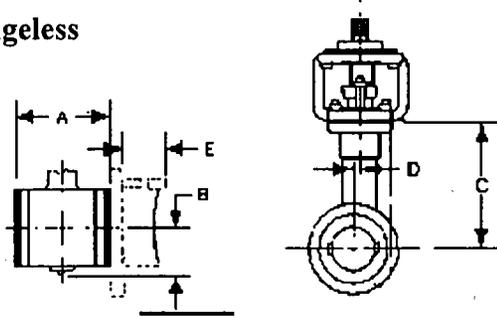
Actuator Size	Actuator Action	Actuator Spring	Output Torque(Ft-lbs)
40	Fail closed	20 psi	31
	Fail open	20 psi	19
	Fail closed	35 psi	58
	Fail open	35 psi	58
	Fail closed	60 psi	95
	Fail open	60 psi	102
55	Fail closed	20 psi	88
	Fail open	20 psi	76
	Fail closed	35 psi	152
	Fail open	35 psi	124
	Fail 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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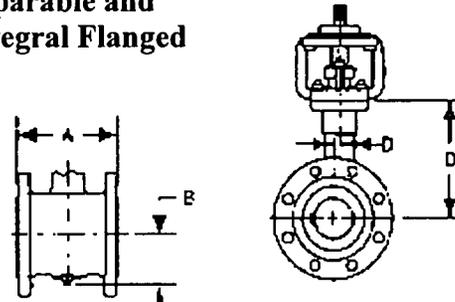
## K-Max Valve Dimensional Data

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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:**

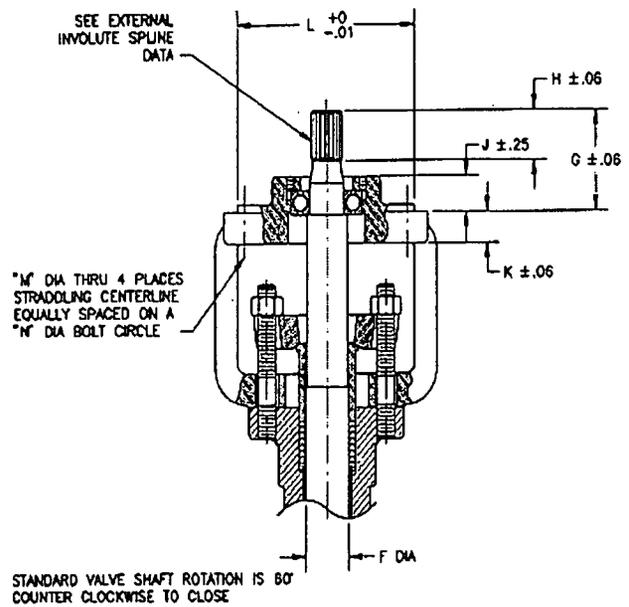
1. All dimension are subject to change without notice. Request certified drawings for use in preparing piping layouts.
2. Flange dimensions conform to ANSI B16.5.
3. Face-to-face dimensions conform to ISA S75.04.

**External Involute Spline Data Table Fillet  
Root Side Fit  
(Dimensions in inches)**

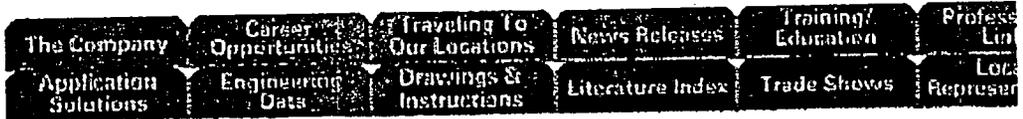
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Feature	Valve Size
---------	------------

	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
<b>Circular Tooth Thickness</b>			
Max effective	0.0393	0.0491	0.0393
Min actual	0.0366	0.0464	0.0361



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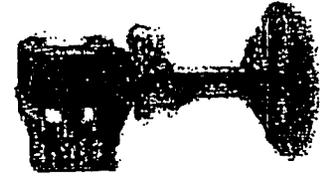
**Products**

- | [Rotary Control Valves](#) |
- | [V-Port Ball Valves](#) |
- | [Globe Control Valves](#) |
- | [Butterfly Valves](#) |
- | [Metal Seated Ball Valves](#) |
- | [Plug Valves](#) |
- | [Knife Gate Valves](#) |
- | [Ported Gate Valves](#) |
- | [Consistency Transmitters](#) |
- | [Valve Diagnostics System](#) |
- | [Severe Duty Control Valves](#) |
- | [Steam Conditioning](#) |
- | [Desuperheaters](#) |
- | [Regulators & Governors](#) |
- | [Intelli-Pulse Digital Valve Controller](#) |
- | [Trim Squirts](#) |
- | [Actuators](#) |
- | [Accessories](#) |



**Manual Gear Actuators**

- [Product Description](#)
- [Actuator Turns](#)
- [Literature](#)
- [Instruction Manuals](#)



**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.

# DeZURIK

250 Riverside Ave. North  
Sartell, MN 56377 - USA  
E-mail: [customerservice@dezurik.com](mailto:customerservice@dezurik.com)  
Web Page: [www.dezurik.com](http://www.dezurik.com)

## FAX Message

Sent by: **TOM SCHOENBERG**  
Fax: 320 / 259 - 2131  
Phone: 320 / 259 - 2273  
E-mail: [tom.schoenberg@dezurik.com](mailto:tom.schoenberg@dezurik.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 sources for information on stainless steels are:

1. Stainless Steel Market Development Office, sponsored by the Specialty Steel Industry of North America (SSINHA), 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
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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



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.

<u>Stainless Steel Alloy</u>	<u>Alloy Description</u>
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**

**Application Data 16.01-7**  
 March, 1997  
 Page 2

**Table I: Chemical Composition of Stainless Steel Alloys**

<u>Alloy</u>	<u>Cr</u>	<u>Ni</u>	<u>Mo</u>	<u>Cu</u>	<u>N</u>	<u>Other</u>
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**

<u>Alloy</u>	<u>Condition</u>	<u>Yeild Strength</u>		<u>Tensile Strength</u>		<u>Elongation</u>
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	(1,171,300 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 3000 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 1/4-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 1/4-inch thick) of Type 304. When welding thicker sections, (over 1/4-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	R <sub>B</sub> 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 imparts 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 specified 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 <sub>B</sub> 85	160 Bhn

ALLEGHENY LUDLUM

# Stainless Steel Handbook



ALLEGHENY LUDLUM STEEL CORPORATION  
Pittsburgh 22, Pennsylvania

# STAINLESS STEEL FINDER

Group	CHROMIUM-NICKEL AUSTENITIC GROUP					
	Type Number	301	302	301	302	304
<b>Analyses — percent:</b>						
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 <sub>2</sub> .25 max	N <sub>2</sub> .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
<b>Physical data:</b>						
Melting range — °F .....			2550-2590	2550-2590	2550-2650	2550-2650
Density — lb/in. <sup>3</sup> .....	.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/in <sup>2</sup> /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 <sup>-6</sup> :						
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)
<b>Electrical properties:</b>						
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
<b>Heat resistance:</b>						
Maximum operating temperature — °F:						
Intermittent service (Note 1) .....	1500	1500	1600	1600	1600	1600
Continuous service .....	1550	1550	1700	1700	1700	1700
<b>Temperatures — working and treating — °F:</b>						
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 .....						
<b>Mechanical properties — annealed:</b>						
Structure annealed .....	A	A	A	A	A	A
Yield strength — lb/in. <sup>2</sup> — min .....	40 000	40 000	35 000	30 000	30 000	25 000
Ultimate strength — lb/in. <sup>2</sup> — 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. <sup>2</sup> x 10 <sup>6</sup> .....	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
<b>Mechanical properties — heat treated:</b>						
Yield strength — lb/in. <sup>2</sup> .....						
Ultimate strength — lb/in. <sup>2</sup> .....	(Note 8)	(Note 8)	(Note 8)	(Note 8)	(Note 8)	(Note 8)
Elongation — % in 2 inches .....						
Hardness — Brinell .....						
Hardness — Rockwell .....						
<b>Creep strength — lb/in.<sup>2</sup> at 1000°F:</b>						
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.

Anal

Ch  
Ni  
Ot  
Ca  
Mn  
Si

Phys

Me  
De  
Sp  
Th

Me

Elec

M  
El

Heat

M

Tem

Fe  
Fo  
An  
Ar  
Hr  
Qu  
Te  
Dr

Mec

St  
Yi  
Ul  
El  
Re  
M  
Hr  
Hr  
Irr

Mec

Yi  
Ul  
El  
Hr  
Hr

Creep

1%  
1%

See pa

See pa

# STAINLESS STEEL FINDER

Group	CHROMIUM-NICKEL AUSTENITIC GROUP					
Type Number	316	316L	317	317	309	310
<b>Analyses — percent:</b>						
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
<b>Physical data:</b>						
Melting range — °F .....	2500-2550	2550-2650	2500-2550	2550-2600	2550-2650	2550-2650
Density — lb/in. <sup>3</sup> .....	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 <sup>2</sup> /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 <sup>-6</sup> :						
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)
<b>Electrical properties:</b>						
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	
<b>Heat resistance:</b>						
Maximum operating temperature — °F:						
Intermittent service (Note 1) .....	1600	1600	1600	1600	1800	1900
Continuous service .....	1700	1700	1700	1700	2000	2100
<b>Temperatures — working and treating — °F:</b>						
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 .....						
<b>Mechanical properties — annealed:</b>						
Structure annealed .....	A	A	A	A	A	A
Yield strength — lb/in. <sup>2</sup> — min .....	30 000	30 000	30 000	30 000	30 000	30 000
Ultimate strength — lb/in. <sup>2</sup> — 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. <sup>2</sup> x 10 <sup>6</sup> .....	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
<b>Mechanical properties — heat treated:</b>						
Yield strength — lb/in. <sup>2</sup> .....	(Note 8)	(Note 8)	(Note 8)	(Note 8)	(Note 8)	(Note 8)
Ultimate strength — lb/in. <sup>2</sup> .....	(Note 8)	(Note 8)	(Note 8)	(Note 8)	(Note 8)	(Note 8)
Elongation — % in 2 inches .....	(Note 8)	(Note 8)	(Note 8)	(Note 8)	(Note 8)	(Note 8)
Hardness — Brinell .....	(Note 8)	(Note 8)	(Note 8)	(Note 8)	(Note 8)	(Note 8)
Hardness — Rockwell .....	(Note 8)	(Note 8)	(Note 8)	(Note 8)	(Note 8)	(Note 8)
<b>Creep strength — lb/in.<sup>2</sup> at 1000°F:</b>						
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.

**CRANE**

**ENGINEERING  
DATA CATALOG**



**CRANE**

**Piping**

**PIPE DATA**  
**Carbon and Alloy Steel — Stainless Steel**

(also see next three pages)

Nom- inal Pipe Size	Outside Diam.  Inches	Identification			Wall Thick- ness (t)  Inches	Inside Diam- eter (d)  Inches	Area of Metal  Square Inches	Transverse Internal Area		Moment of Inertia (I)  Inches <sup>4</sup>	Weight Pipe  Pounds per foot	Weight Water  Pounds per foot of pipe	External Surface  Sq. Ft. per foot of pipe	Section Modulus  ( $\frac{I}{2 \cdot O.D.}$ )
		Steel		Stain- less Steel No.				(a)	(A)					
		Iron Pipe Size	Sched. No.											
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
1 1/4	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
1 1/2	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
2 1/2	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.

# Piping

## PIPE DATA — cont.

Nom- inal Pipe Size  Inches	Outside Diam.  Inches	Identification			Wall Thick- ness (t)  Inches	Inside Diam- eter (d)  Inches	Area of Metal  Square Inches	Transverse Internal Area		Moment of Inertia (I)  Inches <sup>4</sup>	Weight Pipe  Pounds per foot	Weight Water  Pounds per foot of pipe	External Surface  Sq. Ft. per foot of pipe	Section Modulus  ( $\frac{I}{O.D.}$ )
		Steel		Stain- less Steel Sched. No.				(a)	(A)					
		Iron Pipe Size	Sched. No.											
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
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
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
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
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
		...	140	...	1.000	8.750	30.63	60.13	.4176	367.8	104.13	26.06	2.814	68.43
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	40	40S	.375	12.000	14.58	113.10	.7854	279.3	49.56	49.00	3.338	43.8
		...	60	...	.406	11.938	15.77	111.93	.7773	300.3	53.52	48.50	3.338	47.1
		XS	80	80S	.500	11.750	19.24	108.43	.7528	361.5	65.42	46.92	3.338	56.7
		...	100	...	.562	11.626	21.52	106.16	.7372	400.4	73.15	46.00	3.338	62.8
		...	120	...	.688	11.374	26.03	101.64	.7058	475.1	88.63	44.04	3.338	74.6
		...	140	...	.844	11.062	1.53	96.14	.6677	561.6	107.32	41.66	3.338	88.1
12	12.75	...	...	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

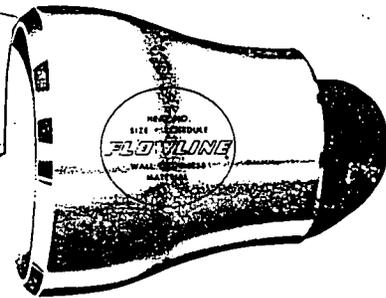
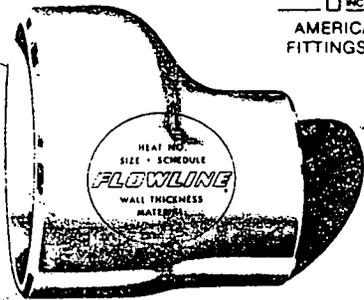
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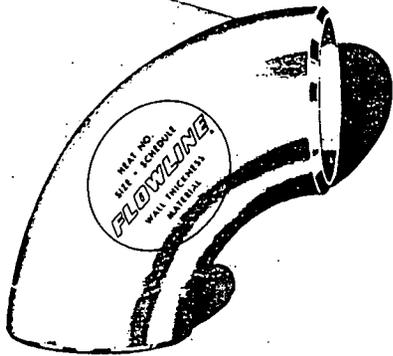
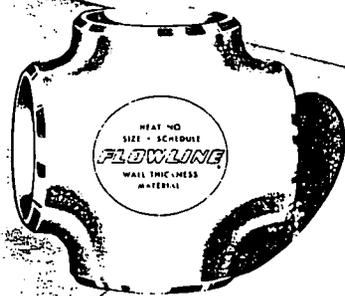
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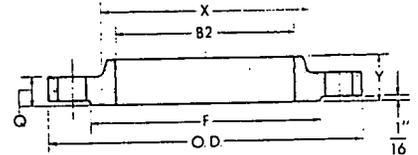
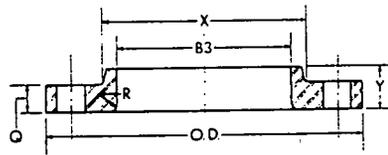
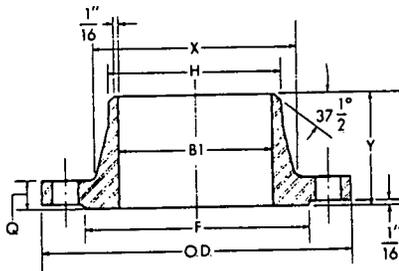
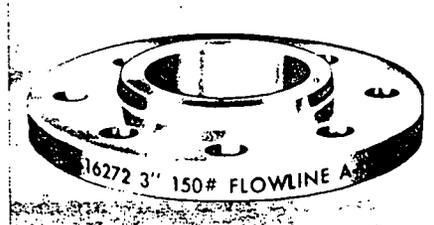
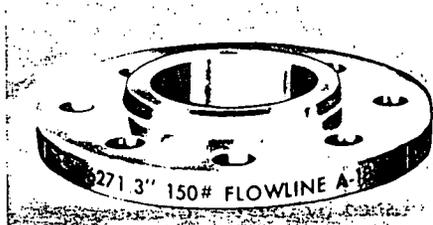
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**WELDING NECK**

**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)			COUNTER BORE OF THREADED FLANGE (C)	DEPTH OF SOCKET	LENGTH THROUGH HUB (Y)		
						(B1) WELDING NECK AND SOCKET WELD (2)	(B2) SLIP-ON AND SOCKET WELD MIN.	(B3) LAP JOINT MIN.			WELDING NECK	SLIP-ON SOCKET WELD AND THREADED	LAP JOINT
1/2	3.50	0.44	1.19	1.38	0.84	0.62	0.88	0.90		0.38	1.88	0.62	0.62
3/4	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 1/4	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 1/2	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 1/2	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 1/2	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
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

NO COUNTER BORE REQUIRED ON 150# THREADED FLANGES

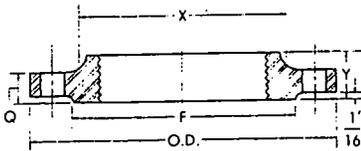
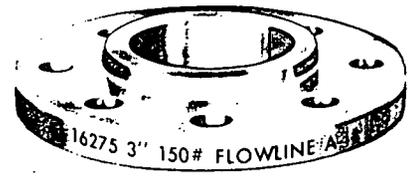
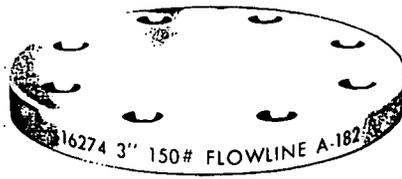
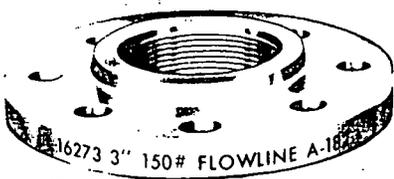
Dimensions are in inches and conform to ANSI B16.5. For dimensional tolerances, see page 65.

(1) Flange Thickness (Q) Includes 1/16" raised face.

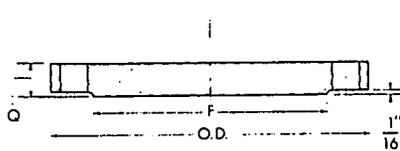
(2) Bore Diameter (B) of welding neck flanges corresponds to matching dimension of Standard Wall/Schedule 40S pipe. Flanges can be bored to match Extra Strong/Schedule 80S pipe.

(3) Every flange is marked to show the **FLOWLINE** trademark, type of metal, pressure rating, and pipe size in accordance with ANSI B16.5.

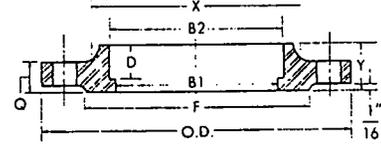
(continued on next page)



**THREADED**



**BLIND**



**SOCKET WELD**

**FORGED FLANGES ANSI B16.5** STAINLESS STEEL INCO® ALLOY ALUMINUM

NOM. PIPE SIZE	DRILLING					BOLTING			
	MIN. THREAD LENGTH	RADIUS (R) LAP JOINT	DIAMETER OF BOLT CIRCLE	NO. OF HOLES	DIA. OF HOLES	DIA. OF BOLTS	MACHINE BOLT LENGTH		STUD BOLT LENGTH
							Raised Face .06"	Raised Face .06"	Ring Joint
1/2	0.62	0.12	2.38	4	0.62	1/2	2.00	2.50	—
3/4	0.62	0.12	2.75	4	0.62	1/2	2.25	2.50	—
1	0.69	0.12	3.12	4	0.62	1/2	2.25	2.75	3.25
1 1/4	0.81	0.19	3.50	4	0.62	1/2	2.50	2.75	3.25
1 1/2	0.88	0.25	3.88	4	0.62	1/2	2.50	3.00	3.50
2	1.00	0.31	4.75	4	0.75	5/8	2.75	3.25	3.75
2 1/2	1.12	0.31	5.50	4	0.75	5/8	3.00	3.50	4.00
3	1.19	0.38	6.00	4	0.75	5/8	3.25	3.75	4.25
3 1/2	1.25	0.38	7.00	8	0.75	5/8	3.25	3.75	4.25
4	1.31	0.44	7.50	8	0.75	5/8	3.25	3.75	4.25
5	1.44	0.50	8.50	8	0.88	3/4	3.25	4.00	4.50
6	1.56	0.50	9.50	8	0.88	3/4	3.50	4.00	4.50
8	1.75	0.50	11.75	8	0.88	3/4	3.75	4.25	4.75
10	1.94	0.50	14.25	12	1.00	7/8	4.00	4.75	5.25
12	2.19	0.50	17.00	12	1.00	7/8	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	1 1/8	5.00	6.00	6.50
20	2.88	0.50	25.00	20	1.25	1 1/8	5.50	6.25	6.75
24	3.25	0.50	29.50	20	1.38	1 1/4	6.00	7.00	7.50

NOM. PIPE SIZE	APPROXIMATE WEIGHT EACH—POUNDS‡			
	WELDING NECK	SLIP-ON, SOCKET WELD AND THREADED	LAP JOINT	BLIND
1/2	1 1/4	1	1	1 1/4
3/4	2	1 1/2	1 1/2	1 1/2
1	2 1/2	2	2	2
1 1/4	3	3	3	3
1 1/2	4 1/4	3	3 1/2	4
2	6	5 1/4	5	6 1/2
2 1/2	10	8	7	9 1/2
3	12	9	9 1/4	12 1/2
3 1/2	12	11	11	13
4	16 1/2	12 1/2	12 1/2	17
5	19	15	15	20
6	25	19	19	26 1/2
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 1/4" 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 producing 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.



The trusted "G" you'll see on every Grinnell product and package. No other supplier can offer you what it represents. Look for it. Depend on it.



**Grinnell**<sup>®</sup>  
SUPPLY SALES COMPANY

# A. S. T. M. PRODUCT CROSS INDEX

Metal	Type	Pipe	Tubing	Welding Fittings <sup>1</sup>	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	B165	B165	B366-WPNC	(2)	Monel #60
	Ni-Cu Inconel—600	B167	B167	B366-WPNCl	(2)	Inconel #62
	Ni-Cr-Fe Alloy B—(Hastelloy)	(2)	(2)	B366-WPHB	(2)	
Ni-Mo Alloy C—(Hastelloy)	(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

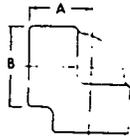
1. When fittings are of welded construction, the fitting manufacturer shall supplement the grade symbol marking with the letter "W".

2. No ASTM specification has been written. However, materials having chemical and physical properties comparable to the other materials listed may be used.

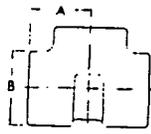
# forged steel threaded

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)

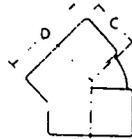
Forged steel threaded fittings conform to ANSI B16.11.



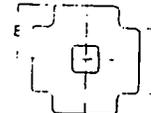
90° elbow



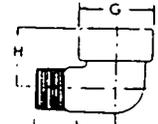
tee



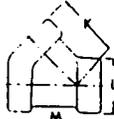
45° elbow



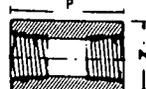
cross



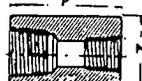
street elbow



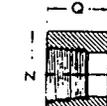
lateral



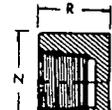
coupling



reducer



half coupling



pipe cap

## 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
<b>Class 2000</b>													
A	1 1/16	1 1/8	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5
B	3/8	1/2	5/8	1	1 1/8	1 1/4	1 3/4	2	2 1/2	3	3 1/2	4	4 1/2
C	1 1/16	1 1/8	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5
D	3/8	1/2	5/8	1	1 1/8	1 1/4	1 3/4	2	2 1/2	3	3 1/2	4	4 1/2
E	1 1/16	1 1/8	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5
F	3/8	1/2	5/8	1	1 1/8	1 1/4	1 3/4	2	2 1/2	3	3 1/2	4	4 1/2

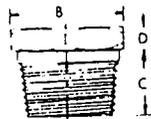
<b>Class 3000</b>													
A	1 1/8	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5	5 1/2
B	3/4	1	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5
C	1 1/8	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5	5 1/2
D	3/4	1	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5
E	1 1/8	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5	5 1/2
F	3/4	1	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5
G	1 1/8	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5	5 1/2
H	3/4	1	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5
J	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5	5 1/2	6
K	1 1/8	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5	5 1/2
L	1 1/8	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5	5 1/2
M	1 1/8	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5	5 1/2
N	3/4	1	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5
P	1 1/8	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5	5 1/2
Q	3/4	1	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5
R	3/4	1	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5

<b>Class 6000</b>													
A	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5	5 1/2	6	6 1/2
B	1	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5	5 1/2
C	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5	5 1/2	6
D	1	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5	5 1/2
E	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5	5 1/2	6	6 1/2
F	1	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5	5 1/2
G	1 1/8	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5	5 1/2
H	3/4	1	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5
J	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5	5 1/2	6
K	1 1/8	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5	5 1/2
L	1 1/8	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5	5 1/2
M	1 1/8	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5	5 1/2
N	3/4	1	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5
P	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5	5 1/2	6
Q	3/4	1	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5
R	3/4	1	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	3	3 1/2	4	4 1/2	5

**forged steel threaded**



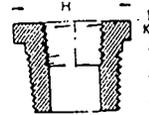
round head plug



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 1/4	2	2 1/2	2 3/4	2 3/4	3	3
B	3/8	5/8	1 1/8	3/4	1 1/8	1 3/8	1 1/2	1	1 1/2	1 3/4	1 1/2	1 3/4	1 3/8
C	3/8	5/8	1 1/8	3/4	1 1/8	1	1	1	1 1/8	1 3/8	1 1/2	1 3/4	1 1/8
D	1/4	1/4	3/8	3/8	3/8	3/8	3/8	3/8	1 1/8	1 1/2	1 1/2	1 1/2	2 1/2
E	3/8	3/8	3/8	3/8	3/8	3/8	3/8	3/8	1 1/8	1 1/2	1 1/2	1 1/2	1 1/4
F	3/8	3/8	3/8	3/8	3/8	3/8	3/8	3/8	1 1/8	1 1/2	1 1/2	1 1/2	1 1/4
G	1/4	1/4	3/8	3/8	3/8	1/2	3/8	3/8	1 1/8	1 1/2	1 1/2	1 1/2	1 1/4
H	.....	3/8	1 1/8	3/8	1 1/8	1 1/8	1 1/8	2	2 1/2	3	3 1/4	4 1/4	5
J	.....	1/2	3/8	1 1/8	3/4	1 3/8	3/8	1 1/8	1	1 3/8	1 3/8	1 3/8	1 1/8
K	.....	1/8	3/8	3/8	1/4	1/4	3/8	3/8	3/8	1/2	1 1/8	1 1/8	1 1/4
L	.....	3/8	3/8	1/2	3/8	3/8	1 1/8	3/4	1 1/8	1 1/8	1 1/8	1 1/8	1 1/4
M	1 1/2	1 1/2	1 1/8	1 1/2	1 1/8	1 1/8	1 1/8	1 1/2	2 1/8	2 1/8	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**

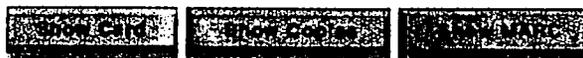
90° ell, fig. 2131	0.37	0.66	1.00	1.59	2.54	3.56	5.88	7.06	13.00	21.78	36.22	37.35	.....
45° ell, fig. 2132	0.25	0.59	0.85	1.34	2.25	2.59	4.56	5.75	9.63	15.46	31.21	26.21	.....
street elbow, fig. 2133	.....	0.40	0.99	1.00	1.63	2.75	3.86	7.23	.....	.....	.....	.....	.....
tee, fig. 2134	0.44	0.92	1.38	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



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- RFI protected
- Intrinsically safe version available



Sensor-Groups

Output	<input type="checkbox"/>	Application	<input type="checkbox"/>
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# Appendix E

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## Analytical Chemistry Protocols

**METHOD #: 150.1**      **Approved for NPDES (Editorial Revision 1978, 1982)**

**TITLE: pH (Electrometric)**

**ANALYTE: pH**

**INSTRUMENTATION: pH Meter**

**STORET No.**

Determined on site 00400

Laboratory 00403

### **1.0 Scope and Application**

1.1 This method is applicable to drinking, surface, and saline waters, domestic and industrial wastes and acid rain (atmospheric deposition).

### **2.0 Summary of Method**

2.1 The pH of a sample is determined electrometrically using either a glass electrode in combination with a reference potential or a combination electrode.

### **3.0 Sample Handling and Preservation**

3.1 Samples should be analyzed as soon as possible preferably in the field at the time of sampling.

3.2 High-purity waters and waters not at equilibrium with the atmosphere are subject to changes when exposed to the atmosphere, therefore the sample containers should be filled completely and kept sealed prior to analysis.

### **4.0 Interferences**

4.1 The glass electrode, in general, is not subject to solution interferences from color, turbidity, colloidal matter, oxidants, reductants or high salinity.

- 4.2 Sodium error at pH levels greater than 10 can be reduced or eliminated by using a "low sodium error" electrode.
- 4.3 Coatings of oily material or particulate matter can impair electrode response. These coatings can usually be removed by gentle wiping or detergent washing, followed by distilled water rinsing. An additional treatment with hydrochloric acid (1 + 9) may be necessary to remove any remaining film.
- 4.4 Temperature effects on the electrometric measurement of pH arise from two sources. The first is caused by the change in electrode output at various temperatures. This interference can be controlled with instruments having temperature compensation or by calibrating the electrode-instrument system at the temperature of the samples. The second source is the change of pH inherent in the sample at various temperatures. This error is sample dependent and cannot be controlled, it should therefore be noted by reporting both the pH and temperature at the time of analysis.

## 5.0 Apparatus

- 5.1 pH Meter-laboratory or field model. A wide variety of instruments are commercially available with various specifications and optional equipment.
- 5.2 Glass electrode.
- 5.3 Reference electrode-a calomel, silver-silver chloride or other reference electrode of constant potential may be used.

NOTE 1: Combination electrodes incorporating both measuring and reference functions are convenient to use and are available with solid, gel type filling materials that require minimal maintenance.

- 5.4 Magnetic stirrer and Teflon-coated stirring bar.
- 5.5 Thermometer or temperature sensor for automatic compensation.

## 6.0 Reagents

- 6.1 Primary standard buffer salts are available from the National Bureau of Standards and should be used in situations where extreme accuracy is necessary.

6.1.1 Preparation of reference solutions from these salts require some special precautions and handling(1) such as low conductivity dilution water, drying

ovens, and carbon dioxide free purge gas. These solutions should be replaced at least once each month.

- 6.2 Secondary standard buffers may be prepared from NBS salts or purchase as a solution from commercial vendors. Use of these commercially available solutions, that have been validated by comparison to NBS standards, are recommended for routine use.

## 7.0 Calibration

- 7.1 Because of the wide variety of pH meters and accessories, detailed operating procedures cannot be incorporated into this method. Each analyst must be acquainted with the operation of each system and familiar with all instrument functions. Special attention to care of the electrodes is recommended.
- 7.2 Each instrument/electrode system must be calibrated at a minimum of two points that bracket the expected pH of the samples and are approximately three pH units or more apart.

7.2.1 Various instrument designs may involve use of a "balance" or "standardize" dial and/or a slope adjustment as outlined in the manufacturer's instructions. Repeat adjustments on successive portions of the two buffer solutions as outlined in procedure 8.2 until readings are within 0.05 pH units of the buffer solution value.

## 8.0 Procedure

- 8.1 Standardize the meter and electrode system as outlined in Section 7.
- 8.2 Place the sample or buffer solution in a clean glass beaker using a sufficient volume to cover the sensing elements of the electrodes and to give adequate clearance for the magnetic stirring bar.
- 8.2.1 If field measurements are being made the electrodes may be immersed directly in the sample stream to an adequate depth and moved in a manner to insure sufficient sample movement across the electrode sensing element as indicated by drift free ( $< 0.1$  pH) readings.
- 8.3 If the sample temperature differs by more than 2-C from the buffer solution the measured pH values must be corrected. Instruments are equipped with automatic or manual

compensators that electronically adjust for temperature differences. Refer to manufacturer's instructions.

- 8.4 After rinsing and gently wiping the electrodes, if necessary, immerse them into the sample beaker or sample stream and stir at a constant rate to provide homogeneity and suspension of solids. Rate of stirring should minimize the air transfer rate at the air water interface of the sample. Note and record sample pH and temperature. Repeat measurement on successive volumes of sample until values differ by less than 0.1 pH units. Two or three volume changes are usually sufficient.
- 8.5 For acid rain samples it is most important that the magnetic stirrer is not used. Instead, swirl the sample gently for a few seconds after the introduction of the electrode(s). Allow the electrode(s) to equilibrate. The air-water interface should not be disturbed while measurement is being made. If the sample is not in equilibrium with the atmosphere, pH values will change as the dissolved gases are either absorbed or desorbed. Record sample pH and temperature.

## 9.0 Calculation

- 9.1 pH meters read directly in pH units. Report pH to the nearest 0.1 unit and temperature to the nearest degree C.

## 10.0 Precision and Accuracy

- 10.1 Forty-four analysts in twenty laboratories analyzed six synthetic water samples containing exact increments of hydrogen-hydroxyl ions, with the following results:

pH Units	pH Units	Accuracy as	
		Bias %	Bias pH Units
3.5	0.10	-0.29	-0.01
3.5	0.11	-0.00	
7.1	0.20	+1.01	+0.07
7.2	0.18	-0.03	-0.002
8.0	0.13	-0.12	-0.01
8.0	0.12	+0.16	+0.01

(FWPCA Method Study 1, Mineral and Physical Analyses)

10.2 In a single laboratory (EMSL), using surface water samples at an average pH of 7.7, the standard deviation was +/- 0.1.

#### Bibliography

1. Standard Methods for the Examination of Water and Wastewater, 14th Edition, p 460, (1975).
2. Annual Book of ASTM Standards, Part 31, "Water", Standard D1293-65, p 178 (1976).
3. Peden, M. E. and Skowron, L. M., Ionic Stability of Precipitation Samples, Atmospheric Environment, Vol. 12, pp. 2343-2349, 1978.

**METHOD #: 160.1**                      **Approved for NPDES (Issued 1971)**

**TITLE: Residue, Filterable (Gravimetric, Dried at 180-C)**

**ANALYTE:**

**Residue, Filterable**

**INSTRUMENTATION: Drying Oven**

**STORET No. 70300**

### **1.0 Scope and Application**

- 1.1 This method is applicable to drinking, surface, and saline waters, domestic and industrial wastes.
- 1.2 The practical range of the determination is 10 mg/L to 20,000 mg/L

### **2.0 Summary of Method**

- 2.1 A well-mixed sample is filtered through a standard glass fiber filter. The filtrate is evaporated and dried to constant weight at 180-C.
- 2.2 If Residue, Non-Filterable is being determined, the filtrate from that method may be used for Residue, Filterable.

### **3.0 Definitions**

- 3.1 Filterable residue is defined as those solids capable of passing through a glass fiber filter and dried to constant weight at 180-C.

### **4.0 Sample Handling and Preservation**

- 4.1 Preservation of the sample is not practical; analysis should begin as soon as possible. Refrigeration or icing to 4-C, to minimize micro-biological decomposition of solids, is recommended.

### **5.0 Interferences**

- 5.1 Highly mineralized waters containing significant concentrations of calcium, magnesium, chloride and/or sulfate may be hygroscopic and will require prolonged drying, desiccation and rapid weighing.
- 5.2 Samples containing high concentrations of bicarbonate will require careful and possibly prolonged drying at 180-C to insure that all the bicarbonate is converted to carbonate.
- 5.3 Too much residue in the evaporating dish will crust over and entrap water that will not be driven off during drying. Total residue should be limited to about 200 mg.

## 6.0 Apparatus

- 6.1 Glass fiber filter discs, 4.7 cm or 2.1 cm, without organic binder, Reeve Angel type 934-AH, Gelman type A/E, or equivalent
- 6.2 Filter holder, membrane filter funnel or Gooch crucible adapter
- 6.3 Suction flask, 500 mL
- 6.4 Gooch crucibles, 25 mL (if 2.1 cm filter is used)
- 6.5 Evaporating dishes, porcelain, 100 mL volume. (Vycor or platinum dishes may be substituted)
- 6.6 Steam bath
- 6.7 Drying oven, 180-C +/- 2-C
- 6.8 Desiccator
- 6.9 Analytical balance, capable of weighing to 0.1 mg

## 7.0 Procedure

- 7.1 Preparation of glass fiber filter disc: Place the disc on the membrane filter apparatus or insert into bottom of a suitable Gooch crucible. While vacuum is applied, wash the disc with three successive 20 mL volumes of distilled water. Remove all traces of water by continuing to apply vacuum after water has passed through. Discard washings.
- 7.2 Preparation of evaporating dishes: If Volatile Residue is also to be measured heat the clean dish to 550 +/- 50-C for one hour in a muffle furnace. If only Filterable Residue is

to be measured heat the clean dish to 180 +/- 2- C for one hour. Cool in desiccator and store until needed. Weigh immediately before use.

- 7.3 Assemble the filtering apparatus and begin suction. Shake the sample vigorously and rapidly transfer 100 mL to the funnel by means of a 100 mL graduated cylinder. If total filterable residue is low, a larger volume may be filtered.
- 7.4 Filter the sample through the glass fiber filter, rinse with three 10 mL portions of distilled water and continue to apply vacuum for about 3 minutes after filtration is complete to remove as much water as possible.
- 7.5 Transfer 100 mL (or a larger volume) of the filtrate to a weighed evaporating dish and evaporate to dryness on a steam bath.
- 7.6 Dry the evaporated sample for at least one hour at 180 plus or minus 2-C. Cool in a desiccator and weigh. Repeat the drying cycle until a constant weight is obtained or until weight loss is less than 0.5 mg.

## 8.0 Calculation

- 8.1 Calculate filterable residue as follows:

$$\text{Filterable residue, mg/L} = \frac{(A - B) * 1,000}{C}$$

where:

A = weight of dried residue + dish in mg

B = weight of dish in mg

C = volume of sample used in mL

## 9.0 Precision and Accuracy

- 9.1 Precision and accuracy are not available at this time.

## Bibliography

1. Standard Methods for the Examination of Water and Wastewater, 14th Edition, p 92, Method 208B, (1975).

METHOD #: 200.7

Test Method (December 1982)

**TITLE: Inductively Coupled Plasma Atomic Emission Spectrometric Method for Trace Element Analysis of Water and Wastes**

Analyte	Symbol	CAS #
Aluminum	Al	7429-90-5
Antimony	Sb	7440-36-0
Arsenic	As	7440-38-2
Barium	Ba	7440-39-3
Beryllium	Be	7440-41-7
Boron	B	7440-42-8
Cadmium	Cd	7440-43-9
Calcium	Ca	7440-70-2
Chromium	Cr	7440-47-3
Cobalt	Co	7440-48-4
Copper	Cu	7440-50-8
Iron	Fe	7439-89-6
Lead	Pb	7439-92-1
Magnesium	Mg	7439-95-4
Manganese	Mn	7439-96-5
Molybdenum	Mo	7439-98-7
Nickel	Ni	7440-02-0
Potassium	K	7440-09-7
Selenium	Se	7782-49-2
Silica	SiO <sub>2</sub>	
Silver	Ag	7440-22-4
Sodium	Na	7440-23-5
Thallium	Th	7440-28-0
Vanadium	V	7440-62-2
Zinc	Zn	7440-66-6

**INSTRUMENTATION: ICP**

**1.0 Scope and Application**

1.1 This method may be used for the determination of dissolved, suspended, or total elements in drinking water, surface water, domestic and industrial wastewaters.

- 1.2 Dissolved elements are determined in filtered and acidified samples. Appropriate steps must be taken in all analyses to ensure that potential interference are taken into account. This is especially true when dissolved solids exceed 1500 mg/L. (See 5.)
- 1.3 Total elements are determined after appropriate digestion procedures are performed. Since digestion techniques increase the dissolved solids content of the samples, appropriate steps must be taken to correct for potential interference effects. (See 5.)
- 1.4 Table 1 lists elements for which this method applies along with recommended wavelengths and typical estimated instrumental detection limits using conventional pneumatic nebulization. Actual working detection limits are sample dependent and as the sample matrix varies, these concentrations may also vary. In time, other elements may be added as more information becomes available and as required.
- 1.5 Because of the differences between various makes and models of satisfactory instruments, no detailed instrumental operating instructions can be provided. Instead, the analyst is referred to the instructions provided by the manufacturer of the particular instrument.

## 2.0 Summary of Method

- 2.1 The method describes a technique for the simultaneous or sequential multi-element determination of trace elements in solution. The basis of the method is the measurement of atomic emission by an optical spectroscopic technique. Samples are nebulized and the aerosol that is produced is transported to the plasma torch where excitation occurs. Characteristic atomic-line emission spectra are produced by a radio-frequency inductively coupled plasma (ICP). The spectra are dispersed by a grating spectrometer and the intensities of the lines are monitored by photomultiplier tubes. The photocurrents from the photomultiplier tubes are processed and controlled by a computer system. A background correction technique is required to compensate for variable background contribution to the determination of trace elements. Background must be measured adjacent to analyte lines on samples during analysis. The position selected for the background intensity measurement, on either or both sides of the analytical line, will be determined by the complexity of the spectrum adjacent to the analyte line. The position used must be free of spectral interference and reflect the same change in background intensity as occurs at the analyte wavelength measured. Background correction is not required in cases of line broadening where a background correction measurement would actually degrade the analytical result. The possibility of additional interferences named in 5.1 (and tests for their presence as described in 5.2) should also be recognized and appropriate corrections made.

### 3.0 Definitions

- 3.1 Dissolved--Those elements which will pass through a 0.45 um membrane filter.
- 3.2 Suspended--Those elements which are retained by a 0.45 um membrane filter.
- 3.3 Total--The concentration determined on an unfiltered sample following vigorous digestion (9.3), or the sum of the dissolved plus suspended concentrations. (9.1 plus 9.2.)
- 3.4 Total recoverable--The concentration determined on an unfiltered sample following treatment with hot, dilute mineral acid (9.4).
- 3.5 Instrumental detection limit-- The concentration equivalent to a signal, due to the analyte, which is equal to three times the standard deviation of a series of ten replicate measurements of a reagent blank signal at the same wavelength.
- 3.6 Sensitivity--The slope of the analytical curve, i.e. functional relationship between emission intensity and concentration.
- 3.7 Instrument check standard multi-element standard of known concentrations prepared by the analyst to monitor and verify instrument performance on a daily basis. (See 7.6.1)
- 3.8 Interference check sample--A solution containing both interfering and analyte elements of known concentration that can be used to verify background and inter-element correction factors. (See 7.6.2)
- 3.9 Quality control sample--A solution obtained from an outside source having known, concentration values to be used to verify the calibration standards. (See 7.6.3)
- 3.10 Calibration standards--a series of know standard solutions used by the analyst for calibration of the instrument (i.e., preparation of the analytical curve). (See 7.4)
- 3.11 Linear dynamic range--The concentration range over which the analytical curve remains linear.
- 3.12 Reagent blank--A volume of deionized, distilled water containing the same acid matrix as the calibration standards carried through the entire analytical scheme. (See 7.5.2)
- 3.13 Calibration blank--A volume of deionized, distilled water acidified with HNO<sub>3</sub> and HCl. (See 7.5.1 )

3.14 Method of standard addition-- The standard addition technique involves the use of the unknown and the unknown plus a known amount of standard.(See 10.6.1)

#### 4.0 Safety

4.1 The toxicity or carcinogenicity of each reagent used in this method has not been precisely defined; however, each chemical compound should be treated as a potential health hazard. From this viewpoint, exposure to these chemicals must be reduced to the lowest possible level by whatever means available. The laboratory is responsible for maintaining a current awareness file of OSHA regulations regarding the safe handling of the chemicals specified in this method. A reference file of material data handling sheets should, also be made available to all personnel involved in the chemical analysis. Additional references to laboratory safety are available and have been identified (14.7, 14.8 and 14.9) for the information of the analyst.

#### 5.0 Interferences

5.1 Several types of interference effects may contribute to inaccuracies in the determination of trace elements. They can be summarized as follows:

5.1.1 Spectral interferences can be categorized as 1) overlap of a spectral line from another element; 2) unresolved overlap of molecular band spectra; 3) background contribution from continuous or recombination phenomena; and 4) background contribution from stray light from the line emission of high concentration elements. The first of these effects can be compensated by utilizing a computer correction of the raw data, requiring the monitoring and measurement of the interfering element. The second effect may require selection of an alternate wavelength. The third and fourth effects can usually be compensated by a background correction adjacent to the analyte line. In addition, users of simultaneous multi-element instrumentation must assume the responsibility of verifying the absence of spectral interference from an element that could occur in a sample but for which there is no channel in the instrument array. Listed in Table 2 are some interference effects for the recommended wavelengths given in Table 1. The data in Table 2 are intended for use only as a rudimentary guide for the indication of potential spectral interferences. For this purpose, linear relations between concentration and intensity for the analytes and the interferents can be assumed. The interference information, which was collected at the Ames Laboratory(USDOE, Iowa State University, Ames Iowa 50011), is expressed at analyte concentration equivalents (i.e. false analyte concentrations) arising from 100 mg/L of the interferent element. The suggested use of this information is as follows: Assume that arsenic (at 193.696 nm) is to be determined in a sample

containing approximately 10 mg/L of aluminum. According to Table 2, 100 mg/L of aluminum would yield a false signal for arsenic equivalent to approximately 1.3 mg/L. Therefore, 10 mg/L of aluminum would result in a false signal for arsenic equivalent to approximately 0.13 mg/L. The reader is cautioned that other analytical systems may exhibit somewhat different levels of interference than those shown in Table 2, and that the interference effects must be evaluated for each individual system. Only those interferents listed were investigated and the blank spaces in Table 2 indicate that measurable interferences were not observed for the interferent concentrations listed in Table 3. Generally, interferences were discernible if they produced peaks or background shifts corresponding to 2-5% of the peaks generated by the analyte concentrations also listed in Table 3. At present, information on the listed silver and potassium wavelengths are not available but it has been reported that second order energy from the magnesium 383.231 nm wavelength interferes with the listed potassium line at 766.491 nm.

5.1.2 Physical interferences are generally considered to be effects associated with the sample nebulization and transport processes. Such properties as change in viscosity and surface tension can cause significant inaccuracies especially in samples which may contain high dissolved solids and/or acid concentrations. The use of a peristaltic pump may lessen these interferences. If these types of interferences are operative, they must be reduced by dilution of the sample and/or utilization of standard addition techniques. Another problem which can occur from high dissolved solids is salt buildup at the tip of the nebulizer. This affects aerosol flow-rate causing instrumental drift. Wetting the argon prior to nebulization, the use of a tip washer, or sample dilution have been used to control this problem. Also, it has been reported that better control of the argon flow rate improves instrument performance. This is accomplished with the use of mass flow controllers.

5.1.3 Chemical Interferences are characterized by molecular compound formation, ionization effects and solute vaporization effects. Normally these effects are not pronounced with the ICP technique, however, if observed they can be minimized by careful selection of operating conditions that is, incident power, observation position, and so forth), by buffering of the sample, by matrix matching, and by standard addition procedures. These types of interferences can be highly dependent on matrix type and the specific analyte element.

- 5.2 It is recommended that whenever a new or unusual sample matrix is encountered, a series of tests be performed prior to reporting concentration data for analyte elements. These tests, as outlined in 5.2.1 through 5.2.4, will ensure the analyst that neither positive nor

negative interference effects are operative on any of the analyte elements thereby distorting the accuracy of the reported values.

5.2.1 Serial dilution--If the analyte concentration is sufficiently high (minimally a factor of 10 above the instrumental detection limit after dilution), an analysis of a dilution should agree within 5 % of the original determination (or within some acceptable control limit (14.3) that has been established for that matrix). If not, a chemical or physical interference effect should be suspected.

5.2.2 Spike addition--The recovery of a spike addition added at a minimum level of 10X the instrumental detection limit (maximum 100X) to the original determination should be recovered to within 90 to 110 percent or within the established control limit for that matrix. If not, a matrix effect should be suspected. The use of a standard addition analysis procedure can usually compensate for this effect. Caution. The standard addition technique does not detect coincident spectral overlap. If suspected, use of computerized compensation, an alternate wavelength, or comparison with an alternate method is recommended. (See 5.2.3)

5.2.3 Comparison with alternate method of analysis--When investigating a new sample matrix, comparison tests may be performed with other analytical techniques such as atomic absorption spectrometry, or other approved methodology.

5.2.4 Wavelength scanning of analyte line region--If the appropriate equipment is available, wavelength scanning can be performed to detect potential spectral interferences.

## 6.0 Apparatus

### 6.1 Inductively Coupled Plasma Atomic Emission Spectrometer.

6.1.1 Computer controlled atomic emission spectrometer with background correction.

6.1.2 Radio frequency generator.

6.1.3 Argon gas supply, welding grade or better.

6.2 Operating conditions--Because of the differences between various makes and models of satisfactory instruments, no detailed operating instructions can be provided. Instead, the

analyst should follow the instructions provided by the manufacturer of the particular instrument. Sensitivity, instrumental detection limit, precision, linear dynamic range, and interference effects must be investigated and established for each individual analyte line on that particular instrument. It is the responsibility of the analyst to verify that the instrument configuration and operating conditions used satisfy the analytical requirements and to maintain quality control data confirming instrument performance and analytical results.

## 7.0 Reagents and standards

- 7.1 Acids used in the preparation of standards and for sample processing must be ultra-high purity grade or equivalent. Redistilled acids are acceptable.
- 7.1.1 Acetic acid, conc. (sp.gr 1.06).
  - 7.1.2 Hydrochloric acid, conc. (sp gr 1.19).
  - 7.1.3 Hydrochloric acid, (1+1): Add 500 mL conc. HCl (sp gr 1.19) to 400 mL deionized, distilled water and dilute to 1 liter.
  - 7.1.4 Nitric acid, conc. (sp gr 1.41).
  - 7.1.5 Nitric acid (1+1): Add 500 mL conc. HNO<sub>3</sub> (sp. gr 1.41) to 400 mL deionized, distilled water and dilute to 1 liter.
- 7.2 Deionized, distilled water. Prepare by passing distilled water through a mixed bed of cation and anion exchange resins. Use deionized, distilled water for the preparation of all reagents, calibration standards and as dilution water. The purity of this water must be equivalent to ASTM Type II reagent water of Specification D 1193 (14.6).
- 7.3 Standard stock solutions may be purchased or prepared from ultra high purity grade chemicals or metals. All salts must be dried for 1 h at 105-C unless otherwise specified. (CAUTION: Many metal salts are extremely toxic and may be fatal if swallowed. Wash hands thoroughly after handling.) Typical stock solution preparation procedures follow:
- 7.3.1 Aluminum solution, stock, 1 mL = 100 ug Al: Dissolve 0.100 g of aluminum metal in an acid mixture of 4 mL of (1+1) HCl and 1 mL of conc. HNO<sub>3</sub> in a beaker. Warm gently to effect solution. When solution is complete, transfer quantitatively to a liter flask, add an additional 10 mL of (1+1) HCl and dilute to 1,000 mL with deionized, distilled water.

7.3.2 Antimony solution stock, 1 mL = 100 ug Sb: Dissolve 0.2669 g K(SbO)C<sub>4</sub>H<sub>4</sub>O<sub>6</sub> in deionized distilled water, add 10 mL (1+1) HCl and dilute to 1000 mL with deionized, distilled water.

7.3.3 Arsenic solution, stock. 1 mL = 100 ug As: Dissolve 0.1320 g of As<sub>2</sub>O<sub>3</sub> in 100 mL of deionized, distilled water containing 0.4 g NaOH. Acidify the solution with 2 mL conc. HNO<sub>3</sub> and dilute to 1,000 mL with deionized, distilled water.

7.3.4 Barium solution, stock, 1 mL = 100 ug Ba: Dissolve 0.1516 g BaCl<sub>2</sub> (dried at 250-C for 2 hrs) in 10 mL deionized, distilled water with 1 mL (1+1) HCl. Add 10.0 mL (1+1) HCl and dilute to 1,000 mL with deionized, distilled water.

7.3.5 Beryllium solution, stock. 1 mL = 100 ug Be: Do not dry Dissolve 1.966 g BeSO<sub>4</sub>·4H<sub>2</sub>O, in deionized, distilled water, add 10.0 mL conc. HNO<sub>3</sub> and dilute to 1,000 mL with deionized, distilled water.

7.3.6 Boron solution. stock. 1 mL = 100 ug B: Do not dry. Dissolve 0.5716 g anhydrous H<sub>3</sub>BO<sub>3</sub> in deionized distilled water dilute to 1,000 mL. Use a reagent meeting ACS specifications, keep the bottle tightly stoppered and store in a desiccator to prevent the entrance of atmospheric moisture.

7.3.7 Cadmium solution, stock. 1 mL = 100 ug Cd: Dissolve 0.1142 g CdO in a minimum amount of (1+1) HNO<sub>3</sub>. Heat to increase rate of dissolution. Add 10.0 mL conc. HNO<sub>3</sub> and dilute to 1,000 mL with deionized, distilled water.

7.3.8 Calcium solution, stock, 1 mL = 100 ug Ca: Suspend 0.2498 g CaCO<sub>3</sub> dried at 180-C for 1 h before weighing in deionized, distilled water and dissolve cautiously with a minimum amount of (1+1) HNO<sub>3</sub>. Add 10.0 mL conc. HNO<sub>3</sub> and dilute to 1,000 mL with deionized, distilled water.

7.3.9 Chromium solution, stock. 1 mL = 100 ug Cr: Dissolve 0.1923 g of CrO<sub>3</sub> in deionized, distilled water. When solution is complete, acidify with 10 mL conc. HNO<sub>3</sub> and dilute to 1,000 mL with deionized, distilled water.

7.3.10 Cobalt solution, stock. 1 mL = 100 ug Co: Dissolve 0.1000 g of cobalt metal in a minimum amount of (1+1) HNO<sub>3</sub>. Add 10.0 mL (1+1) HCl and dilute to 1,000 mL with deionized, distilled water.

7.3.11 Copper solution, stock. 1 mL = 100 ug Cu: Dissolve 0.1252 g CuO in a minimum amount of (1+1) HNO<sub>3</sub>. Add 10.0 mL conc. HNO<sub>3</sub> and dilute to 1,000 mL with deionized, distilled water.

7.3.12 Iron solution. stock. 1 mL = 100 ug Fe: Dissolve 0.1430 g Fe<sub>2</sub>O<sub>3</sub> in a warm mixture of 20 mL (1+1) HCl and 2 mL of conc. HNO<sub>3</sub>. Cool, add an additional 5 mL of conc. HNO<sub>3</sub> and dilute to 1000 mL with deionized, distilled water.

7.3.13 Lead solution. stock, 1 mL = 100 ug Pb: Dissolve 0.1599 g Pb(NO<sub>3</sub>)<sub>2</sub> in minimum amount of (1+1) HNO<sub>3</sub>. Add 10.0 mL conc. HNO<sub>3</sub> and dilute to 1,000 mL with deionized, distilled water.

7.3.14 Magnesium solution, stock. 1 mL = 100 ug Mg: Dissolve 0.1658 g MgO in a minimum amount of (1+1) HNO<sub>3</sub>. Add 10.0 mL conc. HNO<sub>3</sub> and dilute to 1,000 mL with deionized, distilled water.

7.3.15 Manganese solution. stock. 1 mL = 100 ug Mn: Dissolve 0.1000 g of manganese metal in the acid mixture 10 mL conc. HCl and 1 mL conc. HNO<sub>3</sub>, and dilute to 1,000 mL with deionized, distilled water.

7.3.16 Molybdenum solution. stock. 1 mL = 100 ug Mo: Dissolve 0.2043 g (NH<sub>4</sub>)<sub>2</sub>MoO<sub>4</sub> in deionized, distilled water and dilute to 1,000 mL.

7.3.17 Nickel solution, stock, 1 mL = 100 ug Ni: Dissolve 0.1000 g of nickel metal in 10 mL hot conc. HNO<sub>3</sub>, cool and dilute to 1,000 mL with deionized, distilled water.

7.3.18 Potassium solution, stock. 1 mL = 100 ug K: Dissolve 0.1907 g KCl, dried at 110-C, in deionized, distilled water dilute to 1,000 mL.

7.3.19 Selenium solution, stock. 1 mL = 100 ug Se: Do not dry. Dissolve 0.1727 g H<sub>2</sub>SeO<sub>3</sub> (actual assay 94.6%) in deionized, distilled water and dilute to 1,000 mL.

7.3.20 Silica solution. stock, 1 mL = 100 ug SiO<sub>2</sub>: Do not dry. Dissolve 0.4730 g Na<sub>2</sub>SiO<sub>3</sub>·9H<sub>2</sub>O in deionized, distilled water. Add 10.0 mL conc. HNO<sub>3</sub> and dilute to 1,000 mL with deionized, distilled water.

7.3.21 Silver solution, stock. 1 mL = 100 ug Ag: Dissolve 0.1575 g AgNO<sub>3</sub> in 100 mL of deionized, distilled water and 10 mL conc. HNO<sub>3</sub>. Dilute to 1,000 mL with deionized distilled water.

7.3.22 Sodium solution, stock. 1 mL = 100 ug Na: Dissolve 0.2542 g NaCl in deionized, distilled water. Add 10.0 mL conc. HNO<sub>3</sub> and dilute to 1,000 mL with deionized, distilled water.

7.3.23 Thallium solution, stock, 1 mL = 100 ug Tl: Dissolve 0.1303 g TlNO<sub>3</sub> in deionized, distilled water. Add 10.0 mL conc. HNO<sub>3</sub> and dilute to 1,000 mL with deionized, distilled water.

7.3.24 Vanadium solution, stock. 1 mL = 100 ug V: Dissolve 0.2297 g NH<sub>4</sub>VO<sub>3</sub> in a minimum amount of conc. HNO<sub>3</sub>. Heat to increase rate of dissolution. Add 10.0 mL conc. HNO<sub>3</sub> and dilute to 1,000 mL with deionized, distilled water.

7.3.25 Zinc solution, stock. 1 mL = 100 ug Zn: Dissolve 0.1245 g ZnO in a minimum amount of dilute HNO<sub>3</sub>. Add 10.0 mL conc. HNO<sub>3</sub> and dilute to 1,000 mL with deionized, distilled water.

7.4 Mixed calibration standard solutions--Prepare mixed calibration standard solutions by combining appropriate volumes of the stock solutions in volumetric flasks. (See 7.4.1 thru 7.4.5) Add 2 mL of (1+1) HCl and dilute to 100 mL with deionized, distilled water. (See Notes 1 and 6.) Prior to preparing the mixed standards, each stock solution should be analyzed separately to determine possible spectral interference or the presence of impurities. Care should be taken when preparing the mixed standards that the elements are compatible and stable. Transfer the mixed standard solutions to a FEP fluorocarbon or unused polyethylene bottle for storage. Fresh mixed standards should be prepared as needed with the realization that concentration can change on aging. Calibration standards must be initially verified using a quality control sample and monitored weekly for stability (See 7.6.3). Although not specifically required, some typical calibration standard combinations follow when using those specific wavelengths listed in Table 1.

7.4.1 Mixed standard solution I-- Manganese, beryllium, cadmium, lead, and zinc.

7.4.2 Mixed standard solution II-- Barium, copper, iron, vanadium, and cobalt.

- 7.4.3 Mixed standard solution III-- Molybdenum, silica, arsenic, and selenium.
- 7.4.4 Mixed standard solution IV-- Calcium, sodium, potassium, aluminum, chromium and nickel
- 7.4.5 Mixed standard solution V-- Antimony, boron, magnesium, silver and thallium.

NOTE 1: If the addition of silver to the recommended acid combination results in an initial precipitation, add 15 mL of deionized distilled water and warm the flask until the solution clears. Cool and dilute to 100 mL with deionized, distilled water. For this acid combination the silver concentration should be limited to 2 mg/L. Silver under these conditions is stable in a tap water matrix for 30 days. Higher concentrations of silver require additional HCl.

7.5 Two types of blanks are required for the analysis. The calibration blank (3.13) is used in establishing the analytical curve while the reagent blank (3.12) is used to correct for possible contamination resulting from varying amounts of the acids used in the sample processing.

7.5.1 The calibration blank is prepared by diluting 2 mL of (1+1) HNO<sub>3</sub> and 10 mL of (1+1) HCl to 100 mL with deionized, distilled water. (See Note 6.) Prepare a sufficient quantity to be used to flush the system between standards and samples.

7.5.2 The reagent blank must contain all the reagents and in the same volumes as used in the processing of the samples. The reagent blank must be carried through the complete procedure and contain the same acid concentration in the final solution as the sample solution used for analysis.

7.6 In addition to the calibration standards, an instrument check standard (3.7), an interference check sample (3.8) and a quality control sample (3.9) are also required for the analyses.

7.6.1 The instrument check standard is prepared by the analyst by combining compatible elements at a concentration equivalent to the midpoint of their respective calibration curves. (See 12.1.1)

7.6.2 The interference check sample is prepared by the analyst in the following manner. Select a representative sample which contains minimal concentrations of the analytes of interest by known concentration of interfering elements that will

provide an adequate test of the correction factors. Spike the sample with the elements of interest at the approximate concentration of either 100 ug/L or 5 times the estimated detection limits given in Table 1. (For effluent samples of expected high concentrations, spike at an appropriate level.) If the type of samples analyzed are varied, a synthetically prepared sample may be used if the above criteria and intent are met. A limited supply of a synthetic interference check sample will be available from the Quality Assurance Branch of EMSL- Cincinnati. (See 12.1.2)

7.6.3 The quality control sample should be prepared in the same acid matrix as the calibration standards at a concentration near 1 mg/L and in accordance with the instructions provided by the supplier. The Quality Assurance Branch of EMSL- Cincinnati will either supply a quality control sample or information where one of equal quality can be procured. (See 12.1.3)

## 8.0 Sample handling and preservation

8.1 For the determination of trace elements, contamination and loss are of prime concern. Dust in the laboratory environment, impurities in reagents and impurities on laboratory apparatus which the sample contacts are all sources of potential contamination. Sample containers can introduce either positive or negative errors in the measurement of trace elements by (a) contributing contaminants through leaching or surface desorption and (b) by depleting concentrations through adsorption. Thus the collection and treatment of the sample prior to analysis requires particular attention. Laboratory glassware including the sample bottle (whether polyethylene, polypropylene or FEP-fluorocarbon) should be thoroughly washed with detergent and tap water; rinsed with (1+1) nitric acid, tap water, (1+1) hydrochloric acid, tap and finally deionized distilled water in that order (See Notes 2 and 3).

NOTE 2: Chromic acid may be useful to remove organic deposits from glassware; however, the analyst should be cautioned that the glassware must be thoroughly rinsed with water to remove the last traces of chromium. This is especially important if chromium is to be included in the analytical scheme. A commercial product, NOCHROMIX, available from Godax Laboratories, 6 Varick St., New York, NY 10013, may be used in place of chromic acid. Chromic acid should not be used with plastic bottles.

NOTE 3: If it can be documented through an active analytical quality control program using spiked samples and reagent blanks, that certain steps in the cleaning procedure are not required for routine samples, those steps may be eliminated from the procedure.

8.2 Before collection of the sample a decision must be made as to the type of data desired, that is dissolved, suspended or total, so that the appropriate preservation and pretreatment

steps may be accomplished. Filtration, acid preservation, etc., are to be performed at the time the sample is collected or as soon as possible, thereafter.

8.2.1 For the determination of dissolved elements the sample must be filtered through a 0.45-um membrane filter as soon as practical after collection. (Glass or plastic filtering apparatus are recommended to avoid possible contamination.) Use the first 50 -100 mL to rinse the filter flask. Discard this portion and collect the required volume of filtrate. Acidify the filtrate with (1+1 )HNO<sub>3</sub> to a pH of 2 or less. Normally, 3 mL of (1+1) acid per liter should be sufficient to preserve the sample.

8.2.2 For the determination of suspended elements a measured volume of unpreserved sample must be filtered through a 0.45-um membrane filter as soon as practical after collection. The filter plus suspended material should be transferred to a suitable container for storage and/or shipment. No preservative is required.

8.2.3 For the determination of total or total recoverable elements, the sample is acidified with (1+1 ) HNO<sub>3</sub> to pH 2 or less as soon as possible, preferable at the time of collection. The sample is not filtered before processing.

## 9.0 Sample Preparation

9.1 For the determinations of dissolved elements, the filtered, preserved sample may often be analyzed as received. The acid matrix and concentration of the samples and calibration standards must be the same. (See Note 6.) If a precipitate formed upon acidification of the sample or during transit or storage, it must be redissolved before the analysis by adding additional acid and/or by heat as described in 9.3.

9.2 For the determination of suspended elements, transfer the membrane filter containing the insoluble material to a 150-mL Griffin beaker and add 4 mL conc. HNO<sub>3</sub>. Cover the beaker with a watch glass and heat gently. The warm acid will soon dissolve the membrane. Increase the temperature of the hot plate and digest the material. When the acid has nearly evaporated. cool the beaker and watch glass and add another 3 mL of conc. HNO<sub>3</sub>. Cover and continue heating until the digestion is complete, generally indicated by a light colored digestate. Evaporate to near dryness (2 mL), cool, add 10 mL HCl (1+1) and 15 Ml deionized, distilled water per 100 mL dilution and warm the beaker gently for 15 min. to dissolve any precipitated or residue material. Allow to cool, wash down the watch glass and beaker walls with deionized distilled water and filter the sample to remove insoluble material that could clog the nebulizer. (See Note 4.) Adjust the volume based on the expected concentrations of elements present. This volume will

vary depending on the elements to be determined (See Note 6). The sample is now ready for analysis. Concentrations so determined shall be reported as "suspended."

NOTE 4: In place of filtering, the sample after diluting and mixing may be centrifuged or allowed to settle by gravity overnight to remove insoluble material.

9.3 For the determination of total elements, choose a measured, volume of the well mixed acid preserved sample appropriate for the expected level of elements and transfer to a Griffin beaker. (See Note 5.) Add 3 mL of conc. HNO<sub>3</sub>. Place the beaker on a hot plate and evaporate to near dryness cautiously, making certain that the sample does not boil and that no area of the bottom of the beaker is allowed to go dry. Cool the beaker and add another 5 mL portion of conc. HNO<sub>3</sub>. Cover the beaker with a watch glass and return to the hot plate. Increase the temperature of the hot plate so that a gentle reflux action occurs. Continue heating, adding additional acid as necessary, until the digestion is complete (generally indicated when the digestate is light in color or does not change in appearance with continued refluxing.) Again, evaporate to near dryness and cool the beaker. Add 10 mL of 1+1 HCl and 15 mL of deionized, distilled water per 100 mL of final solution and warm the beaker gently for 15 min. to dissolve any precipitate or residue resulting from evaporation. Allow to cool, wash down the beaker walls and watch glass with deionized distilled water and filter the sample to remove insoluble material that could clog the nebulizer. (See Note 4.) Adjust the sample to a predetermined volume based on the expected concentrations of elements present. The sample is now ready for analysis (See Note 6). Concentrations so determined shall be reported as "total."

NOTE 5: If low determinations of boron are critical, quartz glassware should be use.

NOTE 6: If the sample analysis solution has a different acid concentration from that given in 9.4, but does not introduce a physical interference or affect the analytical result, the same calibration standards may be used.

9.4 For the determination of total recoverable elements, choose a measured volume of a well mixed, acid preserved sample appropriate for the expected level of elements and transfer to a Griffin beaker. (See Note 5.) Add 2 mL of (1+1) HNO<sub>3</sub> and 10 mL of (1+1) HCl to the sample and heat on a steam bath or hot plate until the volume has been reduced to near 25 mL making certain the sample does not boil. After this treatment, cool the sample and filter to remove insoluble material that could clog the nebulizer. (See Note 4.) Adjust the volume to 100 mL and mix. The sample is now ready for analysis. Concentrations so determined shall be reported as "total."

## 10.0 Procedure

- 10.1 Set up instrument with proper operating parameters established in 6.2 The instrument must be allowed to become thermally stable before beginning. This usually requires at least 30 min. of operation prior to calibration.
- 10.2 Initiate appropriate operating configuration of computer.
- 10.3 Profile and calibrate instrument according to instrument manufacturer's recommended procedures, using the typical mixed calibration standard solutions described in 7.4. Flush the system with the calibration blank (7.5.1 ) between each standard. (See Note 7.) (The use of the average intensity of multiple exposures for both standardization and sample analysis has been found to reduce random error.)

NOTE 7: For boron concentrations greater than 500 ug/L extended flush times of 1 to 2 min. may be required.

- 10.4 Before beginning the sample run, reanalyze the highest mixed calibration standard as if it were a sample. Concentration values obtained should not deviate from the actual values by more than +/- 5 percent (or the established control limits whichever is lower). If they do, follow the recommendations of the instrument manufacturer to correct for this condition.
- 10.5 Begin the sample run flushing the system with the calibration blank solution (7.5.1 ) between each sample. (See Note 7.) Analyze the instrument check standard (7.6.1) and the calibration blank (7.5.1) each 10 samples.
- 10.6 If it has been found that method of standard addition are required, the following procedure is recommended.

10.6.1 The standard addition technique (14.2) involves preparing new standards in the sample matrix by adding known amounts of standard to one or more aliquots of the processed sample solution. This technique compensates for a sample constituent that enhances or depresses the analyte signal thus producing a different slope from that of the calibration standards. It will not correct for additive interference which causes a baseline shift. The simplest version of this technique is the single-addition method. The procedure is as follows. Two identical aliquots of the sample solution, each of volume  $V(x)$ , are taken. To the first (labeled A) is added a small volume  $V(s)$  of a standard analyte solution of concentration  $c(s)$ . To the second (labeled B) is added the same volume  $V(s)$  of the solvent. The analytical signals of A and B are measured and corrected for nonanalyte signals. The unknown sample concentration  $c(x)$  is calculated:

$$C(x) = S(B)*V(S)*c(s)/ ((S(A) - S(B))* V(x))$$

where S(A) and S(B) are the analytical signals (corrected for the blank) of solutions A and B, respectively. V(s) and c(s) should be chosen so that S(A) is roughly twice S(B) on the average. It is best if V(s) is made much less than V(x), and thus c(s) is much greater than c(x), to avoid excess dilution of the sample matrix. If a separation or concentration step is used, the additions are best made first and carried through the entire procedure. For the results from this technique to be valid, the following limitations must be taken into consideration:

1. The analytical curve must be linear.
2. The chemical form of the analyte added must respond the same as the analyte in the sample.
3. The interference effect must be constant over the working range of concern.
4. The signal must be corrected for any additive interference.

### **11.0 Calculation**

- 11.1 Reagent blanks (7.5.2) should be subtracted from all samples. This is particularly important for digested samples requiring large quantities of acids to complete the digestion.
- 11.2 If dilutions were performed, the appropriate factor must be applied to sample values.
- 11.3 Data should be rounded to the thousandth place and all results should be reported in mg/L up to three significant figures.

### **12.0 Quality Control (Instrumental)**

- 12.1 Check the instrument standardization by analyzing appropriate quality control check standards as follow:

12.1.1 Analyze an appropriate instrument check standard (7.6.1) containing the elements of interest at a frequency of 10%. This check standard is used to determine instrument drift. If agreement is not within +/- 5% of the expected values or within the established control limits, whichever is lower, the analysis is out of control. The analysis should be terminated, the problem corrected, and the instrument recalibrated. Analyze the calibration blank (7.5.1) at a frequency of 10%. The result should be within the established control limits of two standard

deviations of the mean value. If not, repeat the analysis two more times and average the three results. If the average is not within the control limit, terminate the analysis, correct the problem and recalibrate the instrument.

12.1.2 To verify inter-element and background correction factors analyze the interference check sample (7.6.2) at the beginning, end, and at periodic intervals throughout the sample run. Results should fall within the established control limits of 1.5 times the standard deviation of the mean value. If not, terminate the analysis, correct the problem and recalibrate the instrument.

12.1.3 A quality control sample (7.6.3) obtained from an outside source must first be used for the initial verification of the calibration standards. A fresh dilution of this sample shall be analyzed every week thereafter to monitor their stability. If the results are not within +/- 5% of the true value listed for the control sample, prepare a new calibration standard and recalibrate the instrument. If this does not correct the problem, prepare a new stock standard and a new calibration standard and repeat the calibration.

### **13.0 Precision and Accuracy**

13.1 In an EPA round robin phase 1 study, seven laboratories applied the ICP technique to acid-distilled water matrices that had been dosed with various metal concentrates. Table 4 lists the true value, the mean reported value and the mean % relative standard deviation.

### **14.0 References**

1. Winge, R.K., V.J. Peterson, and V.A. Fassel, "Inductively Coupled Plasma-Atomic Emission Spectroscopy: Prominent Lines," EPA-600/4-79-017.
2. Winefordner, J.D., "Trace Analysis: Spectroscopic Methods for Elements," Chemical Analysis, Vol. 46, pp. 41-42.
3. Handbook for Analytical Quality Control in Water and Wastewater Laboratories, EPA-600/4-79-019.
4. Garbarino, J.R. and Taylor, H.E., "An Inductively-Coupled Plasma Atomic Emission Spectrometric Method for Routine Water Quality Testing," Applied Spectroscopy 33, No. 3(1979)
5. "Methods for Chemical Analysis of Water and Wastes," EPA-600/4-79-020.
6. Annual Book of ASTM Standards Part 31.
7. "Carcinogens - Working With Carcinogens," Department of Health, Education, and Welfare, Public Health Service, Center for Disease Control, National Institute for Occupational Safety and Health, Publication No. 77-206, Aug. 1977.

8. "OSHA Safety and Health Standards, General Industry," (29 CFR 1910), Occupational Safety and Health Administration, OSHA 2206, (Revised, January 1976).
9. "Safety in Academic Chemistry Laboratories," American Chemical Society Publication. Committee on Chemical Safety, 3rd Edition, 1979.

**TABLE 1. Recommended Wavelengths(1)  
and Estimated Instrumental Detection Limits Estimated detection**

Element	Wavelength, Nm	Limit, ug/L <sup>2</sup>
Aluminum	308.215	45
Arsenic	193.696	53
Antimony	206.833	32
Barium	455.403	2
Beryllium	313.042	0.3
Boron	249.773	5
Cadmium	226.502	4
Calcium	917.933	10
Chromium	267.716	7
Cobalt	228.616	7
Copper	324.754	6
Iron	259.940	7
Lead	220.353	42
Magnesium	279.079	30
Manganese	257.610	2
Molybdenum	202.030	8
Nickel	231.604	15
Potassium	766.491	See (3)
Selenium	196.026	75
Silica (SiO <sub>2</sub> )	288.158	58
Silver	328.068	7
Sodium	588.995	29
Thallium	190.864	40
Vanadium	292.402	8
Zinc	213.856	2

(1) The wavelengths listed are recommended because of their sensitivity and overall acceptance. Other wavelengths may be substituted if they can provide the needed sensitivity and are treated with the same corrective techniques for spectral interference. (See 5.1.1.).

(2) The estimated instrumental detection limits as shown are taken from "Inductively Coupled Plasma-Atomic Emission Spectroscopy-Prominent Lines," EPA-600/4-79-017. They are given as

a guide for an instrumental limit. The actual method detection limits are sample dependent and may vary as the sample matrix varies.

(3) Highly dependent on operating conditions and plasma position.

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**Table 2. Analyte Concentration Equivalents (mg/L)  
Arising From Interferents at the 100 mg/L Level**

Analyte	Wavelength, nm	Interferent									
		Al	Ca	Cr	Cu	Fe	Mg	Mn	Ni	Ti	V
Aluminum	308.215							0.21			1.4
Antimony	206.833	0.47		2.9		0.08				.25	0.45
Arsenic	193.696	1.3		0.44							1.1
Barium	455.403										
Beryllium	313.042									0.04	0.05
Boron	249.773	0.4				0.32					
Cadmium	226.502					0.03			0.02		
Calcium	317.933			0.08		0.01	0.01	0.04		0.03	0.03
Chromium	267.716					0.003		0.04			0.04
Cobalt	228.616			0.03		0.005			0.03	0.15	
Copper	324.754					0.003				0.05	0.02
Iron	259.940							0.12			
Lead	220.353	0.17									
Magnesium	279.079		0.02	0.11		0.13		0.25		0.07	0.12
Manganese	257.610	0.005		0.01		0.002	0.002				
Molybdenum	202.030	0.05				0.03					
Nickel	231.604										
Selenium	196.026	0.23				0.09					
Silicon	288.158			0.07							0.01
Sodium	588.995									0.08	
Thallium	190.864	0.30									
Vanadium	292.402			0.05		0.005				0.02	
Zinc	213.856				0.14				0.29		

**Table 3. Interferent and Analyte Elemental Concentrations Used for Interference Measurements in Table 2.**

Analytes	(mg/L)	Interferents	(mg/L)
Al	10	Al	1000
As	10	Ca	1000
B	10	Cr	200
Ba	1	Cu	200
Be	1	Fe	1000
Ca	1	Mg	1000
Cd	10	Mn	200
Co	1	Ni	200
Cr	1	Ti	200
Cu	1	V	200
Fe	1		
Mg	1		
Mn	1		
Mo	10		
Na	10		
Ni	10		
Pb	10		
Sb	10		
Se	10		
Si	1		
Tl	10		
V	1		
Zn	10		

**Table 4. ICP Precision and Accuracy Data**

	Sample 1 Mean			Sample 2 Mean			Sample 3 Mean		
	True Value	Reported Value	Mean Percent	True Value	Reported Value	Mean Percent	True Value	Reported Value	Mean Percent
Element	ug/L	ug/L	RSD	ug/L	ug/L	RSD	ug/L	ug/L	RSD
Be	750	733	6.2	20	20	9.8	180	176	5.2
Mn	350	345	2.7	15	15	6.7	100	99	3.3
V	750	749	1.8	70	69	2.9	170	169	1.1
As	200	208	7.5	22	19	23	60	63	17
Cr	150	149	3.8	10	10	18	50	50	3.3
Cu	250	235	5.1	11	11	40	70	67	7.9
Fe	600	594	3.0	20	19	15	180	178	6.0
Al	700	696	5.6	60	62	33	160	161	13
Cd	50	48	12	2.5	2.9	16	14	13	16
Co	500	512	10	20	20	4.1	120	108	21
Ni	250	245	5.8	30	28	11	60	55	14
Pb	250	236	16	24	30	32	80	80	14
Zn	200	201	5.6	16	19	45	80	82	9.4
Se	40	32	21.9	6	8.5	42	10	8.5	8.3

Not all elements were analyzed by all laboratories.

**METHOD #: 305.1**

**Approved for NPDES (Technical Revision 1974)**

**TITLE: Acidity (Titrimetric)**

**ANALYTE:**

Acidity

**INSTRUMENTATION:** Titration

**STORET No.** 70508

### **1.0 Scope and Application**

- 1.1 This method is applicable to surface waters, sewages and industrial wastes, particularly mine drainage and receiving streams, and other waters containing ferrous iron or other polyvalent cations in a reduced state.
- 1.2 The method covers the range from approximately 10 mg/L acidity to approximately 1000 mg/L as CaCO<sub>3</sub>, using a 50 mL sample.

### **2.0 Summary of Method**

- 2.1 The pH of the sample is determined and a measured amount of standard acid is added, as needed, to lower the pH to 4 or less. Hydrogen peroxide is added, the solution boiled for several minutes, cooled, and titrated electrometrically with standard alkali to pH 8.2.

### **3.0 Definitions**

- 3.1 This method measures the mineral acidity of a sample plus the acidity resulting from oxidation and hydrolysis of polyvalent cations, including salts of iron and aluminum.

### **4.0 Interferences**

- 4.1 Suspended matter present in the sample, or precipitates formed during the titration may cause a sluggish electrode response. This may be offset by allowing a 15-20 second pause between additions of titrant or by slow dropwise addition of titrant as the endpoint pH is approached.

### **5.0 Apparatus**

5.1 pH meter, suitable for electrometric titrations.

## 6.0 Reagents

6.1 Hydrogen peroxide (H<sub>2</sub>O<sub>2</sub>, 30% solution).

6.2 Standard sodium hydroxide, 0.02 N.

6.3 Standard sulfuric acid, 0.02 N.

## 7.0 Procedure

7.1 Pipet 50 mL of the sample into a 250 mL beaker.

7.2 Measure the pH of the sample. If the pH is above 4.0, add standard sulfuric acid (6.3) in 5.0 mL increments to lower the pH to 4.0 or less. If the initial pH of the sample is less than 4.0, the incremental addition of sulfuric acid is not required.

7.3 Add 5 drops of hydrogen peroxide (6.1).

7.4 Heat the sample to boiling and continue boiling for 2 to 4 minutes. In some instances, the concentration of ferrous iron in a sample is such that an additional amount of hydrogen peroxide and a slightly longer boiling time may be required.

7.5 Cool the sample to room temperature and titrate electrometrically with standard sodium hydroxide (6.2) to pH 8.2.

## 8.0 Calculations

8.1 Acidity, as mg/L CaCO<sub>3</sub> = 
$$\frac{[(A * B) - (C * D)] * 50,000}{\text{mL of sample}}$$

where:

A = vol. of standard sodium hydroxide used in titration

B = normality of standard sodium hydroxide

C = volume of standard sulfuric acid used to reduce pH to 4 or less

D = normality of standard sulfuric acid

8.2 If it is desired to report acidity in milliequivalents per liter, the reported values as CaCO<sub>3</sub> are divided by 50, as follows:

$$\text{Acidity as meq/L} = \frac{\text{mg/L CaCO}_3}{50}$$

## 9.0 Precision

- 9.1 On a round robin conducted by ASTM on 4 acid mine waters, including concentrations up to 2000 mg/ L , the precision was found to be +/- 10 mg/L.

## Bibliography

1. Annual Book of ASTM Standards, Part 31, "Water", p 116, D 1067, Method E(1976).
2. Standard Methods for the Examination of Water and Wastewater, 14th Edition, p 277, Method 402(4d) (1975).

**METHOD #: 310.1**                      **Approved for NPDES (Editorial Revision 1978)**

**TITLE: Alkalinity (Titrimetric, pH 4.5)**

**ANALYTE:**

**Alkalinity**

**INSTRUMENTATION:** Titration

**STORET No.** 00410

**1.0 Scope and Application**

- 1.1 This method is applicable to drinking, surface, and saline waters, domestic and industrial wastes.
- 1.2 The method is suitable for all concentration ranges of alkalinity; however, appropriate aliquots should be used to avoid a titration volume greater than 50 mL.
- 1.3 Automated titrimetric analysis is equivalent.

**2.0 Summary of Method**

- 2.1 An unaltered sample is titrated to an electrometrically determined end point of pH 4.5. The sample must not be filtered, diluted, concentrated, or altered in any way.

**3.0 Comments**

- 3.1 The sample should be refrigerated at 4-C and run as soon as practical. Do not open sample bottle before analysis.
- 3.2 Substances, such as salts of weak organic and inorganic acids present in large amounts, may cause interference in the electrometric pH measurements.
- 3.4 For samples having high concentrations of mineral acids, such as mine wastes and associated receiving waters, titrate to an electrometric endpoint of pH 3.9, using the procedure in: Annual Book of ASTM Standards, Part 31, "Water", p 115, D- 1067, Method D, ( 1976).

- 3.5 Oil and grease, by coating the pH electrode, may also interfere, causing sluggish response.

#### 4.0 Apparatus

- 4.1 pH meter or electrically operated titrator that uses a glass electrode and can be read to 0.05 pH units. Standardize and calibrate according to manufacturer's instructions. If automatic temperature compensation is not provided, make titration at 25 +/-2- C.
- 4.2 Use an appropriate sized vessel to keep the air space above the solution at a minimum. Use a rubber stopper fitted with holes for the glass electrode, reference electrode (or combination electrode) and buret.
- 4.3 Magnetic stirrer, pipets, flasks and other standard laboratory equipment.
- 4.4 Burets, Pyrex 50, 25 and 10 mL.

#### 5.0 Reagents

- 5.1 Sodium carbonate solution, approximately 0.05 N: Place 2.5 +/-0.2 g (to nearest mg) Na<sub>2</sub>CO<sub>3</sub> (dried at 250-C for 4 hours and cooled in desiccator) into a 1 liter volumetric flask and dilute to the mark.
- 5.2 Standard acid (sulfuric or hydrochloric), 0.1 N: Dilute 3.0 mL conc H<sub>2</sub>SO<sub>4</sub> or 8.3 mL conc HCl to 1 liter with distilled water. Standardize versus 40.0 mL of 0.05 N Na<sub>2</sub>CO<sub>3</sub> solution with about 60 mL distilled water by titrating potentiometrically to pH of about 5. Lift electrode and rinse into beaker. Boil solution gently for 3-5 minutes under a watch glass cover. Cool to room temperature. Rinse cover glass into beaker. Continue titration to the pH inflection point. Calculate normality using:

$$N = \frac{A * B}{53.00 * C}$$

where:

A = g Na<sub>2</sub>CO<sub>3</sub> weighed into 1 liter

B = mL Na<sub>2</sub>CO<sub>3</sub> solution

C = mL acid used to inflection point

- 5.3 Standard acid (sulfuric or hydrochloric), 0.02 N: Dilute 200.0 mL of 0.1000 N standard acid to 1 liter with distilled water. Standardize by potentiometric titration of 15.0 mL 0.05 N Na<sub>2</sub>CO<sub>3</sub> solution as above.

## 6.0 Procedure

### 6.1 Sample size

6.1.1 Use a sufficiently large volume of titrant (> 20 mL in a 50 mL buret) to obtain good precision while keeping volume low enough to permit sharp end point.

6.1.2 For < 1000 mg CaCO<sub>3</sub> /L use 0.02 N titrant

6.1.3 For > 1000 mg CaCO<sub>3</sub> /L use 0.1 N titrant

6.1.4 A preliminary titration is helpful.

### 6.2 Potentiometric titration

6.2.1 Place sample in flask by pipetting with pipet tip near bottom of flask

6.2.2 Measure pH of sample

6.2.3 Add standard acid (5.2 or 5.3), being careful to stir thoroughly but gently to allow needle to obtain equilibrium.

6.2.4 Titrate to pH 4.5. Record volume of titrant.

### 6.3 Potentiometric titration of low alkalinity

6.3.1 For alkalinity of <20 mg/L titrate 100-200 mL as above (6.2) using a 10 mL microburet and 0.02 N acid solution (5.3).

6.3.2 Stop titration at pH in range of 4.3-4.7, record volume and exact pH. Very carefully add titrant to lower pH exactly 0.3 pH units and record volume.

## 7.0 Calculations

7.1 Potentiometric titration to pH 4.5

$$\text{Alkalinity, mg/L CaCO}_3 = \frac{A * N * 50,000}{\text{mL of sample}}$$

where:

A = mL standard acid

N = normality standard acid

## 7.2 Potentiometric titration of low alkalinity:

$$\text{Total alkalinity, mg/L CaCO}_3 = \frac{(2B - C) * N * 50,000}{\text{mL of sample}}$$

where:

B = mL titrant to first recorded pH

C = total mL titrant to reach pH 0.3 units lower

N = normality of acid

## 8.0 Precision and Accuracy

8.1 Forty analysts in seventeen laboratories analyzed synthetic water samples containing increments of bicarbonate, with the following results:

Increment as Alkalinity	Precision as Standard Deviation	Accuracy As	
		Bias	Bias
Mg/liter, CaCO <sub>3</sub>	mg/liter, CaCO <sub>3</sub>	%	mg/L, CaCO <sub>3</sub>
8	1.27	+10.61	+0.85
9	1.14	+22.29	+2.0
113	5.28	-8.19	-9.3
119	5.36	-7.42	-8.8

(FWPCA Method Study 1, Mineral and Physical Analyses)

8.2 In a single laboratory (EMSL) using surface water samples at an average concentration of 122 mg CaCO<sub>3</sub>/L, the standard deviation was +/-3.

## Bibliography

1. Standard Methods for the Examination of Water and Wastewater, 14th Edition, p 278, Method 403, (1975).
2. Annual Book of ASTM Standards, Part 31, "Water", p 113, D-1067, Method B, (1976).



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**Appendix F**

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**Scope of Work and Pricing Schedule, Coring and Grouting Services**

**Technical Specifications, Coring and Grouting Services**

*DRAFT*

**ATTACHMENT I**

**SCOPE OF WORK AND PRICING SCHEDULE**

**CORING AND GROUTING SERVICES**

**FOR**

**WALKER MINE SEAL TESTING AND EVALUATION PROJECT  
PLUMAS COUNTY, CALIFORNIA**

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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. Its cross-section is about 9 feet by 12 feet 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 valves. A sampling port with a pressure transducer is mounted on one of the drain pipes. The transducer is connected to a datalogger 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 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 smallest 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.

## 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 holes, (4) pressure grout the surrounding rock, and (5) tightly backfill the holes with grout. The purposes of the core holes are to observe concrete and rock conditions, collect continuous concrete and rock core samples for laboratory testing, allow for nondestructive testing of the concrete seal and concrete-rock interface using the holes to insert the testing probes, and pressure grout the rock mass surrounding the seal to fill open seepage features and reduce seepage flows along the concrete-rock interface. Approximate boring locations and orientations are shown on Figure 4. 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**

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 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 Contractor.
- Installation of compressed air, water and/or power lines to the mine seal as necessary to perform the work.
- Installation of sanitary and potable water facilities.
- All other necessary preparatory work to be performed prior to the coring and grouting work.

**2.2.2 CORE DRILLING AND GROUTING**

Core drilling and grouting includes but is not limited to the following:

- Installation of seven 4-inch grout pipes, packers, valves and pressure gauges in the mine seal. Grout pipes will be located and oriented as approximately shown on Figure 4 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 of approximately eight feet of concrete and rock through each of the seven grout pipes with a 3-7/8-inch core bit, yielding a 2-11/16-inch core.
- 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 12-hour intervals.
- Providing two full 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.

- At the completion of testing by the Engineer and other subcontractors, washing, water pressure testing, and pressure grouting 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**

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 MODIFICATIONS TO SCOPE OF WORK**

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.

## **3. PRICING SCHEDULE**

Work will be paid at Unit and Lump Sum prices listed in the Pricing Schedule. 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 listed below:

- Core boxes: Core boxes actually used to store and transport rock and concrete cores.
- Microfine cement and portland cement: Measurement for payment for providing microfine and portland cement will be made at the end of each shift, and will include all microfine and portland cement consumed in mixing grout during that shift as determined by the Engineer. No payment will be made for microfine or portland cement used in grout that is wasted or lost due to improper anchorage of grout nipples, or as a result of equipment breakdown. Microfine or portland cement used in grout satisfactorily mixed but not injected for reasons beyond the Contractor'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 microfine cement and portland cement until it is used to produce grout. The unit of measure will be a sack for portland cement (equivalent to 1 cubic foot or 94 pounds of cement) and a bag (equivalent to 64 pounds) for microfine cement.

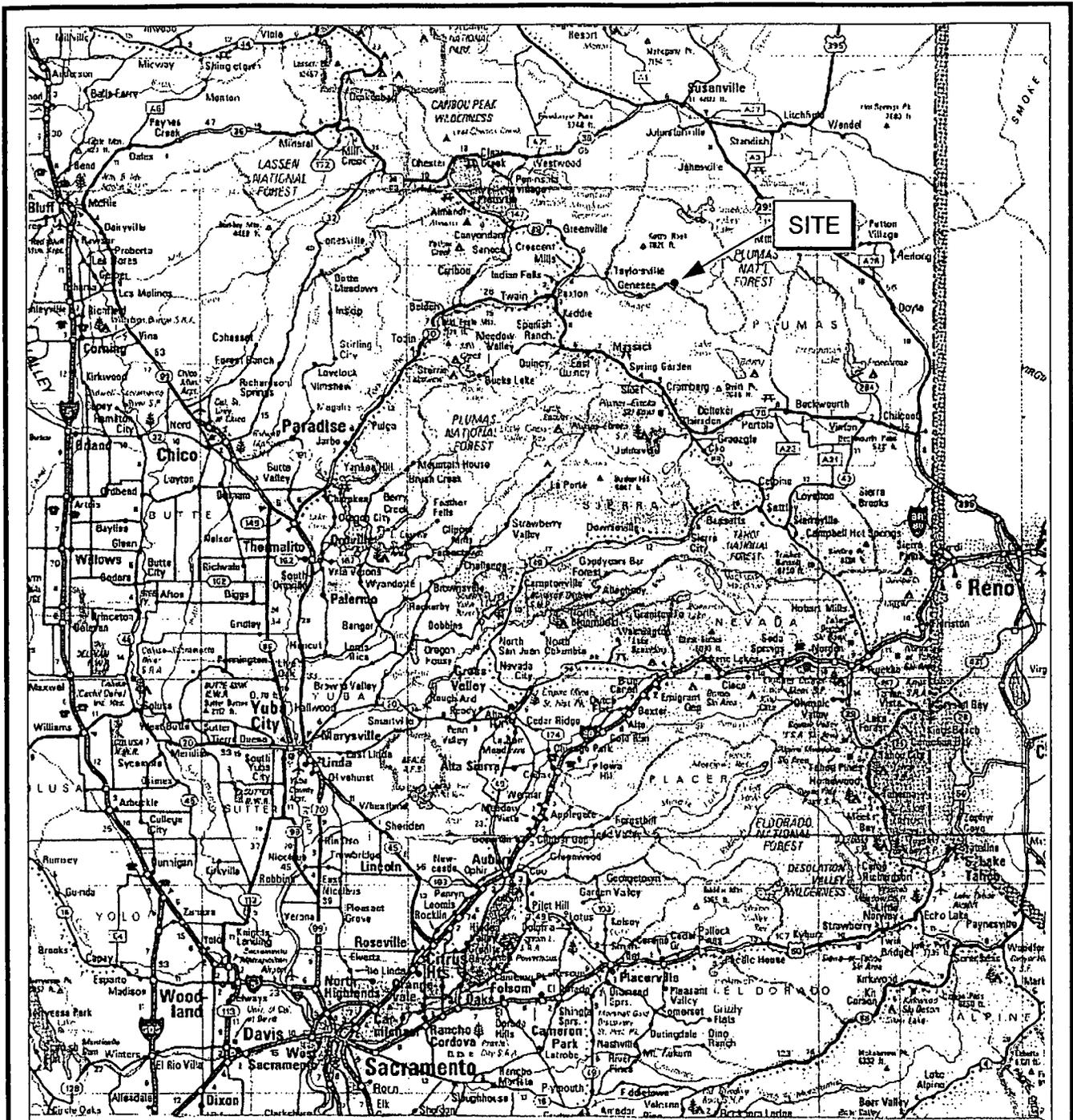
- Add/subtract 8-foot-long core hole: Addition or removal of each 8-foot-long hole to the seven holes included in Item 2. The work includes all activities covered under Item 2 for the added or deleted hole.
- Add/subtract length of core hole: Engineer-directed reduction or increase in the length of drilling 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 reduction or increase in duration of standby time beyond the two full calendar days which are included in Item 2. The work includes all activities covered under Item 2 for the standby time.

Payment will not be made for any of the following:

1. Materials wasted or disposed of in a manner that is not called for in the subcontract.
2. Materials determined as unacceptable before or after placement.
3. Materials not completely unloaded from the transporting vehicle.
4. Core holes drilled beyond the length of the required work as indicated in the subcontract or as directed by the Engineer.
5. Materials remaining on hand after completion of the work.
6. Loading, hauling, handling, and disposal of rejected materials.
7. Standby time due to missing equipment, equipment breakdown, or any other reason except as expressly directed by the Engineer.

## PRICING SCHEDULE

Item No.	Item Description	Units	Quantity	Unit Price	Total Price
1	Mobilization	Lump Sum	1		
2	Core Drilling and Grouting	Lump Sum	1		
3	Demobilization	Lump Sum	1		
4	Core Boxes	Each	7		
5	Microfine Cement	Bag	10		
6	Type II Portland Cement	Sack	10		
7	Add/Subtract an 8-foot-long Core Hole	Each	-		
8	Add/Subtract Length of Core Hole	Foot	-		
9	Add/Subtract Standby Time	Calendar day	-		
TOTAL					



**NOTE**

Map is taken from California road map dated 1998.

LOCATION OF SITE ABOVE

Regional Water Quality Control Board  
Central Valley Region

Walker Mine  
Seal Testing & Evaluation  
Portola, CA

PROJECT LOCATION MAP



GEI Consultants, Inc.

Project 00387

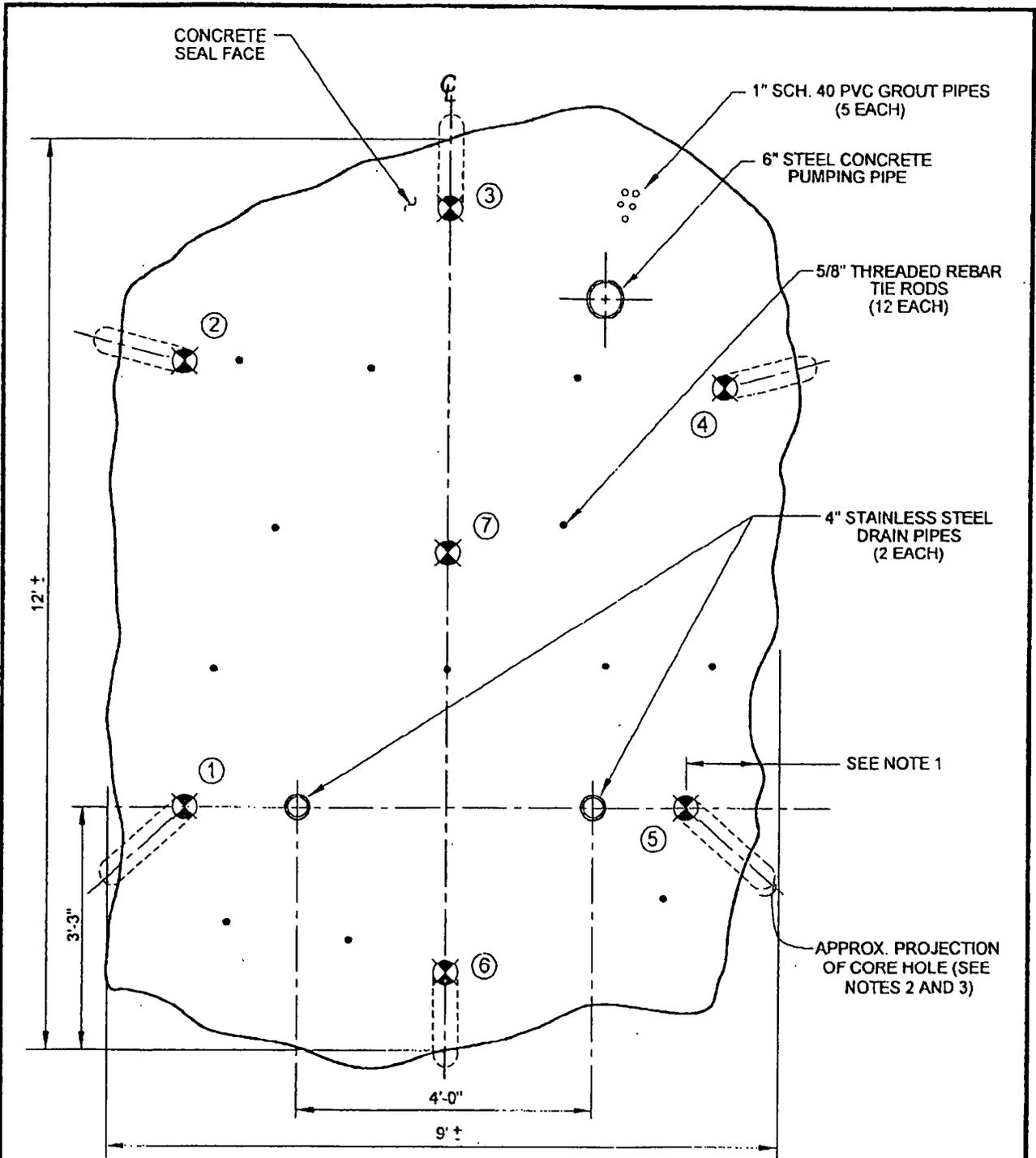
Nov. 2000

Figure 1

Fig-1.dwg 11-30-00 PYM







**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.



Fig-7.dwg 12-04-00 PYM

Regional Water Quality Control Board Central Valley Region	Walker Mine Seal Testing & Evaluation Portola, CA	<b>BOREHOLE LOCATIONS AND ORIENTATIONS</b>	
 <b>GEI Consultants, Inc.</b>	Project 00387	Nov. 2000	Figure 47

*DRAFT*

**ATTACHMENT II**

**TECHNICAL SPECIFICATIONS FOR CORING AND GROUTING**

**FOR**

**WALKER MINE SEAL TESTING AND EVALUATION PROJECT  
PLUMAS COUNTY, CALIFORNIA**

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**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. Proportioning, mixing, pumping and injecting grout into the core holes.
  6. Complete backfilling and drypacking of all core holes.
  7. Protection of the mine seal, drain pipes, valves, monitoring equipment, air duct and other facilities from damage during drilling and grouting operations, and repair of all damaged items to the satisfaction of the Engineer and the RWQCB.
  8. Cleanup of the work areas and safe, legal and environmentally-acceptable disposal of all waste products and materials produced by or resulting from the grouting operations.

- 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. Communications with the Engineer shall be through its 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. 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.
- C. Water-Cement Ratio: Ratio by weight of water to cement.
- D. 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

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.

1.4

#### SUBMITTALS

- A. Injury and Illness Prevention Plan: Provide a project specific Injury and Illness Prevention Plan 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: Provide 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 measured; (e) describe how the angle of the drill holes (versus 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) copies of manufacturer's brochures relating to the equipment; (b) explain how the grout will be mixed and placed in the hole; (c) explain how the specified pressures will be maintained; (d) explain how the grout take will be measured.
  - 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 Data Safety Sheets for all materials that are to be used, 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 schedule for performance of the work.
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. Submit 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 occurred and any other items of interest in connection with the purpose for which the drilling is required.
3. Maintain and submit daily logs of grouting operations, including pressures, volumes and grout mix pumped.

**PART 2      PRODUCTS**

2.1      GENERAL

- A. A 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 zero-bleed neat portland cement grout.

2.2      MATERIALS

- A. Portland cement used for formulation of neat portland cement grout shall conform to ASTM Specification C 150 for Type II portland cement.

- B. Microfine cement shall be a portland/slag cement or portland cement with a grain size range equivalent to Rheocem 800 as supplied by Master Builders Technologies, Cleveland, Ohio, or approved equal.
- C. Water furnished for use in the drilling and grouting operations must 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 defloculator such as Rheobuild 2000 as supplied by Master Builders Technologies, Cleveland, Ohio, or approved equal.
- E. No fast setting chemical grout shall be used at this site.
- F. 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. The hole diameter shall be not less than 3-7/8 inches diameter. Coring must be accomplished in 5-foot advances or less using a 5-foot double-core barrel and diamond bits. Only clean water shall be used as the drilling fluid.
- B. Pressure Testing Equipment: The contractor shall furnish equipment necessary for water pressure testing. The following is a minimum list of type of equipment to be furnished 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 200 psi.
  - 2. Packers: Pneumatic packers, a minimum of 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. 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 amount of grouting equipment 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 200 psi at the collar of the grout hole. In no case will 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 16 mesh screen.
4. Valves for grout lines: They shall be the quick-opening hand operated type diaphragm 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. 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 grout holes in the event that some grout holes interconnect during the grouting procedure.

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. Provide calibrated and clearly marked containers for quick and accurate measurement of the ingredients that are not introduced in full bags in the mixes. 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 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. Submit a site specific Injury and Illness Prevention Plan as required by Cal OSHA before mobilization. Comply with the State Water Resources Control Board's Mine Entry Policy. Throughout the fieldwork conduct daily safety tailgate 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. Discharge the seepage water to the adit floor at a distance of 150 feet or greater from the seal. Take measures to prevent this water from returning to the seal area during the work. Remove all such measures at the completion of the work.

3.2 PREPARATION

No operations will be permitted to commence until the Contractor 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 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 individual holes.
- B. Perform drilling and grouting in accordance with the following sequence requirements:
  - 1. All holes will 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 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.

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 to contain 10 feet of core. Each box shall have metal hasp, hinges, and a lid. Boxes 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.
- 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 200 psi but in no event will pressure in excess of 200 psi will 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), low 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. 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 thick, zero bleed portland cement 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 zero-bleed portland 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 portland cement 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.
7. Flush all grout out of the circulating lines and injection lines, pipes and packers when changing from portland 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 will be required to grout and seal all abandoned holes as specified above.

3.7

#### CLEANUP AND RESTORATION

- A. General: 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, 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. Remove from the adit all waste grout caused by grouting operations. Dispose off site in a legal manner.
- D. Restoration: Any damage to the seal or ancillary equipment caused by or occurring during the grouting operations shall be repaired by a method approved by the Engineer, at no additional cost to the Engineer.



# **Appendix G**

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## **Sonic Logging Technique**

## 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*"

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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## **Appendix H**

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### **ASTM Procedures for Testing of Rock and Concrete Cores**

- **C42**            **Obtaining and Testing Drilled Cores and Sawed Beams of Concrete**
- **D2938**        **Unconfined Compressive Strength of Intact Rock Core Specimens**
- **C597**            **Pulse Velocity Through Concrete**
- **D2845**        **Laboratory Determination of Pulse Velocities and Ultrasonic Elastic Constants of Rock**
- **C856**            **Petrographic Examination of Hardened Concrete**
- **C295**            **Petrographic Examination of Aggregates for Concrete**



# Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete<sup>1</sup>

This standard is issued under the fixed designation C 42; 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 ( $\epsilon$ ) indicates an editorial change since the last revision or reapproval.

*This test method has been approved for use by agencies of the Department of Defense. Consult the DoD Index of Specifications and Standards for the specific year of issue which has been adopted by the Department of Defense.*

## 1. Scope

1.1 This test method covers obtaining, preparing, and testing (1) cores drilled from concrete for length or compressive or splitting tensile strength determinations and (2) beams sawed from concrete for flexural strength determinations.

1.2 The values stated in inch-pound units are to be regarded as the standard.

1.3 *This standard does not purport to address 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:

- C 39 Test Method for Compressive Strength of Cylindrical Concrete Specimens<sup>2</sup>
- C 78 Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)<sup>2</sup>
- C 116 Test Method for Compressive Strength of Concrete Using Portions of Beams Broken in Flexure<sup>2</sup>
- C 174 Test Method for Measuring Length of Drilled Concrete Cores<sup>2</sup>
- C 496 Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens<sup>2</sup>
- C 617 Practice for Capping Cylindrical Concrete Specimens<sup>2</sup>
- C 670 Practice for Preparing Precision and Bias Statements for Test Methods for Construction Materials<sup>2</sup>

### 2.2 ACI Standards:

- 301-89 Specifications for Structural Concrete for Buildings<sup>3</sup>
- 318-89 Building Code Requirements for Reinforced Concrete<sup>3</sup>

## 3. Apparatus

3.1 *Core Drill*, for obtaining cylindrical core specimens. For specimens to be removed by drilling downward perpen-

dicular to a horizontal surface, a shot drill may be satisfactory; for specimens taken by drilling in other directions or when the test specimen diameter is to be accurately determined for more precise calculation of compressive strength, a diamond drill shall be used.

3.2 *Saw*, for cutting beam specimens to size for flexural strength tests. The saw shall have a diamond or silicon-carbide cutting edge and shall be capable of cutting specimens which conform to the prescribed dimensions, without excessive heating or shock.

## 4. Sampling

### 4.1 General:

4.1.1 Samples of hardened concrete for use in the preparation of strength test specimens shall not be taken until the concrete has become hard enough to permit sample removal without disturbing the bond between the mortar and the coarse aggregate. In general, the concrete shall be 14 days old before the specimens are removed. When preparing strength test specimens from samples of hardened concrete, samples that show abnormal defects or samples that have been damaged in the process of removal shall not be used.

4.1.2 Specimens containing embedded reinforcement shall not be used for determining splitting tensile strength and specimens for determining flexural strength shall not be used if reinforcement is embedded in the tensile portion of the specimen.

NOTE 1—Cores for determining compressive strength that contain embedded reinforcement can yield either higher or lower values than cores without embedded steel and should be avoided if possible or trimmed to eliminate the reinforcement provided an  $L/D$  of 1.00 or greater can be attained.

4.2 *Core Drilling*—A core specimen taken perpendicular to a horizontal surface shall be located, when possible, so that its axis is perpendicular to the bed of the concrete as originally placed and not near formed joints or obvious edges of a unit of deposit. A specimen taken perpendicular to a vertical surface, or perpendicular to a surface with a batter, shall be taken from near the middle of a unit of deposit when possible and not near formed joints or obvious edges of a unit of deposit.

4.3 *Slab Removal*—Remove a slab sufficiently large to secure the desired test specimens without the inclusion of any concrete which has been cracked, spalled, undercut, or otherwise damaged.

<sup>1</sup> This test method is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.61 on Testing Concrete for Strength.

Current edition approved Oct. 15, 1994. Published December 1994. Originally published as C 42 - 21 T. Last previous edition C 42 - 90.

<sup>2</sup> *Annual Book of ASTM Standards*, Vol 04.02.

<sup>3</sup> Available from American Concrete Institute, P.O. Box 19150, Detroit, MI 48219.

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**DRILLED CORES**

**5. Length of Drilled Core Specimens**

5.1 *Minimum Diameter*—A core specimen for the determination of length shall have a diameter of at least 4 in. (102 mm).

5.2 *Procedure for Length Determination*—Measure the lengths of cores in accordance with the provisions of Test Method C 174.

**6. Compressive Strength**

6.1 *Test Specimens*—The diameter of core specimens for the determination of compressive strength should preferably be at least three times the nominal maximum size of the coarse aggregate used in the concrete, and *must* be at least twice the maximum nominal size of the coarse aggregate in the core sample. The length of the specimen, when capped, should be as nearly as practicable twice its diameter. A core having a maximum height of less than 95 % of its diameter before capping or a height less than its diameter after capping shall not be tested.

6.2 *End Preparation*—The ends of core specimens to be tested in compression shall be essentially smooth, perpendicular to the longitudinal axis, and of the same diameter as the body of the specimen. If necessary, saw or tool the ends of the specimens until the following requirements are met:

6.2.1 Projections, if any, shall not extend more than 0.2 in. (5 mm) above the end surfaces,

6.2.2 The end surfaces shall not depart from perpendicularity to the longitudinal axis by more than 5°, and

6.2.3 The diameters of the ends shall not depart more than 0.1 in. (2.5 mm) from the mean diameter of the specimen.

6.3 *Moisture Conditioning*—Submerge the test specimens in lime-saturated water at 73.4 ± 3.0°F (23.0 ± 1.7°C) for at least 40 h immediately prior to making the compression test. Test the specimens promptly after removal from water storage. During the period between removal from water storage and testing, keep the specimens moist by covering with a wet blanket of burlap or other suitable absorbent fabric.

6.3.1 When the specifying authority for the project so directs, cores shall be tested in a moisture condition other than achieved by conditioning according to 6.3. Length-diameter correction factors shown in 6.7 are applicable to both dry and soaked concrete. Direction for consideration of testing in a moisture condition other than that achieved by soaking at least 40 h may be found in ACI 301, chapter 17, and ACI 318, chapter 5.

6.4 *Capping*—Before making the compression test, cap the ends of the specimens in conformance with the procedure prescribed in the applicable section of Practice C 617. The capped surfaces of the specimens shall conform to the planeness requirements of Test Method C 39.

6.5 *Measurement*—Prior to testing, measure the length of the capped specimen to the nearest 0.1 in. (2.5 mm) and use this length to compute the length-to-diameter ratio. Determine the average diameter by averaging two measurements taken at right angles to each other about the midheight of the specimen. Measure core diameters to the nearest 0.01 in.

(0.25 mm) whenever possible, but at least to the nearest 0.1 in.

NOTE 2—Most shot-drilled cores will not be smooth enough to justify measuring of diameters closer than the nearest 0.1 in. (2.5 mm).

6.6 *Testing*—Test the specimens in accordance with the applicable provisions of Test Method C 39.

6.7 *Calculation*—Calculate the compressive strength of each specimen using the computed cross-sectional area based on the average diameter of the specimen.

6.7.1 If the ratio of length to diameter of the specimen exceeds 2.10 it shall be reduced in length to fall within the ratio limits of 1.94 to 2.10. Specimens within these ratios require no correction.

6.7.2 If the ratio of the length to diameter of the specimen is less than 1.94 apply correction factors shown in the following table:

Ratio of Length of Cylinder to Diameter $l/d$	Strength Correction Factor <sup>4</sup>
1.75	0.98
1.50	0.96
1.25	0.93
1.00	0.87

<sup>4</sup> These correction factors apply to lightweight concrete weighing between 100 and 120 lb/ft<sup>3</sup> (1600 and 1920 kg/m<sup>3</sup>) and to normal weight concrete. They are applicable to concrete dry or soaked at the time of loading. Values not given in the table shall be determined by interpolation. The correction factors are applicable for nominal concrete strengths from 2000 to 6000 psi (13.8 to 41.4 MPa). (Correction factors depend on various conditions such as strength and elastic moduli. Average values are given in the table.)

6.8 *Report*—Report the results as required by Test Method C 39 with the addition of the following information:

6.8.1 Length of test specimen before and after capping,

6.8.2 Compressive strength to the nearest 10 psi (69 kPa) when the diameter is measured to the nearest 0.01 in. (0.25 mm) and to the nearest 50 psi (345 kPa) when the diameter is measured to the nearest 0.1 in. (2.5 mm), after correction for length-diameter ratio when required,

6.8.3 Direction of application of the load on the specimen with respect to the horizontal plane of the concrete as placed,

6.8.4 The moisture condition at the time of testing, and

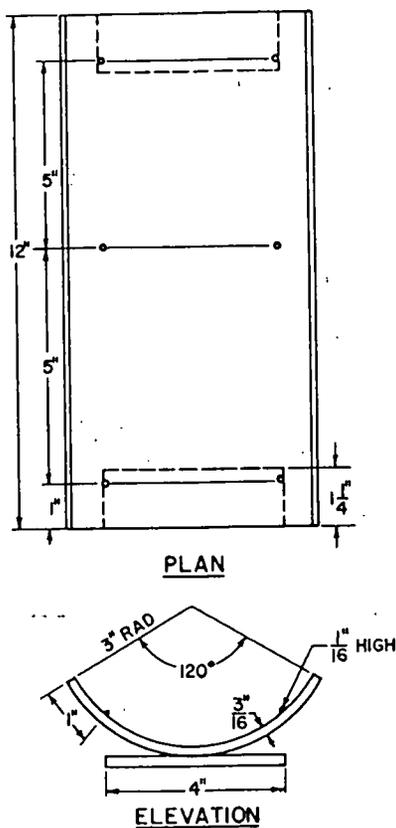
6.8.5 Nominal maximum size of concrete aggregate.

**6.9 Precision**

6.9.1 The single-operator coefficient of variation on cores has been found to be 3.2 %<sup>4</sup> for a range of compressive strength between 4500 (32.0 MPa) and 7000 (48.3 MPa) psi. Therefore, results of two properly conducted tests of single cores by the same operator on the same sample of material should not differ from each other by more than 9 %<sup>4</sup> of their average.

6.9.2 The multi-laboratory coefficient of variation on cores has been found to be 4.7 %<sup>4</sup> for a range of compressive strength between 4500 (32.0 MPa) and 7000 (48.3 MPa) psi. Therefore, results of two properly conducted tests on cores sampled from the same hardened concrete (where a single test is defined as the average of two observations (cores), each made on separate adjacent drilled 4 in. (100 mm) diameter cores), and tested by two different laboratories should not differ from each other by more than 13 %<sup>4</sup> of their average.

<sup>4</sup> These numbers represent, respectively, the (1s %) and (d2s %) limits as described in ASTM Practice C 670.



Metric Equivalents

in.	1/16	3/16	1	1 1/4	3	4	5	12
mm.	1.6	4.8	25	32	76	102	127	305

FIG. 1 Suitable Capping Device for Splitting Tensile Strength Test

6.10 *Bias*—Since there is no accepted reference material suitable for determining the bias for the procedure in this test method, no statement on bias is being made.

### 7. Splitting Tensile Strength

7.1 *Test Specimens*—The specimens shall conform to the dimensional requirements in 6.1, 6.2.1, and 6.2.2. Ends are not to be capped.

7.2 *Moisture Conditioning*—Prior to testing condition the specimens as described in 6.3.

7.3 *Bearing Surfaces*—The line of contact between the specimen and each bearing strip shall be straight and free of any projections or depressions higher or deeper than 0.01 in. (0.25 mm). When the line of contact is not straight or contains projections or depressions having heights or depths greater than 0.01 in., grind or cap the specimen so as to produce bearing lines meeting these requirements. Do not use specimens with projections or depressions greater than 0.1 in. (2.5 mm). When capping is employed the caps shall be as thin as practicable and shall be formed of high-strength gypsum plaster.

NOTE 3—Fig. 1 illustrates a device suitable for applying caps to 6-in. (150-mm) diameter specimens.

7.4 *Testing*—Test the specimens in accordance with the

applicable provisions of Test Method C 496.

7.5 *Calculation and Report*—Calculate the splitting tensile strength and report the results as required in Test Method C 496. When grinding or capping of the bearing surfaces is required, measure the diameter between the finished surfaces. Indicate that the specimen was a core and state its moisture condition at the time of testing.

## SAWED BEAMS

### 8. Flexural Strength

8.1 *Test Specimens*—A beam specimen for the determination of flexural strength shall in general have a cross section of 6 by 6 in. (150 by 150 mm) (Note 4). The specimen shall be at least 21 in. (530 mm) in length, but when two tests for flexural strength are to be made in one beam specimen, it shall be at least 33 in. (840 mm) in length. Perform the sawing operation so that the concrete will not be weakened by shock or by heating. The sawed surfaces shall be smooth, plane, parallel, and free from steps, ridges, and grooves. Take care in handling sawed beam specimens to avoid chipping or cracking.

NOTE 4—In many cases, particularly with prisms cut from pavement slabs, the width will be governed by the size of the coarse aggregate and the depth by the thickness of the slab.

8.2 *Moisture Conditioning*—Submerge the test specimens in lime-saturated water at  $73.4 \pm 3^\circ\text{F}$  ( $23 \pm 1.7^\circ\text{C}$ ) for at least 40 h immediately prior to the flexure test. Test the specimens promptly after removal from water storage. During the period between removal from water storage and testing, keep the specimens moist by covering with a wet blanket of burlap or other suitable absorbent fabric.

8.2.1 When the specifying authority for the project so directs, beams shall be tested in a moisture condition other than that achieved by conditioning in accordance with 8.2. Relatively small amounts of drying of the surface of flexural specimens induce tensile stresses in the extreme fibers that will markedly reduce the indicated flexural strength.

8.3 *Testing*—Test the specimens in accordance with the applicable provisions of Test Method C 78.

NOTE 5—The compressive strengths of portions of beams broken in flexure may be determined by testing such portions as modified cubes in accordance with Test Method C 116.

NOTE 6—Sawing may greatly reduce the indicated flexural strength; beams shall, therefore, be tested with a molded surface in tension whenever possible. The location of the tension face with respect to the position of the concrete as placed and the position of the sawed surfaces should be reported.

8.4 *Report*—Report the results in accordance with the applicable provisions of Test Method C 78 and the requirements of this test method, including the moisture condition at the time of testing. Identify orientation of the specimen's finished, sawed, and tension faces with respect to their positions in the test apparatus.

### 9. Keywords

9.1 compressive strength; concrete; concrete coring; concrete sawing; flexural strength; splitting tensile strength



# Standard Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens<sup>1</sup>

This standard is issued under the fixed designation D 2938; 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 ( $\epsilon$ ) indicates an editorial change since the last revision or reapproval.

## 1. Scope

1.1 This test method specifies the apparatus, instrumentation, and procedures for determining unconfined compressive strength of intact rock core specimens.

1.2 The values stated in inch-pound units are to be regarded as the standard. The SI values given in parentheses are for information only.

1.3 *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 2216 Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock<sup>2</sup>

D 4543 Practice for Preparing Rock Core Specimens and Determining Dimensional and Shape Tolerances<sup>2</sup>

E 4 Practices for Load Verification of Testing Machines<sup>3</sup>

E 122 Practice for Choice of Sample Size to Estimate the Average Quality of a Lot or Process<sup>4</sup>

## 3. Summary of Test Method

3.1 A rock core sample is cut to length and the ends are machined flat. The specimen is placed in a loading frame and, if required, heated to the desired test temperature. Axial load is continuously increased on the specimen until peak load and failure are obtained.

## 4. Significance and Use

4.1 Unconfined compressive strength of rock is used in many design formulas and is sometimes used as an index property to select the appropriate excavation technique.

4.2 The strength of rock cores measured in the laboratory usually do not accurately reflect large-scale *in situ* properties because the latter are strongly influenced by joints, faults, inhomogeneities, weakness planes, and other factors. Therefore, laboratory values for intact specimens must be employed with proper judgement in engineering applications.

## 5. Apparatus

5.1 *Loading Device*, of sufficient capacity to apply load at a rate conforming to the requirements set forth in 9.5. It shall be verified at suitable time intervals in accordance with the procedures given in Practices E 4, and comply with the requirements prescribed therein. The loading device may be equipped with a displacement transducer that can be used to advance the loading ram at a specified rate.

5.2 *Elevated-Temperature Enclosure*—The elevated temperature enclosure may be either an enclosure that fits in the loading apparatus or an external system encompassing the complete test apparatus. The enclosure may be equipped with humidity control for testing specimens in which the moisture content is to be controlled. For high temperatures, a system of heaters, insulation, and temperature measuring devices are normally required to maintain the specified temperature. Temperature shall be measured at three locations, with one sensor near the top, one at midheight, and one near the bottom of the specimen. The average specimen temperature based on the midheight sensor shall be maintained to within  $\pm 1^\circ\text{C}$  of the required test temperature. The maximum temperature difference between the midheight sensor and either end sensor shall not exceed  $3^\circ\text{C}$ .

NOTE 1—An Alternative to measuring the temperature at three locations along the specimen during the test is to determine the temperature distribution in a dummy specimen that has temperature sensors located in drill holes at a minimum of six positions: along both the centerline and specimen periphery at midheight and each end of the specimen. The temperature controller set point shall be adjusted to obtain steady-state temperatures in the dummy specimen that meet the temperature requirements at each test temperature (the centerline temperature at midheight shall be within  $\pm 1^\circ\text{C}$  of the required test temperature, and all other specimen temperatures shall not deviate from this temperatures by more than  $3^\circ\text{C}$ ). The relationship between controller set point and dummy specimen temperature can be used to determine the specimen temperature during testing provided that the output of the temperature feedback sensor (or other fixed-location temperature sensor in the triaxial apparatus) is maintained constant within  $\pm 1^\circ\text{C}$  of the required test temperature. The relationship between temperature controller set point and steady-state specimen temperature shall be verified periodically. The dummy specimen is used solely to determine the temperature distribution in a specimen in the elevated temperature enclosure—it is not to be used to determine compressive strength.

5.3 *Temperature Measuring Device*—Special limits-of-error thermocouples or platinum resistance thermometers (RTDs) having accuracies of at least  $\pm 1^\circ\text{C}$  with a resolution of  $0.1^\circ\text{C}$ .

5.4 *Platens*—Two steel platens are used to transmit the axial load to the ends of the specimen. They shall have a hardness of not less than 58 HRC. The bearing faces shall

<sup>1</sup> This 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. Current edition approved June 15, 1995. Published July 1995. Originally published as D 2938 - 71. Last previous edition D 2938 - 86 $\epsilon$ 1.

<sup>2</sup> *Annual Book of ASTM Standards*, Vol 04.08.

<sup>3</sup> *Annual Book of ASTM Standards*, Vol 03.01.

<sup>4</sup> *Annual Book of ASTM Standards*, Vol 14.02.

not depart from a plane by more than 0.0125 mm when the platens are new and shall be maintained within a permissible variation of 0.025 mm. The platen diameter shall be at least as great as the specimen diameter, but shall not exceed 1.10 times the specimen diameter. The platen thickness shall be at least one-half the specimen diameter.

5.4.1 *Spherical Seating*—One of the platens should be spherically seated and the other a plain rigid platen. The diameter of the spherical seat shall be at least as large as that of the test specimen, but shall not exceed twice the diameter of the test specimen. The center of the sphere in the spherical seat shall coincide with the center of the loaded end of the specimen. The spherical seat shall be lubricated to ensure free movement. The movable portion of the platen shall be held closely in the spherical seat, but the design shall be such that the bearing face can be rotated and tilted through small angles in any direction.

5.4.2 *Rigid Seating*—If a spherical seat is not used, the bearing faces of the platens shall be parallel to 0.0005 mm/mm of platen diameter. This criterion shall be met when the platens are in the loading device and separated by approximately the height of the test specimen.

6. Safety Precautions

6.1 Many rock types fail in a violent manner when loaded to failure in compression. A protective shield should be placed around the test specimen to prevent injury from flying rock fragments. Elevated temperatures increase the risks of electrical shorts and fire.

7. Sampling

7.1 The specimen shall be selected from the cores to represent a valid average of the type of rock under consideration. This can be achieved by visual observations of mineral constituents, grain sizes and shape, partings and defects such as pores and fissures, or by other methods such as ultrasonic velocity measurements.

8. Test Specimens

8.1 Prepare test specimens in accordance with Practice D 4543.

8.2 The moisture condition of the specimen at time of test can have a significant effect upon the deformation of the rock. Good practice generally dictates that laboratory tests be made upon specimens representative of field conditions. Thus, it follows that the field moisture condition of the specimen should be preserved until time of test. On the other hand, there may be reasons for testing specimens at other moisture contents including zero. In any case, tailor the moisture content of the test specimen to the problem at hand and report it in accordance with 11.1.3. If the moisture content of the specimen is to be determined, follow the procedures in Test Method D 2216.

8.3 If moisture condition is to be maintained, and the elevated temperature enclosure is not equipped with humidity control, seal the specimen using a flexible membrane or apply a plastic or silicone rubber coating to the specimen sides.

9. Procedure

9.1 Check the ability of the spherical seat to rotate freely in its socket before each test.

9.2 Place the lower platen on the base or actuator rod of the loading device. Wipe clean the bearing faces of the upper and lower platens and of the test specimen, and place the test specimen on the lower platen. Place the upper platen on the specimen and align properly. A small axial load, approximately 100 N, may be applied to the specimen by means of the loading device to properly seat the bearing parts of the apparatus.

9.3 When appropriate, install elevated-temperature enclosure.

9.4 If testing at elevated temperature, raise the temperature at a rate not exceeding 2°C/min until the required temperature is reached (Note 2). The test specimen shall be considered to have reached temperature equilibrium when all temperature measuring device outputs are stable for at least three readings taken at equal intervals over a period of no less than 30 min (3 min for tests performed at room temperature). Stability is defined as a constant reading showing only the effects of normal instrument and heater unit fluctuations.

NOTE 2—It has been observed that for some rock types microcracking will occur for heating rates above 1°C/min. The operator is cautioned to select a heating rate that microcracking is not significant.

9.5 Apply the axial load continuously and without shock until the load becomes constant, reduces, or a predetermined amount of strain is achieved. Apply the load in such a manner as to produce either a stress rate or a strain rate as constant as feasible throughout the test. Do not permit the stress rate or strain rate at any given time to deviate by more than 10 percent from that selected. The stress rate or strain rate selected should be that which will produce failure in a test time between 2 and 15 min. The selected stress rate or strain rate for a given type shall be adhered to for all tests in a given series of investigation (Note 3). Record the maximum load sustained by the specimen.

NOTE 2—Results of tests by other investigators have shown that strain rates within this range will provide strength values that are reasonably free from rapid loading effects and reproducible within acceptable tolerances. Lower strain rates are permissible, if required by the investigation.

10. Calculation

10.1 Calculate the compressive strength in the test specimen from the maximum compressive load on the specimen and the initial computed cross-sectional area as follows:

$$\sigma = \frac{P}{A}$$

where:

- $\sigma$  = Compressive strength
- $P$  = Maximum load
- $A$  = Cross sectional area.

NOTE 4—Tensile stresses are used as being positive. A consistent application of a compression-positive sign convention may be employed if desired. The sign convention adopted needs to be stated explicitly in the report. The formulas given are for engineering stresses. True stresses may be used, if desired, provided that specimen diameter at the time of peak load is known.

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11. Report

11.1 The report shall include the following:

11.1.1 Source of sample including project name and location (often the location is specified in terms of the drill hole number and depth of specimen from the collar of the hole).

11.1.2 Lithologic description of the rock, formation name, and load direction with respect to lithology.

11.1.3 Moisture condition of specimen before test.

11.1.4 Specimen diameter and height, conformance with dimensional requirements.

11.1.5 Temperature at which test was performed.

11.1.6 Rate of loading or deformation rate.

11.1.7 Unconfined compressive strength.

NOTE 5—If failure is ductile, with the load on the specimen still increasing when the test is terminated, the strain at which the compressive strength was calculated shall be reported.

11.1.8 Type and location of failure. A sketch of the fractured specimen is recommended.

11.1.9 If the actual equipment or procedure has varied from the requirements contained in this test method, each variation and the reasons for it shall be discussed.

12. Precision and Bias

12.1 Data are being evaluated via an interlaboratory test program for rock properties to determine the precision of this test method.

12.2 Bias cannot be determined since there is no standard value of compressive strength that can be used to compare with values determined using this method.

13. Keywords

13.1 compression testing; compressive strength; loading tests; rock

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This standard is subject to revision at any time by the responsible technical committee and must be reviewed every five years and if not revised, either reapproved or withdrawn. Your comments are invited either for revision of this standard or for additional standards and should be addressed to ASTM Headquarters. Your comments will receive careful consideration at a meeting of the responsible technical committee, which you may attend. If you feel that your comments have not received a fair hearing you should make your views known to the ASTM Committee on Standards, 100 Barr Harbor Drive, West Conshohocken, PA 19428.



# Standard Test Method for Pulse Velocity Through Concrete<sup>1</sup>

This standard is issued under the fixed designation C 597; 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 ( $\epsilon$ ) indicates an editorial change since the last revision or reapproval.

*This test method has been approved for use by agencies of the Department of Defense. Consult the DoD Index of Specifications and Standards for the specific year of issue which has been adopted by the Department of Defense.*

<sup>ε1</sup> NOTE—Editorial corrections were made throughout in August 1991.

## 1. Scope

1.1 This test method covers the determination of the pulse velocity of propagation of compressional waves in concrete.

This test method does not apply to the propagation of other vibrations within the concrete.

1.2 The pulse velocity is independent of the dimensions of the body provided reflected waves from boundaries do not complicate the determination of the arrival time of the directly transmitted pulse.

1.3 The pulse velocity  $V$  is related to the physical properties of a solid by the equation:

$$V^2 = (K) \frac{E}{D} \quad (1)$$

where:

$K$  = a constant,

$E$  = the modulus of elasticity, and

$D$  = the density.

The relationship is independent of the frequency of the vibrations.

1.4 The values stated in SI units are to be regarded as the standard.

1.5 *This standard does not purport to address all of the safety problems, 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. Summary of Method

2.1 Pulses of compressional waves are generated by an electro-acoustical transducer that is held in contact with one surface of the concrete under test. After traversing through the concrete, the pulses are received and converted into electrical energy by a second transducer located a distance  $L$  from the transmitting transducer. The transit time  $T$  is measured electronically. The pulse velocity  $V$  is calculated by dividing  $L$  by  $T$ .

## 3. Significance and Use

3.1 This test method may be used to advantage to assess the uniformity and relative quality of concrete, to indicate the presence of voids and cracks, to estimate the depth of cracks, to indicate changes in the properties of concrete, and

in the survey of structures, to estimate the severity of deterioration or cracking.

NOTE 1—Moisture content of concrete can affect pulse velocity.

3.2 The results obtained by the use of this test method should not be considered as a means of measuring strength nor as an adequate test for establishing compliance of the modulus of elasticity of field concrete with that assumed in design.

NOTE 2—When circumstances permit, a velocity - strength (or velocity - modulus) relationship may be established by the determination of pulse velocity and compressive strength (or modulus of elasticity) on a number of samples of a concrete.

This relationship may serve as a basis for the estimation of strength (or modulus of elasticity) by further pulse-velocity tests on that concrete.

3.3 The procedure is applicable in both field and laboratory testing regardless of size or shape of the specimen within the limitations of available pulse-generating sources.

NOTE 3—Presently available test equipment limits path lengths to approximately 50 mm (2 in.) minimum and 15 m (50 ft) maximum, depending, in part, upon the frequency and intensity of the generated signal. The upper limit of the path length depends partly on surface conditions and partly on the characteristics of the interior concrete under investigation. The maximum path length is obtained by using transducers of relatively low vibrational frequencies (10 to 20 kHz) to minimize the attenuation of the signal in the concrete. (The resonant frequency of the transducer assembly, that is, crystals plus backing plate, determines the frequency of vibration in the concrete.) For the shorter path lengths where loss of signal is not the governing factor, it is preferable to use vibrational frequencies of 50 kHz or higher to achieve more accurate transit-time measurements and hence greater sensitivity.

## 4. Apparatus

4.1 The testing apparatus, shown schematically in Fig. 1, consists of a pulse generator, a pair of transducers (transmitter and receiver), an amplifier, a time measuring circuit, a time display unit, and connecting cables.

4.1.1 *Pulse Generator and Transmitting Transducer*—The pulse generator shall consist of circuitry for generating pulses of voltage. The transducer for transforming these electronic pulses into wave bursts of mechanical energy shall have a resonant frequency in the range of 10 to 150 kHz. The pulse generator shall produce repetitive pulses at a rate of not less than 10 pulses per second nor more than 150 pulses per second. The transducer shall be constructed of piezoelectric, magnetostrictive, or other voltage-sensitive material (Rochelle salt, quartz, barium titanate, lead zirconate-titanate (PZT), etc.), housed for protection. The transducer shall be provided with a suitable coupling medium in order to obtain adequate contact with the concrete (Note 4). The use of long

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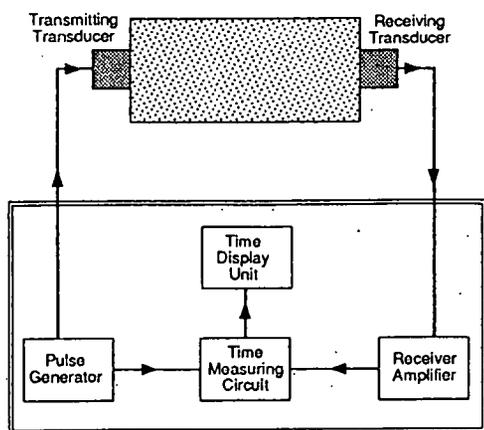
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<sup>1</sup> This test method is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.64 on Nondestructive Testing of Concrete.

Current edition approved Oct. 26, 1983. Published December 1983. Originally published as C 597 - 67T. Last previous edition C 597 - 71(1979).



NOTE—It is advantageous to incorporate the pulse generator, time measuring circuit, receiver amplifier, and time display into one unit.

FIG. 1 Schematic Diagram of Pulse Velocity Testing Circuit

cables connecting the transducers to the pulse generator shall not result in greater than 0.5 % loss of voltage. A triggering pulse shall be produced to start the time measuring circuit.

NOTE 4—Thin, metal diaphragms may be used against smooth, plane surfaces. Distended rubber diaphragms and an oil-filled housing under slight pressure have been found satisfactory on rough surfaces.

4.1.2 *Receiving Transducer and Amplifier*—The receiving transducer shall be similar to the transmitting transducer. The voltage generated by it shall be amplified as necessary to produce triggering pulses to the time-measuring circuit. The amplifier shall have a flat response between 5 kHz and ten times the resonant frequency of the receiving transducer.

4.1.3 *Time-Measuring Circuit*—The time-measuring circuit and the associated triggering pulses shall be capable of providing an overall time-measurement precision of one part in 200 over its specified time range. It should be initiated by a triggering voltage from the pulse generator and should operate at the repetition frequency of the latter. The time-measuring circuit shall provide an output when the received pulse is detected, and this output shall be used to determine the transit time displayed on the time-display unit. The time-measuring circuit shall be insensitive to operating temperature and voltage changes in the power source.

4.1.4 *Display Unit*—Two types of display units are available. One uses a cathode ray tube (CRT) on which the pulses transmitted and received are displayed as deflections of the trace in relation to an established time scale. The other uses an interval timer with a direct-reading digital display.

4.1.5 *Calibration Device*—A calibration device shall be provided for the purpose of checking the proper operation of the time-measuring circuit.

NOTE 5—One form of calibration device for CRT display units consists of a quartz-crystal oscillator with a frequency of 100 kHz synchronized with the pulse-generator circuit. The calibration device is connected to the vertical amplifier for use. Another method suitable for either type of display unit is the use of two or more reference bars for which the transit times are accurately known.

4.1.6 Where pulse-velocity measurements on large structures require the use of long interconnecting cables, the low-capacity, shielded, coaxial type shall be used.

## 5. Sampling

5.1 For best results, the transducers should be located

directly opposite each other. However, because the effective beam width of the transducers is wide, transit times can be measured across corners of a structure but with some loss of sensitivity and accuracy. Measurements along the same surface should not be used unless only one face is accessible since such measurements are indicative only of surface layers. Where possible, measurements in close proximity to steel parallel to the direction of pulse propagation should be avoided.

NOTE 6—Since the pulse velocity in steel could be up to double that in concrete, pulse-velocity measurements in the vicinity of the reinforcing steel in reinforced concrete may be higher than in nonreinforced concrete of the same composition.

## 6. Procedure

### 6.1 Determination of Calibration Correction:

6.1.2 Check the accuracy of the transmit time measurements against the calibration device (section 4.1.5). The difference between the reading taken on the calibration device and the calibration-device constant is the calibration correction, which must be included in the determination of transit time (section 6.3). Check the calibration over a range of transit times that includes the transit times anticipated.

### 6.2 Determination of Zero Correction:

6.2.1 Apply zero-time correction to the measured transit times. The zero correction is equal to the travel time between the transmitting and receiving transducers with zero thickness of concrete between the two. Measure it directly by pressing the two transducers together using a coupling agent and applying the same pressure used in the actual measurement; and measure the transit time. With digital display units it may be preferable to make zero adjustments by coupling the two transducers to opposite ends of a reference bar for which the transit time is accurately known.

NOTE 7—For digital display units, the procedures outlined in section 6.2 should be performed before the procedures outlined in section 6.1.

### 6.3 Determination of Transit Time:

6.3.1 Apply an appropriate coupling agent (such as water, oil, petroleum jelly, grease, or other viscous materials) to the transducer diaphragms, the test surface, or both, to avoid entrapped air between the contact surface of the diaphragms of the transducers and the surface of the concrete. Press the faces of the transducers against the surfaces of the concrete assuring good contact between the two, and measure the transit time (Note 8). Measure the length of the shortest direct path between the centers of the diaphragms.

NOTE 8—Repeat measurement should be made at same location to minimize erroneous readings due to poor contact.

## 7. Calculation

7.1 Calculate the pulse velocity as follows:

$$V = L/T \quad (2)$$

where:

$V$  = pulse velocity, m/s (ft/s),  
 $L$  = distance between transducers, m (ft), and  
 $T$  = effective transit time, s (measured time minus zero time correction and corrected for calibration errors).

## 8. Report

8.1 The report shall consist of at least the following:

- 8.1.1 Location of test or identification of specimen.
- 8.1.2 Distance between transducers, stated to greater precision than 0.5 % of distance.
- 8.1.3 Transit time.
- 8.1.4 Location of transducers.
- 8.1.5 Effective transit time, stated to greater precision than 0.5 %.
- 8.1.6 Pulse velocity.

**9. Precision and Bias**

- 9.1 *Precision:*
- 9.1.1 The quantity measured by this test method is the transit time of a compressional wave passing through concrete. The property usually reported is the velocity with which the waves travel through the concrete. The accuracy of the measurement is dependent upon the ability of the equipment and the operator to determine precisely the distance between transducers and the time of arrival of the wave at the receiver. The strength of the received signal is affected by the travel path length and by the presence and degree of cracking or deterioration in the concrete tested:
- 9.1.2 Tests involving three test instruments and five operators have indicated that for path lengths from 0.3 to 6 m (1 to

20 ft) through sound concrete, different operators using the same instrument or one operator using different instruments will achieve repeatability of test results within 2 %. For longer path lengths through sound concrete, attenuation of the signal will decrease the absolute repeatability of the transit-time measurement, but the longer transit time involved will result in a calculation of velocity having the same order of accuracy.

9.1.3 In the case of tests through badly cracked or deteriorated concrete, the variation of the results are substantially increased. Attenuation is affected by the nature of the deterioration and the resonant frequency of the transducers. Differences between operators or instruments may result in differences in test results as large as 20 %. In such cases, however, calculated velocities will be sufficiently low as to indicate clearly the presence of distress in the concrete tested.

**9.2 Bias:**

9.2.1 Nothing is being said concerning the bias of this standard.

**10. Keywords**

10.1 concrete; nondestructive testing; pulse velocity

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# Standard Test Method for Laboratory Determination of Pulse Velocities and Ultrasonic Elastic Constants of Rock<sup>1</sup>

This standard is issued under the fixed designation D 2845; 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 ( $\epsilon$ ) indicates an editorial change since the last revision or reapproval.

<sup>ε1</sup> NOTE—Paragraph 10.5 was added editorially in January 1997.

## 1. Scope

1.1 This test method describes equipment and procedures for laboratory measurements of the pulse velocities of compression waves and shear waves in rock (1)<sup>2</sup> (Note 1) and the determination of ultrasonic elastic constants (Note 2) of an isotropic rock or one exhibiting slight anisotropy.

NOTE 1—The compression wave velocity as defined here is the dilatational wave velocity. It is the propagation velocity of a longitudinal wave in a medium which is effectively infinite in lateral extent. It should not be confused with the bar or rod velocity.

NOTE 2—The elastic constants determined by this test method are termed ultrasonic since the pulse frequencies used are above the audible range. The terms sonic and dynamic are sometimes applied to these constants but do not describe them precisely (2). It is possible that the ultrasonic elastic constants may differ from those determined by other dynamic methods.

1.2 This test method is valid for wave velocity measurements in both anisotropic and isotropic rocks although the velocities obtained in grossly anisotropic rocks may be influenced by such factors as direction, travel distance, and diameter of transducers.

1.3 The ultrasonic elastic constants are calculated from the measured wave velocities and the bulk density. The limiting degree of anisotropy for which calculations of elastic constants are allowed and procedures for determining the degree of anisotropy are specified.

1.4 The values stated in U.S. customary units are to be regarded as the standard. The metric equivalents of U.S. customary units may be approximate.

## 2. Referenced Documents

2.1 *ASTM Standard:*  
E 691 Practice for Conducting an Interlaboratory Study to Determine the Precision of a Test Method<sup>3</sup>

## 3. Summary of Test Method

3.1 Details of essential procedures for the determination of the ultrasonic velocity, measured in terms of travel time and distance, of compression and shear waves in rock

<sup>1</sup> 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.

Current edition approved Dec. 10, 1995. Published April 1996. Originally published as D 2845 - 69. Last previous edition D 2845 - 90<sup>ε1</sup>.

<sup>2</sup> The boldface numbers in parentheses refer to the list of references at the end of this test method.

<sup>3</sup> *Annual Book of ASTM Standards*, Vol 14.02.

specimens include requirements of instrumentation, suggested types of transducers, methods of preparation, and effects of specimen geometry and grain size. Elastic constants may be calculated for isotropic or slightly anisotropic rocks, while anisotropy is reported in terms of the variation of wave velocity with direction in the rock.

## 4. Significance and Use

4.1 The primary advantages of ultrasonic testing are that it yields compression and shear wave velocities, and ultrasonic values for the elastic constants of intact homogeneous isotropic rock specimens (3). Elastic constants are not to be calculated for rocks having pronounced anisotropy by procedures described in this test method. The values of elastic constants often do not agree with those determined by static laboratory methods or the *in situ* methods. Measured wave velocities likewise may not agree with seismic velocities, but offer good approximations. The ultrasonic evaluation of rock properties is useful for preliminary prediction of static properties. The test method is useful for evaluating the effects of uniaxial stress and water saturation on pulse velocity. These properties are in turn useful in engineering design.

4.2 The test method as described herein is not adequate for measurement of stress-wave attenuation. Also, while pulse velocities can be employed to determine the elastic constants of materials having a high degree of anisotropy, these procedures are not treated herein.

## 5. Apparatus

5.1 *General*—The testing apparatus (Fig. 1) should have impedance matched electronic components and shielded leads to ensure efficient energy transfer. To prevent damage to the apparatus allowable voltage inputs should not be exceeded.

5.2 *Pulse Generator Unit*—This unit shall consist of an electronic pulse generator and external voltage or power amplifiers if needed. A voltage output in the form of either rectangular pulse or a gated sine wave is satisfactory. The generator shall have a voltage output with a maximum value after amplification of at least 50 V into a 50- $\Omega$  impedance load. A variable pulse width, with a range of 1 to 10  $\mu$ s is desirable. The pulse repetition rate may be fixed at 60 repetitions per second or less although a range of 20 to 100 repetitions per second is recommended. The pulse generator shall also have a trigger-pulse output to trigger the oscilloscope. There shall be a variable delay of the main-pulse

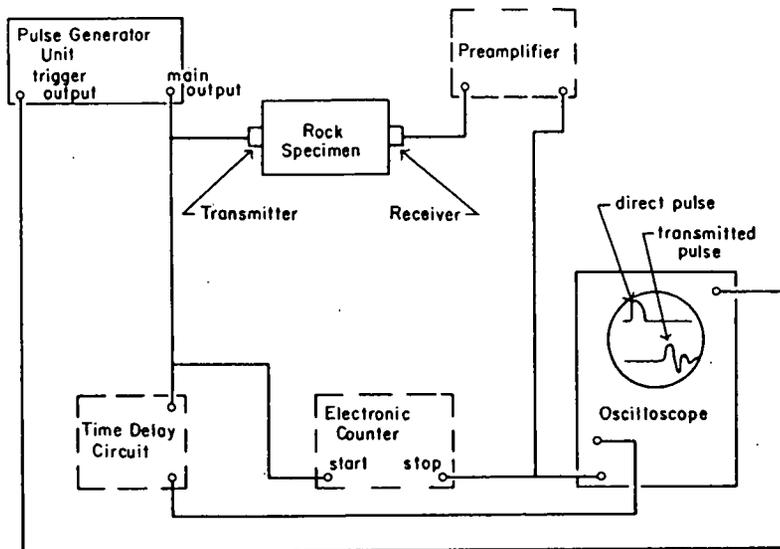
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FIG. 2



NOTE—Components shown by dashed lines are optional, depending on method of travel-time measurement and voltage sensitivity of oscilloscope.

FIG. 1 Schematic Diagram of Typical Apparatus

output with respect to the trigger-pulse output, with a minimum range of 0 to 20  $\mu$ s.

5.3 Transducers—The transducers shall consist of a transmitter which converts electrical pulses into mechanical pulses and a receiver which converts mechanical pulses into electrical pulses. Environmental conditions such as ambient temperature, moisture, humidity, and impact should be considered in selecting the transducer element. Piezoelectric elements are usually recommended, but magnetostrictive elements may be suitable. Thickness-expander piezoelectric elements generate and sense predominately compression-wave energy; thickness-shear piezoelectric elements are preferred for shear-wave measurements. Commonly used piezoelectric materials include ceramics such as lead-zirconate-titanate for either compression or shear, and crystals such as a-c cut quartz for shear. To reduce scattering and poorly defined first arrivals at the receiver, the transmitter shall be designed to generate wavelengths at least three times the average grain size of the rock.

NOTE 3—Wavelength is the wave velocity in the rock specimen divided by the resonance frequency of the transducer. Commonly used frequencies range from 75 kHz to 3 MHz.

5.3.1 In laboratory testing, it may be convenient to use unhoused transducer elements. But if the output voltage of the receiver is low, the element should be housed in metal (grounded) to reduce stray electromagnetic pickup. If protection from mechanical damage is necessary, the transmitter as

well as the receiver may be housed in metal. This also allows special backings for the transducer element to alter its sensitivity or reduce ringing (4). The basic features of a housed element are illustrated in Fig. 2. Energy transmission between the transducer element and test specimen can be improved by (1) machining or lapping the surfaces of the face plates to make them smooth, flat, and parallel, (2) making the face plate from a metal such as magnesium whose characteristic impedance is close to that of common rock types, (3) making the face plate as thin as practicable, and (4) coupling the transducer element to the face plate by a thin layer of an electrically conductive adhesive, an epoxy type being suggested.

5.3.2 Pulse velocities may also be determined for specimens subjected to uniaxial states of stress. The transducer housings in this case will also serve as loading platens and should be designed with thick face plates to assure uniform loading over the ends of the specimen (5).

NOTE 4—The state of stress in many rock types has a marked effect on the wave velocities. Rocks *in situ* are usually in a stressed state and therefore tests under stress have practical significance.

5.4 Pre-amplifier—A voltage preamplifier is required if the voltage output of the receiving transducer is relatively low or if the display and timing units are relatively insensitive. To preserve fast rise times, the frequency response of the preamplifier shall drop no more than 2 dB over a frequency range from 5 kHz to 4  $\times$  the resonance frequency of the receiver. The internal noise and gain must also be considered in selecting a preamplifier. Oscilloscopes having a vertical-signal output can be used to amplify the signal for an electronic counter.

5.5 Display and Timing Unit—The voltage pulse applied to the transmitting transducer and the voltage output from the receiving transducer shall be displayed on a cathode-ray oscilloscope for visual observation of the waveforms. The oscilloscope shall have an essentially flat response between a frequency of 5 kHz and 4  $\times$  the resonance frequency of the transducers. It shall have dual beams or dual traces so that

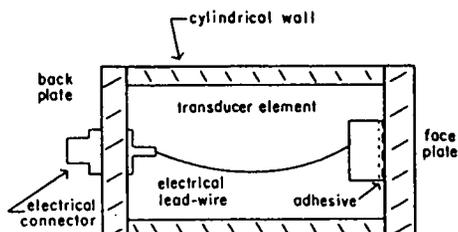
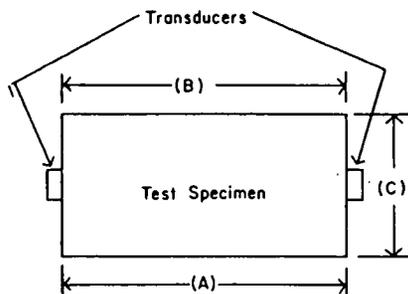


FIG. 2 Basic Features of a Housed Transmitter or Receiver

the two waveforms may be displayed simultaneously and their amplitudes separately controlled. The oscilloscope shall be triggered by a triggering pulse from the pulse generator. The timing unit shall be capable of measuring intervals between 2  $\mu$ s and 5 ms to an accuracy of 1 part in 100. Two alternative classes of timing units are suggested, the respective positions of each being shown as dotted outlines in the block diagram in Fig. 1: (1) an electronic counter with provisions for time interval measurements, or (2) a time-delay circuit such as a continuously variable-delay generator, or a delayed-sweep feature on the oscilloscope. The travel-time measuring circuit shall be calibrated periodically with respect to its accuracy and linearity over the range of the instrument. The calibration shall be checked against signals transmitted by the National Institute of Standards and Technology radio station WWV, or against a crystal controlled time-mark or frequency generator which can be referenced back to the signals from WWV periodically. It is recommended that the calibration of the time measuring circuit be checked at least once a month and after any severe impact which the instrument may receive.

6. Test Specimens

6.1 Preparation—Exercise care in core drilling, handling, sawing, grinding, and lapping the test specimen to minimize the mechanical damage caused by stress and heat. It is recommended that liquids other than water be prevented from contacting the specimen, except when necessary as a coupling medium between specimen and transducer during the test. The surface area under each transducer shall be sufficiently plane that a feeler gage 0.001 in. (0.025 mm) thick will not pass under a straightedge placed on the surface. The two opposite surfaces on which the transducers will be placed shall be parallel to within 0.005 in./in. (0.1 mm/20 mm) of lateral dimension (Fig. 3). If the pulse velocity measurements are to be made along a diameter of a core, the above tolerance then refers to the parallelism of the lines of contact between the transducers and curved surface of the rock core. Moisture content of the test specimen can affect the measured pulse velocities (see 6.2). Pulse velocities may be determined on the velocity test specimen for rocks in the oven-dry state (0 % saturation), in a saturated condition (100 % saturation), or in any intermediate state. If the pulse velocities are to be determined with the rock in the same moisture condition as received or as exists underground, care must be exercised during the preparation procedure so that the moisture content does not change. In this case it is suggested that both the sample and test specimen be stored in moisture-proof bags or coated with wax and that dry



NOTE—(A) must be within 0.1 mm of (B) for each 20 mm of width (C).

FIG. 3 Specification for Parallelism

surface-preparation procedures be employed. If results are desired for specimens in the oven-dried condition, the oven temperature shall not exceed 150°F (66°C). The specimen shall remain submerged in water up to the time of testing when results are desired for the saturated state.

6.2 Limitation on Dimensions—It is recommended that the ratio of the pulse-travel distance to the minimum lateral dimension not exceed 5. Reliable pulse velocities may not be measurable for high values of this ratio. The travel distance of the pulse through the rock shall be at least 10  $\times$  the average grain size so that an accurate average propagation velocity may be determined. The grain size of the rock sample, the natural resonance frequency of the transducers, and the minimum lateral dimension of the specimen are interrelated factors which affect test results. The wavelength corresponding to the dominant frequency of the pulse train in the rock is approximately related to the natural resonance frequency of the transducer and the pulse-propagation velocity, (compression or shear) as follows:

$$\Lambda \approx V/f, \tag{1}$$

where:

- $\Lambda$  = dominant wavelength of pulse train, in. (or m),
- $V$  = pulse propagation velocity (compression or shear), in./s (or m/s), and
- $f$  = natural resonance frequency of transducers, Hz.

The minimum lateral dimension of the test specimen shall be at least 5  $\times$  the wavelength of the compression wave so that the true dilational wave velocity is measured (Note 5), that is,

$$D \geq 5\Lambda, \tag{2}$$

where:

- $D$  = minimum lateral dimension of test specimen, in. (or m).

The wavelength shall be at least 3  $\times$  the average grain size (See 4.3) so that

$$\Lambda \geq 3d, \tag{3}$$

where:

- $d$  = average grain size, in. (or m).

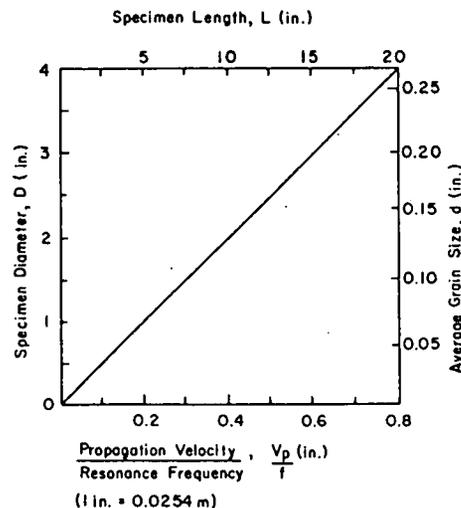


FIG. 4 Graph Showing Allowable Values of Specimen Diameter and Grain Size Versus the Ratio of Propagation Velocity to Resonance Frequency

Equation relation:

where:

$V_p =$  pulse propagation velocity  
 $D$  = specimen diameter  
 Since  $V_p/D$  shall be at least 10  $\times$  the average grain size, the values of  $d$  in Fig. 4, the diagonal line.

NOTE: Transducer Cannady specimen

7. Procedure

7.1 Preparation of the specimen  
 The specimen shall be prepared so that the surface to be tested is perpendicular to the longitudinal axis and its diameter is within 0.1 % of the nominal diameter.

where:

- $\rho$  = density
- $m$  = mass
- $V$  = volume

7.2 Sample preparation  
 7.3. The gauge shall be used to check the specimen diameter. The pulse velocity measurement shall be made using the scope. 7.3. The distance between the transducers shall be within 0.1 % of the nominal distance.

Equations 1, 2, and 3 can be combined to yield the relationship for compression waves as follows:

$$D \geq 5(V_p/f) \geq 15 d, \quad (4)$$

where:

$V_p$  = pulse propagation velocity (compression), in./s (or m/s).

Since  $V_p$  and  $d$  are inherent properties of the material,  $f$  and  $D$  shall be selected to satisfy Eq 4 (Fig. 4) for each test specimen. For any particular value of  $V_p/f$  the permissible values of specimen diameter  $D$  lie above the diagonal line in Fig. 4, while the permissible values of grain size  $d$  lie below the diagonal line. For a particular diameter, the permissible values for specimen length  $L$  lie to the left of the diagonal line.

NOTE 5—Silavaeva and Shamina (6) found the limiting ratio of diameter to wavelength to be about 2 for metal rods. Data obtained by Cannady (3) on rock indicate the limiting ratio is at least 8 for a specimen length-to-diameter ratio of about 8.

### 7. Procedure

7.1 *Determination of Travel Distance and Density*—Mark off the positions of the transducers on the specimen so that the line connecting the centers of the transducer contact areas is not inclined more than 2° (approximately 0.1 in. in 3 in. (1 mm in 30 mm)) with a line perpendicular to either surface. Then measure the pulse-travel distance from center to center of the transducer contact area to within 0.1 %. The density of the test specimen is required in the calculation of the ultrasonic elastic constants (see 7.2). Determine the density of the test specimen from measurements of its mass and its volume calculated from the average external dimensions. Determine the mass and average dimensions within 0.1 %. Calculate the density as follows:

$$\rho = m/V$$

where:

$\rho$  = density, lb sec<sup>2</sup>/in.<sup>4</sup> (or kg/m<sup>3</sup>),  
 $m$  = mass of test specimen, lb sec<sup>2</sup>/386.4 in. (or kg), and  
 $V$  = volume of test specimen, in<sup>3</sup> (or m<sup>3</sup>).

7.2 *Moisture Condition*—The moisture condition of the sample shall be noted and reported as 8.1.3.

#### 7.3 *Determination of Pulse-Travel Time:*

7.3.1 Increase the voltage output of the pulse generator, the gain of the amplifier, and the sensitivity of the oscilloscope and counter to an optimum level, giving a steeper pulse front to permit more accurate time measurements. The optimum level is just below that at which electromagnetic noise reaches an intolerable magnitude or triggers the counter at its lowest triggering sensitivity. The noise level shall not be greater than one tenth of the amplitude of the first peak of the signal from the receiver. Measure the travel time to a precision and accuracy of 1 part in 100 for compression waves and 1 part in 50 for shear waves by (1) using the delaying circuits in conjunction with the oscilloscope (see 7.1.1) or (2) setting the counter to its highest usable precision, (see 6.3.2).

7.3.1.1 The oscilloscope is used with the time-delay circuit to display both the direct pulse and the first arrival of the transmitted pulse, and to measure the travel time. Characteristically, the first arrival displayed on the oscilloscope consists of a curved transition from the horizontal zero-voltage

trace followed by a steep, more or less linear, trace. Select the first break in a consistent manner for both the test measurement and the zero-time determination. Select it either at the beginning of the curved transition region or at the zero-voltage intercept of the straight line portion of the first arrival.

7.3.1.2 The counter is triggered to start by the direct pulse applied to the transmitter and is triggered to stop by the first arrival of the pulse reaching the receiver. Because a voltage change is needed to trigger the counter, it can not accurately detect the first break of a pulse. To make the most accurate time interval measurements possible, increase the counter's triggering sensitivity to an optimum without causing spurious triggering by extraneous electrical noise.

7.3.2 Determine the zero time of the circuit including both transducers and the travel-time measuring device and apply the correction to the measured travel times. This factor will remain constant for a given rock and stress level if the circuit characteristics do not change. Determine the zero time accordingly to detect any changes. Determine it by (1) placing the transducers in direct contact with each other and measuring the delay time directly, or (2) measuring the apparent travel time of some uniform material (such as steel) as a function of length, and then using the zero-length intercept of the line through the data points as the correction factor.

7.3.3 Since the first transmitted arrival is that of the compression wave, its detection is relatively easy. The shear-wave arrival, however, may be obscured by vibrations due to ringing of the transducers and reflections of the compression wave. The amplitude of the shear wave relative to the compression wave may be increased and its arrival time determined more accurately by means of thickness shear-transducer elements. This type of element generates some compressional energy, so that both waves may be detected. Energy transmission between the specimen and each transducer may be improved by using a thin layer of a coupling medium such as phenyl salicylate, high-vacuum grease, or resin, and by pressing the transducer against the specimen with a small seating force.

7.3.4 For specimens subjected to uniaxial stress fields, first arrivals of compression waves are usually well defined. However, the accurate determination of shear-wave first arrivals for specimens under stress is complicated by mode conversions at the interfaces on either side of the face plate and at the free boundary of the specimen (4). Shear-wave arrivals are therefore difficult to determine and experience is required for accurate readings.

7.4 *Ultrasonic Elastic Constants*—The rock must be isotropic or possess only a slight degree of anisotropy if the ultrasonic elastic constants are to be calculated (Section 8). In order to estimate the degree of anisotropy of the rock, measure the compression-wave velocity in three orthogonal directions, and in a fourth direction oriented at 45° from any one of the former three directions if required as a check. Make these measurements with the same geometry, that is, all between parallel flat surfaces or all across diameters. The equations in 8.2 for an isotropic medium shall not be applied if any of the three compression-wave velocities varies by more than 2 % from their average value. The error in  $E$  and  $G$  (see 8.2) due to both anisotropy and experimental error

will then normally not exceed 6 %. The maximum possible error in  $\mu$ ,  $\lambda$ , and  $K$  depends markedly upon the relative values of  $V_p$  and  $V_s$  as well as upon testing errors and anisotropy. In common rock types the respective percent of errors for  $\mu$ ,  $\lambda$ , and  $K$  may be large as or even higher than 24, 36, and 6. For greater anisotropy, the possible percent of error in the elastic constants would be still greater.

8. Calculation

8.1 Calculate the propagation velocities of the compression and shear waves,  $V_p$  and  $V_s$ , respectively, as follows:

$$V_p = L_p/T_p$$

$$V_s = L_s/T_s$$

where:

$V$  = pulse-propagation velocity, in./s (or m/s),  
 $L$  = pulse-travel distance, in. (or m),  
 $T$  = effective pulse-travel time (measured time minus zero time correction), s,

and subscripts  $p$  and  $s$  denote the compression wave and shear wave, respectively.

8.2 If the degree of velocity anisotropy is 2 % or less, as specified in 6.4, calculate the ultrasonic elastic constants as follows:

$$E = [\rho V_s^2(3V_p^2 - 4V_s^2)]/(V_p^2 - V_s^2)$$

where:

$E$  = Young's modulus of elasticity, psi (or Pa), and  
 $\rho$  = density, lb/in.<sup>3</sup> (or kg/m<sup>3</sup>);

$$G = \rho V_s^2$$

where:

$G$  = modulus of rigidity or shear modulus, psi (or Pa);

$$\mu = (V_p^2 - 2V_s^2)/[2(V_p^2 - V_s^2)]$$

where:

$\mu$  = Poisson's ratio;

$$\lambda = \rho(V_p^2 - 2V_s^2)$$

where:

$\lambda$  = Lamé's constant, psi (or Pa); and

$$K = \rho(3V_p^2 - 4V_s^2)/3$$

where:

$K$  = bulk modulus, psi (or Pa).

9. Report

9.1 The report shall include the following:

9.1.1 Identification of the test specimen including rock type and location,

9.1.2 Density of test specimen,

9.1.3 General indication of moisture condition of sample at time of test such as as-received, saturated, laboratory air

dry, or oven dry. It is recommended that the moisture condition be more precisely determined when possible and reported as either water content or degree of saturation.

9.1.4 Degree of anisotropy expressed as the maximum percent deviation of compression-pulse velocity from the average velocity determined from measurements in three directions,

9.1.5 Stress level of specimens,

9.1.6 Calculated pulse velocities for compression and shear waves with direction of measurement,

9.1.7 Calculated ultrasonic elastic constants (if desired and if degree of anisotropy is not greater than specified limit),

9.1.8 Coupling medium between transducers and specimen, and

9.1.9 Other data such as physical properties, composition, petrography, etc., if determined.

10. Precision and Bias

10.1 An interlaboratory study of longitudinal and transverse pulse velocity (LPV and TPV) of intact specimens of four rock types was conducted in accordance with Practice E 691 in six laboratories with five replications per rock type. The results of this study are reported in ISR Research Report No. PS D18.12-R01, 1992, and its Addendum, 1994.

10.2 The repeatability and reproducibility statistics reported in Table 1 refer to within-laboratory and between-laboratory precision, respectively. Each entry in the tables has the dimensions of km/s.

10.3 The probability is approximately 95 % that two test results obtained in the same laboratory on the same material will not differ by more than the repeatability limit  $r$ . Likewise, the probability is approximately 95 % that two test results obtained in different laboratories on the same material will not differ by more than the reproducibility limit  $R$ . The precision statistics are calculated from:

$$r = 2 \sqrt{2} s_r$$

where:

$s_r$  = the repeatability standard deviation, and

$$R = 2 \sqrt{2} s_R$$

where:

$s_R$  = the reproducibility standard deviation.

10.4 It should be noted here that the anisotropy for TPV in Barre Granite is about 5 %, depending on the orientation of the plane of polarization for shear waves. The data presented here are "average" results.

10.5 Bias—There is no accepted reference value for this test method; therefore bias cannot be determined.

TABLE 1 Repeatability and Reproducibility Statistics

	Barre Granite	Berea Sandstone	Salem Limestone	Tennessee Marble
		LPV, km/s		
Mean $\bar{x}$	3.47	2.28	4.15	6.15
Repeatability limit $r$	0.22	0.15	0.27	0.44
Reproducibility limit $R$	0.48	0.30	0.58	0.45
		TPV, km/s		
Mean $\bar{x}$	2.37	1.45	2.30	3.33
Repeatability limit $r$	0.14	0.07	0.21	0.25
Reproducibility limit $R$	0.80	0.58	0.61	0.55

11. Key

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- (1) Simr Elec 1337
- (2) Whit Test Press
- (3) Can Four Min

11. Keywords

11.1 compression testing; isotropy; ultrasonic testing; velocity-pulse

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- (2) Whitehurst, E. A., *Evaluation of Concrete Properties from Sonic Tests*, Am. Concrete Inst., Detroit, Mich., and the Iowa State Univ. Press, Ames, Iowa, 1966, pp. 1-2.
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- (6) Silaeva, O. I., and Shamina, O. G., "The Distribution of Elastic Pulses in Cylindrical Specimens," *USSR Academy of Sciences (Izvestiya), Geophysics Series*, 1958, pp. 32-43, (English ed., Vol 1, No. 1, 1958, pp. 17-24).

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# Standard Practice for Petrographic Examination of Hardened Concrete<sup>1</sup>

This standard is issued under the fixed designation C 856; 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 ( $\epsilon$ ) indicates an editorial change since the last revision or reapproval.

## 1. Scope

1.1 This practice outlines procedures for the petrographic examination of samples of hardened concrete. The samples examined may be taken from concrete constructions, they may be concrete products or portions thereof, or they may be concrete or mortar specimens that have been exposed in natural environments, or to simulated service conditions, or subjected to laboratory tests. The phrase "concrete constructions" is intended to include all sorts of objects, units, or structures that have been built of hydraulic cement concrete.

NOTE—A photographic chart of materials, phenomena, and reaction products discussed in Sections 7 through 12 and Tables 1 through 6 are available as Adjunct C 856.

1.2 The petrographic procedures outlined herein are applicable to the examination of samples of all types of hardened hydraulic-cement mixtures, including concrete, mortar, grout, plaster, stucco, terrazzo, and the like. In this practice, the material for examination is designated as "concrete," even though the commentary may be applicable to the other mixtures, unless the reference is specifically to media other than concrete.

1.3 Annex A1 outlines an uranyl acetate method for identifying locations where alkali-silica gel may be present. It is a requirement that the substances in those locations must be identified using any other more definitive techniques, such as petrographic microscopy.

1.4 The purposes of and procedures for petrographic examination of hardened concrete are given in the following sections:

	Section
Qualifications of Petrographers	3
Purposes of Examination	4
Apparatus	5
Selection and Use of Apparatus	6
Samples	7
Examination of Samples	8
Specimen Preparation	9
Visual and Stereomicroscope Examination	10
Polarizing Microscope Examination	11
Metallographic Microscope Examination	12
Report	13

1.5 The values stated in inch-pound units are to be regarded as the standard. The SI units in parentheses are provided for information purposes only.

1.6 *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 appro-*

*priate safety and health practices and determine the applicability of regulatory limitations prior to use. A specific hazard statement is given in 5.2.10.1.*

## 2. Referenced Documents

### 2.1 ASTM Standards:

- C 42 Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete<sup>2</sup>
- C 215 Test Method for Fundamental Transverse, Longitudinal, and Torsional Frequencies of Concrete Specimens<sup>2</sup>
- C 227 Test Method for Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar-Bar Method)<sup>2</sup>
- C 294 Descriptive Nomenclature of Constituents of Natural Mineral Aggregates<sup>2</sup>
- C 295 Guide for Petrographic Examination of Aggregates for Concrete<sup>2</sup>
- C 342 Test Method for Potential Volume Change of Cement-Aggregate Combinations<sup>2</sup>
- C 441 Test Method for Effectiveness of Mineral Admixtures or Ground Blast-Furnace Slag in Preventing Excessive Expansion of Concrete Due to the Alkali-Silica Reaction<sup>2</sup>
- C 452 Test Method for Potential Expansion of Portland Cement Mortars Exposed to Sulfate<sup>3</sup>
- C 457 Practice for Microscopical Determination of Air-Void Content and Parameters of the Air-Void System in Hardened Concrete<sup>2</sup>
- C 496 Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens<sup>2</sup>
- C 597 Test Method for Pulse Velocity Through Concrete<sup>2</sup>
- C 637 Specification for Aggregates for Radiation-Shielding Concrete<sup>2</sup>
- C 638 Descriptive Nomenclature of Constituents of Aggregates for Radiation-Shielding Concrete<sup>2</sup>
- C 803 Test Method for Penetration Resistance of Hardened Concrete<sup>2</sup>
- C 805 Test Method for Rebound Number of Hardened Concrete<sup>2</sup>
- C 823 Practice for Examination and Sampling of Hardened Concrete in Constructions<sup>2</sup>
- C 944 Test Method for Abrasion Resistance of Concrete or Mortar Surfaces by the Rotating Cutter Method<sup>2</sup>
- C 1012 Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution<sup>3</sup>
- C 1260 Test Method for Potential Alkali Reactivity of Aggregates (Mortar-Bar Method)<sup>2</sup>

<sup>1</sup> This practice is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.65 on Petrography.

Current edition approved Dec. 10, 1995. Published March 1996. Originally published as C 856 - 77. Last previous edition C 856 - 83 (1998)<sup>1</sup>.

<sup>2</sup> Annual Book of ASTM Standards, Vol 04.02.

<sup>3</sup> Annual Book of ASTM Standards, Vol 04.01.

TABLE 1 Visual Examination of Concrete (1)<sup>6</sup>

Coarse Aggregate	+ Fine Aggregate	+ Matrix	+ Air	+ Embedded Items
<i>Composition:</i>				
Maximum dimension, <sup>A</sup> in. or mm, in the range >d>	Type:	color, by comparison with National Research Council <i>Rock Color Chart</i> (1963)	more than 3 % of total, predominantly in spherical voids?	Type, size, location; kinds of metal; other items
Type:	Type:	color distribution:	less than 3 % of total, abundant nonspherical voids?	
1 Gravel	1 Natural sand	1 mottled	voids empty, filled, lined, or partly filled	
2 Crushed stone	2 Manufactured sand	2 even		
3 Mixed 1 and 2	3 Mixed	3 gradational changes		
4 Other (name)	4 Other (name)			
5 Mixed 1 +/or 2 +/or 4	5 Mixed 1 +/or 2 +/or 4			
If Type 1, 2, or 4, homogeneous or heterogeneous	If Type 1, 2, or 4, homogeneous or heterogeneous			
Lithologic types				
Coarse aggregate more than 20, 30, 40, or 50 % of total				
<i>Fabric:</i>				
Shape	distribution	distribution	shape	voids below horizontal or low-angle reinforcement
Distribution	particle shape		distribution	
Packing	grading	as perceptible	grading (as perceptible)	
Grading (even, uneven, excess, or deficiency of size or sizes)	preferred orientation		parallelism of long axes of irregular voids or sheets of voids: with each other; with flat sides or long axes of coarse aggregate	
Parallelism of flat sides or long axes of exposed sections, normal to direction of placement +/or parallel to formed and finished surfaces <sup>B</sup>				
<i>Condition:</i>				
Does it ring when hit lightly with a hammer or give a dull flat sound? Can you break it with your fingers? Cracks? How distributed? Through or around coarse aggregate? With cores or sawed specimens, did the aggregate tear in drilling or sawing? Crack fillings? Surface deposits? If air dry, are there unusually wet or dry looking areas? Rims on aggregate?				clean or corroded? Are cracks associated with embedded items?

<sup>A</sup> A substantial portion of the coarse aggregate has maximum dimensions in the range shown as measured on sawed or broken surfaces.

<sup>B</sup> Sections sawed or drilled close to and parallel to formed surfaces appear to show local turbulence as a result of spading or rodding close to the form. Sections sawed in the plane of bedding (normal to the direction of placement) are likely to have inconspicuous orientation. Sections broken normal to placement in conventionally placed concrete with normal bond tend to have aggregate knobs abundant on the bottom of the upper piece as cast and sockets abundant on the top of the lower piece as cast.

E 3 Methods of Preparation of Metallographic Specimens<sup>4</sup>  
E 883 Practice for Metallographic Photomicrography<sup>4</sup>

2.2 *ASTM Adjunct:*  
Adjunct C 856 A chart of 27 photos<sup>5</sup>

3. Qualifications of Petrographers

3.1 The examination should be performed by persons qualified by education and experience to operate the equipment used and to record and interpret the results obtained. In some cases, the petrographer will have had experience adequate to provide detailed interpretation of the materials' performance with respect to engineering and other consequences of the observations. In others, the interpretation will be made in part by engineers, scientists, or others qualified to relate the observations to the questions to be answered.

3.2 This practice may be used by a petrographer employed directly by those for whom the examination is made. The employer should tell the petrographer, in as much detail as necessary, the purposes and objectives of the examination, the kind of information needed, and the extent of examination desired. Pertinent background information, including results of prior testing, should be made available. If the petrographer is highly experienced, his advice and judgment

should be sought regarding the extent of the examination, and the matters discussed in 3.3.

3.3 This practice may form the basis for establishing arrangements between a purchaser of a consulting service and the consultant. In such a case, the purchaser and the consultant should together determine the kind, extent, and objectives of the examination and analyses to be made, and should record their agreement in writing. The agreement may stipulate specific determinations to be made, observations to be reported, funds to be obligated, or a combination of these or other conditions.

4. Purposes of Examination

4.1 Examples of purposes for which petrographic examination of concrete is used are given in 4.2 through 4.5. The probable usefulness of petrographic examination in specific instances may be determined by discussion with an experienced petrographer of the objectives of the investigation proposed or underway.

4.2 *Concrete from Constructions:*

- 4.2.1 Determination in detail of the condition of concrete in a construction.
- 4.2.2 Determination of the causes of inferior quality, distress, or deterioration of concrete in a construction.

<sup>4</sup> Annual Book of ASTM Standards, Vol 03.01.

<sup>5</sup> Available from ASTM Headquarters, 100 Barr Harbor Drive, West Conshohocken, PA 19428. Request adjunct No. 12-308560-00.

<sup>6</sup> The boldface numbers in parentheses refer to the list of references at the end of this practice.

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**TABLE 2 Outline for Examination of Concrete with a Stereomicroscope (1)**

**NOTE—Condition**—When it is examined at 6 to 10× under good light, the freshly broken surface of a concrete in good physical condition that still retains most of its natural moisture content has a luster that in mineralogical terms is subtranslucent, glistening vitreous.<sup>4</sup> Thin edges of splinters of the paste transmit light; reflections appear to come from many minute points on the surface, and the quality of luster is like that from broken glass but less intense. Concrete in less good physical condition is more opaque on a freshly broken surface, and the luster is dull, subvitreous, going toward chalky. A properly cured laboratory specimen from a concrete mixture of normal proportions cured 28 days that has shown normal compressive or flexural strength and that is broken with a hammer and examined on a new break within a week of the time that it finished curing should provide an example of concrete in good physical condition.

Under the same conditions of examination, when there is reasonable assurance that the concrete does not contain white portland cement or slag cement, the color of the matrix of concrete in good physical condition is definitely gray or definitely tan, except adjoining old cracks or original surfaces.

Coarse Aggregate	Fine Aggregate	Matrix	Voids
Lithologic types and mineralogy as perceptible	Lithologic types, and mineralogy as perceptible	Color	Grading
Surface texture	Shape	Fracture around or through aggregate	Proportion of spherical to nonspherical
Within the piece:	Surface texture	Contact of matrix with aggregate:	Nonspherical, ellipsoidal, irregular, disk-shaped
Grain shape	Grading	close, no opening visible on sawed or broken surface; aggregate not dislodged with fingers or probe; boundary openings frequent, common, rare	Color change from interior surface to matrix
Grain size extreme range observed, mm	Distribution	Width	Interior surface luster like rest of matrix, dull, shining
Median within range — to — mm		Empty	Linings in voids absent, rare, common, in most, complete, partial, colorless, colored, silky tufts, hexagonal tablets, gel, other
Textureless (too fine to resolve)		Filled	Underside voids or sheets of voids uncommon, small, common, abundant
Uniform or variable within the piece		Cracks present, absent, result of specimen preparation, preceding specimen preparation	
From piece to piece:		Mineral admixtures <sup>c</sup>	
Intergranular bond		Contamination	
Porosity and absorption <sup>b</sup>		Bleeding	
If concrete breaks through aggregate, through how much of what kind?			
If boundary voids, along what kind of aggregate? All? All of one kind? More than 50 % of one kind? Several kinds?			
Segregation			

<sup>4</sup> Dana, E. S., *Textbook of Mineralogy*, revised by W. E. Ford, John Wiley & Sons, New York, N. Y. 4th ed., 1932, pp. 273-274.

<sup>b</sup> Pore visible to the naked eye, or at × —, or sucks in water that is dropped on it.

<sup>c</sup> Dark solid spheres or hollow-centered spheres of glass, or of magnetite, or some of glass and some of magnetite, recognizable at magnification of ×9 on sawed or broken surfaces. Other mineral admixtures with characteristic particles visible at low magnification are recognizable. Ground surface of concrete containing portland blast-furnace slag cement are unusually white near-free surfaces but retain greenish or blue-greenish patches, and slag particles can be seen with the stereomicroscope or polarizing microscope.

4.2.3 Determination of the probable future performance of the concrete.

4.2.4 Determination whether the concrete in a construction was or was not as specified. In this case, other tests may be required in conjunction with petrographic examination.

4.2.5 Description of the cementitious matrix, including qualitative determination of the kind of hydraulic binder used, degree of hydration, degree of carbonation if present, evidence of unsoundness of the cement, presence of a mineral admixture, the nature of the hydration products, adequacy of curing, and unusually high water - cement ratio of the paste.

4.2.6 Determination whether alkali - silica or alkali - carbonate reactions, or cement - aggregate reactions, or reactions between contaminants and the matrix have taken place, and their effects upon the concrete.

4.2.7 Determination whether the concrete has been subjected to and affected by sulfate attack, or other chemical attack, or early freezing, or to other harmful effects of freezing and thawing.

4.2.8 Part of a survey of the safety of a structure for a present or proposed use.

4.2.9 Determination whether concrete subjected to fire is essentially undamaged or moderately or seriously damaged.

4.2.10 Investigation of the performance of the coarse or fine aggregate in the structure, or determination of the composition of the aggregate for comparison with aggregate from approved or specified sources.

4.2.11 Determination of the factors that caused a given concrete to serve satisfactorily in the environment in which it was exposed.

4.2.12 Determination of the presence and nature of sur-

face treatments, such as dry shake applications on concrete floors.

4.3 *Test Specimens from Actual or Simulated Service*—Concrete or mortar specimens that have been subjected to actual or simulated service conditions may be examined for most of the purposes listed under Concrete from Constructions.

4.4 *Concrete Products:*

4.4.1 Petrographic examination can be used in investigation of concrete products of any kind, including masonry units, precast structural units, piling, pipe, and building modules. The products or samples of those submitted for examination may be either from current production, from elements in service in constructions, or from elements that have been subjected to tests or to actual or simulated service conditions.

4.4.2 Determination of features like those listed under concrete from constructions.

4.4.3 Determination of effects of manufacturing processes and variables such as procedures for mixing, molding, demolding, consolidation, curing, and handling.

4.4.4 Determination of effects of use of different concrete-making materials, forming and molding procedures, types and amounts of reinforcement, embedded hardware, etc.

4.5 *Laboratory Specimens*—The purposes of petrographic examination of laboratory specimens of concrete, mortar, or cement paste are, in general, to investigate the effects of the test on the test piece or on one or more of its constituents, to provide examples of the effects of a process, and to provide the petrographer with visual evidence of examples of reactions in paste or mortar or concrete of known materials, proportions, age, and history. Specific purposes include:

TABLE 3 Effects of Fire on Characteristics of Concrete

Characteristic	Causes and Effects	Ways of Investigation
Surface hardness	Dehydration to 100°C removes free water; dehydration is essentially complete at 540°C; calcium hydroxide goes to CaO at 450-500°C. Paste expands with thermal coefficient effect and then shrinks, cracks, decrepitates, and becomes soft (4).	Beneath the softened concrete, which can be tested in accordance with Test Method C 805, the concrete is probably normal if it has not undergone color change. Establish by coring for compressive tests, by wear tests (CRD-C 52) (4), and by scratching with a knife.
Cracking	Perpendicular to the face and internal, where heating or cooling caused excess tensile stresses. In some new concrete, resembles large-scale shrinkage cracking; may penetrate up to 100 mm but may heal autogenously (4).	Examination of the surface, ultrasonic tests, coring, petrographic examination (4).
Color change—When concrete has not spalled, observe depth of pink color to estimate the fire exposure.	Concrete made with sedimentary or metamorphic aggregates shows permanent color change on heating. Color normal to 230°C; goes from pink to red from 290 to 590°C; from 590 to 900°C color changes to gray and then to buff (4). For temperatures up to about 500°C temperature distribution is little affected by using carbonate rather than siliceous aggregate (5). At 573°C low quartz inverts to high with 0.85 % increase in volume, producing popouts. Spalling over steel to expose one fourth of the bar at 790°C; white powdered decomposed hydration products at 900°C. Surface crazing about 290°C; deeper cracking about 540°C.	Color change is the factor most useful to the investigator; permits recognizing how deeply a temperature of about 300°C occurred (5).
Aggregate behavior—Aggregate behavior affects strength, modulus, spalling, cracking, surface hardness, and residual thermal strains (4).	Aggregates differ in thermal diffusivity, conductivity, coefficient of expansion. Heat transmission decreases from concrete made with highly siliceous aggregate, sandstone, traprock, limestone, lightweight aggregates (4). Occurs subparallel to free face; followed by breaking off saucer-like pieces especially at corners and edges (4).	Changes on heating are often accompanied by volume change (4).
Spalling	Reduction in strength of concrete containing siliceous gravel after heating, then cooling and testing: Heated to Temperature, °C	Determinations by compressive tests and static modulus of cores; Test Method C 805 for qualitative determination; Test Method C 597 (4).
	Reduction, %	
	180	25
	370	50
	570	80
	Reduction in Modulus Temperature, °C	Reduction, %
	200	25
	430	50
	760	70

Note: Compressive strength and elastic modulus. For concrete at least 1-year old, strength will increase after cooling from 300°C if design strength was attained (5).

4.5.1 To establish whether alkali - silica reaction has taken place, what aggregate constituents were affected, what evidence of the reaction exists, and what were the effects of the reaction on the concrete. The technique in Annex A1 is helpful for identifying locations where alkali-silica gel may be present.

4.5.2 To establish whether one or more alkali - carbonate reactions have taken place, which aggregate constituents were affected and what evidence of the reaction or reactions exists, and the effects of the reaction on the concrete properties.

4.5.3 To establish whether any other cement - aggregate reaction has taken place. In addition to alkali - silica and alkali - carbonate reactions, these include hydration of anhydrous sulfates, rehydration of zeolites, wetting of clays and reactions involving solubility, oxidation, sulfates, and sulfides (see Refs 1, 2, and 3).<sup>6</sup>

4.5.4 To establish whether an aggregate used in a test has been contaminated by a reactive constituent when in fact the aggregate was not reactive.

4.5.5 To establish the effects of a freezing and thawing test or other physical or mechanical exposure of concrete on the aggregate and the matrix.

4.5.6 To establish the extent of reaction, the nature of

reaction products, and effects of reaction produced in exposure to a chemically aggressive environment such as in Test Method C 452 or Test Method C 1012.

4.5.7 To determine the characteristics of moist-cured concrete that has not been subjected to chemical attack or cement - aggregate reaction or freezing and thawing.

4.5.8 By comparison with appropriate laboratory specimens, a petrographer may be able to substantiate the existence of a particular reaction in concrete or determine that the reaction cannot be detected.

5. Apparatus

5.1 The apparatus and supplies employed in making petrographic examinations of hardened concrete depend on the procedures required. The following list includes the equipment generally used. Equipment needed to perform the examinations in Annex A1 is listed therein. Equipment required for field sampling is not listed. Any other useful equipment may be added.

5.2 For Specimen Preparation:

5.2.1 Diamond Saw—Slabbing saw with an automatic feed and blade large enough to make at least a 7-in. (175-mm) cut in one pass.

5.2.2 Cutting Lubricant, for diamond saw.

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TABLE 4 Outline for Examination of Concrete in Thin Sections

Coarse and Fine Aggregate	Relict Cement Grains and Hydration Products	Characteristics of Cement Paste
<p>Mineralogy, texture, fabric, variable or homogeneous.</p> <p>Grading; excess or deficiency of sand sizes is to be judged after examination of a series of thin sections. Grain size and nature of internal boundaries in aggregate. Classification of coarse and fine aggregate.</p> <p>Natural mineral aggregate or crushed stone; natural or manufactured fine aggregate.</p> <p>Bond with matrix; peripheral cracks inside the borders of aggregate grains; internal cracks. General microfractures if one can establish that they existed before thin-sectioning.</p> <p><i>Alkali - carbonate reactions</i>—If the coarse aggregate is a carbonate rock or rocks, are there rims or partial rims depleted in calcium hydroxide? Partly dolomitic rocks that have reacted sometimes are bordered with paste free from calcium hydroxide along the dolomitic portion while the paste along the limestone portion is normal. See other comments in Column 3.</p> <p><i>Alkali - silica reaction</i>—Does the aggregate contain particles of types known to be reactive (chert, novaculite, acid volcanic glass, cristobalite, tridymite, opal, bottle glass)? If quartzite, metamorphosed subgraywacke, argillite, phyllite, or any of those listed in the sentence above, are there internal cracks inside the periphery of the aggregate? Has the aggregate been gelatinized so that it has pulled off during sectioning leaving only a peripheral hull bonded to the mortar? (This last phenomenon also occurs in concrete with air-cooled slag aggregate, where it indicates reaction between cement and slag.) Cracks that appear to be tensile and to narrow from the center toward the border of the particle are also evidence of alkali - silica reaction (6).</p>	<p>In concrete over 2 years old and normally cured, the only residual cement grains are those that were largest, which may be composed of several constituents or be of alite or belite (substituted C<sub>3</sub>S and C<sub>2</sub>S). The latter two may be bordered by one or two layers of gel having different indexes of refraction, or by a layer of calcium hydroxide. The largest relict grains may be truly unhydrated and retain the low (dark gray) birefringence of alite in distorted quasihexagonal sections and the visible birefringence to first-order yellow of the lamellar twins in rounded grains of belite. Interstitial aluminoferrite appears as prismatic grains ranging in color from brown to greenish brown to reddish brown and having a high refractive index and pleochroism masked by the color of the grain. Tricalcium aluminate is usually not recognized in thin section because the cubic form is isotropic or because it hydrates early in the hydration history of the concrete forming submicroscopic ettringite or tetracalcium aluminum sulfate hydrate or other tetracalcium aluminum hydrates with or without other anions. These may be visible in voids in older concrete but are best discriminated by X-ray diffraction. Cements from different sources have different colors of aluminoferrite and the calcium silicates have pale green or yellow or white shades. It should be possible to match cements from one source.</p>	<p>Normal cement paste consists in plane transmitted light of pale tan matter varying somewhat in index of refraction and containing relict unhydrated cement grains. In concrete sectioned at early age or not adequately cured, the paste contains unhydrated cement grains ranging down to a few micrometres in maximum size with an upper limit as large as 100 μm in maximum diameter if the cement was ground in open-circuit mills or was deliberately ground to low surface area to reduce the heat of hydration. With crossed polars, normal paste is black or very dark mottled gray with scattered anhedral poikilitic crystals or small segregations of calcium hydroxide and scattered relict grains of cement. In concrete of high water - cement ratio and siliceous aggregate, the calcium hydroxide crystals are as large as the maximum size of residual cement grains, about 100 μm. In concrete of lower water - cement ratio, higher cement content, and either siliceous or carbonate aggregate, the maximum size of calcium hydroxide crystals is considerably smaller. Regardless of water - cement ratio and type of aggregate, calcium hydroxide crystals occupy space tangential to the undersides of aggregate particles. Where all the aggregate is carbonate rock the maximum size of calcium hydroxide is smaller than in comparable concrete with siliceous aggregate. (Calcium hydroxide is probably epitaxial on calcite.)</p> <p>Cement paste in concrete that has been subjected to prolonged acid leaching is low in calcium hydroxide which is present as recrystallized virtually anhedral grains precipitated near the exterior surfaces.</p> <p>In concrete over 2 or 3 years old made with Type I, II, or III cement, some ettringite is to be expected as rosettes in air voids. This is a normal phenomenon; to demonstrate sulfate attack it must be established chemically that the SO<sub>3</sub> content of the concrete is greater than would be supplied by the original sulfate content of the cement. Ettringite in voids is not ettringite that has damaged concrete although it may accompany submicroscopic ettringite in the paste that has damaged the concrete.</p>

5.2.3 *Horizontal Lap Wheel or Wheels*, steel, cast iron, or other metal lap, preferably at least 16 in. (400 mm) in diameter, large enough to grind at least a 4 by 6-in. (100 by 152-mm) area.

5.2.4 *Free Abrasive Machine*, using abrasive grit in lubricant, with sample holders rotating on a rotating table. This type of grinding machine greatly increases the speed of preparation of finely ground surfaces.

5.2.5 *Polishing Wheel*, at least 8 in. (200 mm) in diameter and preferably two-speed, or a vibratory polisher.

5.2.6 *Hot Plate or Oven*, thermostatically controlled, to permit drying and impregnating specimens with resin or wax for preparing thin sections, ground surfaces, and polished sections.

5.2.7 *Prospector's Pick or Bricklayer's Hammer*, or both.

5.2.8 *Abrasives*—Silicon carbide grits, No. 100 (150-μm), No. 220 (63-μm), No. 320 (31-μm), No. 600 (16-μm), No. 800 (12-μm); optical finishing powders, such as M-303, M-204, M-309; polishing powders as needed.

5.2.9 *Plate-glass Squares*, 12 to 18-in. (300 to 450-mm)

on an edge and at least 3/8 in. (10 mm) thick for hand-finishing specimens.

5.2.10 *Suitable Medium or Media*, for impregnating concrete and mounting thin sections plus appropriate solvent. Canada balsam, Lakeside 70 cement, and flexibilized epoxy formulations have been used.

5.2.10.1 **Warning**—Flexibilized epoxies form strong bonds but have higher indexes of refraction than Canada balsam or Lakeside 70 and are toxic. Do not allow to touch the skin; plastic gloves shall be worn, and the work shall be done under a hood so as not to breathe the fumes.

5.2.11 *Microscope Slides*—Clear, noncorrosive, glass approximately 24 mm wide and at least 45 mm long. Thickness may need to be specified to fit some thin section machines.

5.2.12 *Cover Glasses*, noncorrosive and preferably No. 1 (0.18-mm) thickness.

5.3 *For Specimen Examination:*

5.3.1 *Stereomicroscope*, providing magnifications in the range from 7× to 70× or more.

5.3.2 *Dollies*—Small, wheeled dollies with flat tops and with tops curved to hold a section of core assist in manipu-

TABLE 5 Characteristics of Concrete Observed Using Microscopes

Characteristic	Type of Microscope		
	Stereomicroscope	Petrographic	Metallographic
<b>Aggregate:</b>			
Shape	X	X	X
Grading	X	...	...
Distribution	X	...	...
Texture	X	X	X
Composition	X	X	...
Rock types	X	X	...
Alteration	X	X	...
degree	X	X	...
products	X	X	...
Coatings	X	X	X
Rims	X	X	...
Internal cracking	X	X	...
Contamination	X	X	X
<b>Concrete:</b>			
Air-entrained or not	...	...	...
Air voids	X	X	X
shape	X	...	...
size	X	...	...
distribution	X	...	...
Bleeding	X	...	...
Segregation	X	X	X
Aggregate-paste bond	X	X	X
Fractures			
Embedded items			
size	X	...	...
shape	X	...	...
location	X	...	...
type	X	...	...
Alteration			
degree and type	X	X	X
reaction products			
location	X	X	X
identification	X <sup>A</sup>	X	...
Nature and condition of surface treatments	X	X	...
<b>Paste:</b>			
Color	X	X	...
Hardness	X	...	X
Porosity	X	...	X
Carbonation	X	X	...
Residual cement			
distribution	...	X	X
particle size	...	X	X
abundance	...	X	X
composition	...	X	X
Mineral admixtures	X <sup>B</sup>	X	X
size	...	X	X
abundance	X	X	X
identification	X <sup>C</sup>	X	X
Compounds in hydrated cement			
Contamination			
size	X	X	X
abundance	X	X	X
identification	...	X	X <sup>D</sup>

<sup>A</sup> Secondary ettringite can sometimes be recognized by crystal habit and silky luster.

<sup>B</sup> Fly ash can be detected by color and shape when dark spheres are present. In concrete that has not oxidized the presence of slag may be inferred from the green or blue color of the paste.

<sup>C</sup> Ettringite and calcium hydroxide in voids may be recognized by their crystal habits.

<sup>D</sup> Magnesium oxide and calcium oxide should be identifiable in polished section.

lating concrete specimens under the stereomicroscope.

5.3.3 *Polarizing Microscope*, for examinations in transmitted light, with mechanical stage; low-, medium-, and high-power objectives such as 3.5X, 10X, and 20 to 25X; 43 to 50X with numerical aperture 0.85 or more; assorted eyepieces having appropriate corrections and magnifications for use with each of the objectives; micrometer eyepiece; condenser adjustable to match numerical aperture of objective with highest numerical aperture to be used; full-wave

and quarter-wave compensators, quartz wedge, and other accessories.

5.3.4 *Metallographic Microscope*, with vertical illuminator, mechanical stage, metallographic objectives of low, medium, and high magnification, and appropriate eyepieces to provide a range of magnifications from about 25X to 500X. Reflected polarized light should be available and appropriate compensators provided. Some polarizing microscopes can be equipped with accessories for metallographic

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Hydrated  
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TABLE 6 Secondary Deposits in Concrete<sup>A</sup>

Compound and Mineral Equivalent	Indexes of Refraction	Form and Occurrence
Calcium carbonate (CaCO <sub>3</sub> ); calcite	$\omega$ = 1.658 $\epsilon$ = 1.486	Fine-grained, white or gray masses or coatings in the cement paste, in voids, along fractures, or on exposed surfaces; very common
Calcium carbonate (CaCO <sub>3</sub> ); aragonite	$\alpha$ = 1.530 $\beta$ = 1.680 $\gamma$ = 1.685	Minute, white prisms or needles in voids or fractures in concrete; rare
Calcium carbonate (CaCO <sub>3</sub> ); vaterite	$\omega$ = 1.544–1.550 $E$ = 1.640–1.650	Spherulitic, form-birefringent, white encrustations on moist-stored laboratory specimens (vaterite A); also identified in sound concrete from structures by X-ray diffraction ( $\alpha$ -vaterite); common (7)
6-calcium aluminate trisulfate-32 hydrate (Ca <sub>6</sub> [Al(OH) <sub>6</sub> ] <sub>2</sub> ·24H <sub>2</sub> O)(SO <sub>4</sub> ) <sub>3</sub> ·2H <sub>2</sub> O (8); ettringite	$\omega$ = 1.464–1.469 <sup>B</sup> $\epsilon$ = 1.458–1.462	Fine, white fibers or needles or spherulitic growths in voids, in the cement paste, or in fractures; very common (1, 7)
Tetracalcium aluminate monosulfate-12-hydrate (3CaO·Al <sub>2</sub> O <sub>3</sub> ·CaSO <sub>4</sub> ·12H <sub>2</sub> O)	$\omega$ = 1.504 $\epsilon$ = 1.49	White to colorless, minute, hexagonal plates in voids and fractures; very rare (7)
Tetracalcium aluminate-13-hydrate (Ca <sub>4</sub> Al <sub>2</sub> (OH) <sub>14</sub> ·6H <sub>2</sub> O)	$\omega$ = 1.53 $\epsilon$ = 1.52	Micalike, colorless, pseudo-hexagonal, twinned crystals in voids; very rare (9)
Hydrous sodium carbonate (Na <sub>2</sub> O·CO <sub>2</sub> ·H <sub>2</sub> O); thermonatrite	$\alpha$ = 1.420 $\beta$ = 1.506 $\gamma$ = 1.524	Minute inclusions in alkalic silica gel; rare (7)
Hydrated aluminum sulfate (2Al <sub>2</sub> O <sub>3</sub> ·SO <sub>3</sub> ·15H <sub>2</sub> O); paraluminate	$\alpha$ = 1.463 ± 0.003 $\beta$ = 1.471 $\gamma$ = 1.471	Occurring in cavities in intensely altered concrete; very rare (9)
Calcium sulfate dihydrate (CaSO <sub>4</sub> ·2H <sub>2</sub> O); gypsum	$\alpha$ = 1.521 $\beta$ = 1.523 $\gamma$ = 1.530	White to colorless crystals in voids, in the cement paste, or along the surfaces of aggregate particles in concrete or mortar affected by sulfate or seawater attack; unusual
Calcium hydroxide (Ca(OH) <sub>2</sub> ); portlandite	$\omega$ = 1.574 $\epsilon$ = 1.547	White to colorless, hexagonal plates or tablets in the cement paste, in voids, along fractures; ubiquitous in concrete
Magnesium hydroxide (Mg(OH) <sub>2</sub> ); brucite	$\omega$ = 1.559 $\epsilon$ = 1.580	White to yellow, fine-grained encrustations and fillings in concrete attacked by magnesian solutions or seawater; unusual (10, 11)
Hydrous silica (SiO <sub>2</sub> ·nH <sub>2</sub> O); opal	$\eta$ = 1.43	White to colorless, finely divided, amorphous; resulting from intense leaching
Alkalic silica gel (Na <sub>2</sub> O·K <sub>2</sub> O·CaO·SiO <sub>2</sub> )	varies with water content $\eta$ = 1.46–1.53	or carbonation of cement paste; unusual in recognizable proportions
Hydrated iron oxides (Fe <sub>2</sub> O <sub>3</sub> ·nH <sub>2</sub> O); Limonite	opaque or nearly so	White, yellowish, or colorless; viscous, fluid, waxy, rubbery, hard; in voids, fractures, exudations, aggregate; common (12, 13)
Thaumasite (Ca <sub>6</sub> [Si(OH) <sub>6</sub> ] <sub>2</sub> ·24H <sub>2</sub> O)(SO <sub>4</sub> ) <sub>2</sub> (CO <sub>3</sub> ) <sub>2</sub> (8)	$\omega$ = 1.504 $\epsilon$ = 1.468 ± 0.002 <sup>B</sup>	Brown stain in fractures and on surfaces; common
Syngenite (K <sub>2</sub> Ca(SO <sub>4</sub> ) <sub>2</sub> )·H <sub>2</sub> O	$\alpha$ = 1.501 (15) $\beta$ = 1.51 $\gamma$ = 1.51	Prismatic, hexagonal; capable of growing in continuity with ettringite; in sewer pipe subject to sulfate attack, in grout, in some pavement (14)
Hydrotalcite Mg <sub>3/4</sub> Al <sub>1/4</sub> (OH) <sub>2</sub> (CO <sub>3</sub> ) <sub>1/8</sub> (H <sub>2</sub> O) <sub>1/2</sub> (8)	$\omega$ = 1.510 ± 0.003 $\epsilon$ = 1.495 ± 0.003	Found in cavities and zones peripheral to slate particles, in fibrous form (16)
		Foliated platy to fibrous masses (17, 8)

<sup>A</sup> The literature and private reports include data on many unidentified secondary compounds in concrete; these are not included in the tabulation. Indexes of refraction of common mineralogic types are taken from standard works on mineralogy.

<sup>B</sup> Higher and lower indexes of refraction have been recorded for naturally occurring ettringite (15) and thaumasite (14), but it is not known that the naturally occurring minerals and compounds found in hydrated cement are of the same composition.

examination, if the tube can be raised or the stage lowered to give adequate clearance for the vertical illuminator and the thicker specimens usually employed.

5.3.5 *Eyepiece Micrometer*—Eyepiece micrometers calibrated using a stage micrometer are useful for measuring particles of aggregate, cement grains, calcium hydroxide and other crystals, and crack widths.

5.3.6 *Stage Micrometer*, to calibrate eyepiece micrometers.

5.3.7 *Microscope Lamps*—Many modern polarizing microscopes have built-in illuminators which are convenient and satisfactory if, with the condenser, they can be adjusted to fill the back lens of the objective of highest numerical aperture with light. If the microscope requires a separate illuminator, tungsten ribbon-filament bulbs in suitable adjustable housings are satisfactory. Many kinds of illumina-

tors are available for stereomicroscopes; some can be mounted on the microscope, some stand on their own bases; choice is a question of adequacy of illumination for the tasks intended. Focusable illuminators are preferred.

5.3.8 *Needleholders and Points*—In addition to pin vises and needles from laboratory supply houses, a No. 10 sewing needle mounted in a handle or a selection of insect pins from size 00 to size 4 are useful for prying out reaction products.

5.3.9 *Bottles with Droppers*, for acid, water, and other reagents applied during examination.

5.3.10 *Assorted Forceps*, preferably stainless steel, including fine-pointed watchmaker's forceps.

5.3.11 *Lens Paper*.

5.3.12 *Refractometer, and Immersion Media*, covering the range of refractive indexes from 1.410 to at least 1.785, in steps not larger than 0.005. Stable immersion media, cali-

brated at a known temperature and of known thermal coefficient, are preferable and should be used in a temperature-controlled room. A thermometer graduated in tenths of a degree Celsius should be used to measure air temperature near the microscope stage so that thermal corrections of refractive index can be made if needed.

### 6. Selection and Use of Apparatus

6.1 Laboratories should be equipped to provide photographs, photomicrographs, and photomicrographs to illustrate significant features of the concrete. While ordinary microscope lamps are sometimes satisfactory for photomicrography in transmitted and reflected light, lamps providing intense point or field sources, such as tungsten ribbon-filament bulbs, or zirconium or carbon arcs, are highly desirable. For much useful guidance regarding photomicrography, especially using reflected light, see Practice E 883.

6.2 The minimum equipment for petrographic examination of concrete where both specimen preparation and examination are completed within the laboratory consists of a selection of apparatus and supplies for specimen preparation, a stereomicroscope preferably on a large stand so that 6-in. (152-mm) diameter cores can be conveniently examined, a polarizing microscope and accessories, lamps for each microscope, and stable calibrated immersion media of known thermal coefficient. Specimens for petrographic examination may be obtained by sending samples to individuals or firms that offer custom services in preparing thin or polished sections and finely ground surfaces. It is more convenient to prepare specimens in house, and their prompt availability overrides their probably greater cost.

6.3 X-ray diffraction, X-ray emission, differential thermal analysis, thermogravimetric analysis, analytical chemistry, infrared spectroscopy, scanning electron microscopy, energy or wavelength dispersive analysis, and other techniques may be very useful in obtaining quick and definite answers to relevant questions where microscopy will not do so. Some undesirable constituents of concrete, some hydration products of cement, and some reaction products useful in defining the effects of different exposures, and many contaminating materials may not be identified unless techniques that supplement light microscopy are used. (18, 19). The uranyl-acetate technique given in Annex A1 can be helpful in locating sites where alkali-silica gel may be present.

### 7. Samples

7.1 The minimum size of sample should amount to at least one core, preferably 6 in. (152 mm) in diameter and 1 ft (305 mm) long for each mixture or condition or category of concrete, except that in the case of pavement the full depth of pavement shall be sampled with a 4 or 6-in. (102 or 152-mm) core. Broken fragments of concrete are usually of doubtful use in petrographic examination, because the damage to the concrete cannot be clearly identified as a function of the sampling technique or representative of the real condition of the concrete. Cores smaller in diameter than 6 in. can be used if the aggregate is small enough; in deteriorated concrete, core recovery is much poorer with 2½-in. (54-mm) diameter core than with 6-in. diameter core. While it is desirable in examination and testing to have a

core three times the maximum size of aggregate, this circumstance is a rare occurrence when concrete with aggregate larger than 2 in. is sampled, because of the cost of large bits and the problems of handling large cores.

7.2 *Samples from Constructions*—The most useful samples for petrographic examination of concrete from constructions are diamond-drilled cores with a diameter at least twice (and preferably three times) the maximum size of the coarse aggregate in the concrete. If 6-in. (152-mm) aggregate is used, a core at least 10 in. (250 mm) in diameter is desirable; usually a 6-in. diameter core is the largest provided.

7.2.1 The location and orientation of all cores, including cores or core lengths not sent to the laboratory, should be clearly shown; and each core should be properly labeled. For vertically drilled cores, the elevation or depth at top and bottom of each section should be shown, and core loss and fractures antedating the drilling should be marked. For cores taken horizontally or obliquely, the direction of the vertical plane and the tops and bottoms should be marked. A field log should be provided.

7.2.2 Broken pieces of concrete from extremely deteriorated structures or pieces removed while preparing for repair work are sometimes used for petrographic examination. The samples will be more useful if their original locations in the structure are clearly described or indicated in a sketch or photographs.

7.2.3 The information provided with the samples should include:

7.2.3.1 The location and original orientation of each specimen (see Practice C 823),

7.2.3.2 The mixture proportions of the concrete or concretes,

7.2.3.3 Sources of concrete-making materials and results of tests of samples thereof,

7.2.3.4 Description of mixing, placing, consolidation, and curing methods,

7.2.3.5 Age of the structure, or in case of a structure that required several years to complete, dates of placement of the concrete sampled,

7.2.3.6 Conditions of operation and service exposure,

7.2.3.7 The reason for and objectives of the examination,

7.2.3.8 Symptoms believed to indicate distress or deterioration, and

7.2.3.9 Results of field tests such as measurements of pulse velocity (Test Method C 215), rebound hammer numbers (Test Method C 805) or probe readings (Test Method C 803).

7.3 *Samples from Test Specimens from Natural Exposures, Concrete Products, and Laboratory Specimens:*

7.3.1 Information provided should include: materials used, mixture proportions, curing, age of concrete when placed in service or test, orientation in exposure, present age, condition surveys during exposure, characteristics of the natural or laboratory exposure, and method of manufacture of concrete products. Large concrete products may be sampled like constructions; smaller ones may be represented by one or more showing the range of condition from service or fabrication or both.

7.3.2 The exposure of laboratory specimens should be described with test results, age at test and available test results on the aggregates, hydraulic binders, and admixtures

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used. This information should accompany test specimens from natural exposures and concrete products or samples therefrom, if available.

## 8. Examination of Samples

8.1 *Choice of Procedures*—Specific techniques and procedures employed in examination of a sample depend on the purpose of the examination and the nature of the sample. Procedures to be used should be chosen after the questions that the examination is intended to answer have been clearly formulated. The procedures should be chosen to answer those questions as unequivocally and as economically as possible. The details that need to be resolved will be dictated by the objectives of the examination and will vary for different situations. Consequently, the selection and location of specimens from the samples submitted for examination should be guided by the objectives of the study. Practice C 457 should be referred to for those relevant subjects not described here.

8.2 *Visual Examination and Outline of Additional Examination*—A petrographic examination of concrete, mortar, or cement paste should begin with a review of all the available information about the specimen or specimens, followed by a visual examination of each sample. An outline of information that can be obtained is given in Table 1. That study should be followed by an examination using a stereomicroscope (see Table 2 and the section on Visual and Stereomicroscopic Examination). In some cases, further study is unnecessary, and a report can be prepared. In other cases, specimens are chosen during the visual and stereomicroscope examination for further processing and additional stereomicroscope study, more detailed examination using the petrographic or metallographic microscopes or by X-ray diffraction and other instrumental methods, and for other chemical or physical tests. Methods for specimen preparation are outlined in the Specimen Preparation Section. Tables 2, 3, and 4 summarize characteristics of concrete conveniently observed with stereomicroscopic, petrographic, and metallographic microscopes. Examination using a stereomicroscope is outlined in the Visual and Stereomicroscopic Examination Section. Examination of fire-damaged concrete is outlined in Table 3; using a polarizing microscope in the Polarizing Microscope Examination Section and Table 4; and using a metallographic microscope in the Metallographic Microscope Examination Section. During each kind of study, the petrographer should note specific examinations to be made in detail, later, and may recognize the need to reexamine specimens. Observations possible using different kinds of microscopes are shown in Table 5; properties of some relevant compounds are listed in Table 6. A broad overview of possible locations where alkali-silica gel may be present can be obtained using the uranyl-acetate method outlined in Annex A1.

8.3 *Photography*<sup>2</sup>—It is frequently desirable to maintain a photographic record as illustrations for the report and for future reference. Photographs which may be useful include:

8.3.1 Overlapping close-ups of a core or cores, laid out in sequence, including a dimensional scale. It is convenient to use a view camera at a constant distance using the same lighting in all photographs.

8.3.2 Photographs and photomacrographs of features of

interest on a core surface, a ground surface, or an old or new crack surface, at magnifications of 0.7× to 3× are particularly useful if the specimen will be destroyed during subsequent studies. A camera equipped with a long extension bellows and lenses of focal length from 6.5 to 2.9 in. (165 to 74 mm), and a ground-glass back, is desirable.

8.3.3 Photomacrographs at magnifications of at least 3× to 10×, and sometimes at higher magnifications, may be used to illustrate reaction products or growths on the specimen or significant crack patterns. Black and white photographs such as 4 by 5 in. (102 by 127 mm) taken at 1× magnification on fine-grained film with a long gray scale, convey a great deal of information and may be more useful than a photograph taken at a higher magnification on smaller format. As a general guide, the subject should be illuminated by oblique lighting at a low angle if the relief on the subject surface is low, and at a higher angle if the relief is high. The object of the oblique illumination is to reveal detail and contrast without obscuring features by long shadows. Details can also be enhanced by the use of colored filters such as Wratten G or other yellow filters. Cross lighting from two sides is sometimes successful in revealing detail and obliterating confusing shadows.

8.3.4 Photomicrographs of thin sections, immersion mounts, and polished sections at magnifications from 2× to an upper limit appropriate to the subject and dependent on the quality of the equipment and the skill of the photographer may be used to illustrate textures, compounds, and crack patterns.

8.3.5 The range of photographic techniques used should be adequate to provide a choice useful for illustrations for the report and for the record. The subjects may usually be selected during the visual and stereomicroscopic examinations.

## 9. Specimen Preparation

### 9.1 Preparation for Visual and Stereomicroscope Examination:

9.1.1 Diamond-drilled cores, formed or finished surfaces, freshly broken surfaces, or old crack surfaces should be examined in the condition received. It is sometimes helpful to have drilled surfaces and formed and finished surfaces wetted to increase contrast.

9.1.2 Diamond saw cuts should be oriented with relation to significant features of the concrete, either normal to the bedding directions in conventional concrete, or normal to a formed or finished surface, or to a crack or crack system, in order to reveal the structure and fabric of the concrete and the extent of alteration outward from the crack.

9.1.3 It is useful to prepare at least one sawed surface by grinding it with progressively finer abrasives (as described in Practice C 457) until a smooth matte finish is achieved and to select areas on the matching opposing surface for preparation of thin sections and specimens for optical, chemical, X-ray diffraction, or other examinations.

9.1.4 Specimens obtained by diamond drilling are not ordinarily damaged in the process; however, weak concrete damaged by chemical attack, an alkali - aggregate reaction, freezing and thawing, or several of these, will give poor core recovery with many fractures if it is drilled with a 2 1/8-in. or 54-mm bit and barrel while it will give essentially complete

recovery if drilled with a 6-in. (152-mm) diameter bit and barrel. This difference is particularly important in petrographic examinations made during condition surveys of old structures. Weakened concrete may also break during sawing. The removal and preparation of specimens for laboratory studies usually involves the application of force and sometimes the application of heat to the specimen.

9.1.5 The effects of force can be minimized during specimen preparation by using thicker slices and making only one cut parallel to the long axis of a core section. Fractured or fragile concrete can be supported by partially or completely encasing it in plaster, epoxy resin, or other reinforcing media before sawing.

9.1.6 Heat used while impregnating concrete with thermoplastic wax or resin will cause cracking if the concrete is heated while it is wet, and will alter the optical properties of some compounds, such as ettringite. Artifacts may therefore be produced and compound identification made difficult. These artifacts may be mistaken as original features. Care must therefore be used in evaluating a particular feature and indexing it as original in the specimen, or produced during the removal of the specimen from the structure or during laboratory processing.

9.1.7 When alkali - carbonate reactions are suspected and rims around crushed carbonate aggregate are seen, it is useful to etch a sawed or ground surface in 6 N or weaker hydrochloric acid to see if peripheral rims on coarse aggregate particles are more or less susceptible to etching than the interior of the particle. Since etching destroys the surface, this step should not be taken until all other examinations of the surface have been completed. Etching the ground surface for 30 s in 10 % hydrochloric acid is an appropriate procedure.

9.2 *Preparation of Immersion Mounts*—Secondary products of nondeleterious and deleterious chemical processes may be observed during the stereomicroscopic examination. Finely pointed probes may be used to excavate and transfer them to microscope slides. The material is then covered with a cover glass, immersed in a refractive index liquid, and examined using a polarizing microscope to determine the optical characteristics of the product such as index of refraction, birefringence, and optical character.

9.3 *Preparation of Thin Sections*—The detailed description of thin-section preparation is beyond the scope of this practice. There are many laboratories that provide this service if in-house facilities are not available. The procedure includes slicing the concrete into 1/16-in. (2-mm) thick wafers if the concrete is strong and thicker slices if it is not. It may be necessary to impregnate the concrete with a resin before slicing to prevent disintegration. Diluted flexibilized epoxy resins or thermoplastic resins have been used successfully. The thin concrete slices are then mounted on glass slides with either flexibilized epoxy, Canada balsam, or Lakeside 70, and ground on laps using progressively finer abrasive until a thickness of 30 μm or less is obtained; thickness not greater than 20 μm is required for detailed examination of the paste in transmitted light. It is usually necessary to check the thickness of the section by the use of birefringent colors of common minerals in the aggregate, such as quartz or feldspar, during the final grinding stages. A cover glass is placed on the cleaned, prepared section and secured with

Canada balsam or other media.

9.3.1 Semiautomatic thin-section making machines are available which prepare the original surface of the blank for mounting, trim the excess thickness of the blank after mounting, and grind the section to 50 to 100 μm, leaving little thickness to be removed by hand lapping.

9.4 *Preparation for Examination with the Metallographic Microscope:*

9.4.1 The preparation of specimens for examination with the metallographic microscope is described in Methods E 3.

9.4.2 The procedures described here are intended as guides only. Methods should be used that are appropriate to the varieties of microscopical techniques to be employed, and to specimen condition and composition. It is impossible to provide instructions to suit every possible situation.

9.5 The preparation of the specimens for the uranyl-acetate method given in Annex A1 is provided therein.

## 10. Visual and Stereomicroscope Examination

10.1 If there is more than one specimen, arrange them in logical order to represent: position in the structure and differences in materials, proportions, and exposure, or combinations of these. Photographs and sketches of significant features should be made before specimens are altered. Tables 1 and 2 list some features to be observed during visual and stereomicroscopic examination.

10.2 *Concrete from Constructions (Core Log):*

10.2.1 Fit fragmented cores together and determine if any pieces are missing. Measure cores to verify field data and prepare a diagrammatic log of each core, if necessary. The log should be made to a scale to show relevant features. The log may be used to show fresh and old fractures, reacted particles, reaction products, changes in size or type of coarse and fine aggregates, distribution of coarse aggregate, honeycomb, segregation of components, cold joints or lift or course boundaries, location and direction of steel or other embedded items, changes in color of paste, and other significant features. Notes should be taken and suitably referenced in the log. If cores have been suitably packed so as to preserve the moisture content as drilled (which may or may not be the actual in-place moisture content) specimens should be carefully handled to preserve the moisture content and avoid breakage. Tests that may be made of specimens in the as-received moisture condition include compressive strength, static or dynamic modulus of elasticity, air content by high-pressure meter, permeability, and freezing and thawing.

10.2.2 During the visual examination, general comparison of cores representing different conditions, materials, and extent and nature of deterioration should be made and recorded. The specimens should be sorted into comparable groups based on condition or location of regions designated for more detailed examination. In some cases, during the visual examination or the stereomicroscope examination, one or more specimens from each group may be selected for more detailed studies. If a reduced number of specimens is to be selected for detailed examination, the selection should be made after careful visual and stereomicroscope examinations have been used to define one or more factors that characterize each group.

10.3 *Specimens from Natural Exposures*—These specimens should be laid out in logical order, either by materials,

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10.2 *Concrete from Constructions (Core Log)*:

10.2.1 Fit fragmented cores together and determine if any pieces are missing. Measure cores to verify field data and prepare a diagrammatic log of each core, if necessary. The log should be made to a scale to show relevant features. The log may be used to show fresh and old fractures, reacted particles, reaction products, changes in size or type of coarse and fine aggregates, distribution of coarse aggregate, honeycomb, segregation of components, cold joints or lift or course boundaries, location and direction of steel or other embedded items, changes in color of paste, and other significant features. Notes should be taken and suitably referenced in the log. If cores have been suitably packed so as to preserve the moisture content as drilled (which may or may not be the actual in-place moisture content) specimens should be carefully handled to preserve the moisture content and avoid breakage. Tests that may be made of specimens in the as-received moisture condition include compressive strength, static or dynamic modulus of elasticity, air content by high-pressure meter, permeability, and freezing and thawing.

10.2.2 During the visual examination, general comparison of cores representing different conditions, materials, and extent and nature of deterioration should be made and recorded. The specimens should be sorted into comparable groups based on condition or location of regions designated for more detailed examination. In some cases, during the visual examination or the stereomicroscope examination, one or more specimens from each group may be selected for more detailed studies. If a reduced number of specimens is to be selected for detailed examination, the selection should be made after careful visual and stereomicroscope examinations have been used to define one or more factors that characterize each group.

10.3 *Specimens from Natural Exposures*—These specimens should be laid out in logical order, either by materials,

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proportions, age, or combinations of these, and compared with respect to composition and condition. Significant features for more detailed examination should be marked and noted. Table 3 shows features of fire-damaged concrete.

#### 10.4 Concrete Products:

10.4.1 The samples may consist of complete units when they have relatively small dimensions (such as tile, block, or brick), or portions removed from units by coring or sawing. The samples should be organized and oriented in a manner appropriate to the objectives of the examination. So, for example, portions taken from a single product unit should be grouped together, and each sample should be described by visual observation and measurement of dimensions in relation to markings or labels. The following information should be recorded or clearly marked on the samples: location in the product unit (top, side, end, interior, etc.); vertical and horizontal directions as cast or molded; outer and inner ends; position in the product unit during curing period; position of the sample relative to highly stressed portions of the concrete adjacent to tendon anchorages; and location in the unit as put in place in construction, etc. Sketches or photographs of pertinent features should be recorded to show such conditions as cracking, staining, chemical deposits, presence of foreign matter, segregation, surface defects, and the like.

10.4.2 Procedures such as those set forth in 10.1 and 10.2 are applicable in the examination of concrete products. The investigation may be directed toward features resulting from the specific manufacturing operations involved, such as mixing, molding, demolding, curing, and any prestressing procedures. For example, lack of uniformity might originate in incomplete mixing, improper consolidation during molding, or incomplete or nonuniform curing. Surface defects and poor appearance may result from improper application of form-release agents, possibly giving rise to inhibition of hydration of the cement in a near-surface zone. Features like these can be detected and described by visual and microscopical examination of formed or molded surfaces, sawed and lapped surfaces, or fracture surfaces across the sample.

10.5 *Laboratory Specimens*—These should be laid out in logical order and compared with respect to composition and condition, and to features influenced or expected to be influenced by the test process.

10.6 *Stereomicroscope Examination*—The stereomicroscope examination reveals additional details at magnifications from 5× to 150× (see Tables 1 and 2). The stereomicroscope examination and visual examination are often carried on alternately. Examinations at low magnifications reveal characteristics of formed, finished, deteriorated, broken, sawed, or ground surfaces. Old cracks may be opened and their surfaces examined to detect reaction products and secondary alteration. Old cracks are frequently of different color than the mass of the concrete, and appear blurred by secondary deposits.

10.6.1 Freshly broken surfaces may follow structural weaknesses and reveal significant features that have not been masked by secondary alteration.

10.6.2 Sawed and ground surfaces are examined to detect fine cracks and trace networks of cracks that are not perceptible on drilled or sawed surfaces, to examine filling

and partial filling in voids and cracks, and to detect rims on aggregate particles that may indicate chemical reactions between the cement and the aggregate. Rims on gravel or natural sand should be presumed to be a result of weathering in the deposit, unless samples of aggregates used are available and show that rims were not present before the aggregate was used in the concrete. Rims produced in the concrete on particles of sand and gravel are absent or relatively thin and faint at locations where the particle is in contact with an air void. Rims produced by weathering may mask rims produced by alkali - silica reaction. Rimmed crushed stone in concrete usually indicates alteration in the concrete, as alkali - silica reaction or alkali - carbonate reaction (12, 20, 21). Pale rims in mortar bordering coarse aggregate (7, 19) and pale areas in the mortar may be gel-soaked paste (6) or highly carbonated paste adjoining carbonate aggregate that has undergone an alkali - carbonate reaction. The method in Annex A1 is helpful in locating deposits of alkali-silica gel.

10.6.3 Deposits in old cracks may include calcite or ettringite or calcium hydroxide, or combinations of these compounds, or more unusual crystalline substances, but sometimes include sand, silt, clay, and freshwater or salt-water organisms. Such organisms may provide evidence that a part of a structure that is not ordinarily submerged has been submerged.

10.6.4 Quantitative determination of constituents of concrete can be made by microscopical point-count or linear-traverse procedures in general accord with the requirements of Practice C 457. A total analysis may include the proportional amounts of coarse aggregate, fine aggregate, cementitious matrix, and air voids. The differentiation of coarse and fine aggregate is not accurate unless the two fractions are distinguishable lithologically, since the maximum dimension of the particles usually is not intersected by the prepared surface. Likewise, the proportion of individual rock types can be determined quantitatively by these methods, such as, the content of unsound or reactive constituents in the coarse or fine aggregate or both; proportions of lightweight and normal weight aggregate; and the abundance of granular contaminating substances.

## 11. Polarizing Microscope Examination

11.1 *Characteristics of Good Concrete Thin Sections*—Features that characterize good, thin sections of concrete or mortar are as follows: the mounted lower surface is free of abrasive and has been ground to a smooth matte finish ordinarily obtained by finishing the surface with optical alumina or equivalent; the lower surface of the blank is entire; air voids have complete peripheries; there is a sharp boundary between the void and surrounding mortar; deposits in voids are preserved; and aggregate that has been observed under the stereomicroscope to be entire when the blank was selected has remained entire and microfractures have not been produced in sectioning. It is very difficult to avoid production of microfractures when the aggregate is principally quartzite and quartz, if it is desired to reduce the section to a thickness of 30 μm so that the brightest birefringence color of quartz is first-order white. Some cracks will be formed and some loss of paste which is softer than quartz aggregate will occur.

11.1.1 The desirable thickness of the thin sections ranges

from about 40 to 20  $\mu\text{m}$ . Thinner sections may be required for detailed examination of the cement paste matrix. It is sometimes necessary to give up normal thickness and use a thicker section to preserve fragile deposits in voids such as alkali - silica gel, ettringite, calcium hydroxide, calcite, aragonite, or thaumasite. When it is desired to preserve the contacts between mortar and coarse aggregate, it may be preferable to leave the section thicker than normal with the highest birefringence color of the quartz pale first-order yellow. In normal concrete, the material bordering the aggregate consists of amorphous cement gel, with calcium hydroxide crystals quite evenly distributed but with slight concentrations along the undersides of coarse aggregate and fine aggregate, if the section is cut parallel to the placement direction. In cases of alkali - silica reaction, there is usually a zone depleted in or free of calcium hydroxide surrounding reacted aggregate particles; sometimes gel is present surrounding the aggregate or soaking the paste so that it is completely dark with crossed polars and brown grading hazily to more normal paste in plane polarized light. It is fairly easy to recognize gel-soaked paste; it is possible with experience to recognize depleted calcium hydroxide around a reacted aggregate particle, so long as the fine aggregate or any crusher dust present does not contain much colorless mica. Thin flakes of muscovite can be confused with thin tablets of calcium hydroxide when both are viewed on edge; they can be distinguished if the higher index of the mica is observed.

11.1.2 It is sometimes preferable to lose part of the section or crack the quartz in order to distinguish among quartz (birefringence 0.009), calcium hydroxide (birefringence 0.027), and calcite (birefringence 0.172). When the three materials occur together, the quartz particles will be shaped like sand grains or rock fragments, while calcium hydroxide will be present either as tablets tangential to aggregate or as poikilitic crystals in the paste enclosing residual cement grains or areas of gel. The birefringence of calcium hydroxide is three times that of quartz; the birefringence of calcite is 6.4 times that of calcium hydroxide; and birefringence of aragonite is 5.7 times that of calcium hydroxide. While aragonite is fairly uncommon in concrete, calcite is common as a product of carbonation and as an ingredient of aggregate. At normal thickness of 30  $\mu\text{m}$ , the highest birefringence of calcium hydroxide viewed parallel to the cleavage is bright first-order yellow with an occasional orange or first-order red area. Calcite in the same section has high-order white birefringence except in very minute grains in which it is very thin, approximately rhombic in shape, and the birefringence color is first-order white. The birefringence relations discussed in this paragraph are the basis that makes possible much of the interpretation of concrete thin sections.

11.2 *Choice of Areas for Thin Sections of Concrete*—Areas from which thin sections are to be prepared usually should be chosen after examining the sawed or sawed and ground surface with a stereomicroscope. Selection of the area may depend on the features to which the examination is directed, or the choice may be affected in the case of highly deteriorated concrete by the desire to choose a volume strong enough to endure sectioning even after impregnation. In that second case, an area of mortar with coarse aggregate at the corners or along the sides may be the successful choice. If it is

desired to examine the aggregate to compare it with aggregate of a known source to establish whether the two probably came from the same deposit or quarry, coarse aggregate should be chosen in sufficient number to cover the range of varieties present and their physical condition. If the concrete has been subjected to alkali - silica or alkali - carbonate reactions, areas selected for sectioning should include coarse aggregate with cracks inside the periphery but surrounding the center and cracks in the middle of the particle which narrow toward the border which may or may not contain gel in the case of alkali - silica reaction. Reaction-rimmed particles may be chosen for sectioning in the case of either kind of reaction, but it is ordinarily a futile effort; the rims distinct to the naked eye or the low power of the stereomicroscope may not be visible in thin section.

11.2.1 Features visible in the examination of thin sections of concrete are shown in Table 4.

## 12. Metallographic Microscope Examination (22)

12.1 There are two advantages of examining concrete with a metallographic microscope: only one prepared surface is required; the polished surface enhances and allows etching residual cement. When the aim of the petrographic examination is particularly concerned with the composition and characteristics of the cement, polished sections that are produced by grinding and polishing a sawed piece of concrete are particularly useful in detection of unusual amounts of free CaO or MgO or their hydration products and other cement constituents. Although the coarser relict cement grains may be far from a representative sample, they can reveal characteristics that, if correlated with the behavior and history of the concrete, can be extremely helpful in interpreting its behavior. Table 5 lists constituents of concrete visible, after preparation, using different microscopes. Table 6 is a list of substances reported as occurring in concrete.

## 13. Report

13.1 The report of the examination should include the following:

13.1.1 Location and orientation of the samples in the construction or products or type of specimen,

13.1.2 History of the samples insofar as is available,

13.1.3 Physical and chemical tests made on the samples, with their results,

13.1.4 Description of the samples and a report on mixture proportions, if available or if estimated, workmanship, construction practice, and original quality of the concrete in the constructions, insofar as such information is available, and

13.1.5 Interpretation, insofar as possible, of the nature of the materials and the chemical and physical events that have led to the success or distress of the concrete.

## 14. Keywords

14.1 aggregates; air voids; alkali-silica reaction; analysis; deterioration; examination; fire effects; hardened concrete; microscopy; paste; petrographic; secondary deposits

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## Standard Guide for Petrographic Examination of Aggregates for Concrete<sup>1</sup>

This standard is issued under the fixed designation C 295; 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 reappraisal. A superscript epsilon ( $\epsilon$ ) indicates an editorial change since the last revision or reappraisal.

*This standard has been approved for use by agencies of the Department of Defense. Consult the DoD Index of Specifications and Standards for the specific year of issue which has been adopted by the Department of Defense.*

### 1. Scope

1.1 This guide<sup>2</sup> outlines procedures for the petrographic examination of samples representative of materials proposed for use as aggregates in concrete.

1.2 This guide outlines the extent to which petrographic techniques should be used, the selection of properties that should be looked for, and the manner in which such techniques may be employed in the examination of samples of aggregates for concrete.

1.3 This guide does not attempt to describe the techniques of petrographic work since it is assumed that the guide will be used by persons who are qualified by education and experience to employ such techniques for the recognition of the characteristic properties of rocks and minerals and to describe and classify the constituents of an aggregate sample.

1.4 The rock and mineral names given in Descriptive Nomenclature C 294 should be used insofar as they are appropriate in reports prepared according to this guide.

1.5 *This standard does not purport to address the safety problems 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.*

1.6 The values stated in SI units are to be regarded as the standard.

### 2. Referenced Documents

#### 2.1 ASTM Standards:

- C 33 Specification for Concrete Aggregates<sup>3</sup>
- C 117 Test Method for Materials Finer than 75- $\mu$ m (No. 200) Sieve in Mineral Aggregates by Washing<sup>3</sup>
- C 136 Test Method for Sieve Analysis of Fine and Coarse Aggregates<sup>3</sup>
- C 294 Descriptive Nomenclature of Constituents of Natural Mineral Aggregates<sup>3</sup>
- C 702 Practice for Reducing Field Samples of Aggregate to Testing Size<sup>3</sup>
- D 75 Practice for Sampling Aggregates<sup>4</sup>

<sup>1</sup> This guide is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.65 on Petrography of Concrete and Aggregates.

Current edition approved Oct. 26, 1990. Published December 1990. Originally published as C 295 - 54. Last previous edition C 295 - 85.

<sup>2</sup> This guide is based on the "Method of Petrographic Examination of Aggregates for Concrete," by Katharine Mather and Bryant Mather, *Proceedings, ASTM, ASTEA, Vol 50, 1950, pp. 1288-1312.*

<sup>3</sup> *Annual Book of ASTM Standards, Vol 04.02.*

<sup>4</sup> *Annual Book of ASTM Standards, Vol 04.03.*

E 11 Specification for Wire-Cloth Sieves for Testing Purposes<sup>5</sup>

E 883 Guide for Metallographic Photomicrography<sup>6</sup>

### 3. Summary of Method

3.1 The specific procedures employed in the petrographic examination of any sample will depend to a large extent on the purpose of the examination and the nature of the sample. In most cases the examination will require the use of optical microscopy. Complete petrographic examinations for particular purposes and to investigate particular problems may require examination of aggregates or of selected constituents by means of additional procedures, such as X-ray diffraction analysis, differential thermal analysis, infrared spectroscopy, or others; in some instances, such procedures are more rapid and more definitive than are microscopical methods.

3.2 Identification of the constituents of a sample is usually a necessary step towards recognition of the properties that may be expected to influence the behavior of the material in its intended use, but identification is not an end in itself. The value of any petrographic examination will depend to a large extent on the representativeness of the samples examined, the completeness and accuracy of the information provided to the petrographer concerning the source and proposed use of the material, and the petrographer's ability to correlate these data with the findings of the examination.

3.3 It is assumed that the examination will be made by persons qualified by education and experience to operate the equipment used and to record and interpret the results obtained. In some cases, the petrographer will have had experience adequate to provide detailed interpretation of the materials' performance with respect to engineering and other consequences of the observations. In others, the interpretation will be made in part by engineers, scientists, or others qualified to relate the observations to the questions to be answered.

### 4. Significance and Use

4.1 Petrographic examinations are made for the following purposes:

4.1.1 To determine the physical and chemical characteristics of the material that may be observed by petrographic methods and that have a bearing on the performance of the material in its intended use.

<sup>5</sup> *Annual Book of ASTM Standards, Vol 14.02.*

<sup>6</sup> *Annual Book of ASTM Standards, Vol 03.01.*

4.1.2 To describe and classify the constituents of the sample,

4.1.3 To determine the relative amounts of the constituents of the sample, which are essential for proper evaluation of the sample, when the constituents differ significantly in properties that have a bearing on the performance of the material in its intended use, and

4.1.4 To compare samples of aggregate from new sources with samples of aggregate from one or more sources, for which test data or performance records are available.

4.2 This guide may be used by a petrographer employed directly by those for whom the examination is made. The employer should tell the petrographer, in as much detail as necessary, the purposes and objectives of the examination, the kind of information needed, and the extent of examination desired. Pertinent background information, including results of prior testing, should be made available. The petrographer's advice and judgment should be sought regarding the extent of the examination.

4.3 This guide may form the basis for establishing arrangements between a purchaser of consulting petrographic service and the petrographer. In such a case, the purchaser and the consultant should together determine the kind, extent, and objectives of the examination and analyses to be made, and should record their agreement in writing. The agreement may stipulate specific determinations to be made, observations to be reported, funds to be obligated, or a combination of these or other conditions.

4.4 Petrographic examination of aggregate considered for use in hydraulic cement concrete is one aspect of the evaluation of aggregate, but petrographic examination is also used for many other purposes. Petrographic examinations provide identification of types and varieties of rocks present in potential aggregates. However, as noted above, identification of every rock and mineral present in an aggregate source is not required.

4.5 The petrographic examination should establish whether the aggregate contains chemically unstable minerals such as soluble sulfates, unstable sulfides that may form sulfuric acid or create distress in concrete exposed to high temperatures during service, or volumetrically unstable materials such as smectites (also known as the montmorillonite-saponite group of minerals or swelling clays). Specifications may limit the quartz content of aggregates for use in concrete that may be subject to high temperature (purposefully or accidentally) because of the conversion to beta-quartz at 573°C (1063°F), with accompanying volume increase.

4.6 Petrographic examination should identify the portion of each coarse aggregate that is composed of weathered or otherwise altered particles and the extent of that weathering or alteration, whether it is severe, moderate, or slight, and should determine the proportion of each rock type in each condition. If the concrete in which the aggregate may be used will be exposed to freezing and thawing in a critically saturated condition, finely porous and highly weathered or otherwise altered rocks should be discriminated because they will be especially susceptible to damage by freezing and thawing and will cause the aggregate portion of the concrete to fail in freezing and thawing. This will ultimately destroy the concrete because such aggregates cannot be protected by adequately air-entrained sound mortar. Finely porous aggregates

near the concrete surface are also likely to form popouts, which are blemishes on pavements and walls.

4.7 Petrographic examinations may also be used to determine the proportions of cubic, spherical, ellipsoidal, pyramidal, tabular, flat, and elongated particles in an aggregate sample or samples. Flat, elongated, and thin chip-like particles in aggregate increase the mixing water requirement and decrease concrete strength.

4.8 Petrographic examination should identify and call attention to potentially alkali-silica reactive and alkali-carbonate reactive constituents, determine such constituents quantitatively, and recommend additional tests to confirm or refute the presence in significant amounts of aggregate constituents capable of alkali reaction in concrete. This guide is referred to in the Appendix of Specification C 33. Alkali-silica reactive constituents found in aggregates include: opal, cristobalite, tridymite, siliceous and some intermediate volcanic glass, chert, glassy to cryptocrystalline acid volcanic rocks, synthetic siliceous glasses, some argillites, phyllites, metamorphic graywackes, rocks containing highly metamorphic quartz such as graywackes, phyllites, schists, gneisses, gneissic granites, vein quartz, quartzite, and sandstone. Criteria are available in the mineralogic literature for identifying the minerals in the list above by optical properties or X-ray diffraction, or both, and in petrographic and petrologic literature for identifying the rocks in the list by mineral composition and texture in thin section, sometimes assisted by X-ray diffraction for mineral composition. Potentially deleterious alkali-carbonate reactive rocks are usually calcareous dolomites or dolomitic limestones with clayey insoluble residues. Some dolomites essentially free of clay and some very fine-grained limestones free of clay and with minor insoluble residue, mostly quartz, are also capable of some alkali-carbonate reactions, however, such reactions have not yet been found deleterious.

4.9 Petrographic examination may be directed specifically at the possible presence of contaminants in aggregates, such as synthetic glass, cinders, clinker, or coal ash, magnesium oxide, calcium oxide, or both, soil, hydrocarbons, chemicals that may affect the setting behavior of concrete or the properties of the aggregate, animal excrement, plants or rotten vegetation, and any other contaminant that may prove undesirable in concrete.

4.10 These objectives for which this guide was prepared will have been attained if those involved with the evaluation of aggregate materials for use in concrete construction have reasonable assurance that the petrographic examination results wherever and whenever obtained, as corrected, may confidently be compared.

## 5. Apparatus and Supplies

5.1 The apparatus and supplies listed below comprise a selection that will permit the use of all of the procedures described in this guide. All specific items listed have been used, in connection with the performance of petrographic examinations, by the procedures described herein; it is not, however, intended to imply that other items cannot be substituted to serve similar functions. Whenever possible the selection of particular apparatus and supplies should be left to the judgment of the petrographer who is to perform the work so that the items obtained will be those with the use of

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which he has the greatest experience and familiarity. The minimum equipment regarded as essential to the making of petrographic examinations of aggregate samples are those items, or equivalent apparatus or supplies that will serve the same purpose, that are indicated by asterisks in the lists given below.

#### 5.1.1 Apparatus and Supplies for Preparation of Specimens:

5.1.1.1 *Rock-Cutting Saw*,\* preferably with 350-mm or larger diamond blade, and automatic feed.

5.1.1.2 *Horizontal Grinding Wheel*,\* preferably 400 mm in diameter.

5.1.1.3 *Polishing Wheel*, preferably 200 to 300 mm in diameter.

5.1.1.4 *Abrasives*<sup>7</sup>—\*Silicon carbide grit No. 100 (122  $\mu$ m), 220 (63  $\mu$ m), 320 (31  $\mu$ m), 600 (16  $\mu$ m), and 800 (12  $\mu$ m); alumina M-305 (5  $\mu$ m).

5.1.1.5 *Geologist's Pick or Hammer*.

5.1.1.6 *Microscope Slides*,\* clear, noncorrosive, 25 by 45 mm in size.

5.1.1.7 *Canada Balsam*,\* neutral, in xylene, or suitable low-viscosity epoxies, or Lakeside 70.

5.1.1.8 *Xylene*.\*

5.1.1.9 *Mounting Medium*,\* suitable for mounting rock slices for thin sections.

5.1.1.10 *Laboratory Oven*.\*

5.1.1.11 *Plate-Glass Squares*,\* about 300 mm on an edge for thin-section grinding.

5.1.1.12 *Sample Splitter* with pans.\*

5.1.1.13 *Micro Cover Glasses*,\* noncorrosive, square, 12 to 18 mm, 25 mm, etc.

5.1.1.14 *Plattner Mortar*.

#### 5.1.2 Apparatus and Supplies for Examination of Specimens:

5.1.2.1 *Polarizing Microscope*,\* with mechanical stage; low-, medium-, high-power objectives, and objective-centering devices; eyepieces of various powers; full- and quarter-wave compensators; quartz wedge.

5.1.2.2 *Microscope Lamps*\* (preferably including a sodium arc lamp).

5.1.2.3 *Stereoscopic Microscope*,\* with objectives and oculars to give final magnifications from about 6 $\times$  to about 150 $\times$ .

5.1.2.4 *Magnet*,\* preferably Alnico, or an electromagnet.

5.1.2.5 *Needleholder and Points*.\*

5.1.2.6 *Dropping Bottle*, 60-mL capacity.

5.1.2.7 *Petri Culture Dishes*.

5.1.2.8 *Forceps*, smooth, straightpointed.

5.1.2.9 *Lens Paper*.\*

5.1.2.10 *Immersion Media*,\*  $n = 1.410$  to  $n = 1.785$  in steps of 0.005.

5.1.2.11 *Counter*.

5.1.2.12 *Photomicrographic Camera* and accessories.

5.2 The items under Apparatus and Supplies include those used to make thin sections. Semiautomatic thin section machines are now available, and there are several thin-section makers who advertise in *Geotimes*, the *American*

*Mineralogist*, and other mineralogical or geological journals. Laboratories may find it reasonable to buy a thin-section machine or use a commercial thin-section maker. Remotely located laboratories have more need to be able to make their own thin sections.

5.3 It is necessary that facilities be available to the petrographer to check the index of refraction of the immersion media. If accurate identification of materials is to be attempted, as for example the differentiation of quartz and chalcedony, or the differentiation of basic from intermediate volcanic glass, the indices of refraction of the media need to be known with accuracy. Media will not be stable for very long periods of time and are subject to considerable variation due to temperature change. In laboratories not provided with close temperature control, it is often necessary to recalibrate immersion media several times during the course of a single day when accurate identifications are required. The equipment needed for checking immersion media consists of an Abbé refractometer. The refractometer should be equipped with compensating prisms to read indices for sodium light from white light, or it should be used with a sodium arc lamp.

5.4 A laboratory that undertakes any considerable amount of petrographic work should be provided with facilities to make photomicrographic records of such features as cannot adequately be described in words. Photomicrographs can be taken using standard microscope lamps for illumination; however, it is recommended that whenever possible a zirconium arc lamp be provided for this purpose. For illustrations of typical apparatus, reference may be made to the paper by Mather and Mather,<sup>2</sup> and manufacturers of microscopes equipped with cameras and photomicrographic equipment may be consulted. Much useful guidance regarding photomicrography, especially using reflected light, is found in Guide E 883.

## 6. Sampling

6.1 Samples for petrographic examination should be taken by or under the direct supervision of a geologist familiar with the requirements for random sampling of aggregates for concrete and in general following the requirements of Practice D 75. The exact location from which the sample was taken, the geology of the site, and other pertinent data should be submitted with the sample. The amount of material actually studied in the petrographic examination will be determined by the nature of the examination to be made and the nature of the material to be examined, as discussed below.

6.1.1 *Undeveloped quarries* should be sampled by means of cores drilled through the entire depth expected to be exploited. Drilling of such cores should be in a direction that is essentially perpendicular to the dominant structural feature of the rock. Massive material may be sampled by "NX" (53-mm (2 $\frac{1}{8}$ -in.) diameter) cores. Thinly bedded or complex material should be represented by cores not less than 100 mm (4 in.) in diameter. There should be an adequate number of cores to cover the limits of the deposit proposed for the project. The entire footage of the recovered core should be included in the sample and accurate data given as to elevations, depths, and core losses.

6.1.2 *Operating quarries and operating sand and gravel*

<sup>7</sup> The values given in micrometres are the approximate average grain size of commercial silicon carbide grit in the designated size classification.

**TABLE 1** Minimum Sizes for Samples from Undeveloped Sand and Gravel Deposits

Sieve Size	Quantity		
	kg	(lb)	Pieces
Larger than 150-mm (6-in.)	...	...	...
75 to 150-mm (3 to 6-in.)	...	...	300 <sup>A</sup>
37.5 to 75-mm (1½ to 3-in.)	180	(400)	...
19.0 to 37.5-mm (¾ to 1½-in.)	90	(200)	...
4.75 to 19.0-mm (No. 4 to ¾-in.)	45	(100)	...
Finer than 4.75-mm (No. 4) <sup>B</sup>	23	(50)	...

<sup>A</sup> Not less than one piece from each apparent type of rock.

<sup>B</sup> Fine aggregate.

deposits, in which stock piles of the material produced are available, should be represented by not less than 45 kg (100 lb) or 300 pieces, whichever is larger, of each size of material to be examined. Samples from stock piles should be composed of representative portions of larger samples collected with due consideration given to segregation in the piles.

6.1.3 *Exposed faces of nonproducing quarries*, where stock piles of processed material are not available, should be represented by not less than 2 kg (4 lb) from each distinctive stratum or bed, with no piece weighing less than 0.5 kg (1 lb), or by a drilled core as described above.

6.1.4 *Undeveloped sand and gravel deposits* should be sampled by means of test pits dug to the anticipated depth of future economic production. Samples should consist of not less than the quantities of material indicated in Table 1, selected so as to be representative of the deposits.

**NATURAL GRAVEL AND SAND**

**7. Procedure**

7.1 *Selection of Samples for Examination*—Samples of gravel and natural sand for petrographic examination should be dry sieved in accordance with Method C 136 to provide samples of each sieve size. In the case of sands an additional portion should then be tested in accordance with Test Method C 117, with the wash water being saved and removed by drying in order to provide a sample of the material passing the 75-µm (No. 200) sieve. The results of the sieve analysis of each sample made in accordance with Method C 136 should be provided to the petrographer making the examination and used in calculating results of the petrographic examination. Each sieve fraction should be examined separately, starting with the largest size available. Rocks are more easily recognized in larger pieces; the breakdown of a heterogeneous type present in the larger sizes may have provided particles of several apparently different types in the smaller sizes. Some important and easily confused types may be recognizable using the stereoscopic microscope if they are first recognized and separated in the larger sizes, but may require examination using the petrographic microscope if they are first encountered in the smaller sizes.

7.2 The number of particles of each sieve fraction to be examined will be fixed by the required precision of determination of the less abundant constituents. Assuming that the field sampling and laboratory sampling procedures are accurate and reliable, the number of particles examined, identified, and counted in each sieve fraction will depend on the required accuracy of the estimate of constituents present in small quantities. The numbers given in this method are

minimal. They are based on experience and on statistical considerations.<sup>8</sup> It is believed that at least 150 particles of each sieve fraction should be identified and counted in order to obtain reliable results. Precise determinations of small quantities of an important constituent will require counts of larger numbers of particles. If the sample of a sieve fraction contains many more particles than need to be identified, the sample shall be reduced in accordance with one of the procedures in Practice C 702, so as to contain a proper number of particles for examination.

**8. Procedure for Examination of Natural Gravel**

8.1 *Coatings*—The pebbles should be examined to establish whether exterior coatings are present. If they are, it should be determined whether the coatings consist of materials likely to be deleterious in concrete (opal, gypsum, easily soluble salts, organic matter). It should also be determined qualitatively how firmly the coatings are bonded to the pebbles.

8.2 *Rock Types*—The sieve fraction should be sorted into rock types by visual examination. If all or most of the groups present are types easily identifiable in hand specimen by examination of a natural or broken surface, and by scratch and acid tests, no further identification may be needed. Fine-grained rocks that cannot be identified macroscopically or that may consist of or contain constituents known to be deleterious in concrete should be checked by examination with the stereoscopic microscope. If they cannot be identified by that means, they should be examined by means of the petrographic microscope. The amount of work done in identifying fine-grained rocks should be adapted to the information needed about the particular sample. Careful examination of one size of a sample, or study of information from previous examination of samples from the same source, will usually reveal the amount of additional detailed microscopical work required to obtain information adequate for the purpose. In some instances, petrographic methods other than microscopy, such as X-ray diffraction, may be required or might most rapidly serve to identify fine-grained rock materials.

8.3 *Condition*—The separated groups belonging to each rock type should be examined to determine whether a further separation by physical condition is necessary. If all of the particles of a rock type are in a comparable condition, that fact should be noted. More frequently, particles in several degrees of weathering will be found in a group. They should be sorted into categories based on condition and on the expectation of comparable behavior in concrete. The types of categories intended are: (1) fresh, dense; (2) moderately weathered; (3) very weathered; or (1) dense; (2) porous (or porous and friable). It usually is not practicable to recognize more than three conditions per rock type, and one or two may be sufficient. An important constituent present in larger quantities may sometimes require separation into four groups by condition. The conspicuous example is chert

<sup>8</sup> Simpson, G. G., and Rowe, A., *Quantitative Zoology*, McGraw-Hill Co., Inc., New York, NY, 1939, pp. 182-185.

Dryden, A. L., Jr., "Accuracy in Percentage Representation of Heavy Mineral Frequencies," *Proceedings*, U. S. Nat. Academy Sciences, Vol 17, No. 5, May 1931, pp. 233-238.

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when it is the major constituent of a gravel sample. It may be present as dense, unfractured chert; as vuggy chert; as porous chert; and as dense but highly fractured chert. The determination of which of these four conditions characterizes a particle may be expected to have an important influence on prediction of the behavior of the particle in concrete.

#### 8.4 Record:

8.4.1 Notes should be taken during the examination. Each rock type should be described; the relevant features may include the following:

8.4.1.1 Particle shape,

8.4.1.2 Particle surface, texture,

8.4.1.3 Grain size,

8.4.1.4 Internal structure, including observations of pore space, packing of grains, cementation of grains,

8.4.1.5 Color,

8.4.1.6 Mineral composition,

8.4.1.7 Significant heterogeneities,

8.4.1.8 General physical condition of the rock type in the sample,

8.4.1.9 Coatings or incrustations, and

8.4.1.10 Presence of constituents known to cause deleterious chemical reaction in concrete.

8.4.2 Particle counts should be recorded so that tables can be made for inclusion in the report. When the examination has been completed, the notes should contain enough information to permit the preparation of tables and descriptions. Tables should be prepared showing the composition and condition of the samples by sieve fractions, and the weighted average composition, based on the grading of the sample as received and on the distribution of constituents by sieve fractions. Descriptions of constituent groups should be prepared containing the relevant features among those enumerated in the preceding list.

### 9. Procedure for Examination of Natural Sand

9.1 The procedure for the examination of natural sand is similar to that for the examination of gravel, with the modifications necessitated by the differences in particle size.

9.1.1 *Sizes Coarser than 600  $\mu\text{m}$  (No. 30)*—Each sieve fraction present that is coarser than the 600- $\mu\text{m}$  (No. 30) sieve should be reduced in accordance with one of the procedures in Practice C 702 until a split or splits containing at least 150 particles are obtained. The reduced sample of each sieve fraction should be examined, and its constituents identified and counted, using the stereoscopic microscope. It is convenient to spread out the sample in a flat-bottom glass dish such as a Petri dish and manipulate the grains with a forceps and dissecting needle. The identification of grains in the coarser sand sizes is often easier when the grains are just submerged in water. The submergence lessens reflection from the outer surfaces and may show diagnostic features that cannot be seen when the grains are dry. There are exceptions to this generalization. Where identification is difficult, the examination includes examination of the natural surface (dry and wet), examination of a broken surface (dry and wet), and scratch and acid tests. Only after all of these steps have been taken and the grain is still unidentified should the petrographer resort to the petrographic microscope. Grains that cannot be identified using the stereoscopic microscope, or that are suspected of consisting of or con-

taining substances known to react deleteriously in concrete, should be set aside to be examined with the petrographic microscope. If the question of reaction with the alkalis (sodium and potassium) of portland cement paste is important in the examination of the sample, certain additions to the procedure are indicated. If the coarser sand sizes contain fine-grained, possibly glassy igneous rocks, several typical particles of each variety of such rocks should be selected for a more thorough examination. The petrographer should determine the presence or absence of glass by crushing typical grains and examining them in immersion media, using the petrographic microscope. In difficult or especially important cases, it may be necessary to break suspected grains and to make immersion mounts of part of the grain and a thin section of another part. Where the sand contains chert and the potential reactivity of the chert is an important consideration, a number of chert particles from the fractions retained on the 600- $\mu\text{m}$  (No. 30) sieve should be set aside for determinations of the index of refraction. Methylene blue staining techniques may be useful in identifying the presence of smectite.<sup>9</sup>

9.1.2 *Sizes Finer than 600  $\mu\text{m}$  (No. 30)*—The sieve fractions finer than the 600- $\mu\text{m}$  (No. 30) sieve should each be reduced in a sample splitter or by quartering to about 4 or 5 g. The volume will usually be less than a level teaspoonful. In some gradings the fractions retained on the 150- $\mu\text{m}$  (No. 100) and 75- $\mu\text{m}$  (No. 200) sieves may be present in such small amounts that reduction is unnecessary. These splits should be further reduced on a miniature sample splitter or by coning and quartering with a spatula on a clean sheet of paper. The examination may be conducted as in 9.1.1. As required, the petrographic microscope should be used. For this purpose, a representative portion of each reduced split should be mounted in immersion oil on a clean glass slide and covered with a clean cover glass. No entirely satisfactory method of reducing a split to a predetermined number of grains is known. The reduced split can be sampled by spreading it in a thin layer on glass or clean paper, dragging the end of a dissecting needle moistened in immersion oil through the sample and transferring the grains that adhere to the needle to a drop of immersion oil on a clean slide. If this is done carefully, a fairly representative sample will be obtained. If the dissecting needle is made of magnetized steel, a concentration of magnetic minerals may result. It is usually necessary to make several mounts of the 300- $\mu\text{m}$  (No. 50) and 150- $\mu\text{m}$  (No. 100) sieve fractions to obtain at least 150 grains of each. The index of the immersion oil should be selected to make the identification of the important constituents as easy and as definite as possible. The use of an immersion oil with an index of or just below the lower index of quartz (1.544) is recommended. The slide should be mounted on a petrographic microscope equipped with a mechanical stage. Several traverses should be made, and each grain that passes under the cross hair should be identified and counted. Care should be taken to move the slide on the north-south adjustment between traverses so that no grain will be counted twice. Each sieve fraction passing the

<sup>9</sup> Higgs, Nelson B., "Preliminary Studies of Methylene Blue Adsorption as a Method of Evaluating Degradable Smectite-Bearing Concrete Aggregate Sands," *Cement and Concrete Research*, Vol. 16, 1986, pp. 528-534.

600- $\mu$ m (No. 30) and retained on the 75- $\mu$ m (No. 200) sieve should be examined. Ordinarily, the material passing the 75- $\mu$ m (No. 200) sieve is mounted on a slide following the procedure described above, examined by means of the petrographic microscope, and its composition estimated. If an unusually large amount of this size is present, or if it contains constituents that may be expected to have an important effect on the suitability of the aggregate for the intended use, it should be counted. In this event, it is suggested that the fraction passing the 75- $\mu$ m (No. 200) sieve be washed over the 45- $\mu$ m (No. 325) sieve before being counted.

9.1.3 Grain thin sections using an epoxy as mounting medium also may be useful in classifying particles passing the 300- $\mu$ m (No. 50) sieve.

**DRILLED CORE, LEDGE ROCK, CRUSHED STONE, AND MANUFACTURED SAND**

**10. Procedure for Examination of Drilled Core**

10.1 Each core should be examined and a log prepared showing footage of core recovered, core loss and location; location and spacing of fractures and parting planes; lithologic type or types; alternation of types; physical condition and variations in condition; toughness, hardness<sup>10</sup>, coherence; obvious porosity; grain size, texture, variations in grain size and texture; type or types of breakage; and presence of constituents capable of deleterious reaction in concrete. If the size of the core permits, the probability that the rock will make aggregate of the required maximum size should be considered. If the surface of the core being examined is wetted, it is usually easier to recognize significant features and changes in lithology. Most of the information usually required can be obtained by careful visual examination, scratch and acid tests, and hitting the core with a hammer. In the case of fine-grained rocks, it may be necessary to examine parts of the core, using the stereoscopic microscope, or to prepare thin sections of selected portions. Some considerations and procedures are more applicable to particular rock types than to others. Ordinarily, the layered rocks considered for concrete aggregate will be limestone, and occasionally metamorphic rocks, such as phyllite, gneiss, or schist. One of the most important questions arising in the examination of limestone is that of the presence, type, and distribution of argillaceous impurities. Limestones that contain intercalated thin beds of soft shale may make suitable sources of aggregate if the shale is so distributed that it does not prevent manufacture of the required maximum size, and if the shale can be eliminated or reduced in processing. Where argillaceous impurities are present, it should be determined whether they actually consist of clay minerals or of other minerals in clay sizes. If they do consist of clay minerals, it should be established whether the clay minerals include swelling clays (that is, smectites, also known as the montmorillonite-saponite group of minerals).<sup>11</sup> X-ray diffraction analysis is especially valuable in identification and

quantitative determination of clay minerals. Methylene blue staining techniques are useful in identifying smectite. In the examination of fine-grained igneous rocks, particular attention should be directed to the nature of the ground-mass. This examination should include determination of the presence or absence of opal, chalcedony, natural glass, and swelling clays; if any of these are found the amount should be estimated; if natural glass is found the type should be determined.

**11. Procedure for Examination of Ledge Rock**

11.1 The procedure used in examination should be the same as for core samples to the extent that the spacing of samples and size of the individual pieces allow. If the sample consists of a relatively large quantity of broken stone produced by blasting, it is desirable to inspect the whole sample, estimate the relative abundance of rock types or varieties present, and sample each type before further processing. Subsequent procedure should be the same as given below for crushed stone.

**12. Procedure for Examination of Crushed Stone**

12.1 The procedure for examination of crushed stone should be similar to that for core, except that necessary quantitative data should be obtained by particle counts of the separated sieve fractions obtained as described in the section on Natural Gravel and Sand.

**13. Procedure for Examination of Manufactured Sand**

13.1 The examination procedure should be similar to that for natural sand, with particular emphasis on the amount and extent of fracturing and the amount and nature of rock dust developed by the milling operations. If a sample of the rock from which the sand was produced is available, examination of it will provide useful information.

**CALCULATION AND REPORT**

**14. Calculation**

14.1 Calculate the composition of each sieve fraction of a heterogeneous sample and the weighted average composition of the whole sample as follows:

14.1.1 Express the composition of each sieve fraction by summing the total number of particles of that fraction counted, and calculating each constituent in each condition as a percentage of the total amount (as number of particles in percent, in each sieve fraction). It is convenient to calculate and record the percentages to tenths at this stage. An example of these calculations is given in the upper half of Table 2.

14.1.2 Obtain the weight percent of the sieve fraction in the whole sample (individual percentages retained on consecutive sieves) from the grading of the sample as determined by Method C 136.

14.1.3 By multiplying the percentage of the constituent in the sieve fraction, determined as described above by the percentage of the sieve fraction in the whole sample, obtained as described above, calculate the percentage in the whole sample of that constituent in that size (weighted percentage of constituents in sieve fraction, Table 2). It is

<sup>10</sup> Woolf, D. O., "Methods for the Determination of Soft Pieces in Aggregate," *Proceedings, ASTM*, Vol 47, 1947, p. 967.

<sup>11</sup> Carroll, Dorothy, "Clay Minerals: A Guide to Their X-ray Identification," *The Geological Society of America*, Special Paper 126, 1970.

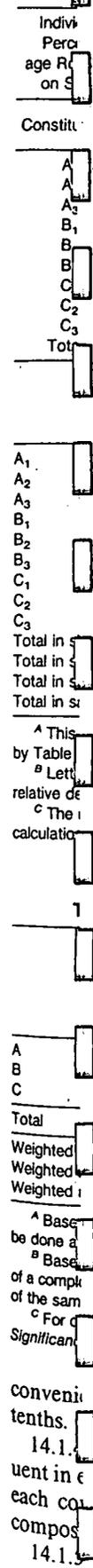


TABLE 2 Calculation of Results of Particle Counts<sup>A</sup>

Individual Percent- age Retained on Sieve	Composition of Fractions Retained on Sieves Shown Below							
	19.0-mm (¾-in.)		12.5-mm (½-in.)		9.5-mm (¾-in.)		4.75-mm (No. 4)	
	17.4		32.6		29.5		20.5	
Constituents <sup>B</sup>	Number of Particles	%	Number of Particles	%	Number of Particles	%	Number of Particles	%
A <sub>1</sub>	250	50.0	200	40.0	150	30.0	50	10.0
A <sub>2</sub>	50	10.0	100	20.0	125	25.0	100	20.0
A <sub>3</sub>	10	2.0	50	10.0	75	15.0	100	20.0
B <sub>1</sub>	107	21.4	70	14.0	62	12.4	32	6.4
B <sub>2</sub>	76	15.2	53	10.6	19	3.8	87	17.4
B <sub>3</sub>	...	...	20	4.0	43	8.6	96	19.2
C <sub>1</sub>	5	1.0	5	1.0	20	4.0	20	4.0
C <sub>2</sub>	2	0.4	2	0.4	6	1.2	10	2.0
C <sub>3</sub>	...	...	...	...	...	...	5	1.0
Totals	500 <sup>C</sup>	100	500 <sup>C</sup>	100	500 <sup>C</sup>	100	500 <sup>C</sup>	100

Weighted Percentages of Constituents in Each Sieve Fraction

	Weighted Percentages of Constituents in Each Sieve Fraction				Weighted Composition of Sample
	19.0-mm (¾-in.)	12.5-mm (½-in.)	9.5-mm (¾-in.)	4.75-mm (No. 4)	
A <sub>1</sub>	8.7	13.0	8.9	2.1	32.7 64.6 (Total A)
A <sub>2</sub>	1.7	6.5	7.4	4.1	19.7
A <sub>3</sub>	0.4	3.3	4.4	4.1	12.2
B <sub>1</sub>	3.7	4.6	3.7	1.3	13.3 31.8 (Total B)
B <sub>2</sub>	2.6	3.5	1.1	3.6	10.8
B <sub>3</sub>	...	1.3	2.5	3.9	7.7
C <sub>1</sub>	0.2	0.3	1.2	0.8	2.5 3.6 (Total C)
C <sub>2</sub>	0.1	0.1	0.3	0.4	0.9
C <sub>3</sub>	...	...	...	0.2	0.2
Total in sieve fraction	17.4	32.6	29.5	20.5	
Total in sample, condition 1					48.5
Total in sample, condition 2					31.4
Total in sample, condition 3					20.1

<sup>A</sup> This table indicates a convenient method of setting up a work sheet for recording results and calculations. The results developed here are entered in the form indicated by Table 3. Table 3 is included in the petrographic report. Table 2 is not.

<sup>B</sup> Letters (A, B, C) refer to the various constituents found, subscript numbers (1, 2, 3) refer to the various conditions in which each constituent has been found, such as relative degree of weathering.

<sup>C</sup> The recommendation concerning the number of particles to be counted has been met; the selection of 500 particles per fraction for the example is to illustrate the calculation; it is not intended to suggest that a predetermined number of particles per fraction should be selected.

TABLE 3 Composition and Condition of an Aggregate Sample (Table Constructed from Calculations Shown in Table 2)<sup>C</sup>

Constituents	Amount, as Number of Particles in Percent							
	In Fractions Retained on Sieves Shown Below <sup>A</sup>				In Whole Sample <sup>B</sup>			Totals
	19.0-mm (¾-in.)	12.5-mm (½-in.)	9.5-mm (¾-in.)	4.75-mm (No. 4)	Condition 1	Condition 2	Condition 3	
A	62	70	70	50	33	20	12	65
B	37	29	25	43	13	11	8	32
C	1	1	5	7	2	1	tr	3
Total	100	100	100	100	...	...	...	100
Weighted average, condition 1					48	...	...	...
Weighted average, condition 2					...	32	...	...
Weighted average, condition 3					...	...	20	...

<sup>A</sup> Based on count of 500 particles in each sieve fraction. (The number of particles of each sieve fraction counted should be shown in the report and this may conveniently be done as a footnote to the table.)

<sup>B</sup> Based on grading of the sample as received, and on the distribution of constituents by sieve fractions shown at the left above. (If the petrographic report forms part of a complete investigation of the sample, including a report of the grading, the grading need not be shown. If the petrographic report is to be submitted alone, the grading of the sample should be included with it.)

<sup>C</sup> For other forms of presentation of results of petrographic analysis, see Tables 1-4 in Mielenz, R. C., "Petrographic Examination of Concrete Aggregates," Chapter 33, *Significance of Tests and Properties of Concrete and Concrete-Making Materials*, ASTM STP 169 B, p. 197.

convenient to calculate and record these percentages to tenths.

14.1.4 By adding the weighted percentages of each constituent in each sieve fraction, obtain the weighted percentage of each constituent in the whole sample (see under weighted composition of sample in Table 2).

14.1.5 Construct a table to show the composition of each

sieve fraction and the weighted composition of the whole sample. Report values to the nearest whole number. Report constituents amounting to 0.5 % or less of a sieve fraction or of the whole sample as traces. Table 3 is an example constructed from the data obtained in Table 2. As a convention, the total in each sieve fraction and the total in the whole sample shall each be 100 % without the traces.

Difficulties in abiding by this convention can usually be avoided by grouping minor constituents of little engineering importance. It is preferable to tabulate constituents known to react deleteriously in concrete so that their distribution will be apparent from inspection of the table, even though the amount in the whole sample or in any fraction is very small.

**15. Report**

15.1 The report of the petrographic examination should summarize the essential data needed to identify the sample as to source and proposed use, and include a description giving the essential data on composition and properties of the material as revealed by the examination. The report should record the test procedures employed, and give a description of the nature and features of each important constituent of the sample, accompanied by such tables and photographs as may be required. The findings and conclusions should be expressed in terms likely to be intelligible to those who must make decisions as to the suitability of a material for use as concrete aggregate.

15.2 When the sample has been found to possess properties or constituents that are known to have specific unfavorable effects in concrete, those properties or constituents should be described qualitatively and, to the extent practicable, quantitatively. The unfavorable effects that may be expected to ensue in concrete should be mentioned. When appropriate, it should be stated that a given sample was not found to contain any undesirable features. When such is the case it may also be appropriate, especially if the report of the petrographic examination is not accompanied by reports of

results of physical and chemical tests for which numerical limits may be applicable, to add that the material appears acceptable for use provided the applicable acceptance tests are made and the results are within the appropriate limits. The report should not, however, contain conclusions other than those based upon the finding of the examination unless the additional data to support such conclusions are included in or with the petrographic report and the petrographer has been authorized to analyze the other relevant nonpetrographic data.

15.3 The petrographic report should include recommendations regarding any additional petrographic, chemical, physical, or geological investigations that may be required to evaluate adverse properties that are indicated by the petrographic examination that has been performed. Supplementary petrographic investigations might include qualitative or quantitative analysis of the aggregate or of selected portions thereof by X-ray diffraction, differential thermal methods, or other procedures that are directed to identification and description of the constituents of the aggregate.

**16. Precision and Bias**

16.1 Those test methods that are used in connection with this guide that have been standardized in ASTM are subject to having precision and bias sections. Those that have not been standardized will each be provided with such a section, if and when standardized. None of the nonstandardized procedures mentioned for optional use in this practice are used in ways that lend themselves to the preparation of precision and bias statements.

*The American Society for Testing and Materials takes no position respecting the validity of any patent rights asserted in connection with any item mentioned in this standard. Users of this standard are expressly advised that determination of the validity of any such patent rights, and the risk of infringement of such rights, are entirely their own responsibility.*

*This standard is subject to revision at any time by the responsible technical committee and must be reviewed every five years and if not revised, either reapproved or withdrawn. Your comments are invited either for revision of this standard or for additional standards and should be addressed to ASTM Headquarters. Your comments will receive careful consideration at a meeting of the responsible technical committee, which you may attend. If you feel that your comments have not received a fair hearing you should make your views known to the ASTM Committee on Standards, 1916 Race St., Philadelphia, PA 19103.*



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# Appendix I

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## Valve Catalog Sheet

## 87-100 Series

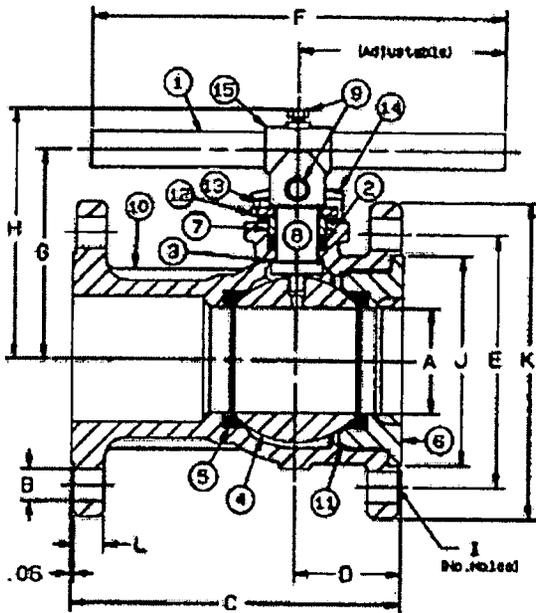
### Stainless Steel ANSI Class 150 Flanged Ball Valve

ANSI Class 150 Flanged Ends, 275 psig WOG, Cold Non-Shock. 150 psig Saturated Steam. Vacuum Service to 29 inches Hg.

Federal Specification: WW-V-35C, Type: II, Composition: SS, Style: 3.

#### FEATURES (3" to 10")

- Graphite stem packing
- Adjustable packing gland
- RPTFE seats and seals
- Actuator mounting provisions
- Certified to API standard 607, 4th Edition
- Meets NACE MR-01-75
- Unibody construction
- Corrosion resistant
- Blow-out-proof stem design
- Field repairable
- Positive shut-off



#### SS ANSI CLASS 150 FLANGED BALL VALVE (3" TO 10")

NUMBER	SIZE	A	B	C	D	E	F	G	H	I	J	K	L	WT.
87-100-01	3"	2.50	.75	8.00	2.56	6.00	18.00	5.12	6.38	4	5.00	7.50	.75	35.8
87-10A-01	4"	3.25	.75	9.00	3.22	7.50	18.00	6.22	7.38	8	6.18	9.00	.93	55.0
87-10C-01	6"	4.50	.87	10.50	4.12	9.50	18.00	7.09	8.25	8	8.50	11.00	1.00	104.0
87-10E-01	8"	6.00	.87	11.50	5.75	11.75	18.00	9.31	10.43	8	10.62	13.50	1.18	185.0
87-10G-01	10"	7.50	1.00	13.00	6.50	14.25	30.00	10.50	11.87	12	12.75	16.00	1.31	280.0

## STANDARD MATERIAL LIST

1. Handle	Galvanized pipe
2. Stem packing	Graphite
3. Stem bearing	RPTFE
4. Ball *	A351-CF8M
5. Seat (2)	RPTFE
6. Retainer	A351-CF8M
7. Packing gland	A276-316
8. Stem	A276-316
9. Hex cap screw	18-8 SS
10. Body	A351-CF8M
11. Body seal	RPTFE
12. Gland plate	304 SS
13. Gland plate screw	18-8 SS
14. Stop	A276-316
15. Handle adapter	A351-CF8M

\*Hollow balls are standard on 8" to 10" valves.

### OPTIONS AVAILABLE:

Option Explanations>>

(SUFFIX)	OPTION	SIZES
-14-	Vented Ball	3" to 10"
-19-	Lock Plate	3" to 10"
-21-	UHMWPE Seats (Non-PTFE)	3" to 10"
-35-	PTFE Trim	3" to 10"
-49-	Assembled Dry	3" to 10"
-57-	Cleaned For Gaseous Oxygen	3" to 10"
-60-	Static Grounded Ball & Stem	3" to 10"
-65-	Multifill Seats & Graphite Packing	3" to 10"
-68-	4" Stem Extension	3" to 6"
-69-	Purge Ports	3" to 10"
-MG-	Gear Operator & Hand Wheel	3" to 10"