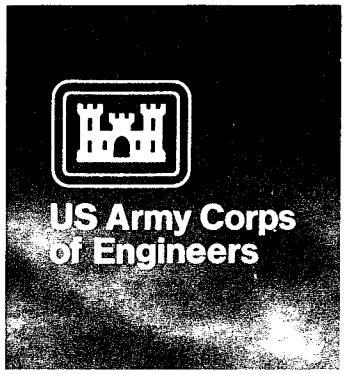

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TECHNICAL REPORT REMR-CS-16

REPAIR OF DAM INTAKE STRUCTURES AND CONDUITS. CASE HISTORIES

by

Roy L. Campbell, Sr., Dennis L. Bean

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Waterways Experiment Station, Corps of Engineers
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COVER PHOTOS:

TOP — Drilling of grout hole in conduit invert, Arkabutla Dam

BOTTOM — Chinking joint with lead wool, Arkabutla Dam

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Based on a survey of inspection reports, 29 percent of the maintenance and repair problems at Corps dams were observed in intake structures and conduits. Repairs to these structures did not perform as desired in better than 40 percent of the reported efforts and with better than 21 percent reported as failed. A number of products whose manufacturer's literature indicated that their products were suited for application in a wet environment such as that found in intake structures and conduits failed. In some instances, the repair technique was at fault. In others, the product failed to perform as indicated.

This report documents selected repair efforts to intake structures and conduits and presents them in a case history format that includes a project description and a history of the repair efforts. The project description identifies principal project features and gives a detail description of the deficiency being repaired to include its history and

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cause (if known). The descriptions of repair efforts are presented chronologically for each project and include a detailed description of the repair products and techniques used and the performance of the repair (if known).

The two most common types of concrete deficiencies being repaired were leakage from cracks and joints and cavitation erosion damage to conduit passageways immediately downstream of gates. The most successful method documented for reducing leakage through both cracks and joints was chemical (urethane) grouting. The most successful method documented for withstanding cavitation damage was resurfacing of damaged area using a product call Belzona Magma Quartz. However, before a recommendation can be made, laboratory testing is needed to further substantiate the quartz product's potential for cavitation repair. Also, because this product is extremely expensive, it may be more economical to repeat the resurfacing on a routine basis using an inexpensive patching material. Evaluation of repairs to other types of deficiencies were not conclusive.

PREFACE

The study reported on herein was authorized by Headquarters, US Army Corps of Engineers (HQUSACE), under Civil Works Research Work Unit 32304, "Maintenance and Repair of Intake Structures and Conduits," for which Mr. Roy L. Campbell, Sr., is Principal Investigator. This work unit is part of the Concrete and Steel Structures Problem Area of the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program sponsored by HQUSACE. The Overview Committee at HQUSACE for the REMR Research Program consists of Mr. James E. Crews and Dr. Tony C. Liu. Technical Monitor for this study was Dr. Liu.

The study was performed at the US Army Engineer Waterways Experiment Station (WES) under the general supervision of Mr. Bryant Mather, Chief, Structures Laboratory (SL), and Mr. John M. Scanlon, Chief, Concrete Technology Division (CTD), and under the direct supervision of Mr. Roy L. Campbell, Sr., Civil Engineer, CTD. Program Manager for REMR is Mr. William F. McCleese, CTD. Problem Area Leader for Concrete and Steel Structures is Mr. James E. McDonald. This report was prepared by Messrs. Roy L. Campbell, Sr. and Dennis L. Bean.

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CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENTS

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic metres
degrees (angle)	0.01745329	radians
Fahrenheit degrees	5/9	Celsius degrees or kelvins*
feet	0.3048	metres
gallons (US liquid)	3.785412	litres
horsepower (550 foot-pounds (force) per second)	745.6999	watts
inches	25.4	millimetres
miles (US statute)	1.609347	kilometres
mils	0.0254	millimetres
ounces (US fluid)	0.02957353	litres
pounds (force)	4.448222	newtons
pounds (force) per square foot	47.88026	pascals
pounds (force) per square inch	0.006894757	megapascals
quarts (US liquid)	0.9463529	litres
square feet	0.09290304	square metres
tons (force)	8.896444	kilonewtons

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain kelvin (K) readings, use: $K = (5/9)(F - 32) + 273.15$.

REPAIR OF DAM INTAKE STRUCTURES AND CONDUITS: CASE HISTORIES

PART I: INTRODUCTION

Background

1. A search of the US Army Engineer Waterways Experiment Station (WES) damage and repair data base (McDonald and Campbell 1985) indicated that 29 percent of the maintenance and repair problems at Corps dams were observed in intake structures and conduits. It was found that the most frequent occurrences of deficiencies reported were: cracking, 41 percent; seepage, 24 percent; spalling, 14 percent; and erosion, 8 percent. Twenty-one percent of the total number of deficiencies reported were classified as needing monitoring or remedial action. The number of repair and maintenance activities reported was 43 percent of those classified as needing attention.

2. For reported repairs at locations other than joints of intake structures and conduits, the following results were found:

   a. Fifty-four percent of the repairs made to intake structures were reported as performing less than satisfactorily with 29 percent reported as failed.

   b. The most frequent type of deficiencies being repaired in intake structures were: cracking, 36 percent; erosion, 21 percent; and spalling, 13 percent.

   c. Forty percent of the repairs made to conduits were reported as performing less than satisfactorily with 21 percent reported as failed.

   d. The most frequent type of deficiencies being repaired in conduits were: erosion, 43 percent; cracking, 19 percent; and spalling, 17 percent.

3. It should be noted that seepage often accompanied cracking as a co-deficiency and was most often the primary reason for repair being made although this was not generally reported as such.

Purpose

4. The purpose of this study is to provide detailed information on current practices in the repair of intake structures and conduits that can be

used in the evaluation and selection of repair materials and techniques and in the planning and managing of the repair work.

Scope

5. A search of the damage and repair data base was performed to identify Corps projects where repairs were made to either the intake structure or conduit. Details of repairs were collected through reviews of periodic inspection reports, contacts with district offices and material vendors, and visits to ongoing repair projects. Information considered beneficial to future repairs was documented in the form of repair case histories and reported herein.

PART II: CASE HISTORIES

Applegate Dam

Background

6. Applegate Dam was completed in 1980 and is located in the Portland US Army Engineer District (USAED), in southwest Oregon on the Applegate River about 23 miles* southwest of Medford. The principal project features include a 244-ft-high, 1,200-ft-long, zoned gravel embankment with a central impervious core; a controlled outlet works through the embankment; gated spillway on left abutment with a flip bucket chute; and fish facilities at the toe of left abutment. The outlet works consists of a reinforced concrete intake structure, outlet conduit, primary stilling basin and secondary stilling basin. The outlet conduit is 930 ft long and has an oblong-shaped 9-ft-wide, 14.5-ft-high cross section. A 20-in., 20-oz copper waterstop is located in the center of the 4-ft-thick reinforced concrete walls at each monolith joint of the conduit.

7. During the May 1981 periodic inspection, significant leakage was observed through three monolith joints in the outlet conduit. Leakage at joint 15/16 was reportedly flowing at 3 to 5 gpm from near the upper left side of the joint. Water was assumed to be flowing from the foundation rock along the joint through either a break in the waterstop or a zone of porous concrete around the water stop. The impervious core of the dam is in direct contact with the conduit in this area. The potential for erosion of the core material was felt to be serious enough to warrant immediate repair. Leaks at the other two joints were located under the cobble and boulder zone and were not considered a threat to the integrity of the embankment. Leakage at joint 12/13 was reported during the June 1984 periodic inspection. Because of its location near the dam core, it also was recommended for immediate repair.

Repairs

8. Remedial grouting of monolith joints in the conduit was performed during the period between June 1981 and July 1984 using CR-250, a hyrophilic

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* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

urethane chemical grout manufactured by 3M Company of Minneapolis, Minnesota. These repairs were described in detail by McDonald (1986). The conduit was inspected in June 1985 and no leakage at repaired joints was reported.

Arkabutla Dam

Background

9. Arkabutla Dam was completed in 1943 and is located in northwestern Mississippi on the Coldwater River approximately 12 miles northwest of Coldwater, Mississippi, in the Vicksburg District. The main dam is a rolled earthfill embankment approximately 10,700 ft long with a maximum height of 67 ft above the valley floor. The outlet works is located near the south abutment of the main dam. It consists of reinforced concrete structures that include an approach channel, three-gate control structure, transition, single-barrel conduit, chute, and stilling basin. The conduit is made up of thirteen 25 ft monoliths each having a modified inverted egg-shaped cross section (Figure 1) that is 18.25 ft high and 16 ft wide with a ring thickness of 3 ft at the crown and 3.5 ft at the spring line and invert. The monolith joints of the conduit have 9-in. rubber water stops in the crown and side walls that extend below field joint, but not into the invert (Figure 2).

10. It appears that the omission of water stops in the invert is the major contributing factor in the leakage and associated deficiencies observed at the monolith joints of the conduit. During the early operations of the outlet works, leakage was observed at the upstream joints between monoliths 1 and 7. Leakage continued after remedial grouting in 1950 and was accompanied by loss of foundation material and differential movement. Movements of 0.04 ft or greater were reported in 1969 for monolith joints 3/4, 6/7, 9/10, and 10/11. Also reported were spalling and missing filler at various joint locations. Remedial grouting of all conduit joints was performed in 1970. In 1974, a sinkhole developed in the embankment above the downstream end of the conduit near the west edge of the toe road. This sinkhole was cleaned out and backfilled with sandy clay after an investigation concluded that the sinkhole had developed from earlier foundation material losses through openings which were grouted in 1970. Leakage continued and remedial grouting was performed in 1977. In total, leakage has occurred to varying degrees at all monoliths

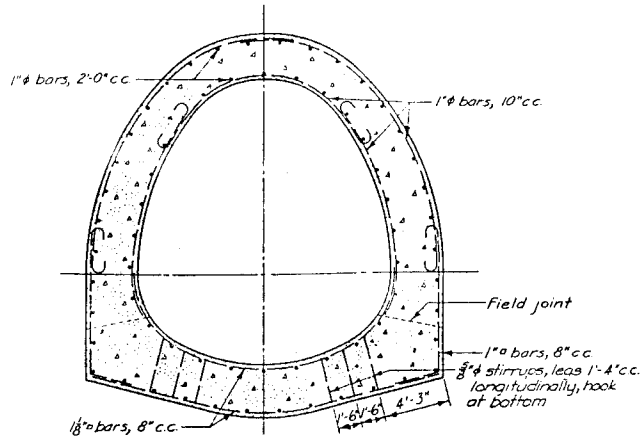
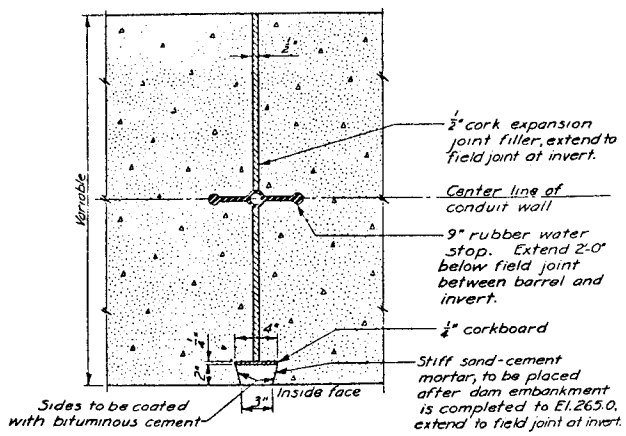
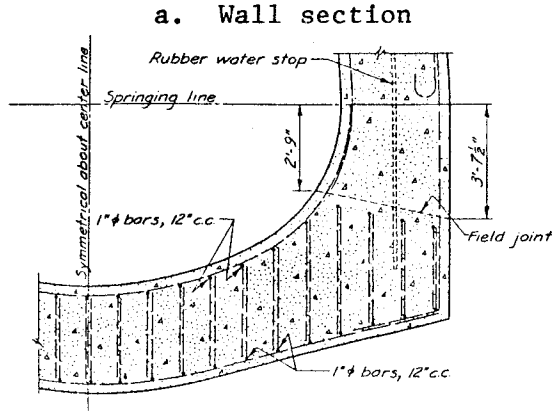


Figure 1. Typical conduit section, Arkabutla Dam



a. Wall section



b. Conduit section

Figure 2. Waterstop location in joint, Arkabutla Dam

joints of the conduit with significant losses of foundation material and movement occurring at a number of the joints.

Repairs

11. 1950. Remedial grouting (USAED, Vicksburg 1969) was performed by Vicksburg District personnel in November 1950 to stop leakage at joints between monoliths 1 through 8 and at a midmonolith crack in monolith 3. Holes were drilled into the lower part of the conduit. Pipe sleeves were placed in the holes and grouted in place for injection ports. Depths of holes varied between 3.75 and 5.6 ft for joints and 3 and 3.2 ft for crack. A grout mixture composed of 6.5 gal of water, one 94-lb sack of portland cement, and 2 lb of voloclay bentonite was used. Maximum grouting pressures varied between 190 and 350 psi. Joints 5/6 and 6/7 took 80 percent of grout used in the repair with individual takes of 57 percent pumped at pressures of 190 to 200 psi and 23 percent at 206 to 210 psi, respectively. The midmonolith crack took 2.5 sacks of cement pumped at pressures of 190 to 250 psi. Overall, a total of 171.5 sacks of cement were pumped during the repair. A mixture of one part cement and one part sand was used in patching and sealing of joints.

12. Leakage was observed during a 1969 periodic inspection at all of the joints grouted in 1950, with flowing water occurring at joints 2/3 and 5/6 and heavy flow at joint 6/7. Also reported were differential movements of 0.04 ft or greater occurring at grouted joints 3/4 and 6/7 and spalling and missing joint filler at various joint locations. These results indicated the repair had failed and that additional remedial work was needed.

13. 1970. Remedial grouting* was performed by Vicksburg District personnel in 1970 at all 12 monolith joints and at midmonolith cracks in monoliths 3, 5, and 6 of the conduit. Grouting was accomplished in three stages occurring in January, June, and October-November 1970 due to the limited length of time in which flow from the reservoir could be interrupted. A total of 592.3 sacks of portland cement (296.3 cu ft of solid volume, including fly ash and bentonite) were placed during the three stages. Approximately 98.3 percent of the cement was placed at joints 3/4, 5/6, 6/7, and 12/13 with percentages of 69.2, 5.6, 11.4, and 12.1, respectively.

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* A. R. Bourquard. 12 February 1971. "Arkabutla Lake, Report of Grouting Operations, Outlet Works Conduit," Letter Report, US Army Engineer District, Vicksburg, Miss.

14. The grouting operations utilized both new grout holes and those ports installed in 1950 for placement of the grout. The general procedure for drilling in the invert was to redrill existing ports, and to drill required new holes. The new holes were drilled with a 2-1/2-in.-diameter bit to a depth of 1.0 ft. Pipe nipples with a 1-1/2-in. inside diameter were placed 0.8 ft into each hole and cemented in place. The holes were then completed by drilling completely through the conduit concrete using a 1-3/8-in.-diameter rock bit. The grout holes along the joints were started about a foot from the joint and were slanted so as to intersect the joint near the bottom of the monolith. The crown grout holes were drilled vertically into the joint with a 3-1/4-in.-diameter, thin-wall bit and 2-1/2-in. inside diameter nipples were cemented into place. The holes were then completed by drilling, with a 2-in.-diameter rock bit, completely through the conduit concrete. The grout take in the crown was negligible in each case. For this reason, there were only three holes made in the crown of the conduit. For the midmonolith cracks, holes were drilled normal to the conduit surface at the cracks approximately 1 ft deep, and 1-1/2-in. inside diameter nipples were cemented into the holes. The midmonolith holes were not drilled through the conduit.

15. Operating guides called for grouting with a maximum pressure of 46 psi, beginning with a relatively thin mixture (a water-cement ratio of 4:1) and thickening the mixture in stages as necessary to a 1:1 mix. A portland-cement grout was used during the January and June operations with bentonite and fly ash added to mixtures used in the October-November operation.

16. Upon completion of grouting operations, all pipe nipples were capped and any holes around the nipples were filled with Skim-Kote (a two-component epoxy grout manufactured by Hallemite, Lehn, and Fink Industrial Products Div. of Sterling Drug, Inc.). Existing joint material and caulking were removed from the monolith joints in the invert of the conduit. The joints were cleaned and where possible dried. The joints which were dried were sealed with Colma Joint Sealer (a two-component, polysulfide system manufactured by Sika Chemical Corporation of Lyndhurst, New Jersey) and capped with Skim-Kote. In joints which could not be dried, the entire joint was filled and sealed with Skim-Kote. The joint sealer was tied to the ends of the waterstop at the bottom of the sidewalls of the conduit. The surface area containing the midmonolith cracks were not treated.

17. Inspection of the conduit after grouting was completed indicated a successful repair with only small clear water seeps. However, leakage at the joints worsened over the following years to the point that remedial grouting was recommended and performed in 1977 to stop the loss of foundation material. The pregrouting inspection in 1977 recorded minor leakage at most of the joints. The most significant leakage was through monolith joint 2/3 where approximately 4 gpm of water and an estimated two cups (assumed to be 8-oz capacity cups) of foundation sand per day were escaping. All other flows were 1 gpm or less of clear water.

18. 1977. Remedial grouting (USAED, Vicksburg 1979) was performed by the Vicksburg District personnel in June 1977 at joints and cracks of monoliths 1 through 9. After closure of the control structures gates, drilling and grouting equipment were placed in the conduit. A 2-ft-high sandbag dike was constructed across the transition section (Figure 3) and two 3-in.-diameter drain pipes extended from there to near the end of the conduit to pass water leaking from around the gates through the repair area.

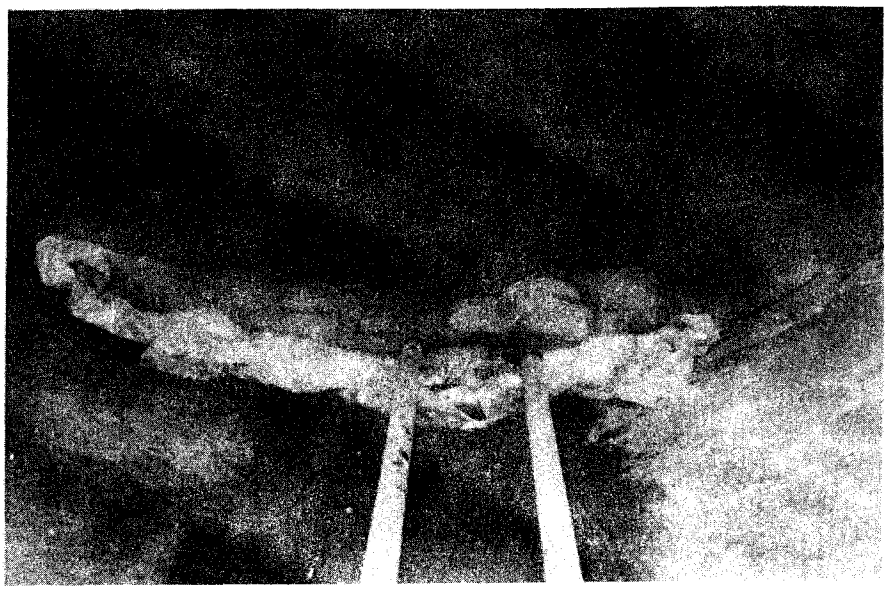

Figure 3. Sandbag dike and drain pipes, Arkabutla Dam, 1977

19. Ports from previous grouting operations were utilized whenever possible. New 2-1/2-in.-diameter grout holes were percussion (jackhammer) drilled

in the invert to a depth of 22 in. (Figure 4). A 1-1/2-in. inside diameter pipe coupling with nipple (flared at the bottom) was grouted in each hole. After the grout had set, a 1-3/8-in.-diameter hole was drilled to refusal (steel rebar) or through the conduit concrete to foundation sand and a 1-in. inside diameter cutoff valve installed.

Figure 4. Drilling of grout hole in conduit invert, Arkabutla Dam, 1977

20. An exploratory hole was drilled in the crown at monolith joint 12/13 mainly to check the area near the bottom of the sinkhole that developed in the embankment close to the downstream end of the conduit in 1974. An air-powered rotary drill equipped with a 2-1/2-in.-diameter, thin-wall diamond bit was used to drill 2 ft into the joint. The drill was mounted and braced between the top of a wooden scaffold and the top of the conduit (Figure 5). When the waterstop was penetrated, no water was encountered. A 2-in. inside diameter, 20-in.-long, pipe coupling and nipple with a flared end were grouted in place.

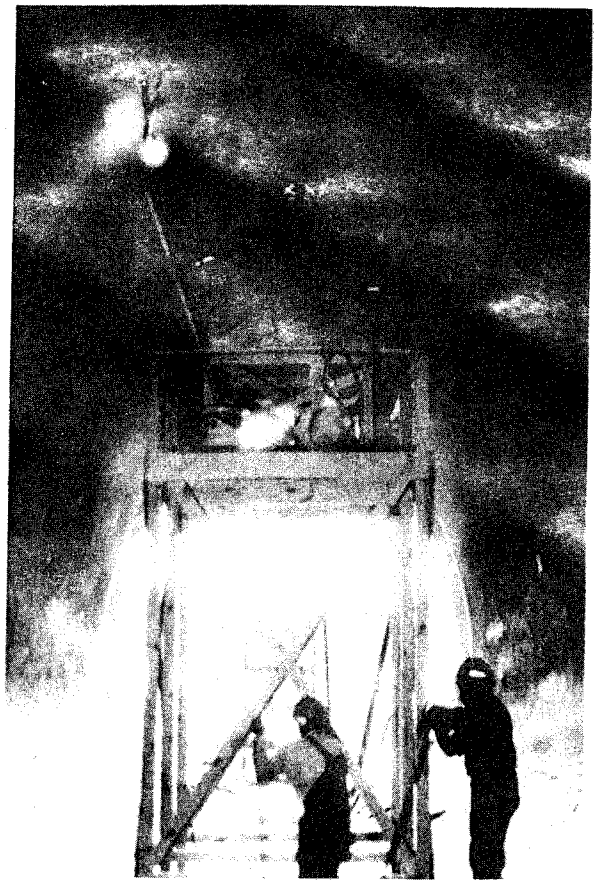

Figure 5. Drilling exploratory hole in conduit crown, Arkabutla Dam, 1977

After the grout had set, a 1-3/16-in.-diameter, thin-wall diamond bit was used to advance the hole. Total depth of the hole was about 36 in. when a pea gravel filter was encountered and the drilling terminated. It was not desirable to pump grout into the pea gravel filter; therefore, this hole was sealed with a pipe plug and no further work was done in this area.

21. Three basic cement grout mixtures were used: a 4:1 in narrow cracks and areas of minor seepage, a 1:1 in holes with back pressure or potential significant take, and a 3:1 mixture for in-between conditions. Two pounds of bentonite (aquagel) were added per sack of portland cement to help keep the cement grains in suspension and reduce shrinkage. The maximum allowable grouting pressure for each hole was calculated based on the thickness of the overburden and estimated water table at that location. Grout pressures were in

the range of 30 to 46 psi (approximately 80 percent of the calculated maximum allowable).

22. A hydraulically operated dual mixing bin grout machine with a 20-cu-ft sump tank capacity was located on the embankment surface about 20 ft from the right stilling basin wall. Approximately 300 ft of 1-1/2-in. inside diameter hose extended from the grout machine to a set of control valves (Figure 6). An additional 300 ft of the hose was used to complete the loop back to the machine for recirculation of the grout during periods when the grouting operation had to be interrupted. At the control valves, a 50-ft-long, 1-1/2-in. inside diameter line was used to connect to injection ports. Grout pressure was indicated by a gauge at the set of control valves (Figure 6) and a second gauge at the injection port (Figure 7). Lead wool was used to chink grout leaks in the conduit (Figure 8).

Figure 6. Control valve, Arkabutla Dam, 1977

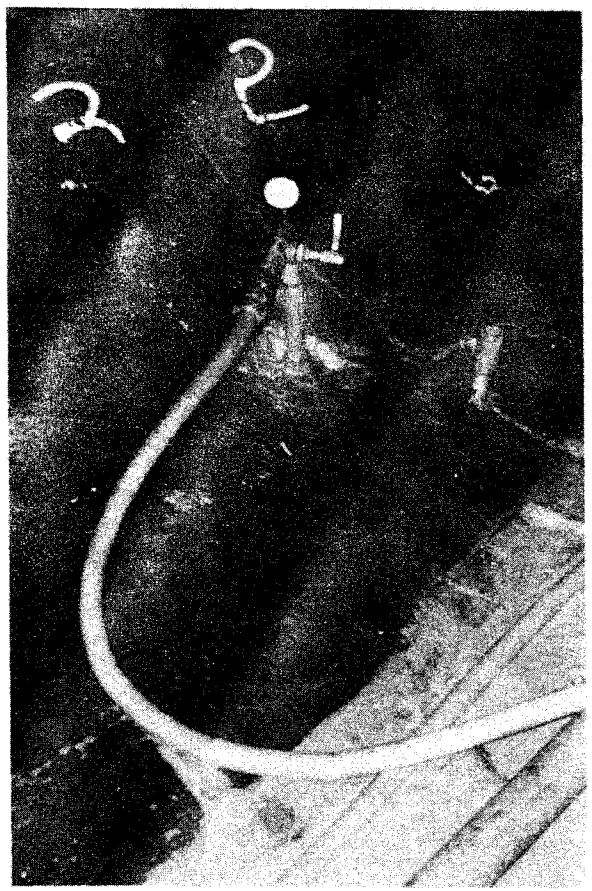

Figure 7. Pressure gauge and cutoff valve, Arkabutla Dam, 1977

Figure 8. Chinking joint with lead wool, Arkabutla Dam, 1977

23. Immediately prior to grouting any port having back pressure, the cut-off valve was opened and a 1/2-in. inside diameter, 6-ft-long, flushing pipe inserted (Figure 9). Grout was jetted through the pipe into the hole to

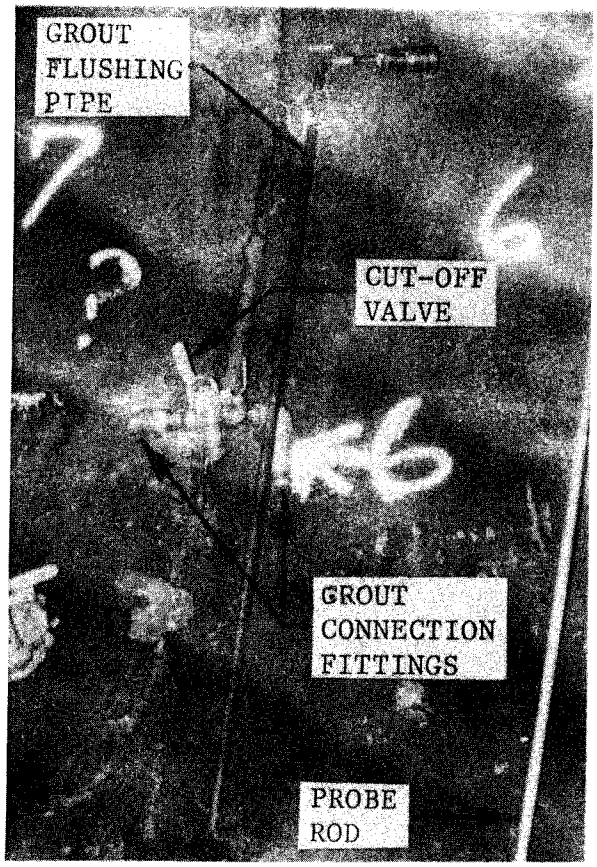

Figure 9. Grout hole with cutoff valve, Arkabutla Dam, 1977

remove foundation sands that had filled the port (Figure 10). After the port was cleared, the pipe was quickly removed and the valve closed. When the regular grout connection was made, the hole was unobstructed and could freely pass grout to adjacent voids.

24. At the end of the grouting day, all grouted holes were inspected. Water dripping from a hole when the valve was slightly opened indicated grout had not sealed the leak. A 4-ft-tall standpipe was connected to the hole and filled with 1:1 grout (Figure 11). If water was observed dripping from the top of the standpipe the next day, adjacent ports were cleaned out, jetted with water, and then pressure grouted with a 1:1 mix.

Figure 10. Flushing of grout hole, Arkabutla Dam, 1977

Figure 11. Grouting using standpipe, Arkabutla Dam, 1977

25. At the end of the repair, all leakage of any consequence through monolith joints and cracks had been successfully stopped. A total of 33 cu ft of cement had been placed with 92 percent being taken at joints 2/3, 5/6, and 6/7 with takes of 13.7, 6.9, and 9.8 cu ft, respectively.

26. In May 1978 the tunnel was unwatered and inspected by Arkabutla Lake personnel. A minor water leak was noted at monolith joint 2/3. There was no indication of any foundation material being lost. No other noteworthy leakage was noted. A periodic inspection in 1979 reported small leaks with no sand and clear flows at joints 3/4, 4/5, 5/6, and 8/9. Periodic inspection in 1984 reported minor leaks at joints 1/2, 2/3, 3/4, 4/5, 5/6, 7/8, and 8/9 and at cracks in monoliths 3, 5, and 6. Flows were clear with no foundation material being passed. Although there has been some increase in leakage, the performance of the 1977 repair was considered good at the time of this report.

Barren River Lake Dam

Background

27. Barren River Lake Dam was completed in 1964 and is located in the Louisville District on the Barren River in south central Kentucky 13 miles southwest of Glasgow, Kentucky. The project consists of an earthfill dam, a gate controlled outlet works along the base of the left abutment, and uncontrolled spillway through the right abutment. The dam embankment is 3,970 ft long, 146 ft high, and 30 ft wide at its top (elevation 618 ft) and consists of compacted impervious fill with a downstream random rockfill zone. The outlet works consists of a control tower (wet type) with three 6.5- by 14-ft hydraulically operated vertical slide gates, two 36-in.-diameter low-flow bypass pipes, and four 4- by 4-ft multilevel inlets; a 17- by 17-ft semielliptical conduit; and a stilling basin. The spillway is 300 ft wide with a crest elevation of 590 ft.

28. During the March 1985 periodic inspection, leakage was noted in the wet well of bypass valve number 2 of the control tower at elevations 512 and 523 ft. At both elevations leakage was found coming from a single hole in an area of honeycombed or rotten concrete with no cracking evident around either of the holes. At that time, both holes were approximately 5/8 in. in diameter and 4 to 6 in. deep.

Repairs

29. 1985. Remedial grouting to stop leakage at two elevations in the control tower was performed by district personnel in December 1985 using FLEX 44, a chemical grout manufactured by DeNeef America, Inc. of St. Louis, Michigan (Lewis and Brockman 1987). The grout is a hydrophobic polyurethane liquid that reacts with water, expanding and curing to a tough, flexible closed-cell foam. At the time of the repair, the lake was at elevation 537.7 ft, resulting in a hydrostatic head of 25.7 ft on the outside of the tower at elevation 512 ft and 14.7 ft at elevation 523 ft. Leakage rates at these elevations were approximately 1 and 2 gpm, respectively.

30. A pneumatic-powered masonry drill with a 1/2-in.-diameter bit was used to drill a 6-in.-deep hole within 4 in. of the leaking hole at elevation 512 ft. During the drilling, the deteriorated concrete easily gave way, creating an oversized hole. The recommended packer was installed but could not be adjusted to provide a watertight compressive seal around the tubing due to the size of the hole.

31. A cutoff valve was connected to the packer and placed in the closed position. Three gallons of FLEX 44 was mixed with 5 percent Flexcat accelerator. The pump hose was connected to the cutoff valve and a slight amount of pump pressure applied before the cutoff valve opened. Grout was pumped into the port using a hand operated, 500-psi capacity, volume delivery bucket pump (Alemite pump, model 7181). Rags were held tightly against the packer in an effort to reduce the leakage from around the packer. However, there was still too much leakage to allow the grout sufficient time to gel before exiting the concrete. This resulted in most of the grout being washed out of the hole and the termination of the repair effort.

32. The cost of materials procured was $384 which included 5 gal of FLEX 44, 1 qt of Flexcat accelerator, 6 gal of washing agent, and 10 mechanical packers. The cost of the Alemite pump was approximately $200.

33. 1986. Remedial grouting to stop leakage at two elevations in the control tower was performed by a three-man crew of district personnel in March 1986 using TACSS-020 NF, a chemical grout manufactured by DeNeef America, Inc. of St. Louis, Michigan (Lewis and Brockman 1987). The grout is a hydrophobic polyurethane liquid that reacts with water, expanding and curing to a tough, rigid closed-cell foam. At the time of repair, the lake was at elevation

526.1 ft, resulting in a hydrostatic head of 14.1 ft on the outside of the tower at elevation 512 ft and 3.1 ft at elevation 523 ft. Flow rates at these elevations were approximately 1 and 2 gpm, respectively. The water temperature was 51° F.

34. Repair work began at the lower elevation (512 ft) where a 12-in. length of 1/2-in.-diameter copper tubing was inserted approximately 6 in. into the existing hole. The wall thickness at this location was 2 ft 11 in. Water Plug, a fast-setting hydraulic cement manufactured by Thoro Systems Products of Miami, Florida, was kneaded into a fairly stiff ball and then packed around the copper tubing. The hydraulic cement had to be packed around the tubing a total of three times before all of the leakage was diverted through the tubing. The cement was allowed to cure for 45 min before grouting was begun.

35. Two gallons of TACSS-020 NF was mixed with 10 percent C-852 accelerator to obtain a minimum gel time of 4 min 30 sec at given water temperature. Mixing was done manually in the pump bucket. Precautions were taken to ensure that water did not enter the pump or the open end of the hose. The cutoff valve connected to the end of the copper tubing was closed and the hose from the pump attached. A small amount of pressure was applied in the line before the cutoff valve was reopened and the grout pumped into the void. Bubbles appeared around the port indicating the grout was reacting. The cutoff valve was closed and hose disconnected after allowing approximately 10 to 15 min for grout to gel. After an hour, the tubing was removed flush with the wall surface. No leakage was noted after the repair.

36. The repair work began at the higher elevation (523 ft) where leakage was occurring at an existing hole. To ensure proper penetration of the grout into the concrete, the hole was drilled to a 12-in. depth. A 1/2-in.-diameter piece of copper tubing was inserted 6 in. into the hole and Water Plug packed around the tubing. This time the leakage could not be totally diverted through the tubing. As the grout was pumped into the void, rags were pressed against the surface around the tubing in an effort to reduce leakage. However, this effort proved unsuccessful as most of the grout was washed out of the hole.

37. The tubing and Water Plug were removed from the hole and the tubing reinserted. Oakum was packed around the tubing and with very little effort all leakage was stopped. The grouting operation was resumed and approximately

2 gal of grout was pumped into the void. It was evident that complete penetration of the 2 ft 11 in. wall had occurred as grout could be seen surfacing to the top of the water on the outside of the tower directly above the repair. The grout was allowed 10 to 15 min to gel before the hose from the pump was disconnected. After an hour, the tubing was removed flush with the wall surface. No leakage was noted after the repair.

38. The estimated cost of the repair was as follows:

Material	$ 400.00
Pump	200.00
Labor	800.00
Total	$1,400.00

Beltzville Dam

Background

39. Beltzville Dam was completed in 1972 and is located on Pohopoco Creek near Lehighton, Pennsylvania, in the Philadelphia District. The principal project features include a 4,560-ft-long, earthfill embankment with a maximum height of 170 ft; a reinforced concrete outlet works; and unlined emergency spillway excavated into rock. The outlet works consists of an intake tower with two gated inlets, a 7-ft-diameter circular outlet conduit, and a stilling basin.

40. During the third periodic inspection of the project in September 1972, minor leakage was observed in the interior surfaces of the intake tower at the horizontal construction joints and form tie locations below pool elevation. The leakage water was collecting in the electrical conduits and boxes that were essential to the operation of the tower. This condition worsened over the next few years, being manifested by numerous instances of minor leakage, extensive calcite formation, and corrosion of embedded connection plates that support interior framing. Overall, the leakage had progressed to a level that the structures ability to perform as designed was being threatened by reductions in the reliability and performance of the electrical systems and increased corrosion of the operating machinery and appurtenances.

41. During a 1984 investigation into the cause of cracking, abundant evidence of alkali-silica reaction was found in concrete cores taken from the

intake tower.* The alkali-silica reaction was concluded to be largely or wholly responsible for the cracking of the concrete. It was also reported that the potential for further expansion and cracking due to alkali-silica reaction still existed.

Repairs

42. September 1977. In September 1977, the first of five repairs (McKenzie and Campbell 1985) was undertaken. A surface treatment manufactured by Vandex (USA), Inc. of Stamford, Connecticut, was applied to the negative side (interior face) of two test sections of the tower wall. The first test section involved sealing of a 25-ft-long by 10-ft-high area of tower wall. The second section involved sealing 30 ft of construction joints in another area of tower wall. The repair sections were located at elevation 530 ft and the pool elevation at the time of repair was approximately 628 ft.

43. During the repair, calcite formations were removed using a chipping hammer, and joints and cracks were routed out approximately 1 to 2 in. to sound concrete, producing a dovetail groove where possible. Four application layers were applied to the routed surfaces in the following order: a slurry coat of Vandex Super, a layer of dry mix containing Vandex Special or Vandex Pure, a layer of wet mix containing Vandex Super and Vandex Premix, and a layer of wet mix containing Vandex Premix. Application began at the highest points of the cracks and continued downward to the lowest points where drainage ports had previously been installed. The drainage ports were later plugged using a wet mix of Vandex Super and Premix. All adjacent areas were coated with a slurry of Vandex Super followed by a slurry of Vandex Premix.

44. A 1984 petrographic analysis of a core taken from the repair area showed that the penetration of the Vandex into the concrete was less than 1 mm. A second application of Vandex within a year of the first had been recommended by the product's representative. However, the performance of the repair was not considered effective enough in stopping the leakage and deterring the formation of calcite (Figure 12) to warrant further use.

45. September 1980. The second repair attempt, in September 1980, involved a test section in which joints and cracks were injected with Concresive 1380, a two-component, low-viscosity liquid adhesive manufactured by

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* A. D. Buck. 24 May 1984. "Test and Examination of Concrete Cores from Beltzville Dam Control Tower and Spillway," letter report, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

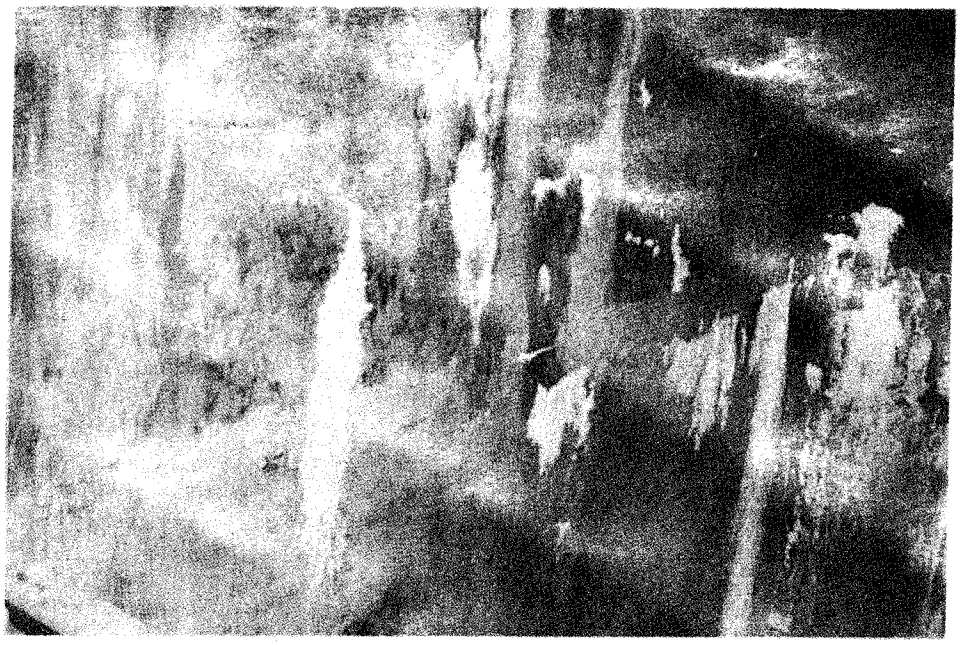


Figure 12. Area of interior wall treated with Vandex, Beltzville Dam, June 1983

Adhesive Engineering Co. of San Carlos, California. After surfaces bordering the joints and cracks were cleaned and sealed, injection ports were drilled and packers installed. The joints and cracks were then injected using 100-psi pressure. As with the repair with the Vandex treatment, the performance of the Concresive 1380 repair was not considered effective enough in stopping the intake tower leakage and deterring the formation of calcite to warrant further use.

46. December 1982-March 1983. In the third repair, Vibraspray S-80, a two-component polyurethane membrane manufactured by Uniroyal, Inc., was applied to the positive side (exterior face) of the upper third of the normally submerged portion of the tower. The work was performed during a 4-month period beginning in December 1982 when the lake had been partially drawn down. The repair required sandblasting the surfaces to be treated, erecting scaffolding and a temporary enclosure over the area to be repaired, heating the area inside the temporary enclosure (necessary because of the cold weather), patching cracks and spalled concrete, applying a bonding agent, applying two

successive 40-mm coats of Vibraspray S-80, patching bug holes, applying a final 30-mm coat of Vibraspray S-80, and maintaining sufficient temperature to ensure proper curing. Inspection of the interior of the tower in June 1983 revealed that the leakage had lessened in the area of application.

47. June 1984. A fourth repair was made in June 1984 during a two-day contract operation in which TACSS-020 NF, a hydrophobic, polyurethane chemical grout manufactured by DeNeef America, Inc. of St. Louis, Michigan, was used to inject construction joints and cracks within selected test areas of the tower. Ambient conditions during the repair were noted as being around 70° F and 70 percent relative humidity.

48. Injection ports 1/2 in. in diameter and 6 in. in depth were drilled into the joints and cracks at intervals of 6 to 36 in. A packer was placed into each port, and fluorescein dye was injected into the port through a brass valve. The dye was used to flush the joint or crack and determine the time of flow between fluid entry and exit. The amount of accelerator needed to achieve the desired reaction time was then determined, based on the dye's time of flow. The TACSS-020 NF and accelerator C-852 were mixed in a 5-gal pail and hand-pumped through a hose to the injection valve and port. Injection began at 100 psi and increased to 400 psi. This pressure was maintained until refusal, at which time the valve was closed until the reaction was completed. The injection process was continued at the next port located adjacent to or above the just completed port.

49. The joints and cracks being repaired were very fine and required the use of a low-viscosity injection material to allow for adequate penetration and sealing of the cracks. TACSS-020 NF is a low-viscosity grout that reacts with water to form a polyurethane foam. It was speculated that during the injection process the grout forced water needed for the reaction out of the joints and cracks and, thereby, slowed reaction time. Also, it was suspected that the temperature in the joints and cracks was low enough to further retard the reaction. The injection process was time-consuming and judged economically unacceptable for the remaining crack repair work in the intake tower.

50. September 1984. The fifth repair was undertaken in September 1984, when 750 linear ft of joints and cracks were injected with Sikadur 52 Injection Resin, a two-component, low-viscosity, epoxy-resin system manufactured by Sika Corporation of Lyndhurst, New Jersey.

51. Chipping hammers and wire brushes were used to clean the surfaces and remove loose materials bordering the cracks. Injection ports 3/4 in. in diameter and 6 in. in depth were drilled into each joint and crack, and packers installed. Sika High Mod Gel, a two-component, structural adhesive, epoxy-resin system was used to seal the surfaces between ports. The Sikadur 52 Injection Resin was then injected at 100 psi into the successive ports.

52. Inspection of the interior of the tower in January 1985 showed that leakage had recurred in some of the injected areas (Figure 13). Overall performance of the injection repair was judged to be not fully successful.

53. Summary. Leakage was reduced to some degree after each of these repairs. The repairs made using Vibraspray S-80 membrane and Sikadur 52 Injection Resin were the most successful. Though neither of these repairs proved to be 100 percent effective, they did help reduce the amount of leakage in treated areas to a more acceptable level. As a result of the repairs and a modification to the heating and ventilation system in the tower, moisture levels in the intake tower have been significantly reduced, with some seasonal variations.

54. Costs for the repairs described above were as follows:

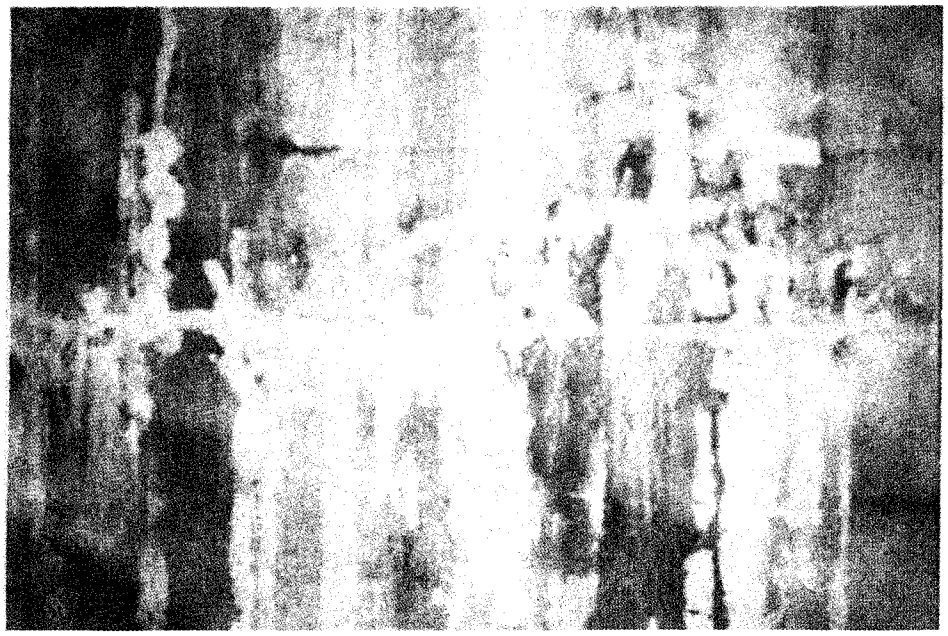
Repair	Material	Unit Cost	Total Cost
1	Vandex Treatment	$0.75/sq ft $20/lin ft	$ 0*
2	Concresive 1380	$700/day	9,800
3	Vibraspray S-80	$22.76/sq ft	227,560**
4	TACSS-020 NF	$1,225/day	2,450
5	Sikadur 52 Injection Resin	$50/lin ft	37,500†

* This was a demonstration effort performed by the manufacturer's representative at no cost to the government.

** Erecting and heating the enclosure (necessitated by cold weather) approximately doubled the cost of this repair.

† This does not include the cost of scaffolding used to inject areas 90 ft above the floor.

a) West wall, elevation 612 ft



b) North wall, elevation 600 ft

Figure 13. Areas repaired with Sikadur 52 Injection Resin, Beltzville Dam, January 1985

Blakely Mountain Dam

Background

55. Blakely Mountain Dam was completed in 1955 and is located in the Vicksburg District on the Ouachita River near Hot Springs, Arkansas. The principal project features include an earth embankment, a flood control outlet works, a hydroelectric plant, and an uncontrolled emergency spillway. The embankment is 231 ft high, 1,100 ft long and contains an impervious core surrounded by a random fill. The flood-control outlet works consists of a reinforced concrete intake control tower with three 8- by 15-ft service gates; a 19-ft-diameter, 1,640-ft-long, reinforced concrete lined tunnel; and a stilling basin. The power outlet works consist of a reinforced concrete intake control tower with two 12- by 24-ft service gates; a 30-ft-diameter, 1,440-ft-long, reinforced concrete lined power tunnel; a cylindrical shaped, steel surge tower; a 24-ft-diameter, 700-ft-long, steel power conduit located inside the power tunnel; a powerhouse; and stilling basin. The flood-control and power tunnels were constructed between August 1948 and May 1950.

56. Leakage at joints, holes, and cracks in the flood control tunnel occurred early in the service life of the tunnel. The leakage increased to the extent that cement grouting was needed and performed in 1956. A total of 290 sacks of cement were used in making the repair. Work was performed by hired labor.

57. In March 1970 water was reported leaking from several holes (grout pipes and other apparently formed openings). Leakage was also noted through cracks and monolith joints. Where practical, these areas of leakage were caulked in 1970.

58. During the September 1976 inspection significant leakage of water was noted coming from the crown at monolith joint 2/3 and the invert at monolith joint 6/7. It was recommended that these two joints be grouted to stop the leakage. Money was appropriated and the work scheduled for the summer of 1978. A prework inspection of the tunnel was made in May 1978 in which significant leakage was observed coming from the crown of monolith joint 13/14. This joint was also recommended for repair. At the time of repair, leakage was reported to be 10 gpm at joint 2/3, 2 gpm at joint 6/7, and 3 gpm at joint 13/14. Minor leakage of less than 0.25 gpm was noted from hairline cracks near the springline of monolith joint 1 (transition area of tunnel).

Cement grouting at the noted joints and cracks was completed in July 1978 with reportedly successful results.

59. A visual inspection was made of the interior of the flood-control tunnel in September 1981 at which time monolith joint 13/14 (crown) was leaking water at a rate of 1 to 3 gpm and leaks were also located near the upstream end of monolith 1. In April 1985 an inspection of the flood-control tunnel was made at which time a leak was observed in the east gate bay of the intake structure approximately 7 ft downstream of the emergency gate. This leak was flowing between 2 and 4 gpm. It was determined that chemical grouting should be performed to decrease the leakage. The work was performed in February 1986.

Repairs

60. 1978. Remedial grouting (USAED, Vicksburg 1981) was performed by district personnel in July 1978 using a portland-cement grout with 2 lb of bentonite added per 94-lb sack of cement to stop leakage in the flood-control tunnel at monolith joints 2/3, 6/7, and 13/14 and at hairline cracks in monolith 1. Gate closures were made using two service gates and one emergency bulkhead gate. This arrangement provided the maximum air draft possible through the intake structure and flood control tunnel. Leakage pass the gates was estimated to be 3 gpm.

61. Three 10,000-cfm fans were positioned at the top of the intake structure to act as exhaust fans. They served to draw air through the tunnel and up the intake structure. However, when drilling began, it was noted that the natural air flow seemed to be toward the downstream end of the tunnel and the fans were not aiding but hindering the flow. Therefore, the fans were reversed and air flow through the tunnel was more than adequate to prevent the buildup of any fumes. Another problem was the fluctuating water levels in the tunnel. Less than an inch of water was present in the tunnel invert at monolith 1 but this increased to about a foot at the downstream end of the tunnel. During power generation the water level rose approximately 5 in. in the tunnel.

62. A crane was used to lower heavy equipment to the entrance at the downstream end of the flood-control tunnel and a ladder secured to the side of the outlet provided access for the personnel. A scaffold was constructed on a truck bed to serve as a working platform for the repairs. Hole locations were measured along the circumference of the tunnel. The first hole was located 4.5 ft to the right of crown centerline at monolith joint 13/14. An attempt

was made to drill the hole with a 2-7/8-in.-diameter roller rock bit. However, the long length of the drill stem allowed the bit to move around far too much. Consequently, a 2-7/8-in.-diameter thin-wall diamond bit was used to drill the holes which were approximately 15 in. deep. When the neoprene water stop was encountered (usually at a depth of 12 in.) the bit was removed and a sharpened pipe was inserted to cut through the waterstop. A 2-in. inside diameter nipple pipe, flared at the bottom, was grouted in place as an injection port. This was accomplished by chinking between the pipe and the hole with lead wool and then forcing thick grout through the pipe into the area of the hole outside the pipe. Water flowing through and around each nipple hole washed some grout out, but repeated filling of the hole with grout provided sufficient grout to secure each nipple. After the grout had set, an 1-3/8-in.-diameter thin-wall diamond bit was used to extend each hole an additional 6 in.

63. During drilling at joint 2/3, when the water stop was cut, the flow increased to approximately 15 gpm. This made securing and grouting the nipple in place most difficult. The problem was solved by installing a dual coupling nipple with a short nipple between the two couplings. Considerable lead wool was used to chink the nipple in place and then sufficient grout was pumped through the nipple to bond the nipple in place.

64. The procedure for the repair of monolith joint 6/7 was modified slightly in that a pneumatic hammer with a 2-1/2-in.-diameter bit was used in the invert of the joint to drill a 15-in.-deep hole. The remainder of the technique was the same as for the previously described joints.

65. Three holes were drilled in leaking hairline cracks in the left face of monolith 1 near the upstream joint and one in the right face. A 2-7/8-in.-diameter thin-wall bit was used to drill a 7-in.-deep hole in which a 6-in.-long nipple was secured. A 1-3/8-in.-diameter thin-wall bit was used to extend each hole until it no longer followed the crack. The holes were pressured grouted. The take was very low. It was then desired to extend two of the holes all of the way through the tunnel concrete plus any grout that had been placed in previous years in order to inject grout into voids that might have existed between the concrete and the surrounding rock. One of these was drilled through 2-1/3 ft of concrete, 1 ft of old grout, and 1 ft into foundation shale. Each hole flowed an estimated 6 to 8 gpm after it had been drilled. Back pressure was measured to be 52 psi.

66. The maximum allowable grout pressure for each holes was calculated based on the thickness of the overburden and the estimated water level at the location. Starting pressures for the grouting were usually between 20 and 60 psi. The pressure was increased as the leaks were sealed. Maximum pressure ranged from 90 to 115 psi but never exceeded 80 percent of the calculated maximum allowable pressure.

67. A gasoline-driven, dual mixing bin grout machine with 20-cu-ft sump tank was used. It was allowed to be operated in the tunnel because adequate ventilation was present to remove the carbon monoxide fumes emitted by the gasoline engine. A forklift was used to transport sacks of cement from the downstream end of the tunnel up to the grout machine. A dual grout line of 1-1/2-in. inside diameter fire hose tested to 400 psi was laid to monolith 2 to circulate the grout. From the end of the dual line, a single grout line was extended to connect each hole to be grouted. Grout pressure was monitored with a 120 psi gauge at the grout hole. The viscosity of the grout was varied to accommodate the differing situations, with resultant water-cement ratios of 4:1, 3:1, and 1:1 (by volume).

68. Generally, a thin mixture (4:1 or 3:1) was used to start the grouting of a hole. If it was evident during grouting that the hole would accept a thicker mix, the mixture was thickened. During the grouting of most of the holes, grout leaks would break out along the monolith joint seam. Lead wool was used to chink the leaks. However, joint 2/3 was started with a 1:1 mixture because of the large volume of water flowing out and the obvious open condition of the hole. It took grout rapidly for an hour and then there was slow take for another hour before the void was filled. Total take for joint 2/3 was 34.4 sacks of cement which was 20 sacks more than the next highest take. Joint 13/14 was difficult to grout because of numerous leaks breaking out around the joint. As a result, it was necessary to chink all the way around the joint and grout a total of four times before joint was finally sealed. Monolith joint 6/7 was grouted with a 3:1 mixture. The next day minor leaks were found coming from joint 6/7. The grout hole was cleaned out and regrouted with a 1:1 mixture which stopped the leakage. For holes located in the hairline cracks of monolith 1, a 4:1 grout mixture was used initially but thicker grout was used when those holes were regrouted. Some interconnection between holes and joints was noted.

69. After the repair all of the holes that had been grouted were inspected. A firm to hard grout plug was present in each and the leakage had been stopped. Many hairline cracks that slowly weeped water were noted throughout the tunnel but they were so small that it would have been futile to try to grout them. In September 1981 an inspection was made of the flood control tunnel at which time monolith joint 13/14 (crown) was observed flowing at a rate of 1 to 3 gpm and leaks were noted near the upstream end of Monolith 1. In April 1985 during an inspection of the flood tunnel, leakage was noted at monolith joints 2/3, 3/4, 4/5, 10/11, 11/12, 13/14, and 20/21 and at three cracks located the lower two transition monoliths.

70. 1986. Remedial grouting* was performed by district personnel in February 1986 using two chemical grouts, TACSS-020 NF and FLEX 44, to reduce leakage in the flood-control tunnel at monolith joints 3/4, 4/5, 13/14, and 20/21; at cracks in the transition monoliths; and at the joint at the intake structure. Both grouts are manufactured by DeNeef America, Inc. of St. Louis, Michigan, and are hydrophobic polyurethanes that react with water to form expansive adhesive solids. The TACSS-020 NF reacts with water to form a non-elastomeric foam (much like a polystyrene). The FLEX 44 reacts with water to form a flexible material (much like a flexible joint sealer). A rider was added to the material purchase contract for 5 days of consultant work on product application by material supplier H. H. Horil and Associates, Inc. of New Orleans, Louisiana. Waterplug, a fast setting hydraulic cement, was used to seal leaking joints at the tunnel surface during the repair. It is manufactured by Thoro Systems Products of Miami, Florida.

71. A crane was used to lower a stake body truck, equipment, and materials to the entrance at the downstream end of the flood-control tunnel. Scaffolding was erected on the truck bed to serve as a work platform. Additional ventilation within the tunnel was not considered necessary due to a natural draft of air through the tunnel and intake tower. The only water in the tunnel during the repair work was that flowing from the leaks within the tunnel.

72. The air compressor for supplying compressed air for the drilling and grouting operations was located at the top of the intake structure.

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* D. A. Goss. 21 May 1986. "Chemical Grouting of Blakely Flood Control Tunnel," Trip Report, US Army Engineer District, Vicksburg, Miss.

Five hundred and fifty feet of hose was used to supply air to monolith joint 20/21 where the drilling was started. Two Chicago pneumatic rotary hammer drills, model number 9A, were used to drill the grout holes. The grout holes were 5/8 in. in diameter and were drilled using 18 and 36-in.-long masonry bits that were carbide tipped and had SDS (SR) series shanks (Figure 14).

73. Holes to be drilled were measured from the center of crown along the tunnel circumference and marked. Drilling of the grout holes (Figure 15) was started on 30 January 1986 at monolith joint 20/21. The joint was weeping at two locations, 9.5 ft right of the centerline of the crown and 5 ft left of the centerline of the crown looking downstream. Five holes were located as follows: 5 ft right (weeped), 7.5 ft right (flowed), 7.5 ft left (flowed), 5 ft left (weeping), and 2.5 ft left (flowed). These grout holes were drilled at a 40- to 45-deg angle to the concrete surface and slanted toward the joint with the holes starting 12 to 16 in. downstream of the joint. The purpose for this was to intercept the joint behind the water stop so that the surface of the joint would not have to be sealed to retain the chemical grout. To drill a 30-in.-deep hole required 45 min to 1 hr. This length of time was required frequent withdrawal of the bit to assist in the removal of the cuttings and the additional effort required to puncture the water stop. Bits were not vented to blow cuttings out of the hole.

74. Monolith joint 13/14 had the greatest flow (1 to 3 gpm) of those which were to be grouted. The water was flowing from the joint between 5 ft right to 2 ft left of the centerline of the crown. Six 30-in.-deep holes were drilled at 1.5 ft right (no flow), 3 ft right (flowing), 8 ft right (weeping), 2 ft left (weeping), 4 ft left (high flow), and 10 ft left (high flow). Four 16-in.-deep holes were drilled at 1 ft right (weeping), 1 ft left (flowing), 4 ft right (no flow), and 3 ft left (no flow). The 16-in.-deep holes were started at 6 to 9 in. from the joint and slanted with the intention of intercepting the joint in front of the water stop.

75. Monolith joint 4/5 was leaking at two locations; one at 3 ft right and the other at 7 ft left. Four holes were drilled to intercept the joint behind the water stop and were located at 2.5 ft right (flow), 5 ft right (no flow), 5 ft left (flow), and 8 ft left (flow).

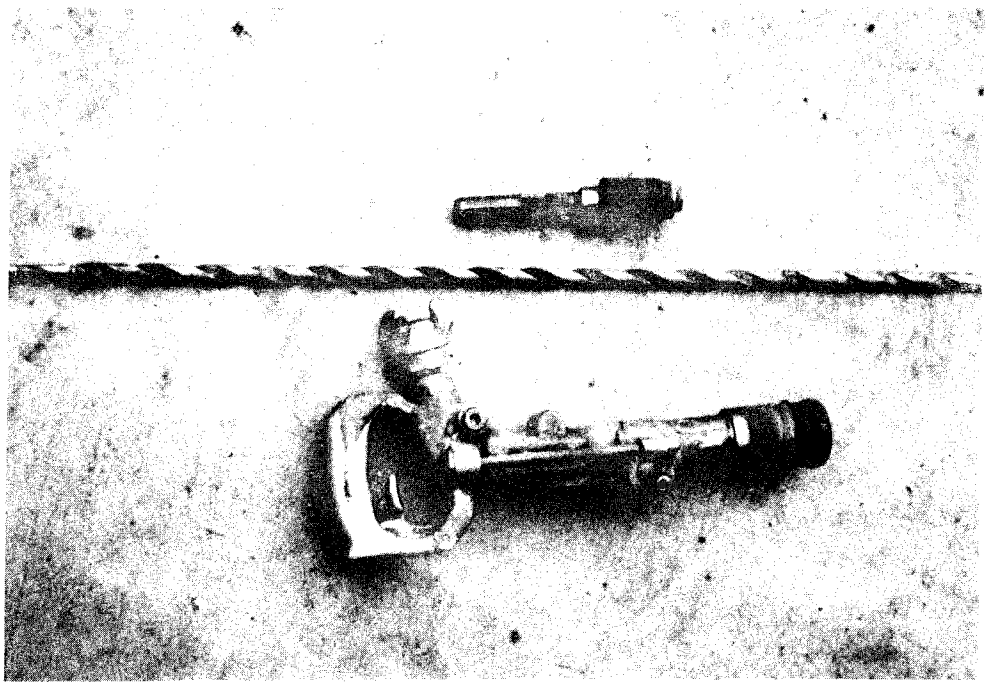

Figure 14. Rotary hammer drill, adapter, and bit, Blakely Mountain Dam, 1986

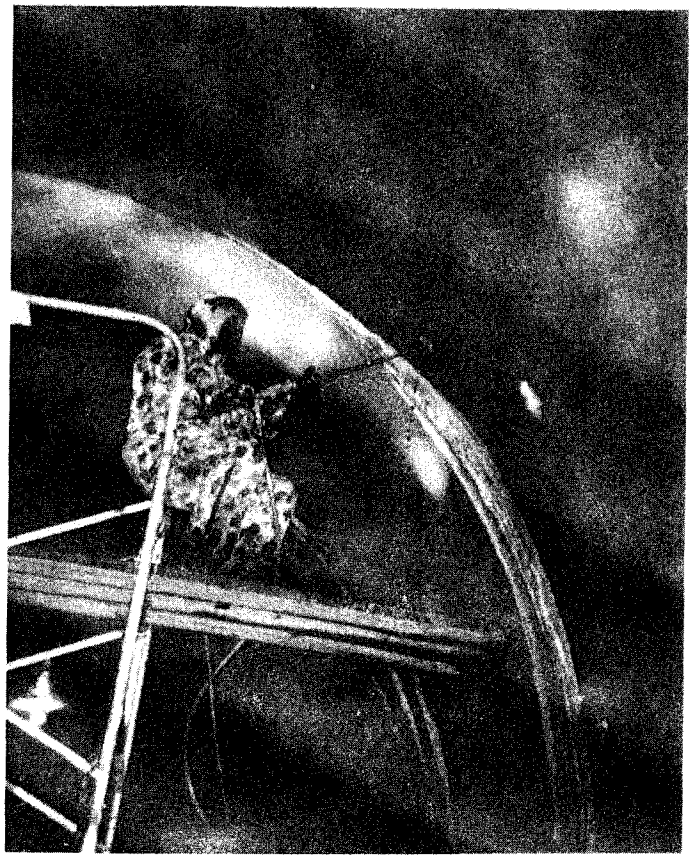

Figure 15. Drilling of grout hole, Blakely Mountain Dam, 1986

76. Monolith joint 3/4 was leaking at 7 ft right and 5 ft left. Grout holes were drilled at 8 ft right (flowing), 6 ft right (no flow), 4 ft left (flowing), and 8 ft left (flowing).

77. In 1978 cement grout had been injected into three cracks in two monoliths of the transition section. Two of these cracks were flowing at a combined rate of 1 to 3 gpm. Grout holes were drilled with the expectation of intercepting the cracks with at least two of every three holes drilled. However, only three of the six holes drilled had water flowing from them. A decision was made to stop drilling at this location until the material consultant arrived to give further instructions on intercepting cracks and the required spacing between holes to efficiently seal the cracks.

78. Minor weeping was observed at the junction of a crack and the first tunnel monolith joint downstream of the east service gate. Six holes were drilled. Three of the holes were in the vertical wall and were located 1 ft, 2.5 ft, and 5 ft from the ceiling with only the lower hole weeping. Three other holes were drilled into the area of the crack in the ceiling with none of them indicating interception of an open crack. All of these grout holes were 18 in. deep.

79. On 3 February 1986, the injection of the chemical grout was begun with a total of 15.0 gal of FLEX 44 and 15.0 gal of TACSS 020 NF being used. Approximately 3.0 gal of each grout were wasted due to loss through joints and loss due to the humid air in the tunnel reacting with the chemical grouts at their exposed surface in opened pails. The grout was injected into the joints and cracks through a packer patented by Avanti International, another chemical grout supplier (Figure 16). These injectors were possibly not the best to use with hairline cracks since they required 400- to 600-psi pressure to open the grease fittings. A modified Graco Airless President Hydra-Spray Pump, capable of spraying heavy viscous materials, was used to inject the chemical grouts (Figure 17). The spray gun modification consisted of a high-pressure pipe cross onto which was connected a dump valve, a 3,000-psi fluid gage, and a 3-ft-long pressure hose with a grease fitting connector (female) on the free end. A ground wire was connected to the pump for a positive ground, in accordance with the pump literature for use with paint. The ground wire for the grout pump was run from the tunnel outlet to monolith 1 and up the intake structure by the project personnel.

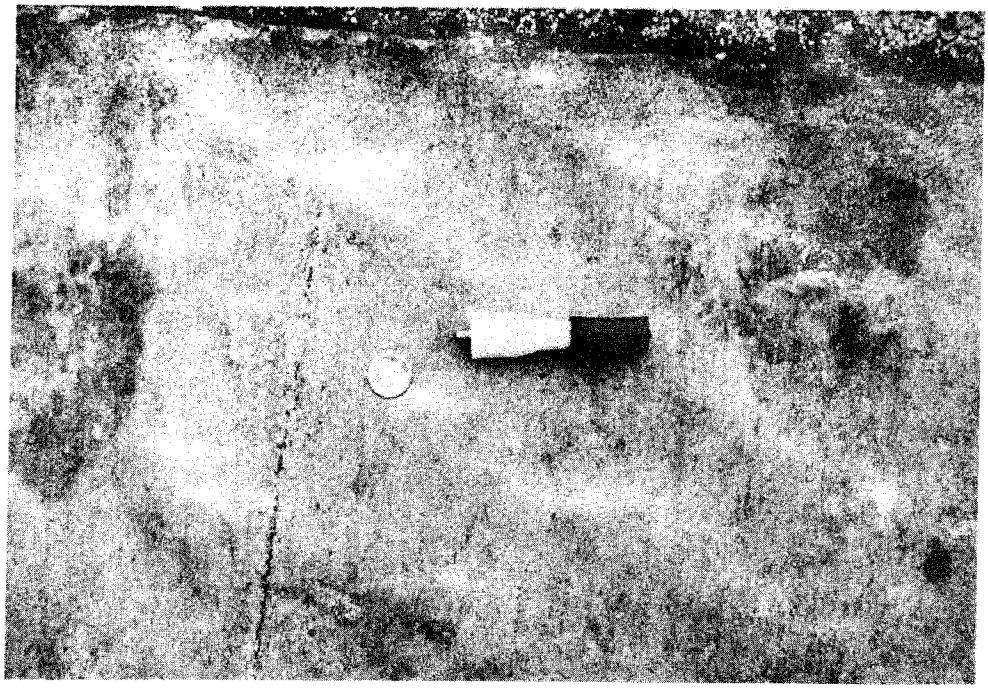

Figure 16. Injection packer, Blakely Mountain Dam, 1986

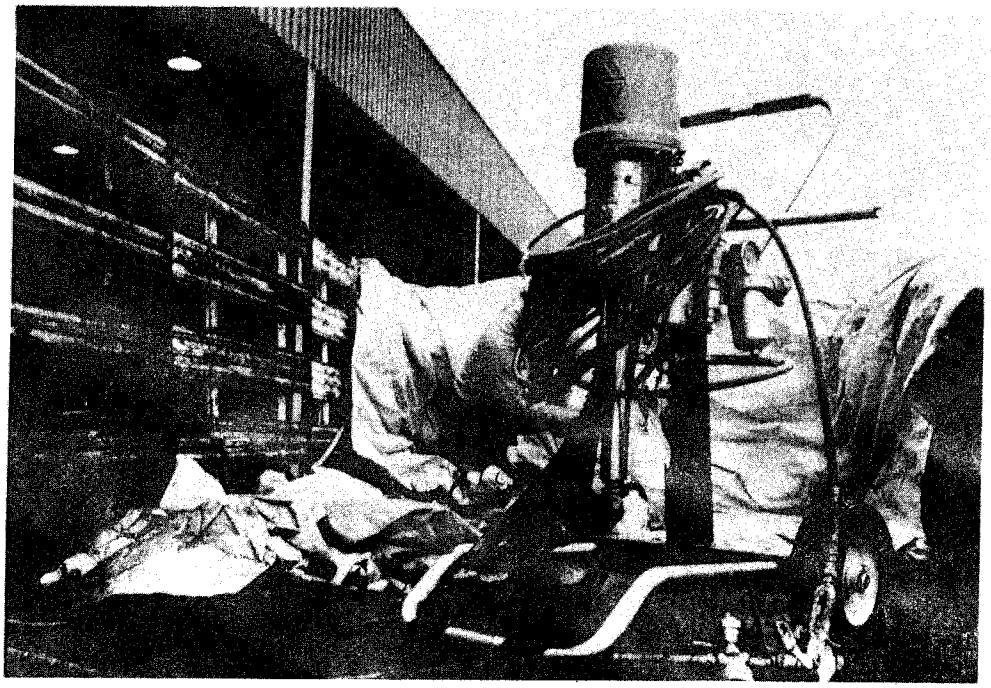

Figure 17. Grout pump, Blakely Mountain Dam, 1986

80. The Graco pump ordered was to have teflon seals substituted for the leather seals in the liquid pump. The pump received did not have the requested seals. As a result, the pump failed causing a loss of 1 day of work while efforts were made to obtain replacement parts. Also, the female connector to the grease fitting had a seal that was not compatible with solvents. The grout crew needed six to eight of these females connectors at the start of the job. One or two connectors were replaced each day depending on how many times the pump was flushed out with a cleaner.

81. It was determined that between 400- and 600-psi pressure was required to open the grease fitting in the injectors. This pressure was recorded on the injection gage when allowing the grout to flow through the injectors held over a waste pail. The external water pressure was determined to be approximately 90 psi. A maximum injection pressure of twice the external water pressure (180 psi) plus the pressure required to open the injector was used as a cutoff pressure when there was no grout take. If the grout-take was decreasing and the leak was not stopping, reaching this maximum pressure was an indication of a plugged port and the need to move to the next injector to seal off the leak.

82. Grouting with FLEX 44 was started at joint 20/21 with the injector located at 7.5 ft left of the crown centerline. Approximately 3.0 gal of FLEX 44 which had 2.5 percent accelerator (by volume) added were injected. A white coloration of the water seeping from the joint was observed at the beginning of the grouting followed by a decrease in the water flow from the joint. There was no grout take at the injector located at 5 ft right of the crown centerline. Grouting was moved to the injector located at 7.5 ft right where 0.5 gal was injected. The grouting was moved to the injector located at 2.5 ft left where another 0.5 gal was injected until a popping sound within the joint and adjacent cracks was heard. This sound was later determined to be the joint opening up due to the excessive hydraulic pressure in the joint created by the grout filling up the joint since there were minimum voids outside of joint. The popping was speculated to be caused by the unbonding of the calcite deposited in the joints and cracks. Thirty minutes after the grouting of this joint was completed some FLEX 44 was observed at the springline on the right side of the tunnel with only slight weeping occurring from a spot 5 ft left of the crown centerline and at the springline right.

83. At monolith joint 13/14 grouting was started 3 ft right of the crown centerline (30-in.-deep hole) then moved to 2-ft left (30-in.-deep hole) injecting 1.0 gal of FLEX 44 grout in each injector which stopped the flow from crown of the tunnel. However, the point of exit of the seepage moved down the right side of the tunnel to approximately 5 ft right of the crown centerline. The grout mixture was changed to 50-percent FLEX 44 and 50-percent TACSS 020 NF with 2.5 percent accelerator. Approximately 2.5 gal of FLEX 44 left from the day before, were mixed with 2.5 gal of TACSS 020 NF. One gallon of FLEX 44 was not usable due to the humid air in the tunnel reacting with grout in the open pail overnight. While starting the pump it was observed that the pump was not priming properly. The manufacturer was contacted and it was learned that the pump had not been modified as requested. The grout crew was able to inject 3.0 gal of TACSS/FLEX into the joint before the pumping operation was terminated. The seepage from the joint was stopped except for minor weeping. A summary of the TACSS/FLEX grout take at this joint, in the order it was injected, is as follows: 1 ft right (16-in.-deep hole), 0.5 gal; 1 ft left (16-in.-deep hole), 1.0 gal; 4 ft left (30-in.-deep hole), 1.0 gal; and 10 ft left (30-in.-deep hole), 0.5 gal.

84. On 5 February 1986, no grouting was performed due to the leather seals in the pump binding the fluid pump piston. The replacement Teflon seals were received at 5 p.m. The grout pump was repaired on 6 February 1986 and the crew was back grouting by 10 a.m.

85. Grouting was resumed at joint 4/5 with TACSS/FLEX mixture with 2.5 percent FLEX accelerator. Injection was started at 2.5 ft right where 3.0 gal of the TACSS/FLEX mixture were placed. Two additional 18-in.-deep holes were drilled at 1 ft right (flow) and 4 ft right (flow) and the joint was sealed with Waterplug from the crown centerline to 6 ft right. Injection was then continued at 4 ft right, where 0.5 gal TACSS/FLEX was injected, and at 1 ft right, where 0.5 gal of TACSS/FLEX was injected. This quantity of grout nearly stopped the seepage on the right side of the tunnel.

86. A switch was made to TACSS 020 NF to obtain a more rapid gel time with the maximum dosage of 10 percent accelerator. Injection of grout was moved to 8 ft left where 8.0 gal of grout were injected with 2.0 gal coming through joint (Figure 18). The left side of the tunnel started to seal and the injection of grout was moved to 5 ft left where 2.0 gal of grout were

Figure 18. Injection of monolith joint 4/5 with chemical grout leaking out of joint, Blakely Mountain Dam, 1986

injected. The grouting did not completely stop the seepage; however, it was decreased.

87. Grouting of monolith joint 3/4 was started at 8 ft right of the crown centerline where 2.0 gal of TACSS/FLEX was injected and then the grout injection was moved to 8 ft left. At 8 ft left the grout crew injected 1.0 gal and then the grout injection was moved to 4 ft left. At 4 ft left the grout crew injected 1.0 gal TACSS/FLEX grout before a popping sound within the joint was heard. The maximum grout pressure was not reached; however, excessive hydraulic pressure within the joint was reached. The grouting was stopped. The seepage from the joint was not completely stopped.

88. An effort was made to reduce the seepage from the two cracks in the lower gooseneck monoliths. A total of 0.5 gal of TACSS/FLEX mixture and

0.5 gal of FLEX 44 were injected with little loss of grout. However, the seepage from the cracks did not decrease. The consultants and district personnel concluded that additional grout holes would be needed to create an effective seal at both cracks. The grout crew proceeded to the intake structure with plans to return on 7 February 1986 to complete grouting of these cracks.

89. The grout crew injected approximately 0.5 gal of FLEX 44 and 0.5 gal TACSS/FLEX into the joint at the intake structure. During the grouting the exit point for the leakage moved down the joint. Grouting was stopped for the day with plans to drill additional holes to complete the seal at the intake structure.

90. A decision was made by consultants and district personnel to discontinue grouting of the tunnel until later in the summer to give the tunnel a chance to stabilize under the new pressure conditions created by the remedial grouting. The high injection pressures along with a reduction in leakage caused a localized increase in hydrostatic pressures. This increase caused leakage to appear at other locations such as adjacent openings, as was the case at joint 4/5 (Figures 19 and 20). Monolith 19 and the right springline joint of monolith 12 were not grouted since experience on the job indicated that the Graco pump was not the proper pump to perform the grouting of hairline cracks. A hand piston pump was thought to be more suited for this type of grouting.

91. After all grouting operations were completed, it was estimated that the overall leakage in the tunnel had been reduced by 75 percent. Only minor weeping was observed from joint 20/21 at approximately 5 ft left of the crown centerline and at the right springline. The flow from joint 13/14 had been nearly stopped; however, the flow had increased to 0.5 gpm by 7 February 1986. No observable change in the seepage from the springline at monolith 12 or the crack in monolith 19 had occurred since the beginning of the grout injection on 3 February 1986.

92. The flow from joint 4/5 had been nearly stopped on 6 February 1986. However, this joint was leaking again at 3 ft right of crown centerline. Also additional weeping was observed from cracks in the crown of both monoliths 4 and 5 (Figures 19 and 20). The seepage at joint 3/4 had increased some from 6 February 1986 with the seepage occurring at 3 to 5-ft right of the crown

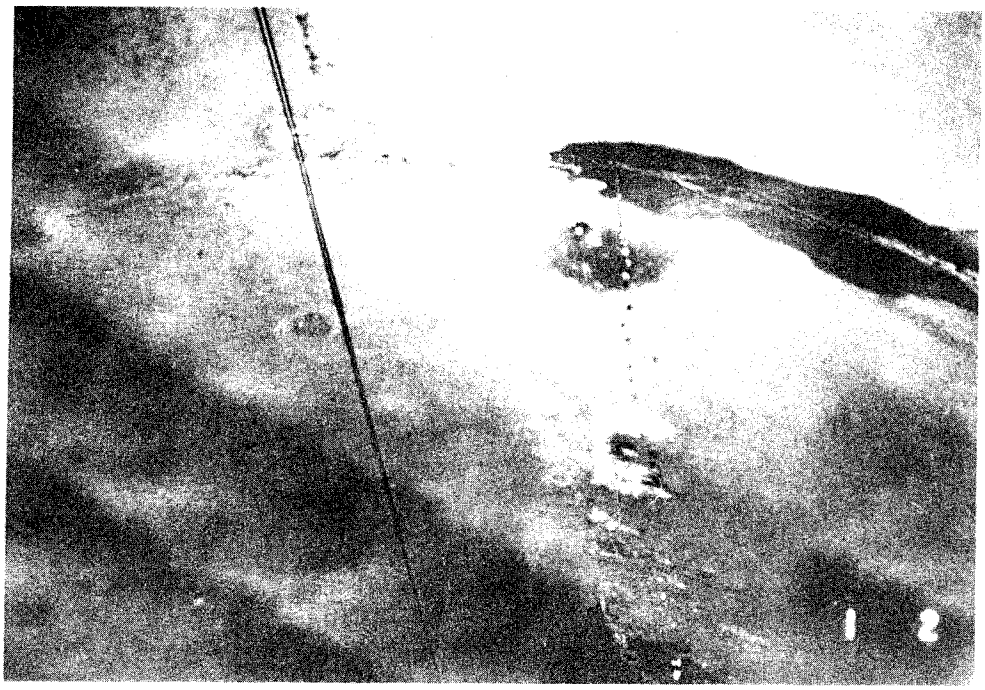

Figure 19. Monolith joint 4/5 before grouting, Blakely Mountain Dam, 1986

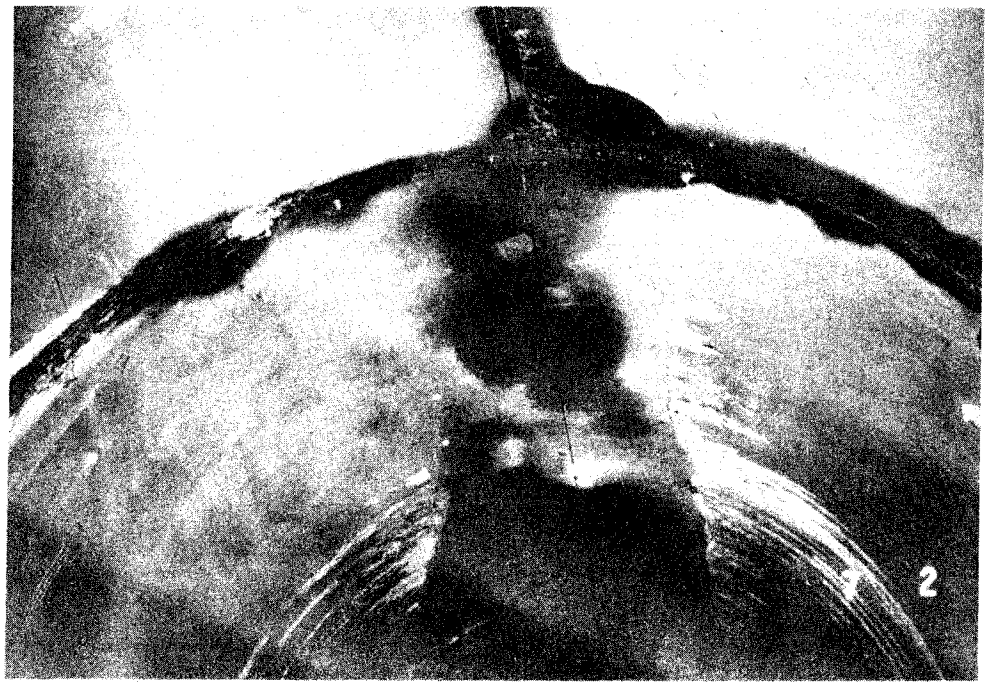

Figure 20. Monolith joint 4/5 after grouting, Blakely Mountain Dam, 1986

centerline. The seepage from the joints and cracks in the intake structure had not increased. However, the joints and cracks within the gooseneck monoliths seemed to be weeping more than prior to the remedial grouting. An inspection of the tunnel in September 1986 showed that the leakage had decreased with only minor seepage noted at joint 4/5.

Buckhorn Lake Dam

Background

93. Buckhorn Lake Dam was completed in 1960 and is located in southeastern Kentucky on the Middle Fork Kentucky River near Buckhorn, Kentucky, in the Louisville District. The project consists of an earthfill dam, a controlled outlet works along the base of the abutment, and a variable width, gate controlled spillway through the right abutment. The dam embankment is 1,020 ft long, 160 ft high, and 30 ft wide at its top (elevation 877 ft) and consists of a compacted random rock with a central impervious core. The spillway has a crest elevation of 820 ft with four 33- by 23-ft vertical lift gates providing total flood control storage to elevation 840 ft. The outlet works consists of a control tower (dry type), with three 5.5- by 11-ft hydraulically operated vertical slide gates (invert elevation 724 ft), two 24-in.-diameter low-flow bypass pipes, a 14- by 14-ft semielliptical main conduit, and a stilling basin.

94. In 1962, leakage was reported in the leftside wall of the main conduit at monolith joint 2/3, approximately 40 ft downstream of the main gates. Leakage from the joint gradually increased to the point that routine monitoring was required in 1980. An attempt to stop the leakage was made by district personnel in 1981 using Water Plug, a fast-setting hydraulic cement manufactured by Thoro Systems Products of Miami, Florida. The rate of leakage was such at the time of the repair that the product did not have time to set before being washed out of the joint. The rate of flow at that time was approximately 5 gpm. The rate of flow continued to increase with a flow rate reaching peaks of 15 to 20 gpm in 1986.

Repair

95. Remedial grouting of monolith joint 2/3 was performed by district personnel in late August 1986 using TACSS-020 NF, a chemical grout manufactured

by DeNeef America, Inc. of St. Louis, Michigan (Lewis and Brockman 1987). TACSS-020 NF is a hydrophobic polyurethane that reacts with water to form a tough, rigid closed-celled foam. Oakum, a loosely twisted hemp impregnated with tar, and Preco Plug, a fast-setting cement, were packed into openings to divert leakage through valve controlled ports. The Preco Plug is manufactured by Fosrco-Preco Industries of Plainview, New York. Injection ports were fabricated by district personnel. Each port consisted of a 1/2-in.-diameter copper tubing with cutoff valve and appropriate fitting to facilitate attachment to the delivery hose. Grout was pumped into ports using a hand operated, 500-psi capacity, volume delivery bucket pump (Alemite pump, model 7181).

96. For safety reasons, no electrical power cords or motors were used in the conduit. Lighting was provided by battery-powered lights and portable gas lanterns. Compressed air was supplied to the work area by an air hose through an air vent in the conduit from a compressor located outside on the service bridge.

97. Two holes in which ports were installed existed prior to this repair effort. One of these holes was drilled in 1962 during a repair made to cracks and honeycombed areas in the conduit floor. This hole penetrated approximately 15 in. into the concrete, 9 in. beyond the water stop. The other, just upstream of the water stop, was an irregularly shaped hole of less than 4-in. penetrable depth in 1981. The depth of the irregularly shaped hole increased over the next several years to the extent that a piece of 1/2-in.-diameter copper tubing could be inserted to a depth of 18 in. or more in 1986.

98. Three, 3/4-in.-diameter holes were drilled to reduce or relieve water pressures during the sealing of injection ports. These holes (Figure 21) were drilled to the depth of the waterstop (approximately 6 in.), but not penetrating it. Port tubings were inserted near full depth of each of the five holes to provide a longer flow path, thereby, increasing the time in which the grout had to set before exiting the concrete. Oakum was packed into cracks and holes around ports until the majority of leakage was flowing through ports (Figure 22). This process had to be done in a persistent manner, since pressure increased as each small quantity of flow was shut off by the packing of the oakum. At times one closure would cause a previously sealed area to seep again. After the packing of the oakum was completed, valves were closed to check effectiveness of packing (Figure 23). Preco Plug

Figure 21. Drilling of injection port using pneumatic drill, Buckhorn Lake Dam, August 1986

Figure 22. Packing oakum around injection ports, Buckhorn Lake Dam, August 1986

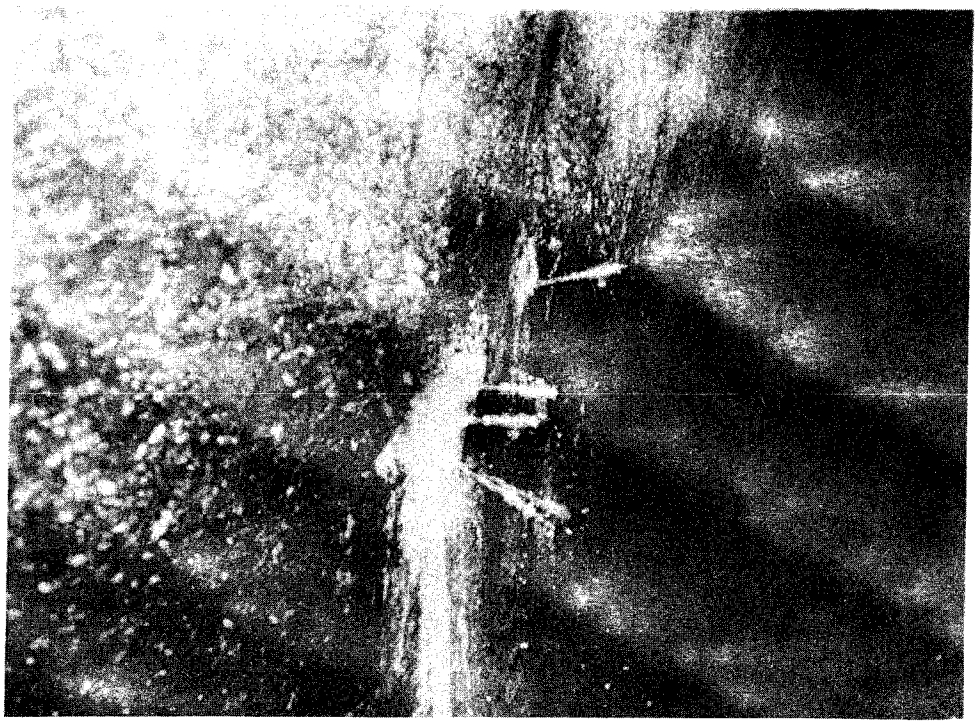

Figure 23. Valves closed to check the effectiveness of packing, Buckhorn Lake Dam, August 1986

was packed into the same areas to provide additional resistance against blowout.

99. After the Preco Plug had set for about 20 min, 2-1/2 gal of TACSS-020 NF was mixed with 5 percent of C-852 accelerator. With the other four valves closed, grout was pumped through the open valve of the port occupying the nondrilled hole (Figure 24). There were a number of small seeps that existed prior to the start of the grouting operation. After 2 to 3 min of pumping these seeps stopped with grout solidifying on the interior wall at some of the seep locations. After all of the approximately 2-1/2 gal of grout had been pumped, the port valve was closed to allow time for grout to react and solidify. Another 2-1/2 gal of grout was prepared and the pump connected to the single port downstream of the joint. The pumping of grout into the port was initiated, but no grout was taken. With valves closed and hose removed, the previously injected grout was allowed to gel for an additional 30 min. Ports were then cut off slightly below the wall surface and projecting oakum removed. Preco Plug was mixed and spread over the work area to form a protective seal (Figure 25).

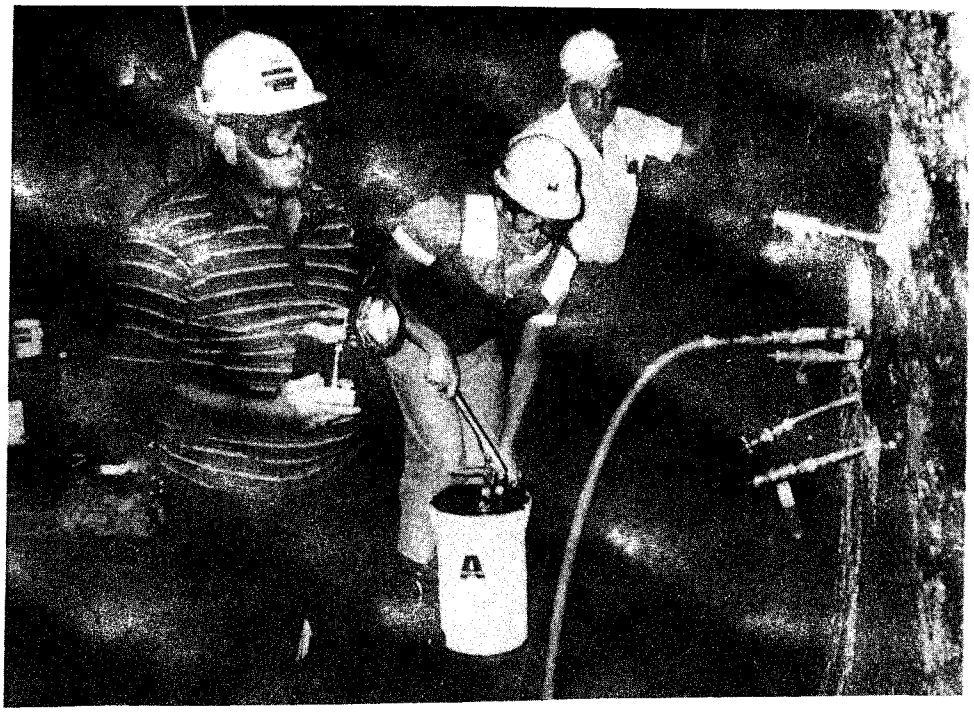

Figure 24. Injection of TACSS-020 NF mix into port occupying nondrilled hole, Buckhorn Lake Dam, August 1986

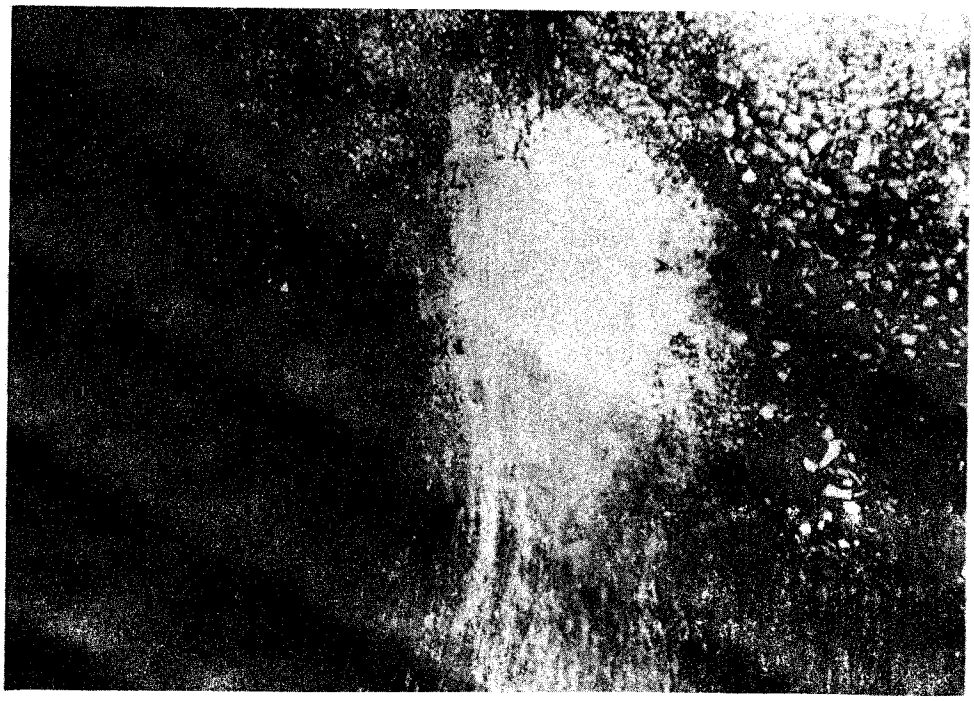

Figure 25. Surface after repair, Buckhorn Lake Dam, August 1986

100. The work was inspected after about 2 days of normal discharge operation with no leakage noted.

101. The estimated cost of the repair was as follows (excluding time and travel expenses incurred by Engineering):

Material	$ 500
Pump	200
Labor	1,500
Total	$2,200

Cave Run Dam

Background

102. Cave Run Dam was completed in 1974 and is located in the Louisville District on the Licking River, 173.6 miles above its junction with the Ohio River and 4 miles upstream from Farmers, Kentucky. The principal projects features include an earthfill and rockfill embankment, a concrete outlet works, and an uncontrolled, 650-ft-long spillway in the left abutment. The embankment is 140 ft high, 2,700 ft long and consists of earthfill and rockfill over a compacted impervious core. The outlet works is located along the base of the left abutment and consists of an intake control tower, a 15-ft-diameter outlet conduit, and a stilling basin. Flow through the outlet works is controlled by two hydraulically operated 6.75- by 15-ft, sluice gates and two 24-in.-diameter, low-flow bypass pipes, all located in the control tower.

103. During a 1980 inspection, erosion damage was observed in the surface of the concrete in the two gate passages and transition of the outlet works. In the right passage, cavitation damage had occurred immediately downstream of the bypass outlet (Figure 26). The eroded area was approximately 6 ft long, 3 ft wide, and 8 in. deep at the deepest point with exposed reinforcing bars. In the left passage, cavitation damage (Figure 27) had also occurred immediately downstream of the bypass outlet and was approximately 4 ft long and 2 ft wide with a maximum depth of about 4 in. Abrasion damage had occurred in the transition between end of the passages and the beginning of the conduit. The damage consisted of two eroded areas downstream of the

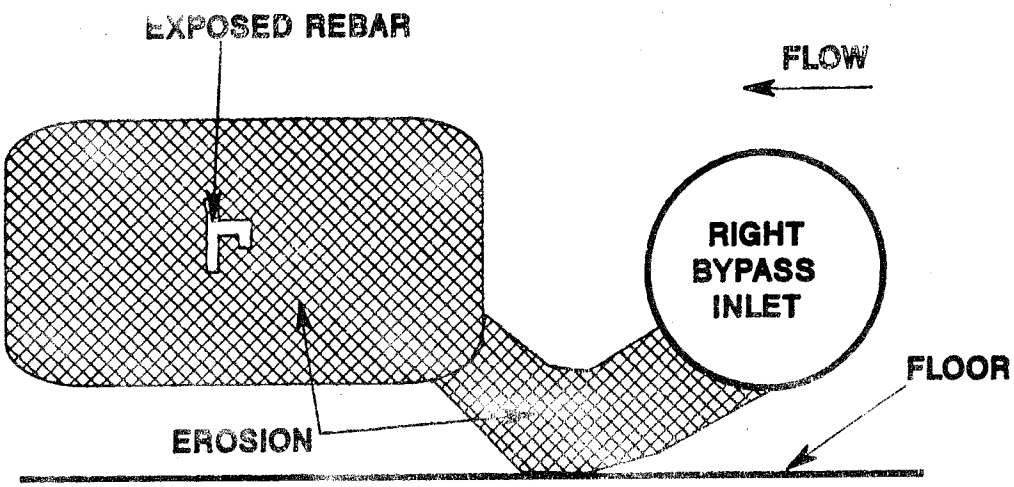

Figure 26. Sketch of cavitation damage in wall of right passage, Cave Run Dam, 1982

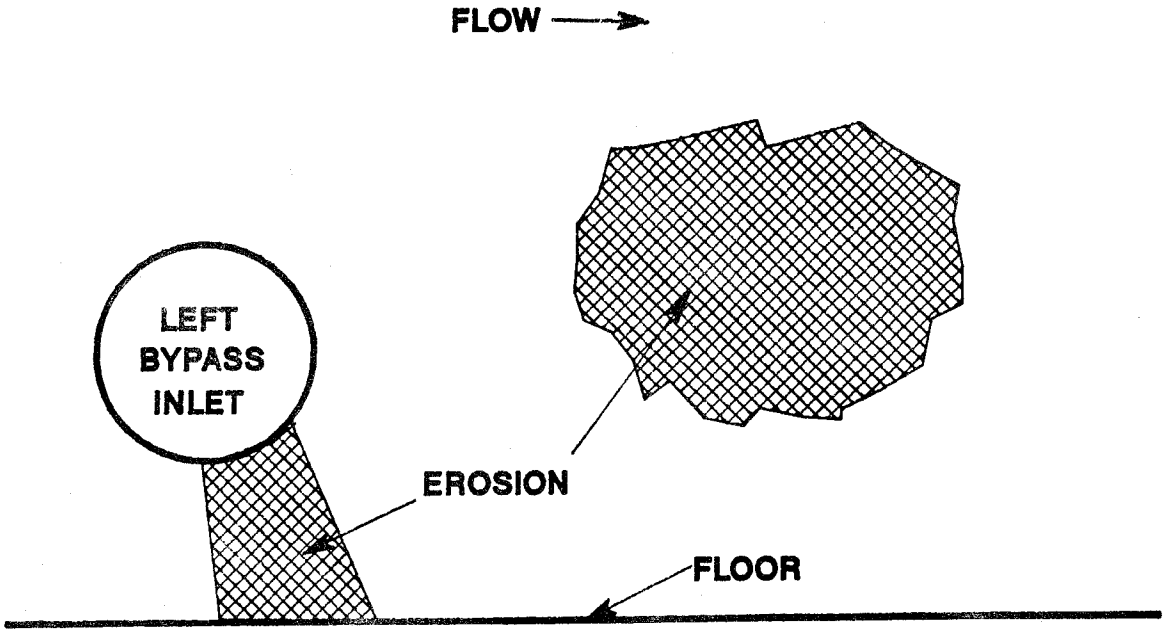

Figure 27. Sketch of cavitation damage in wall of left gate passage, Cave Run Dam, 1982

nose of the pier partition that separates the two passages. One area was located to the right of the pier nose and was eroded 2 to 3 in. deep with a length of 2 ft. The other area was located to the left of the pier and was eroded to a depth of 2 to 3 in. with a length of about 1 ft. A sketch of erosion damage in the transition is presented in Figure 28.

Repair

104. 1982. Cavitation-erosion-damaged concrete in the right gate passage of the outlet works was repaired in November 1982 by district personnel using SikaTop 122, a two-component, prepackaged, acrylic mortar, manufactured by Sika Corporation of Lyndhurst, New Jersey*. The repair was terminated after the mortar failed to set. Inspection of the product containers revealed that no expiration dates were present. It was suspected that the shelf life for the mortar used had been exceeded and, therefore, was the cause of the product's failure to set. The Sika Corporation agreed to replace the undated material. It should be noted that all employees involved complained of minor headaches caused by product fumes and lack of ventilation.

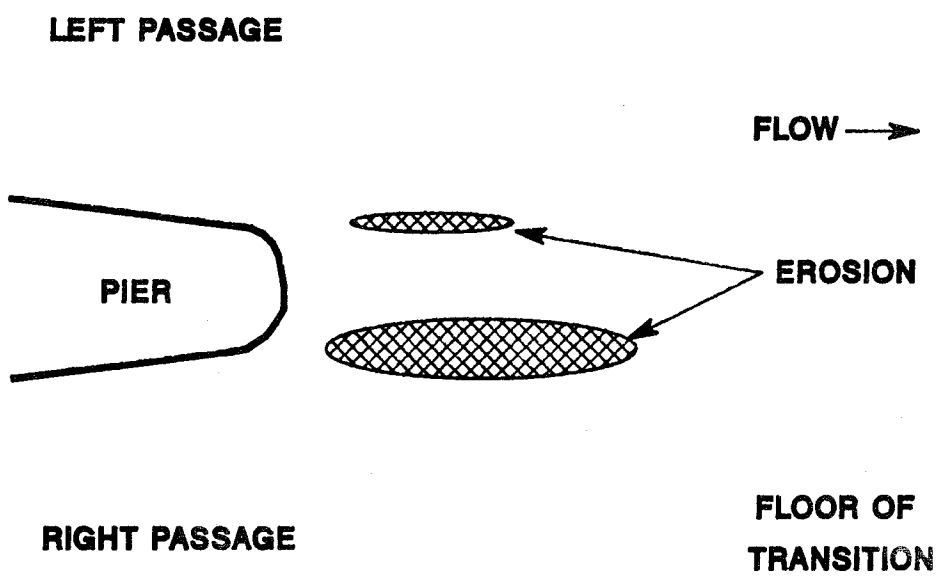

Figure 28. Sketch of abrasion damage in floor of transition, Cave Run Dam, 1982

* B. Bryson. 26 November 1982. "Attempt to Repair Conduit Cavitation - Cave Run Lake," Disposition Form, US Army Engineer, Louisville, Ky.

105. 1983. Erosion-damaged concrete in the gate passages and transition of the outlet works was repaired by project personnel using SikaTop 123 and Preco Plug*. The SikaTop 123 is a polymer-modified, cementitious, two-component, fast-setting mortar, manufactured by Sika Corporation of Lyndhurst, New Jersey. The SikaTop 123 was recommended instead of the SikaTop 122 because it was thicker and more suitable for vertical surfaces. The Preco Plug is a quick setting cement designed to seal leaking joints and cracks. It is manufactured by Fosroc-Preco Industries of Plainview, New York.

106. Preparation for the work included placing sandbags around the right bypass inlet and sealing them with drillers mud. Sandbags were also placed at the mouth of the bypass with a pipe placed to carry leakage from the bypass gate away from the work area. Lighting for the job was provided by fluorescent lights of the type designed for recreational vehicles. The lights were mounted on stands fabricated from rebar. Power for the lights was provided by a 12-v battery. A battery charger was placed on the top floor of the tower and a cord dropped through the tower's ventilation shaft and connected to the battery to offset the draw of the lights.

107. Cavitation-erosion damage in the right passage was repaired using the SikaTop 123 product. A stiff brush was used to clean the surface of the concrete. A work force of seven men was used to make the repair: two men to keep water bailed out of the work area, three men to do the mixing, and two men to place the mortar. A wooden box and paddle had been fabricated for the mixing. The mortar was mixed and placed in small batches. Four units of SikaTop were needed to complete the repair. Water was not allowed to enter the repair area until the mortar had set.

108. Cavitation damage in the left passage and abrasion damage in the transition were repaired using Preco Plug. Dewatering of the work area was not required because of the products ability to set in running water. A stiff brush was used to clean the concrete surface. A work force of three men was used to do the mixing and placing. The areas of abrasion in the transition were repaired with Preco Plug under about 3 in. of water. One 5-gal bucket of Preco mixed with one half sack of portland cement was needed to repair the

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* B. Bryson. 26 July 1983. "Repair of Conduit Cavitation - Cave Run Lake," Disposition Form, US Army Engineer District, Louisville, Ky.

area on the left passage (where the cavitation damage was about one half as large in area as on the right passage). Mixing was done in a wheelbarrow.

109. It was noted by district personnel that the Preco Plug sets much faster than the SikaTop 123. Some of the problems noted with using the SikaTop were: the product became easily diluted with water; harmful fumes were emitted; skin irritations were possible; precise measurements were required for small batches; and the 3-min maximum mixing time seemed to be too short a period. With the Preco Plug there was no waste involved due to water contact such as occurred with the SikaTop, and there were no fumes although a similar warning about skin irritation appeared on the instructions.

110. The cost of the SikaTop 123 was $45.90 per unit. The cost of the Preco Plug was $25.75 per 5-gal bucket. One 5-gal bucket of the Preco Plug is roughly equivalent to one unit of the SikaTop mortar.

111. A summary of time and cost involved in the repairs follows:

Item	Man Hours	Labor Cost	Material Cost
Equipment setup and gate operations	13.0	$166.53	------
Right side repair (SikaTop)	45.5	565.10	$183.60
Left side repair (Preco Plug)	7.0	51.72	28.75
Equipment removal and gate operations	13.0	173.52	------
	78.5	$956.87	$212.35

112. An inspection of the passages and transition in June 1985 revealed only minor deterioration of the featheredges of the repaired areas. However, it should be noted that flows since the repair have not been as severe as those that cause the erosion damage observed in 1980 and, therefore, the performances of the repairs were considered inconclusive at the time of this report.

Eau Galle Dam

Background

113. Eau Galle Dam was completed in 1968 and is located in the St. Paul District in west-central Wisconsin on the Eau Galle River, immediately upstream of Spring Valley, Wisconsin. The principal project features include a zoned

earthfill embankment with a central clay core, a reinforced concrete outlet works, and an emergency spillway with a concrete overflow section. The earthfill embankment has a crest length of 1,800 ft and a maximum height of 122 ft above streambed.

114. The outlet works (Figure 29) includes a morning glory intake, diversion and low-flow conduit and intake, horseshoe conduit, and a stilling basin at the horseshoe conduit outlet. The morning glory intake is a reinforced concrete gravity structure with an ogee crest section and trash rack at the inlet. The diversion and low flow conduit is 80 ft long with a trash rack at its inlet and a reinforced concrete gated intake structure at its outlet into the horseshoe conduit. The horseshoe conduit has a 9.75-ft equivalent diameter and consists of thirty-two 20-ft-long reinforced concrete monoliths. The thickness of the horseshoe and floor portions of conduit varies between 1.5 and 2 ft and 1.5 and 3.75 ft, respectively. The stilling basin walls and slab were designed as an integral U-shaped reinforced concrete structure.

115. Progressive longitudinal cracking (Figure 30) was observed in the crown of monoliths 11 through 16 of the horseshoe conduit during the first periodic inspection in October 1968. By 1978 the cracking was observed in monoliths 5 through 25. Measured crack widths in some monoliths were greater than the width of crack associated with yielding of the reinforcing steel. At that time the highest pool level obtained was 8 ft above morning glory crest. The maximum level possible is 80 ft above the crest.

116. A 1981 structural investigation of the horseshoe conduit (USAED, St. Paul 1981) found the as-constructed loading conditions to be significantly different than those used in the original design. It was concluded that the as-constructed loading conditions produced smaller horizontal restraining loads than those of the design and that the reduction in the horizontal loads decreased the load-carrying capacity of the horseshoe portion of the conduit to a level that resulted in the observed cracking.

Repairs

117. 1979-80. A trial section containing longitudinal cracking was injected in the winter of 1979-80 using materials and technique developed by Adhesive Engineering Company of San Carlos, California. The injection product was Concressive 1380, a two-component, low-viscosity epoxy-resin adhesive.

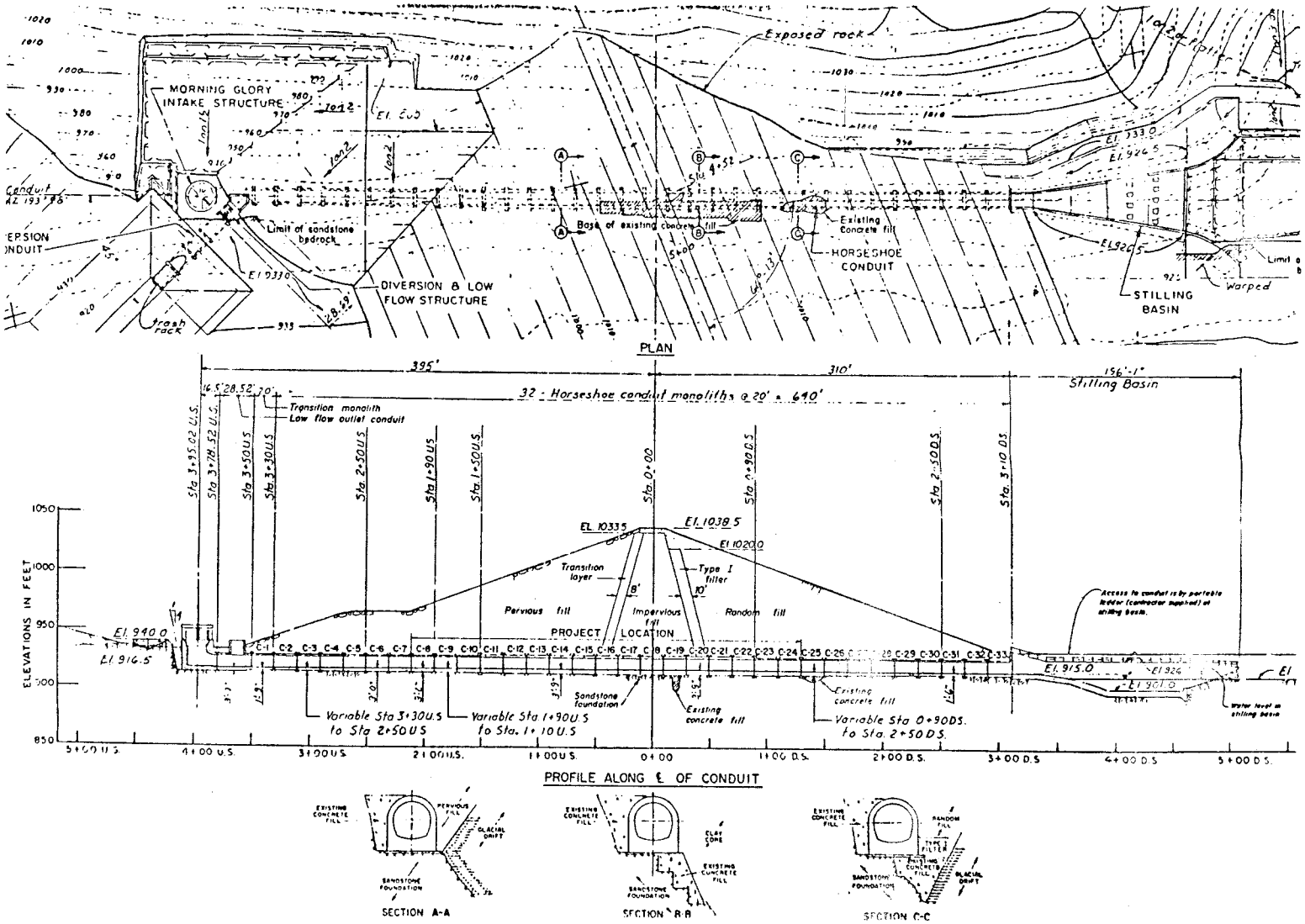
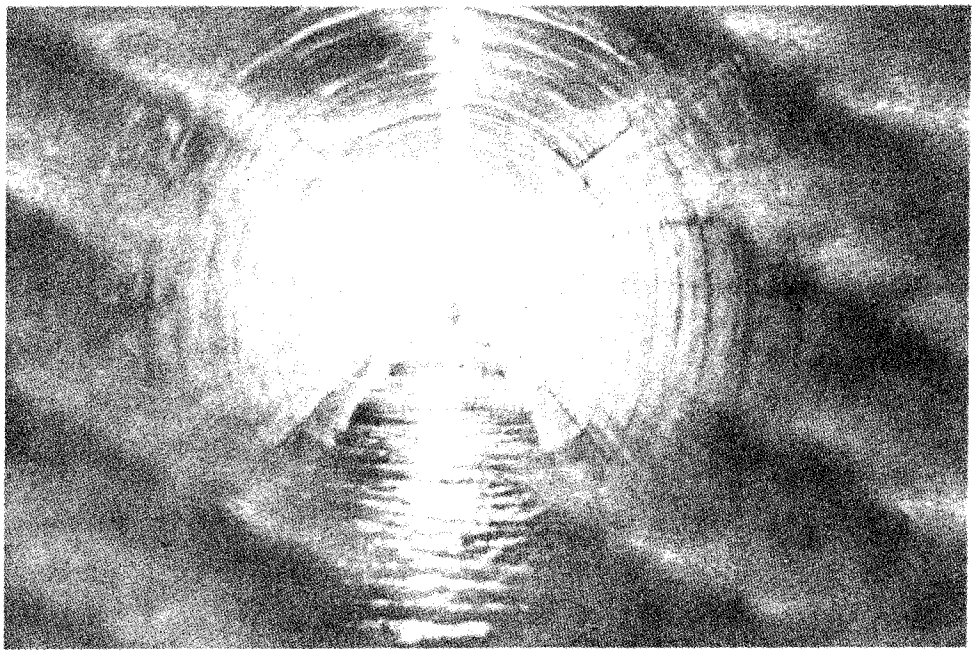


Figure 29. Eau Galle Dam outlet works



a. Longitudinal crack in crown

b. Delamination at reinforcement

Figure 30. Cracking in crown of horseshoe conduit, Eau Galle Dam

The injection was performed using a technique called the Structural Concrete Bonding (SCB) Process. The general repair procedure was to remove loose concrete and clean and seal surface areas containing cracking, drill injection ports into cracks, install packers into ports, inject ports in successive order and remove packers and seal ports after adhesive has set.

118. During the repair, difficulty was encountered in injecting sufficient quantities of liquid adhesive into cracks. This was believed to have been the result of the near 37° F temperatures in the conduit at the time of the repair causing a significant increase in the viscosity of the resin, thereby, limiting its ability to penetrate. Analysis of concrete cores taken from the repair area showed insufficient penetration of adhesive into the crack to warrant further use.

119. 1981. As a part of the 1981 structural investigation a composite concrete-steel liner was designed to reestablish the integrity of the conduit and prevent the collapse of the conduit under projected load increases. The liner was installed that same year in monoliths 8 through 24 of the conduit.

120. The composite liner (Figure 31) consisted of a 3/8-in.-thick steel-plate liner and a 5-5/8-in.-thick layer of nonshrink concrete pumped between the steel and the existing concrete. Steel studs 3/4 in. in diameter (Figure 31) were used to tie the existing concrete, nonshrink concrete, and steel plate together to form a composite structure. The end sections of the repair, monoliths 8 and 24 (Figure 32), consisted of a tapered composite liner of nonshrink concrete and epoxy grout abutted at its end with an epoxy grout liner. The tapered composite liner was integrated with the existing concrete by a ring of dowels located in the nonshrink concrete 2 ft from the joint at the end of the concrete-steel liner and by epoxy bonding agent at the interface.

121. The specifications for the nonshrink concrete required aluminum powder as the expansion agent; 3/8-in. maximum size aggregate; maximum slump of 4 in.; Type I, II, or III portland cement; and a minimum compressive strength of 4,200 psi. The epoxy grout specifications required the epoxy binder to be moisture insensitive and proportioned to no more than three parts white silica sand with one part binder. The epoxy binder specified was TE 2001 Supercrete Bonder manufactured by Technical Sealants and Adhesives of St. Paul, Minnesota.

122. In a few of the monoliths containing the most severely delaminated concrete, the damaged concrete in the crown of the conduit was removed and

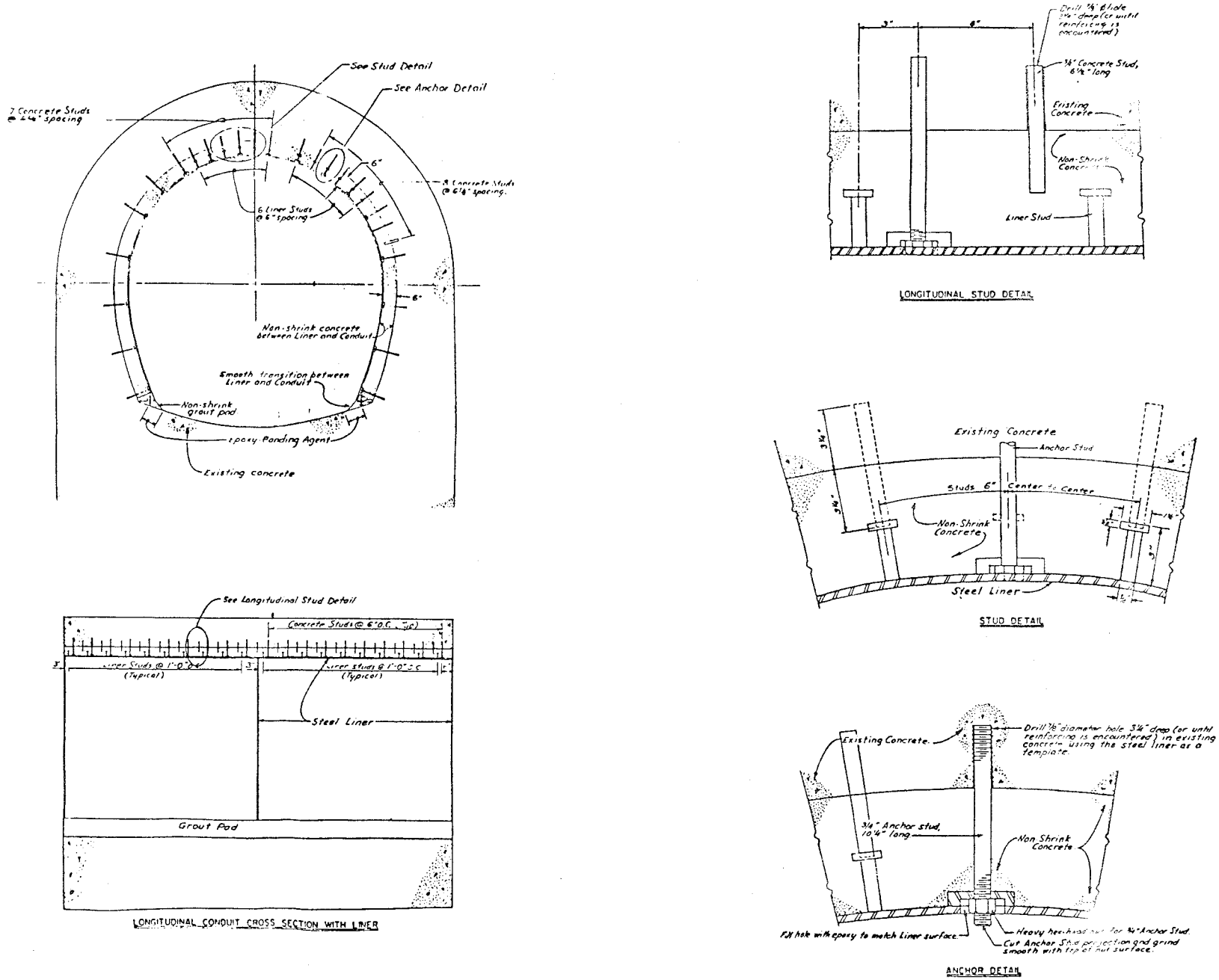


Figure 31. Anchor details for steel liner and nonshrink concrete fill, Eau Galle Dam

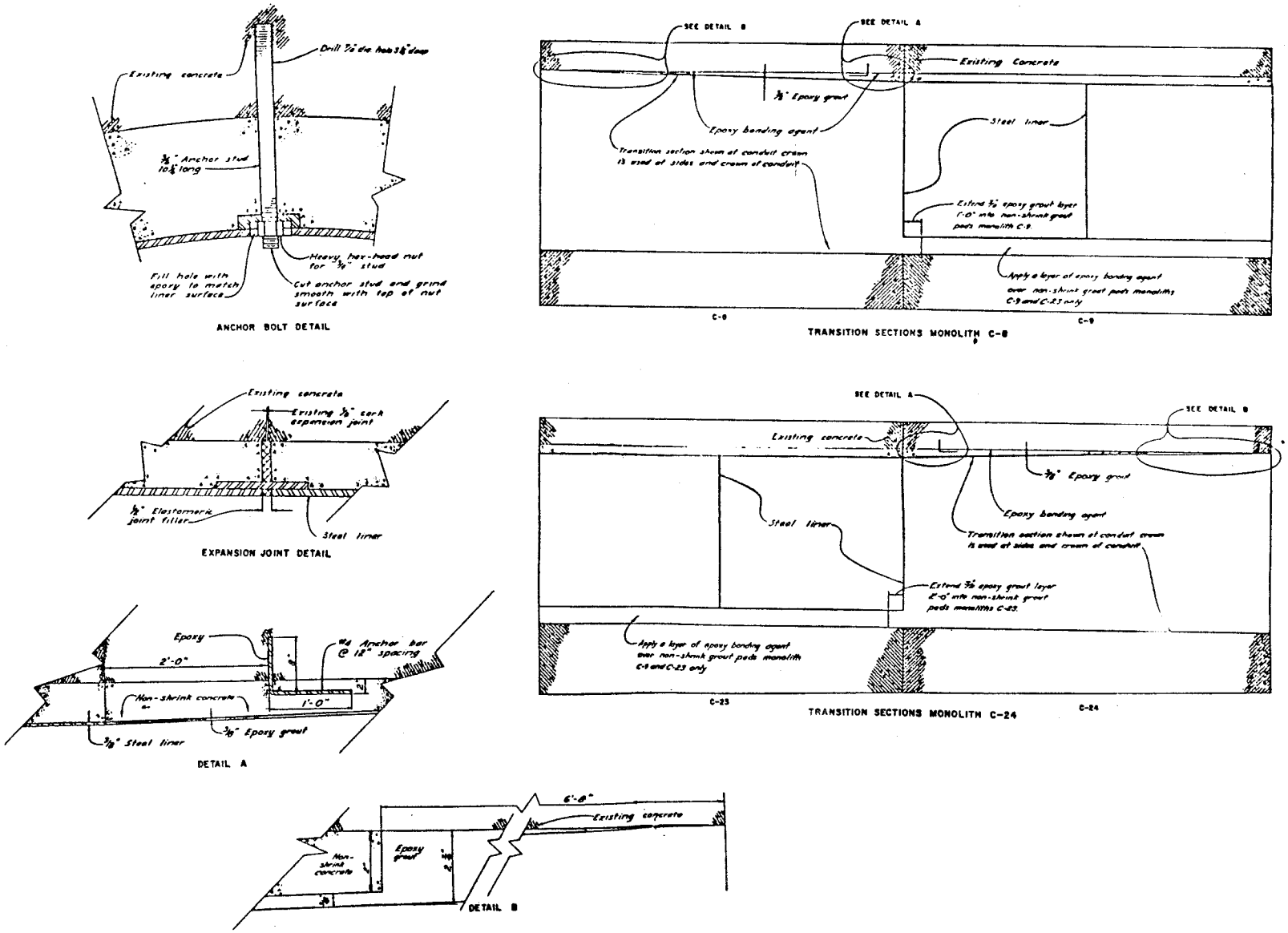


Figure 32. Details for transition sections of repair, Eau Galle Dam

replaced with a nonshrink grout prior to installation of the steel liner (Figure 33). The work was performed in reaches of 5 ft and required 80-ton jacks at the ends of each reach to support the ceiling during the repair. The specifications for the nonshrink grout required aluminum powder as expansive agent, No. 4 maximum size aggregate, Type III portland cement, and minimum compressive strength of 4,200 psi.

123. Conduit surfaces to be in contact with nonshrink concrete or epoxy grout were required to be cleaned by sandblasting or high-pressure waterjetting. Steel liner surfaces to be in contact with nonshrink concrete were required to be cleaned by wire brushing or by sandblasting within 24 hr of placement of the concrete. Each section of steel liner (Figure 34) was required to be aligned at the ends with adjoining liners to within 1/8 in. in both the longitudinal and circumferential directions. After installation of anchor bolts and end forms, nonshrink concrete was pumped between the existing concrete and the steel liner. Forms were required to remain in place for a minimum of 24 hr after placement. Concrete curing periods were specified as 7 days for Type I portland cement, 14 days for Type II, and 3 days for Type III.

124. During the repair, the conduit reinforcing steel and the liner were instrumented to monitor strains. As of July 1985, there has been a slight overall increase in the strains with some variations due to seasonal temperature changes. There have been no periods of high water in the reservoir since the installation of the liner; therefore, the performance of the liner under high water conditions is unknown at the time of this report.

Hidden Dam

Background

125. Hidden Dam was completed in 1975 and is located on the Fresno River about 20 miles north of Fresno, California, in the Sacramento District. The principal project features include a rolled earthfill embankment, outlet works, and spillway. The embankment is 184 ft high and 5,730 ft long and contains an impervious core. The outlet works consists of an approach channel, a combined intake and control tower, a transition section, an oblong reinforced concrete conduit, an exit structure, and return channel. The intake is located at the upstream face of the base tower with two separate intake passages provided through the base of the tower. Releases are controlled by two

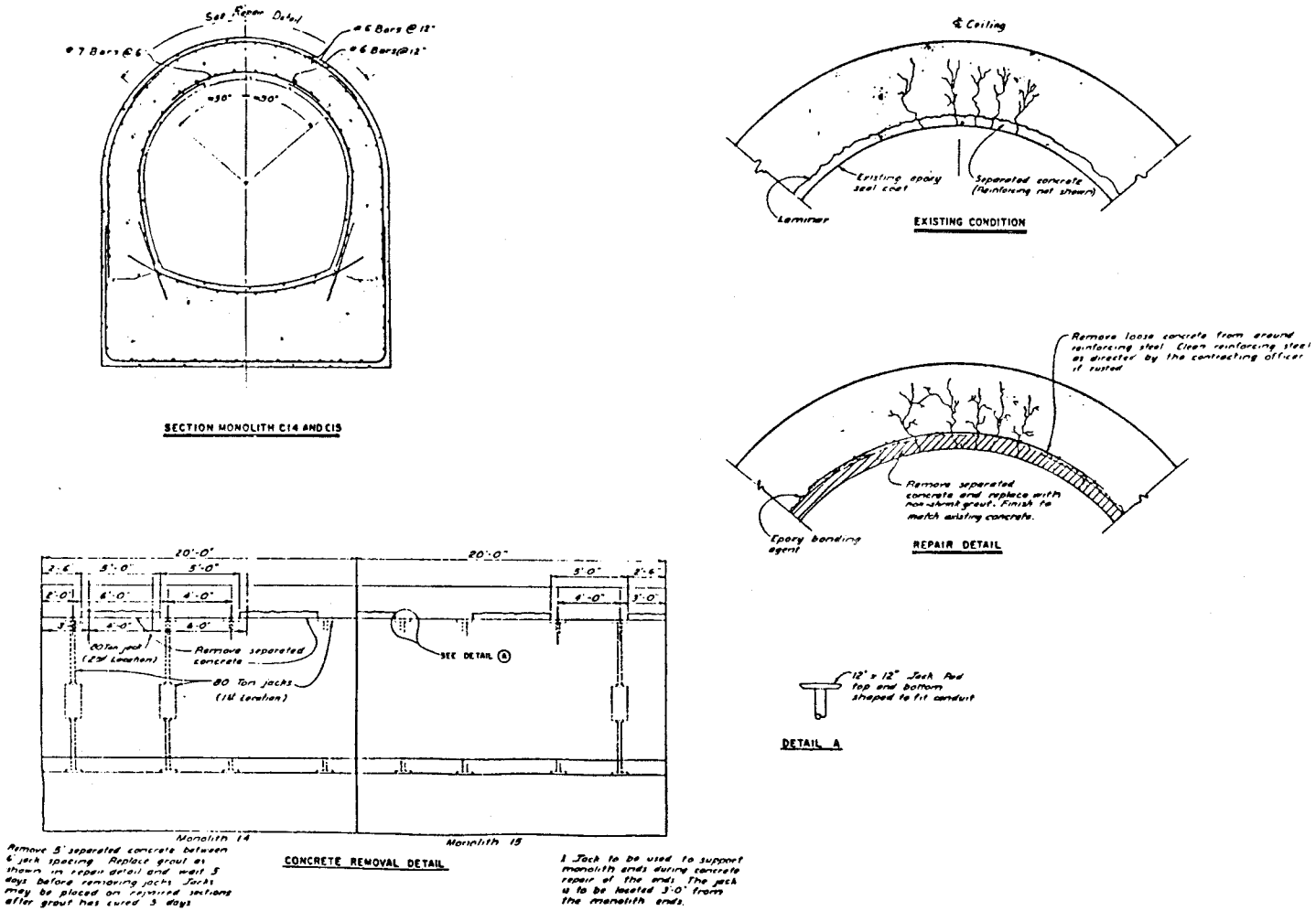


Figure 33. Details of removal and replacement concrete in crown of conduit, Eau Galle Dam

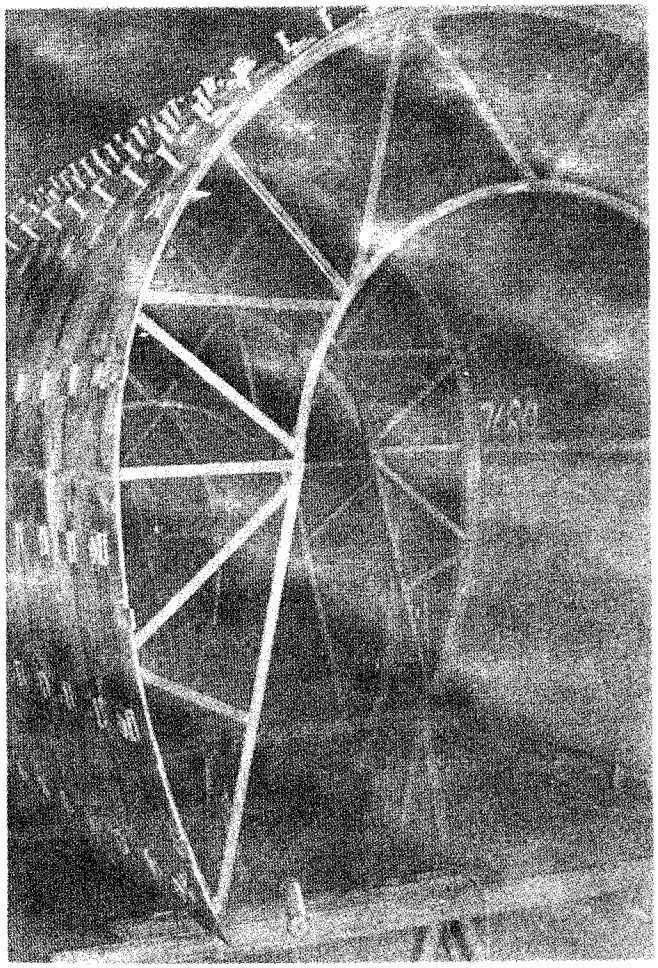


Figure 34. Steel plate liner, Eau Galle Dam

5- by 11.75-ft hydraulically operated service gates. The right service gate also has a 10- by 12-in. low-flow (piggyback) gate for use whenever very small releases are required. The spillway is located adjacent to the left abutment of the dam and has an unlined channel with a 575-ft concrete control sill at the crest.

126. Offsets in the adjoining concrete surfaces at the two farthest downstream joints of the outlet conduit were up to 1/4 in. at the invert. It was suspected that cavitation damage was possible at these joints due to the offsets.

Repair

127. In 1984, district personnel rented a high-speed, car buffer type grinder with a diamond stone. The higher surface at each of the two joints was ground to the level of the lower surface at the joint and cut back on a 1 to 12 slope away from the joint (Figure 35). This produced a smooth surface transition across the joints, thereby, reducing the possibility of cavitation damage occurring during high-flow releases.

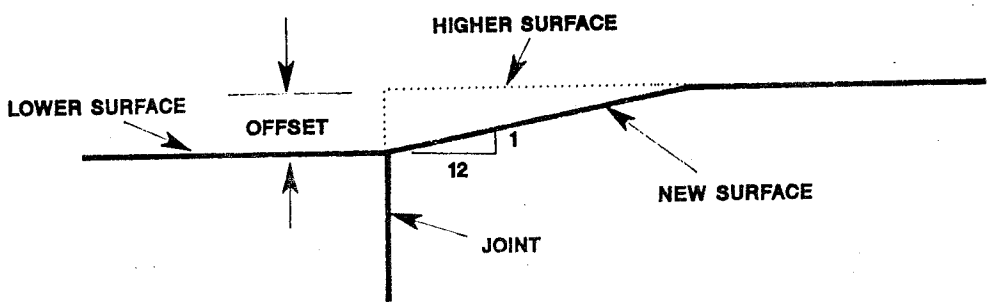


Figure 35. Sketch of repair of offset, Hidden Lake Dam

Isabella Dam

Background

128. Isabella Dam was completed in 1953 and is located northeast of Bakersville, California on the Kern River in the Sacramento District. The principal project features include a main dam, outlet works, auxiliary dam, power canal, and spillway. The main dam is a rolled earthfill structure 185 ft high and 1,695 ft long with an impervious core. The outlet works is located in the left abutment of the main dam and consist of an unlined approach channel, inlet structure, concrete-lined circular tunnel 14.75 ft in diameter and 895 ft long, intake structure with associated machinery, outlet conduit, and a channel to Kern River. Flood-control and irrigation releases are regulated by three 5-ft 8-in. by 10-ft service gates and three emergency hydraulically operated slide gates in tandem pairs in the gate chambers of the control tower. The auxiliary dam is an earthfill structure 100 ft high and 3,257 ft long with its inner body being made up of impervious material. Water in the power canal to Borel Power Plant passes through the auxiliary dam by way of a twin barreled reinforced concrete outlet works. An auxiliary inlet structure equipped with a manually controlled tainter gate is located just

upstream from the auxiliary dam and introduces water to the canal for conveyance downstream during changeovers from either pressure or gravity flow. The spillway is located at the left abutment of the main dam and consists of an unlined approach channel; 140-ft-wide, ungated concrete control section; and an unlined discharge channel to carry spillway flows into the Kern River below the dam.

129. Progressive cavitation-erosion damage to the gate chambers of the intake structure of the main dam was reported during the first three periodic inspections dated November 1969, September 1974, and March 1980, respectively. During the September 1983 preflood inspection, extensive damage to the gates, steel liner, and concrete was found in two of the three chambers. Erosion in the steel gates measured up to 1-1/2 in. deep and in the concrete downstream of the steel liners up to 1 ft with exposed reinforcing steel. These damages were attributed to cavitation.

Repairs

130. January 1974. Cavitation-erosion damaged steel in the three gate chambers and pinhole leaks through the emergency gates of the main dam intake structure were repaired using Devcon WR-2, Sikagard High-Build, and Sikagard Aqua-Top (Farris 1974). Devcon WR-2 is a steel-filled, putty epoxy manufactured by Devcon Corporation of Danvers, Maine. Sikagard High-Build is a two-component, epoxy resin and Aqua-Top is a two-component, epoxy-resin based, marine coating. Both Sikagard products were manufactured by Sika Corporation of Lyndhurst, New Jersey. Work was performed through a contract with Coastal Coatings of Seattle, Washington.

131. During the repair a 365-cfm compressor was used to provide air for ventilation and sandblasting. Lighting was provided by a 35-v system. With all gates closed, leakage averaged about 0.85 cfs per gate. The hydrostatic pressure head on the gates was 94 ft.

132. Caulking the gates was accomplished using a combination of oakum, lead wool, and shingles. The best results were obtained by starting the caulking at the top and working down, leaving the invert until last. The caulking did not totally stop the leakage; however, it did eliminate sprays of water. The most troublesome area was the invert seal. One gate was left partially open so that a minimum 5-cfs flow was maintained to meet downstream discharge requirements. This flow created a drainage problem in the two

remaining chambers. Because the transition area inverts are level, sandbags were placed across the end of the transition to prevent backflow. However, water seeping past the gates kept the invert flooded. A pump was used with little success. Besides creating a noise problem, it could not keep the invert dewatered and dry. The ultimate solution was two plywood baffles cemented in place; one about 4 in. downstream of the emergency gate and one about 2 ft downstream of the area to be coated. The two baffles were connected by a piece of 2-in. polyvinyl chloride (PVC) pipe through which water could drain. The upstream baffle would catch the water flowing past the gate and transfer it via the PVC pipe to the downstream baffle which served as a dike against backflow. This system allowed the entire invert, with the exception of a few inches covered by the upstream baffle, to dry out. Surface preparation, patching, and coating were facilitated by dryer conditions and the flexibility of the PVC pipe.

133. All steel surfaces to be coated were chipped to remove existing coatings and then sandblasted. The old coating of coal tar enamel was easily removed for the most part by hammer and chisel. There were a few problem areas, notably the guides, where a pneumatic chipper was employed. A light residue of coal tar remained after chipping, but was readily removed by sandblasting. A 600-psi capacity, Sandstorm, sandblasting machine, was positioned outside the tower. Inlet pressure was 110 psi. The sand was transferred to the tunnels through 250 ft of 3-in. flexible line. Delivery was controlled at either the pot or the No. 6 venturi-type blasting nozzle. Fifteen tons of 20 grit, No. 2 Monterey Crystal Amber sand was used at a rate of about 17 psf. This gave the entire surface a uniform white metal appearance. No sand was recycled. Waste sand was washed downstream upon the completion of the job. Respiration for the sandblaster was provided by a 3/4-hp Binks oilless air compressor.

134. Ventilation during application of epoxies was provided by two Coppus Jectair airmovers, used in suction, with the discharge up the airvents. The combined capacity of the two airmovers was 5,300 cfm. Several checks with an explosimeter were performed during application of epoxies, but these failed to indicate the existence of any explosive fumes. Before application of the epoxies to the metal, the surface was swept over with sand to remove any oxidation that may have set up during the interval between blasting and application. The surfaces were then cleaned with compressed air.

135. Patching of cavitation pockets, pits and depressions was accomplished after sandblasting by using the Devcon WR-2, putty type epoxy. Minor pits and depressions, 1/8 in. deep or less, were not patched. There were numerous pits of this size, especially in the invert. It was felt that the final coating would effectively cover and protect minor pits, also more patching would enhance the possibility of subsequent failure. The largest cavitation pockets were at the lower upstream surface of the bonnets. These pockets each took from 5 to 7 lb of filler to patch. A total of 30 lb of filler were used on the three chambers. After curing, the patches were smoothed down using a pneumatic grinder and steel files.

136. Most of the epoxy coatings were applied with short-nap rollers. In areas such as guide recesses and bonnets, the epoxies were applied with brushes.

137. There were several small pinhole leaks through the emergency gates that resisted sealing efforts. The most active of these leaks would form a drop of water about every ten minutes, the least active about every 30 min. This was fast enough to prevent the coating from sealing over the pinhole. At the completion of the job, about half the holes had sealed themselves.

138. A total of 44 gal of epoxy was used in the repair; 40 gal of Sikagard High-Build and 4 gal of Sikagard Aqua-Top, the High-Build being used on dry surfaces and the Aqua-Top on wet surfaces. The High-Build required a minimum of 3 coats to attain the 18 mils specified. Some areas required 4 coats. Two coats of Aqua-Top gave 18 mils. Tack-free cure time for both averaged about 12 hr in the relatively constant ambient temperature of 55° F. Runs and drops which formed on the walls, ceilings and bottom edges of the gates were removed after initial curing time by the use of a Stanley Sure Form rasp. After the job was complete, the gates were opened several inches and water was allowed to flow through for 20 min. Reinspection indicated no failures with the exception of the pinholes previously mentioned through the emergency gates. However, it was later reported that the repair to areas of erosion damaged steel lasted only a short time before it failed.

139. The total time expended at the site was 718 man-hr. Of this time, approximately 30 percent was taken by sealing the emergency gates and drying out the work area; 5 percent by chipping operation; 15 percent by sandblasting; 15 percent by patching; 30 percent by application of the coating; and 5 percent by mobilization and demobilization.

140. January 1983. Trial sections of cavitation damaged concrete in the three gate chambers of the main dam intake structure were repaired in January 1983 by project personnel. Damaged areas were chipped out using a jack hammer and then water blasted. Temperatures in the chambers at the time of the repair were near 50° F. Wall patches were made using Sikadur 31, Hi-Mod Gel, a two-component, epoxy-resin system manufactured by Sika Corporation of Lyndhurst, New Jersey, and Set 45, a one component, magnesium phosphate mortar manufactured by Set Products, Inc. of Macedonia, Ohio. The invert was patched using a pre-mix concrete. All repairs were cured for 2 days after placement.

141. A total of eight areas of localized damage were repaired. These areas varied between 0.3 and 9.5 sq ft with depths between 2 and 9 in. The repair was made over a period of 5 days requiring 90 man-hr at an approximate labor cost of $2,760.00.

142. Both the Hi-Mod Gel and the Set 45 promptly washed down the river after the outlet works was put back in service. The premix concrete floor patch also failed. The wet environment and 50° F temperature were suspected of being the contributing causes of these failures.

143. December 1983. Cavitation-erosion damage in the three gate chambers of the intake structure of the main dam was repaired in December 1983 using products manufactured by Belzona Molecular, Inc. of Long Island, New Jersey.* The repair was performed through a contract with a product distributor, Western Industrial Technology, Inc. of Sun Valley, California. The application of the products was experimental and similar to the repairs performed at Pine Flat Dam in August 1983.

144. Prior to the repair, the district contracted Sargent Industries of City of Industry, California, to design and fabricate two airbags. The airbags were to be used to seal the ends of the chambers being repaired for the purpose of maintaining temperature and moisture levels in the repair areas. The design of the airbags were such that they could also be used at three other district projects. The total cost of the two bags was $7,000 ($2,000 for the design and $5,000 for the fabrication).

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* H. R. Spanier. 31 July 1984 and 10 October 1984. Personal Communication, Western Industrial Technology, Inc., Sun Valley, Calif.

145. Cofferdams (Figure 36) were constructed by project personnel upstream and downstream of the work area with two 3-in. PVC pipes between them to keep the water leaking pass the emergency gate from entering the work area. One airbag was installed at the upstream coffer dam to prevent water that was spraying from around the emergency gate from entering the repair area (Figure 37).

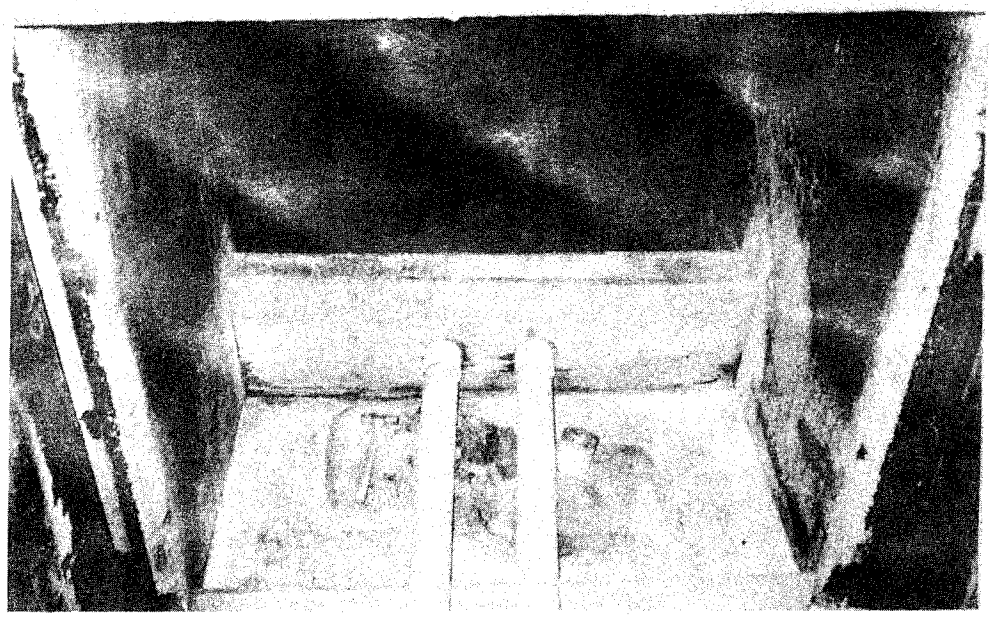

Figure 36. Cofferdam, Isabella Dam, December 1983

146. The repair surfaces were grit blasted by contract personnel with a 10-50 slag grit using a compressor capable of supplying 125 cfm of air at 90-psi pressure. The grit and waste material were quickly carried out of the conduit by the air being sucked down the manhole due to the draft created by the water rushing past in the open end of the chamber. This also served to quickly dry out the area. After grit blasting operation was completed, project personnel placed the downstream airbag.

147. An electric heater blower was installed and ducted into the repair area to provide a continuous flow of warm, dry air. The temperature in the repair area was maintained at 70° F through the night. The next morning the area was inspected and found to be dry. The concrete surface was warm to the touch.

148. The large holes in the concrete at the invert and base of the wall were conditioned with Belzona Molecular Magma Conditioner (Figure 38). The

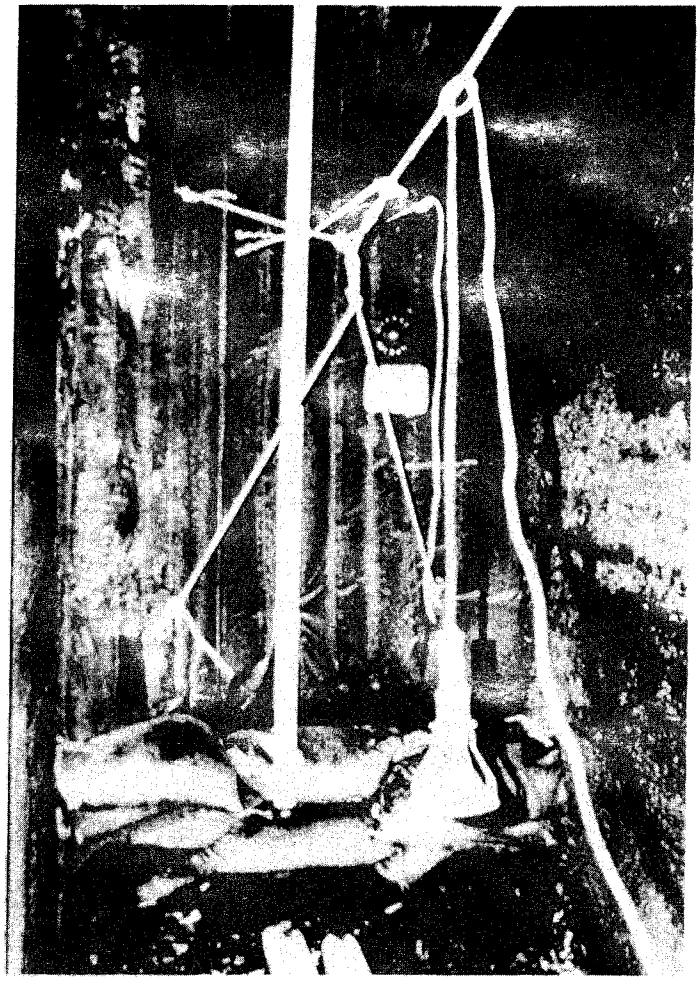

Figure 37. Airbag, Isabella Dam, December 1983

application was done with brushes having half the length of their bristles cut away to increase brush stiffness. Belzona Molecular Magma Fill, a bulk filler material, was mixed and placed into the holes until they were within 1/8 to 1/4 in. of being level with the floor.

149. The vertical surfaces required shuttering. Wooden boards wrapped in polyethylene sheeting were used because the Magma Fill would not bond to the sheeting when it cured. The Magma Fill was allowed to cure for several hours until it hardened. The polyethylene covered boards were removed and uneven areas of the fill on the vertical surfaces were ground flush to the wall. Belzona Molecular Magma Conditioner was applied to the Magma Fill surfaces with stiff bristled brushes. Magma Quartz was applied over the coated surfaces (Figure 39) using conventional concrete trowels to a thickness of 1/8 to 1/4 in. When the application was completed, it was allowed to cure for a

Figure 38. Applying surface conditioner, Isabella Dam, December 1983

Figure 39. Applying and finishing surface layer, Isabella Dam, December 1983

minimum of 8 hr before anyone walked over the finished surfaces. Warm air being blown into the gate chamber helped accelerate the cure.

150. In several repair areas, tiny blisters appeared in the surface. It was speculated that seepage was pushing trapped air through repair material. Since these blisters were so tiny and hard it was concluded that they would not jeopardize the application.

151. There were several areas in the invert and at the joint where the invert meets the wall at which seepage was noticed coming from the repair. In these areas the newly placed Quartz was ground off and replaced with Belzona Molecular E-Metal. The material application consisted of spreading E-Metal on one side of Belzona Molecular Reinforcement Tape, placing the coated side of tape against the repair surface, and applying another coat of E-Metal to the top of the tape. The repair was held down with a piece of polyethylene between the hand and E-Metal while pressure was applied for 5 min. After the E-Metal had cured the hardened surface was ground back and Ceramic R-Metal was applied over the E-Metal where necessary to even out the surface.

152. Repairs to the metal surfaces included the lower seal of the service gate, the recess areas in the ceiling, and the two plates on the two vertical surfaces on the downstream side of the service gates. These surfaces were grit blasted twice. The first blast removed rust, mil scale, and oxidation creating a 3-mil profile in the metal. The metal was allowed to leach for a minimum of 24 hr. The second blast was done just prior to the application of the Belzona Molecular Ceramic R-Metal and E.C. Barrier. The purpose of the second blast was to remove any oxidation and minerals that had leached to the surface of the metal.

153. The metal surface of the walls and the ceiling area were coated with Belzona Molecular E.C. Barrier using stiff bristled brushes. A second coat was supplied approximately 1 hr after the application of the first coat.

154. The gate-seal area was repaired by first applying Belzona Molecular Release Agent to the floor plate that mates with the lower seal area of the gate. The bottom of the gate was coated with Belzona Molecular Ceramic R-Metal and then reinforcement tape impregnated with Ceramic R-Metal was placed along the entire lower seal of the edge of the gate. After allowing the Ceramic R-Metal to cure for approximately 15 min, the gate was lowered completely so as to extrude Ceramic R-Metal. The following day the gate was raised and the excess extruded Ceramic R-Metal was ground off. It should be

noted that during the time the gate was lowered water pressure was being applied to the repair due to leakage pass the emergency gate.

155. The total cost of the repair was $82,100 with approximately $48,000 for materials and $34,100 for labor and equipment. The unit costs of the fill, quartz, and metal products were approximately $1, $2, and $4 per cu in., respectively.

156. The outlet works was inspected in August 1984. Of the three repaired gate chambers, chamber 3 had received the most use since its repair in December 1983 (210 days with peak flows from 490 to 1,639 cfs) and served as the test model. Both the concrete and the steel were damaged. The concrete area was eroded approximately 2 ft wide along the right side of the invert for the entire 15 ft of repair. The deepest erosion in this area was 4-1/2 in. just downstream of the gate liner (Figure 40).

157. A delaminated section of the invert repair was pried loose and examined. The Quartz was firmly attached to the concrete indicating the bond was good and that the concrete beneath the bond had failed. It was speculated that seepage at the joint during the repair had reduced the bond causing the Magma Quartz at the joint to spall, thereby, setting up an abrupt surface irregularity for cavitation to occur. Water then progressively undermined the Magma

Figure 40. Erosion damage, Isabella Dam, December 1983

Fill and eroded the underlying concrete since the bond between the Magma Quartz and concrete was observed to be intact in the repair areas that remained.

158. The steel repair fared worse than the concrete. Extensive cavitation marked the area just downstream of the gate slots indicating that the Ceramic Metals were no different than the steel in resisting the effects of cavitation. The R-Metal on the bottom of the gates was nearly gone with only a few pieces of the embedded reinforcing tape remaining.

159. Repairs in the other two gate chambers were also unsuccessful although to a much lesser degree due to less frequent use.

160. September 1984. Cavitation damage to areas of gate chamber previously repaired in December 1983 was repaired by project personnel using Belzona Molecular products under the supervision of product representative from Western Industrial Technology, Inc., the contractor for 1983 repair.* The repair area in chamber 3 was dewatered in the same manner as the 1983 repair. In chambers 1 and 2 only the emergency gate closures were used to dewater the chambers. Water leaking pass the emergency gates in chambers 1 and 2 was flowing over the invert areas to be repaired, thereby, requiring underwater materials and techniques to be employed.

161. Two days prior to contract supervision, project personnel used Custom Plug to repair a small area of damage (4 by 7 in., 1/4-in. deep) in the concrete at the invert of chamber 1 approximately 1 ft downstream of the steel liner.

162. A small area of damage (12 by 12 in., 1/4 in. deep) in the concrete at the invert of chamber 2 was repaired underwater using Belzona Underwater Metal. The surface to be repaired was ground using an air grinder (Figure 41). The repair material was placed in the area and worked level with the invert surface (Figure 42).

163. Repairs in chamber 3 were made to the concrete at the invert downstream of the steel liner using Magma Conditioner, Fill, and Quartz in the same manner as the 1983 repair. The exceptions to this were that sandblasting was used to prepare surfaces and the application procedure included the addition of 1/8-in.-diameter weep holes in areas that exhibited seepage. The weep holes were intended to divert water from the interface of the repair, thereby,

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* H. R. Spanier, op. cit., page 64.

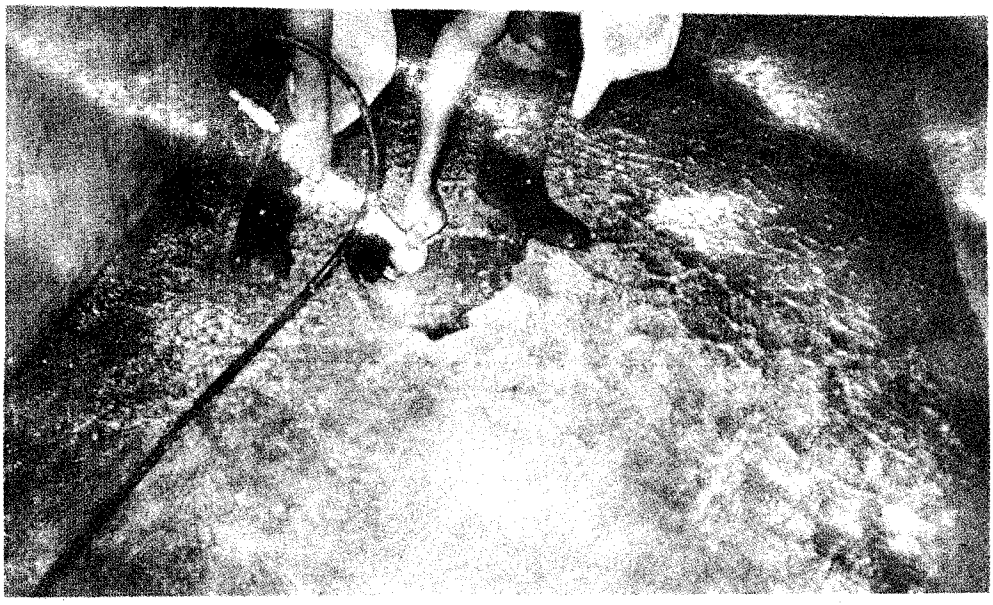


Figure 41. Underwater grinding, Isabella Dam, September 1984

Figure 42. Underwater finishing, Isabella Dam, September 1984

improving the conditions for bonding and reducing the buildup of pressures behind the repair while the chamber was in service.

164. The E.C. Barrier on the gate slots had been scoured on both sides in August 1984, approximately 12, 36, and 48 in. above the floor. C.B.R. Rubber was used to repair these spots.

165. Inspection of chambers 2 and 3 in January 1985 revealed that the Magma Quartz was in good condition. Evidence of wall scour, 1/8 in. deep, was detected in the concrete immediately above the Magma Quartz. The concrete's coarse aggregate was holding, while its fine-grained material had scoured. There was no damage to the Magma Quartz at these locations. The 12-in. square repaired area in the concrete at the invert of chamber 2 had about half of the Underwater Metal missing. The C.B.R. Rubber used to repair the gate slots had been washed out.

166. January 1985. Repairs (Nicks 1985) were made in January 1985 to chamber 3 by project personnel under supervision of the products representative from Western Industrial Technology, Inc. Underwater Metal was used to fill a hole in the concrete of the chamber invert at the end of the steel liner, and a hole just above the Magma Quartz located 22-in. downstream of the liner. R-Metal was used to cover the 1/8-in.-deep scour in the concrete behind the top edge of the left walls of Magma Quartz. The eroded areas on the right gate slot were repaired with Underwater Metal. On the left gate slot Underwater Metal was used at 48 in. above the floor, R-Metal at 36 in., and D&A Rubber at 12 in.

167. Other. For a period following the January 1985 repair, project personnel were routinely making repairs to areas of minor cavitation damage in concrete using repair products from the local hardware stores, such as Custom Plug and Rock Tite. It was recognized that these products would not give as long lasting of a repair as the Belzona Quartz and would require more frequent repair thereafter. However, they are much cheaper and easier to apply which makes them an economically feasible alternative to the much more expensive type of repairs, such as the Belzona Quartz.

168. Stainless steel welding is presently being considered as a means of repairing the steel areas of the gate chambers. It was used in September 1983 to repair areas of cavitation damage in three of the service gates at Coyote Valley Dam in the San Francisco District. The work was performed by a local welder at a cost of $2,550. The only problems reported were that the

bottom corners of gates were difficult to repair because of limited work space and that irregular surfaces were produced that caused minor leakage problems when the gates were closed.

169. Summary. A summary of the materials used to repair the gate chambers at Isabella Dam and their performances as of June 1986 are as follows:

Date	Material	Location	Performance
January 1974	Devcon WR-2, Sikagard High-Build, and Sikagard Aqua-Top	Steel gates and liners	Failed
January 1983	Sikadur 31, Hi-Mod Gel	Concrete walls and floors downstream of steel liner	Failed
January 1983	Set 45	Concrete walls and floors downstream of steel liner	Failed
December 1983	Magma Quartz	Concrete walls downstream of steel liner	Fair
December 1983	Magma Quartz	Concrete floor downstream of steel liner	Failed
December 1983	E-Metal embedded with Reinforcement Tape and R-Metal	Concrete floor areas that exhibited seepage	Failed
December 1983	R-Metal with embedded with Reinforcement Tape	Bottom of steel gates	Failed
December 1983	E.C. Barrier	Walls and ceiling at steel gate slots	Failed
September 1984	Custom Plug	Concrete floor downstream of steel liner in chamber 1	Good

(Continued)

Date	Material	Location	Performance
September 1984	Underwater Metal	Concrete floor downstream of steel liner in chamber 2	*
September 1984	Magma Quartz	Concrete floor downstream of steel liner in chamber 3	Good
September 1984	C.B.R. Rubber	Walls and ceilings at steel gate slots in chamber 3	Failed
January 1985	Underwater Metal	Concrete wall and and floor downstream of steel liner in chamber 3	Good
January 1985	R-Metal	Concrete wall downstream of steel liner in chamber 3	Good
January 1985	Underwater Metal	Left gate slot 48 in. above floor in chamber 3	*
January 1985	R-Metal	Left gate slot 36 in. above floor in chamber 3	**
January 1985	D&A Rubber	Left gate slot 12 in. above floor in chamber 3	Failed

* Half of repair material missing in January 1985. Other half still remains in place.

** Repaired area has not been subjected to cavitation forces. (Gate has only been raised 12 in. above floor due to low flow conditions that have existed since repair.)

Milford Dam

Background

170. Milford Dam was completed in 1965 and is located in the Kansas City District at mile 8.3 on the Republican River in the northwest corner of Geary County, Kansas, about 4 miles northwest of Junction City. The principal project features include an earthfill embankment, an outlet works at the toe of the right abutment, and an uncontrolled service chute spillway in the right abutment. The outlet works consists of an approach channel, an intake control tower, 21-ft-diameter horseshoe shaped conduit, and stilling basin. Two 10.5- by 21-ft hydraulically operated wheeled service gates, and two hydraulically operated wheeled emergency gates (in tandem with the service gates) control discharges through the outlet works. A 2-ft, low-flow gate placed within the leaf of each service gate provides for low flows.

171. Prior to 1964 the Kansas City District had installed corrosion-resistant steel liners to protect the concrete in the gate passages of the intake structures from erosion at the critical locations. As an experiment, epoxy resin coatings were to be applied to the gate passages at Milford Dam during construction to determine if they could be an economical alternative to steel liners. Based on a laboratory investigation (USAED, Kansas City 1965) to evaluate potential coatings, a brush-on epoxy and two epoxy mortars were selected for field application. It was estimated at the time that the cost of a 3/4-in. epoxy mortar liner would be approximately one-fifth the cost of a 3/4-in. corrosion-resistant steel liner.

172. In 1964, epoxy resin coatings were installed in lieu of steel liners in the two gate passages of the intake control tower (USAED, Kansas City 1965). The left passage was lined with a brush-on epoxy coating that was manufactured by George W. Whitesides Company of Louisville, Kentucky. The right passage was lined with two epoxy mortars. One was a 3/8-in. mortar applied to the left wall and the other a 3/4-in. mortar applied to the invert and right wall. The ingredients of the mortars included an epoxy binder manufactured by Steelcote Manufacturing Company of St. Louis, Missouri; a fumed silica thixotropic agent (Cab-o-Sil) manufactured by Godfrey L. Cabot Company of Boston, Massachusetts; and asbestos fibers manufactured by Philip Carey Manufacturing Company of Cincinnati, Ohio. The mixture proportions for the mortars are presented in Tables 1 and 2.

Table 1

Mixture Proportion for 3/8-in. Mortar

Material	Weight, lb
Epoxy resin	4.72
Epoxy hardener	2.53
Asbestos fibers	0.48
Sand (dry)	41.48
Total batch weight	49.21

Table 2

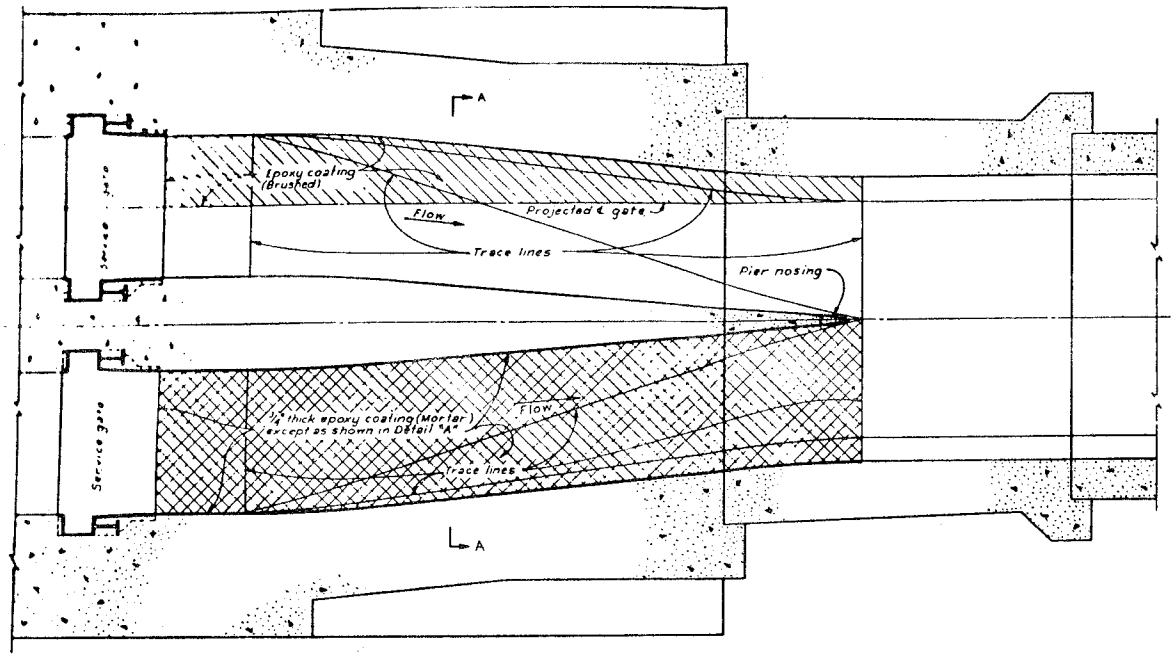
Mixture Proportions for 3/4-in. Mortar

Material	Weight, lb
Epoxy resin	4.72
Epoxy hardener	2.53
Asbestos fibers	0.48
Sand (dry)	34.56
Total batch weight	42.29

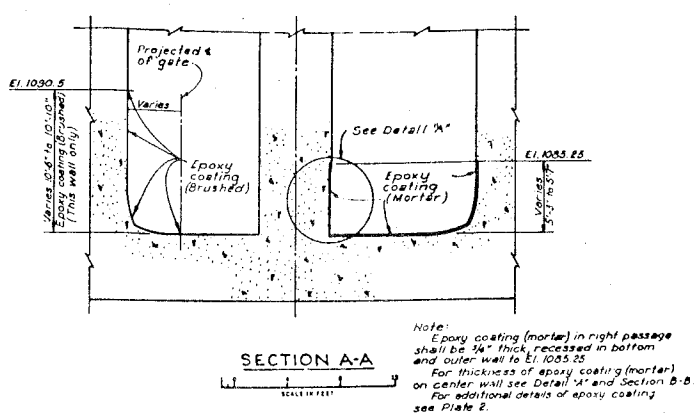
173. All concrete surfaces to which epoxy coatings were to be applied were sandblasted to expose sand particles in the concrete. At the time of application the surfaces were dry and free of dust, oil, and grease. Vacuum cleaning was performed as a part of the final surface preparation. A temperature of 70° F or above was specified in the work area during application and for seven days, thereafter. The area was also required to be kept dry during this period.

174. In the left gate passage, the brush-on epoxy was applied to the wall and floor left of the passage centerline (Figure 43). The epoxy was not applied to the right half of the passage to provide a base from which the effectiveness of the coating could be evaluated.

175. The brush-on epoxy was more difficult to apply than expected. The epoxy ran when applied to walls, and persisted in doing so even when repeatedly brushed back into place, until too stiff to move with a brush. Bristles were



HORIZONTAL SECTION THRU GATE PASSAGES



SECTION A-A

Note:
Epoxy coating (mortar) in right passage shall be 3/4" thick recessed in bottom and outer wall to El. 1085.25
For thickness of epoxy coating (mortar) on center wall see Detail "A" and Section B-B.
For additional details of epoxy coating see Plate 2.

Figure 43. Locations of applied coatings, Milford Dam

pulled from the brushes and became incorporated into the coating. Stiff daubers designed for spreading roofing tar also proved to be unsatisfactory. The applicator finally adopted was the wooden-back fiber brush.

176. During application it was difficult to attain a uniform thickness, or the specified thickness of 1/32 in. From an investigation of the problem, it was found that the epoxy delivered to the contractor differed from that specified, by having 50 percent thixotropic filler, rather than 25 percent. The original epoxy was returned to the manufacture and replaced with epoxy having 25 percent thixotropic filler. However, there was little improvement in the finished coating using the replacement epoxy.

177. The 42-ft-long passage was marked off in 10-ft increments for control purposes. The application was continuous, as specified, and the entire wall and floor surfaces was completed in just over 24 hr. Although adequate for its purpose, the brush-on epoxy was far from attractive and would not be acceptable in a location exposed to view. During the hours following the application, tears and sags developed. These defects were repaired by sandblasting and recoating.

178. In the right gate passage, the floor and right wall were overlaid with epoxy mortar to a thickness of 3/4 in. The left wall of the passage was overlaid with epoxy mortar to a thickness of 3/8 in. Areas of application are shown in Figure 38. The walls were partitioned into 5-ft sections. The boundary of each section was masked with tape before priming. The area primed was restricted to that which could be covered with mortar while the prime coat was still tacky. Within the limits of the primed area, wooden screed boards were installed as vertical forms to control mortar thickness.

179. After the prime coat was applied, the mortar was mixed with a variable speed food mixing machine (Hobart type) having a 1/3-hp motor and a round bottom bowl with a capacity of about 5 gal. The mixing procedure was as follows:

a. Epoxy resin was poured in clean mixing bowl. (The resin was thick and had to be scraped from container.)

b. Asbestos fibers were slowly added while mixing for approximately 4 min.

c. Epoxy hardener was added and mixed for approximately 1 min.

d. Sand was slowly added while mixing for approximately 10 min.

e. Mortar was removed from bowl and mixing bowl cleaned with toluene.

180. The mortar was first applied in a thin layer and worked into the tacky prime coat, after which additional mortar was applied to obtain full thickness. If the prime coat was too thick or if the mortar was applied before the prime coat was sufficiently tacky, the mortar had a tendency to sag.

181. Sagging created an objectional crack where the epoxy mortar joined the top of the blockout. To correct this, a bevel groove was cut along the top of the mortar after the mortar had stiffened sufficiently to hold its contour. The groove was then refilled with fresh mortar. At the end of placement each day, the leading edge was cut on a 1 on 3 bevel. The finished mortar, upon attaining surface tackiness, was sealed with a brush coat of combined epoxy resin binder and hardener. A straightedge was used to check the smoothness of the finished surface. The maximum variation from the theoretical plane was approximately 1/4 in. in 5 ft.

182. Because of the effort required to trowel the stiff epoxy mortar to a smooth, dense surface, it was found advantageous to use two alternating teams of finishers, working about a 1/2 hr at a time. In this manner it was possible to complete approximately 50 to 60 sq ft of surface per 8-hr day.

183. As was expected, the 3/8-in.-thick mortar applied to the passage wall proceeded with much less difficulty than the 3/4-in. Application of mortar to the invert was accomplished from a bridge spanning the area to be covered each day. This application was less difficult than for either of the wall sections.

184. It should be noted that a large percentage of the personnel involved with the application of the epoxies developed dermatitis and complained of lung congestion, headaches, odd taste, and dizziness. Blowers were used to supply ventilation through the work area. It was speculated that increased ventilation would have reduced the number of reported illnesses.

185. Progressive delamination of the brush-on epoxy in the left gate passage was reported during 1965 and 1967 inspections. The delamination appeared as small pockets 2 to 3 in. square. Examination of samples obtained during the 1965 inspection showed the failure occurring in the concrete. However, samples obtained in 1967 showed the failure at the concrete-epoxy interface.

186. In 1974, over 90 percent of the brush-on epoxy loss was from the invert. It was speculated by district personnel that this loss was due to the invert surface being more likely to (a) develop laitance, (b) collect dust under gravity forces prior to the placing of the epoxy, and (c) remain saturated once the reservoir elevations were maintained above the invert elevations. The major area of loss in the invert was approximately 20-ft downstream of the service gate steel liner and was centered about a crack which crossed the invert. This crack probably existed when the epoxy was placed. Another large area of loss was in the concrete at the end of the steel liner.

187. The construction joints, monolith joints, cracks and the interface between the steel gate liner and concrete appeared to be locations where the failure of the brush-on epoxy began. District personnel concluded that the locations may have contributed to the loss in one, or a combination, of the following ways: (a) by having significant differential movements, thus cracking the epoxy, (b) by concentrating the movement of moisture from underlying areas through the epoxy, and (c) by harboring significant amounts of moisture when the epoxy was placed, that reduced the bond strength of the epoxy. Possible solutions to some of these problems were suggested to be: (a) heavier sandblasting to remove weak concrete surfaces; (b) taking extreme care to insure the surfaces are free of dust; (c) employing new formulations of epoxy which can be placed in moist conditions; (d) employing epoxies which have sufficient elasticity to span areas of movement; and (e) installing joints in the coating to allow underlying moisture to escape.

188. The concrete surfaces of the uncoated half of the left gate passage appeared to be in good condition with only minimal wear. It was speculated that the high strength of the concrete was an important factor in the performance of the uncoated portion of the passage. The concrete mixture proportions included 6.5 sacks of cement with 20 percent (by volume) fly ash replacement, 1-1/2-in. maximum size aggregate, and 3 to 4 percent entrained air. The compressive strengths of the concrete at various ages were as follows:

Age	Compressive Strength, psi
7 days	3,200
28 days	4,330
90 days	6,380

(Continued)

Age	Compressive Strength, psi
1 yr	7,400
2 yr	8,020
5 yr	8,280

189. In 1974, approximately 80 percent of the 3/4-in.-thick epoxy mortar in the invert was missing with delamination evident in some of the areas where the mortar remained. The area of loss was primarily between the service gate steel liner and the construction joint between the tower and first conduit monolith (Figure 44). The epoxy mortar on the walls was intact and appeared sound (no drumming was detected when tapped with a hammer). It was suspected that the loss of epoxy mortar was the result of strains caused by differences in thermal expansion between the epoxy mortar and concrete and strains caused by cavitation forces produced by the high discharges that occurred in the spring of 1973.

190. Four small epoxy mortar samples from the invert were analyzed by the Missouri River Division (MRD) Laboratory. Little deterioration was found in the epoxy mortar. The bond between the epoxy and the concrete was found to be good in all but one of the samples. In this one sample, about a third of the bonding surface had failed. The quality of the concrete attached to the samples was not good. Present in the concrete were crazing cracks and either a thin layer of dust from sandblasting or laitance. The thermal coefficient of expansion for the epoxy mortar was determined to be approximately three times greater than that of the concrete.

191. The loss of epoxy mortar liner in the invert resulted in an abrupt change in the surface immediately downstream of the steel liner. This surface irregularity caused turbulent flows that resulted in a continuation of the cavitation damage into the exposed concrete.

Repair

192. 1983. An epoxy resin mortar was used in September of 1983 to repair the area of cavitation erosion damage immediately downstream of the steel liner in the concrete at the invert of the right gate passage*. The epoxy resin used in the mortar was Sikadur Lo-Mod Gel, a low-modulus,

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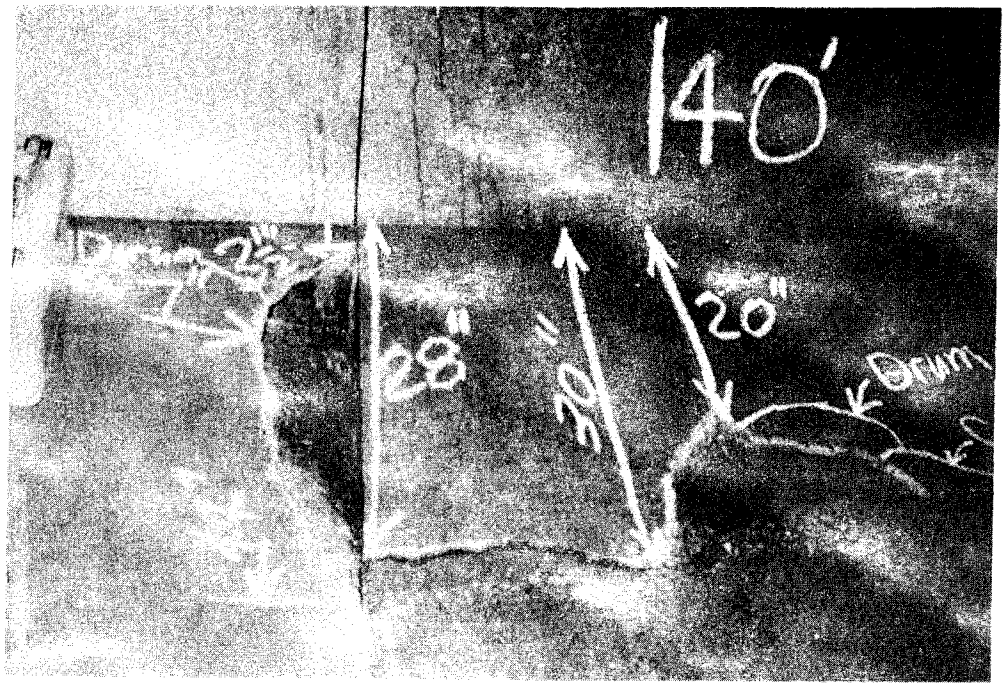
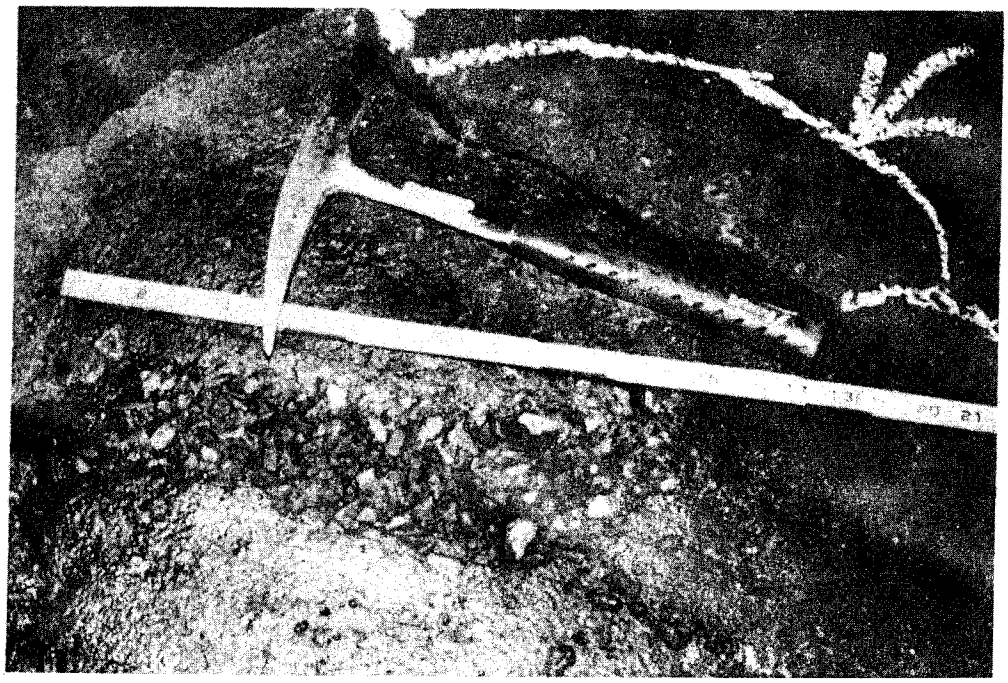
* P. D. Barber. 4 February 1983. "Milford Lake, Repair of the Right Water Passageway of the Intake Tower," Letter, US Army Engineer District, Kansas City, Mo.



a. Downstream end of passage



b. Closeup

Figure 44. Mortar loss, Milford Dam

gap-filling, moisture-insensitive, epoxy adhesive manufactured by Sika Corporation of Lyndhurst, New Jersey.

193. The repair area was dewatered using a sandbag dam with pipes to carry gate leakage downstream of the repair. Portland cement was used to plug leaks in the sandbag dam. Three men using sponge and paper towels removed water that was ponded in the repair area. A saw cut was made across the invert, 1 in. deep, parallel to and 12 in. downstream of the end of the steel liner. The concrete in this area was removed to a minimum depth of 1 in. (Figure 45).

194. The remaining epoxy liner in the right passage invert was saw-cut at the junction for walls and invert along the length of the passage and removed in an effort to prevent the delamination and mortar loss from extending into the liner of the wall. The mortar liner at the upstream edge of conduit monolith 1 was ground to within 1/4 in. of the adjoining upstream surface that had lost its mortar (Figure 46). This was done to provide a smooth surface transition, thereby, eliminating a condition that could result in turbulent flows and cavitation damage.

195. The epoxy resin mortar was placed by contract personnel from the product distributor, Carter Waters of Kansas City, Missouri. The components of the Sikadur Lo-Mod Gel were mixed together on cardboard squares using spatulas. The epoxy was then mixed with Colma Quartzite Aggregate to make an epoxy mortar. The mortar was placed in two layers (Figure 45), to minimize exothermic and shrinkage effects. The dimensions of the placement were about 1 in. by 12 in. by 10 ft. The time required to place the first layer was approximately 2 hr and 45 min. The second layer was placed while the first layer was still tacky (approximately 15 min after the completion of the first). Water was prevented from contacting the epoxy mortar until initial set which was about 4 hr after placement. The temperature at the time of placement was estimated to be in the upper 70° F range.

196. In May 1985, 21 months later, the patch was inspected. The epoxy mortar surface had settled or flowed, before hardening, to about 1/4 to 3/8 in. below the level of the steel. It appeared that the bond was still intact and that the mortar surface had little sign of wear.

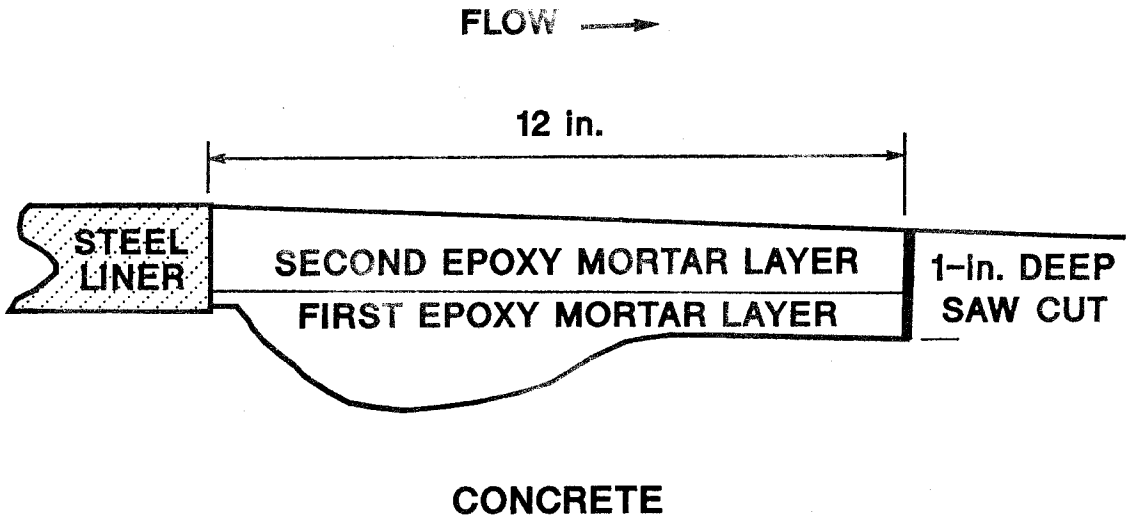


Figure 45. Repair immediately downstream of steel liner, Milford Dam

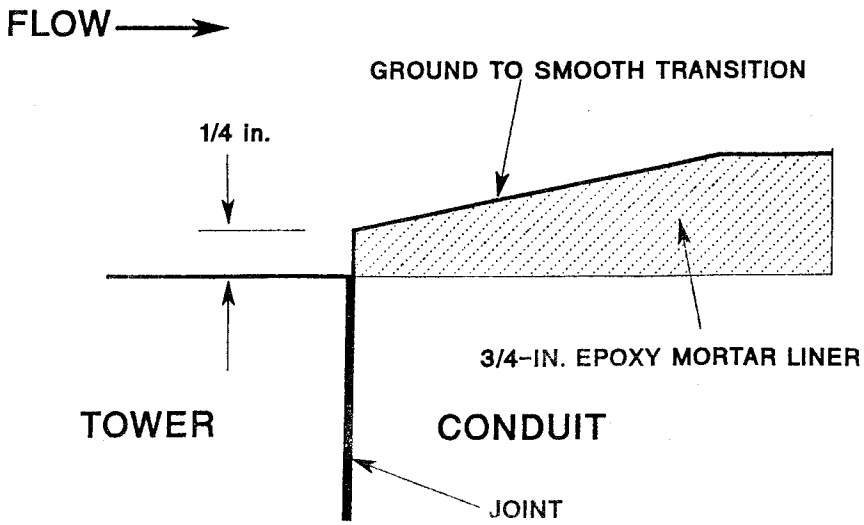


Figure 46. Edge ground to provide smooth transition at monolith joint, Milford Dam

Mulberry Dam

Background

197. Mulberry Dam was completed in 1982 and is a part of the El Paso Flood Protection Project. The dam is located in the Albuquerque District on an arroyo near El Paso, Texas. The original dam was constructed for the City of El Paso in 1957 by a private contractor as three separate embankments with a common reservoir area and a spillway between two of the embankments. This configuration was essentially retained by the Corps of Engineers during reconstruction of the dam with the embankments being raised about 15 ft and a new spillway and outlet works being incorporated into the project in a fourth embankment in the area of the old spillway.

198. The outlet works consists of an uncontrolled two-way riser, an outlet conduit, and an impact type energy dissipator. The conduit is 284 ft long with a 3-ft inside diameter. The outside of the conduit is rectangular in section except for two 20-ft monoliths under the dam axis which are trapezoidal. The design wall thickness is 10 in. at the crown, 11 in. at the invert, and 8 in. at the spring lines. The spillway consists of a 250-ft-long uncontrolled concrete ogee section and discharges into an arroyo adjacent to the main arroyo in which the dam was built.

199. The metal forms that were used during placement were removed from the interior of the conduit for the first time in 1984. About two-thirds of the way up the conduit there was a 60 to 80-ft reach of honeycomb areas and cavities along the invert. In one spot there was a triangular shaped cavity partly covered with a piece of metal plate. The cavity extended through the full depth of the slab and appeared to be wider at the bottom. At another monolith upstream, there were two cavities that penetrated the full depth of slab. One cavity in the middle of the monolith (Figure 47) was about 6 ft long with a varying width of about 10 in. at the maximum. The second cavity located at the upstream end of this monolith was about 12 in. wide by 18 in. long. At the joint, the water stop was firmly embedded into the adjacent monolith, but fully exposed on the side of the cavity. There were two or three other V shaped cavities 5 to 7 ft long and varying in depth from about 1-1/2 to 8 in. About two-fifths of the way up the conduit, the end of a 3- by 3-in. piece of wood was embedded about 8 in. left of invert. Except for an area of scattered pits (approximately 1-1/2 in. in diameter by 1/2 in. deep)

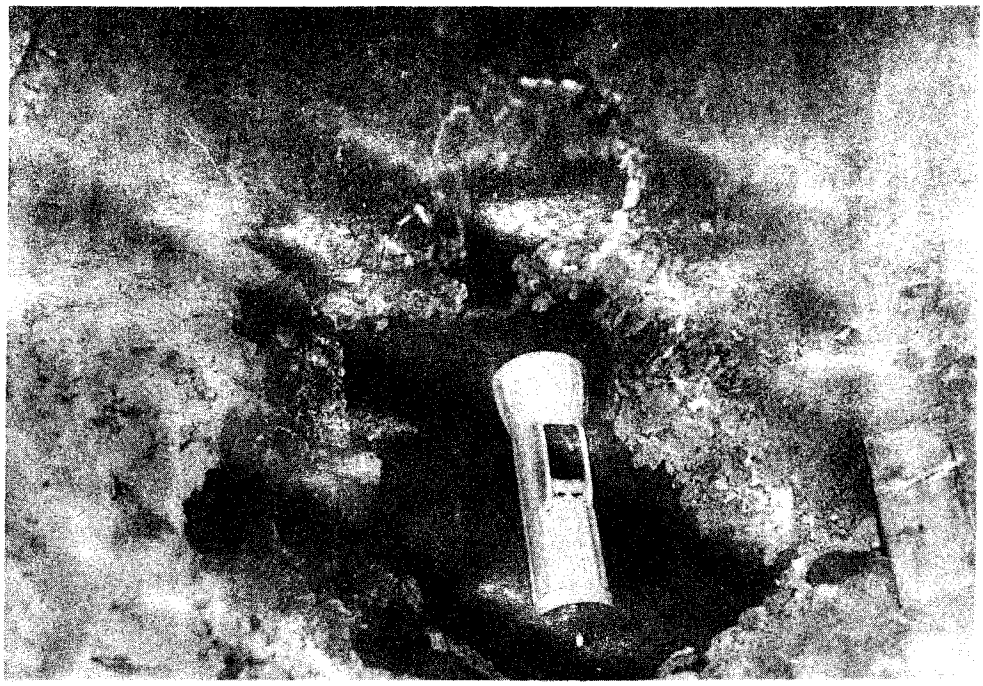


Figure 47. Hole through conduit at invert, Mulberry Dam

on the bottom of one of the monoliths, the concrete surface beyond the honeycomb and cavities areas appeared to be in very good condition. No erosion of soil below the conduit slab was detected.

Repair

200. Honeycomb areas and cavities in the concrete along the invert of the conduit were repaired by contract personnel in November 1984 using Set 45, a one-component, magnesium phosphate concrete manufactured by Set Products, Inc. of Macedonia, Ohio.* Areas to be repaired were chipped out and reinforcing bar cleaned with wire brush before filling voids. The Set 45 when placed was very fluid and had to be frequently trowelled to maintain alignment of the repair surface with that of the surrounding concrete. The set of the Set 45 occurred almost instantaneously.

201. During the 1985 periodic inspection, these repaired areas were found to be in good condition. It should be noted that flows through this conduit are generally infrequent and of low velocities.

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* J. B. Rael. 26 November 1984. "Inspection of Construction by Engineering Personnel," Trip Report, US Army Engineer District, Albuquerque, Albuquerque, N. Mex.

Pine Creek Dam

Background

202. Pine Creek Dam was completed in June 1969 and is located in the Tulsa District on Little River about 5 miles northwest of Wright City, Oklahoma. The embankment is a rolled impervious earthfill, 7,510 ft long, rising 124 ft above the streambed with a top width of 32 ft. The dike is 14,150 ft long, 38 ft high, and 10 ft wide at its crest. The spillway is located in the right abutment and consists of an uncontrolled gravity ogee weir, a flip bucket, and an end sill. The outlet works, located on the right abutment of the river section, includes an intake structure, 13-ft-diameter conduit, a 48-in.-diameter water supply and water quality bypass, and a 36-in.-diameter water supply static head line. Flow through the conduit is controlled by two 5-ft-8-in. by 13-ft hydraulic slide gates operated in tandem.

203. In 1970 there was unacceptable leakage at monolith joints 2/3 and 8/9 of 5 and 20 gpm, respectively. It was suspected that slight differential movement within the conduit caused the joints to open slightly, thereby, permitting water passage. Remedial grouting was performed in 1970 and again in 1976.

Repairs

204. 1970. Remedial grouting (USAED, Tulsa 1975) was performed in March and April of 1970 by district personnel to stop leakage at monolith joints 2/3 and 8/9 in the outlet conduit using a portland cement grout. The drilling equipment consisted of one portable air drill and AX-size (1-7/8-in.-diameter) diamond bits. The grouting equipment included mixing and sump tanks with air-operated agitators, a Moyno pump operated by Ingersol-Rand air motor, an air compressor, a water pump, and various hoses, connections, packers, and gauges. Portland cement Type II was used for grouting and backfilling.

205. The grouting was accomplished by drilling holes through the concrete and pumping grout around the exterior of the conduit and into the monolith joints. All holes were drilled before grouting was begun. Holes that did not vent water after drilling were not grouted. The grout was injected through a packer set in the top of the hole. The minimum allowable pressure was used to reduce the travel of the grout. Mixtures with a water-cement ratio of 1:1 and 0.75:1 by volume were used. The grouting pressures usually

ranged from 15 psi to 30 psi. To provide a barrier and prevent grout leakage the construction joints were caulked with lead-wool. All holes were backfilled or packed with cement.

206. Four holes were drilled near the left spring line at monolith joint 2/3; three of the holes were grouted with 42.6 sacks of cement or 53.25 cu ft of grout. Nine holes were drilled around the joint 8/9, with the majority drilled on the upstream side; four of the holes were grouted with 90.7 sacks of cement or 119.85 cu ft of grout. Figures 48 and 49 show drill hole locations, grout takes, and grouting sequence for joints 2/3 and 8/9.

207. The 5-gpm flow from the left spring line leak at monolith joint 2/3 was reduced to 2 gpm or less. The flow from the joint 8/9 was approximately 15 to 20 gpm. This was reduced to 1 gpm or less. A total of 133.3 sacks of cement or 173.1 cu ft of grout were used at the two joints. Total cost of repair was estimated to be $8,000.

208. In December 1974 leakage was reported to be approximately 1 gpm or less at these joints.

209. 1976. Remedial grouting (USAED, Tulsa 1976) using a portland-cement (Type II) grout was performed in July 1976 by district personnel to stop continuing leakage at monolith joints 2/3 and 8/9 and new leakage at joints 5/6, 12/13, and 13/14. Approximately eight holes were drilled around the periphery of each joint using air drills and 1-7/8-in.-diameter diamond bits. If contact was made with the leaking joint before the conduit wall was penetrated, drilling was stopped and the hole grouted. Usually several holes were drilled before grouting began. Prior to grouting, all holes were pressure tested using the maximum grouting pressure. Holes that would not take water were not grouted. The grout was injected through a packer set in the top of the hole. Mixtures with a water-cement ratio ranging from 4:1 and 0.75:1 by volume were used. The majority of the holes were grouted with a 1:1 mixture. Grouting pressures ranged from 15 to 30 psi. To prevent grout leakage, the joints were caulked with lead wool. All holes were backfilled or packed with cement. A total of 127.8 sacks of cement or 250.5 cu ft of grout were used in grouting the leaks. The total cost of the repair was estimated to be $25,000.

210. In November 1984 during the sixth periodic inspection, leakage was reported to be 1 gpm or less at individual joints with no evidence of embankment material being washed through joints.

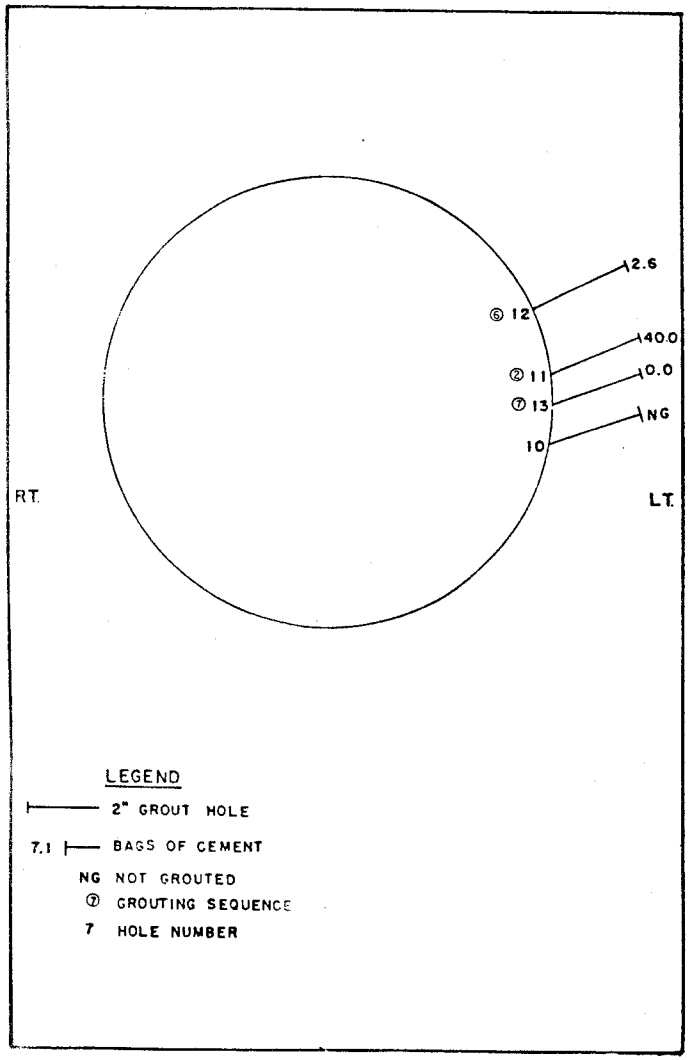

Figure 48. Details of grouting for joint 2/3, Pine Creek Dam, 1976

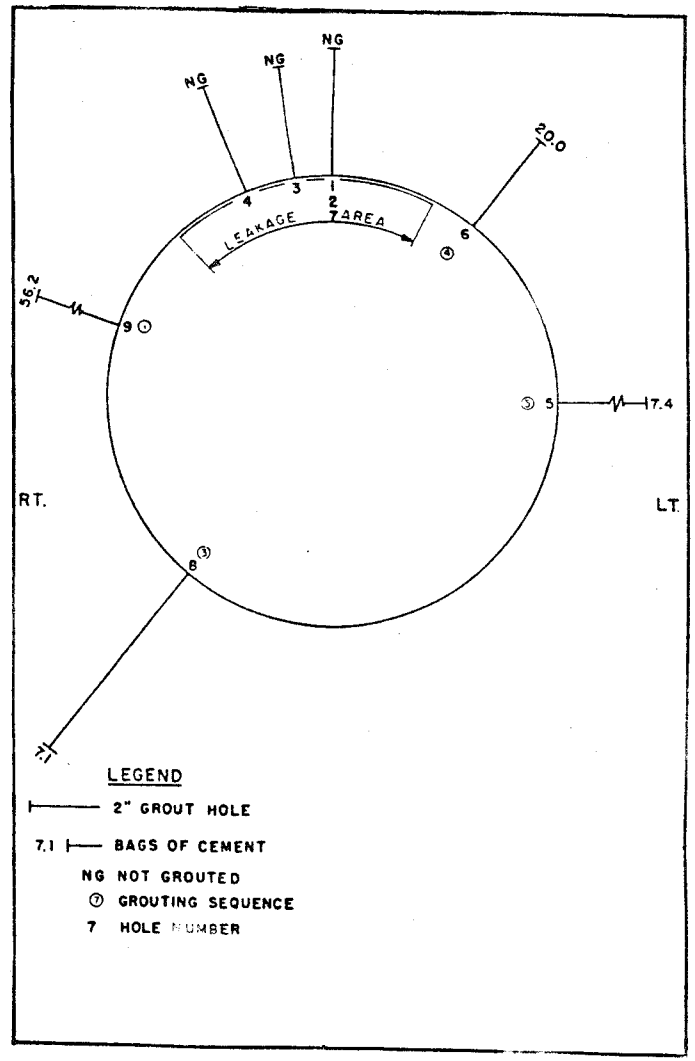

Figure 49. Details of grouting for joint 8/9, Pine Creek Dam, 1976

Warm Springs Dam

Background

211. Warm Springs Dam was completed in 1980 under the supervision of the San Francisco District and is located in the Sacramento District at the confluence of Dry Creek and Warm Springs Creek in Sonoma County near Healdsburg, California. The principal project features include a random zoned earthfill embankment with a central impervious core; tunnelled outlet works; and uncontrolled ogee weir spillway. The outlet works of the dam (Figure 50) is located in the left abutment and consist of a short approach channel; submerged intake structure at the upstream end of the flood conduit; four controlled inlet conduits, three low-flows and one flood; an intake control structure (Figure 51); an outlet conduit; and a stilling basin at the outlet conduit exit.

212. During construction the outlet works of the dam was excavated in rock for most of its length. The exposed rock resulting from excavation of the intake control structure was braced with steel and then shotcreted to maintain the stability of the rock face. The inside face of the structure was formed and concrete placed between the shotcrete and the formed face. After placement the top of each lift was sand blasted in preparation for the next lift. No bonding material was used between lifts due to the rich concrete mixture being placed. The use of water stops at the lift joints had been considered in the design but were deleted. Defects in the interior face were patched and given a cosmetic finish.

213. Leakage from horizontal construction joints and vertical cracks in the wall of the intake control structure was noted during the first periodic inspection in November of 1982. Groundwater was the leakage source since the embankment of the dam had just been completed and no water had been impounded. The extent of the leakage at the joints was highly variable, ranging from very slight, isolated flows covering 0.5 ft to 3 ft of the joint to much heavier flows wetting virtually the entire joint circumference. Many of the vertical cracks were essentially continuous across horizontal construction joints and extended over 100 ft in length. At the lower levels substantial portions of the interior surface were continuously wet and at many locations covered with mineral deposits and efflorescence. The floors at these levels were ponding water.

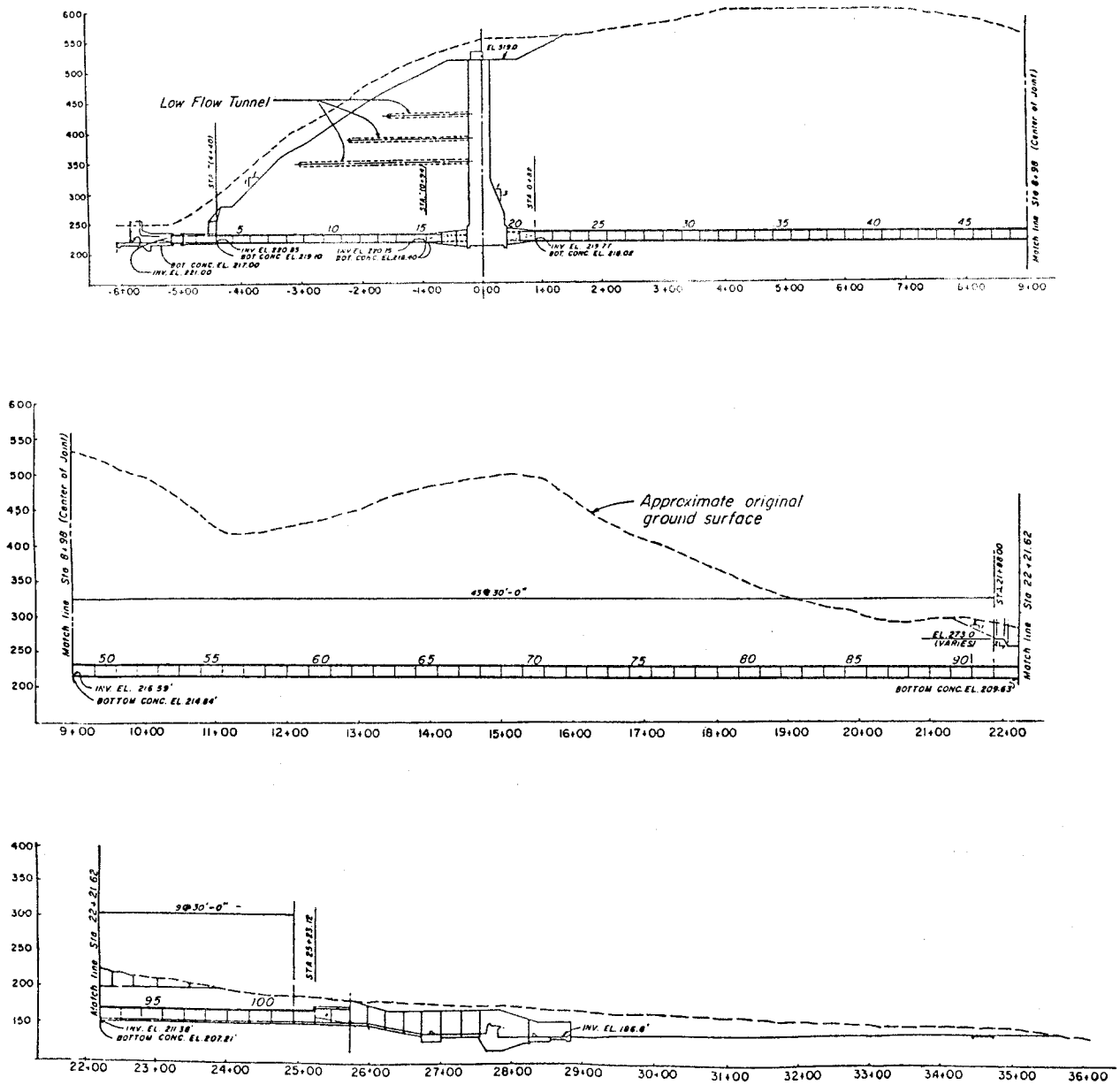

Figure 50. Centerline profile of outlet works, Warm Springs Dam

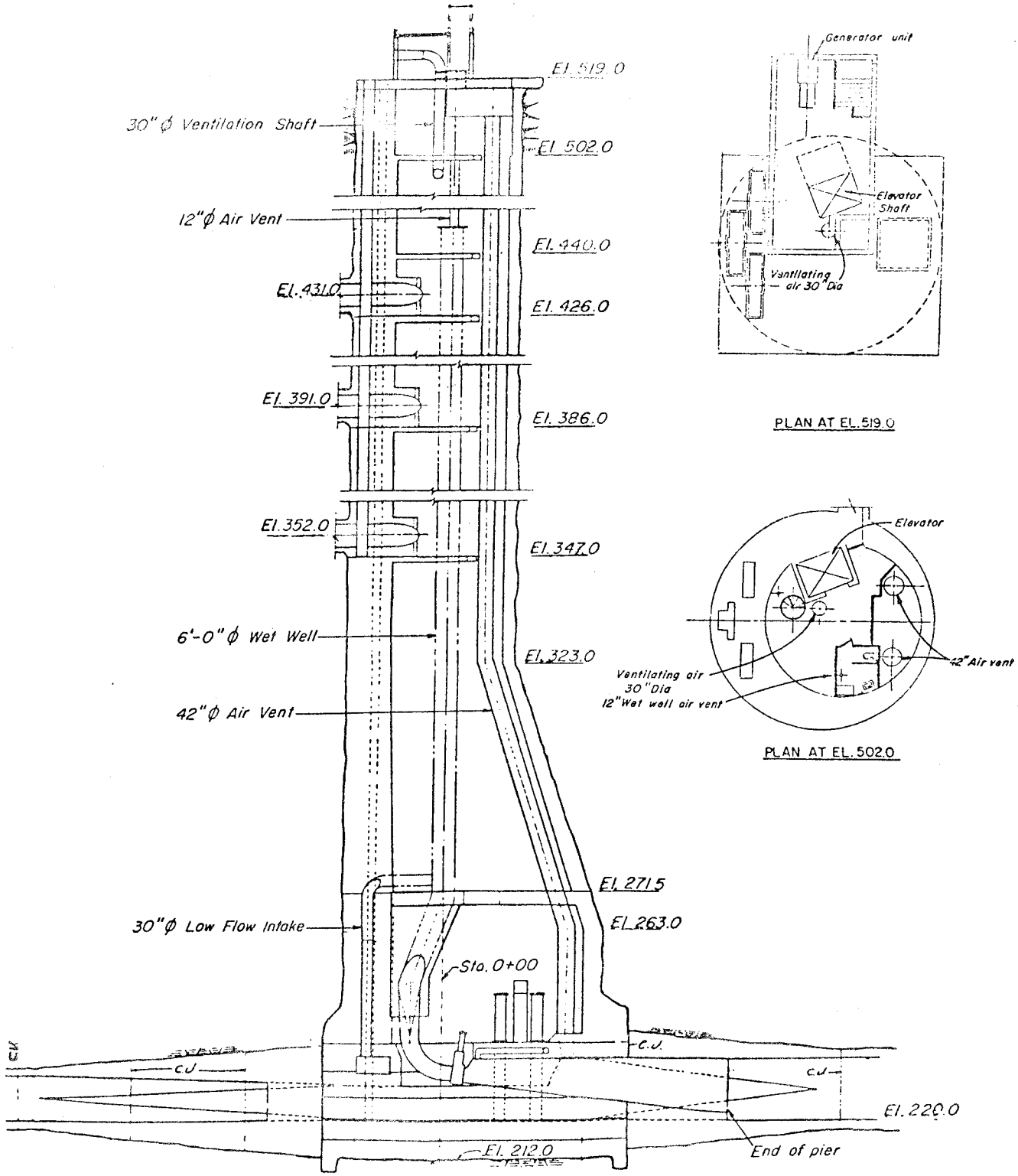

Figure 51. Centerline elevation view of intake control structure, Warm Springs Dam

214. The leakage posed no threat to the structures safety, but was threatening the structure's ability to perform as designed. The leakage was resulting in significant reductions in the reliability and performance of the electrical system and increased corrosion of the operating machinery and appurtenances.

Repair

215. Epoxy injection of leaking construction joints and cracks was performed by personnel from Adhesive Engineering Company of San Carlos, California, between November 1983 and May 1984 (USAED, Sacramento 1986). Surface cleaning began at the top of intake shaft and progressed downward. Porting, sealing, and injection operations all began at the lowest elevation and progressed upward. During the repair, deposits were water blasted off the concrete faces using 5,000-psi water pressure. A hand-held pneumatic chisel was used to remove unsound concrete at the joints and cracks being repaired. Initially, 1- to 2-in.-deep, 0.555-in.-diameter ports were drilled horizontally into the joints and cracks at 36-in. spacings and 1/8-in. inside diameter, 4-in.-long pipe nipples installed in ports. Waterplug, a fast-setting hydraulic cement manufactured by Thoro Systems Products of Miami, Florida, was used to seal the surfaces containing the joints and cracks. Joint and crack openings were estimated to be between 2 and 3 mils. Concressive-1468, a two component epoxy manufactured by Adhesive Engineering Company of San Carlos, California, was used as the injection material.

216. For vertical cracks the injection of epoxy began at the lowest entry port. As the injected material traveled upward and appeared at the next higher port, injection at the lower port was discontinued and initiated at the next higher port. The procedure for the joints was essentially the same as that employed for the vertical cracks except that the ports were injected successively on a horizontal plane.

217. The epoxy was injected at 160-psi pressure. At this pressure the patches and cosmetic finishes placed along the joints and cracks during construction were spalled from the concrete face. These spalls revealed that the joints varied significantly in elevation along their lengths and were filled with debris. It was speculated that during construction of the shaft a screed strip was located at the top of each lift (Figure 52) and that the screed strip formed a blockout that functioned as a sump or trap for debris when the top surface was being prepared for the placement of the next lift.

218. Only small volumes of epoxy could be injected at the horizontal ports indicating limited penetration of the epoxy into the joints. The contractor improved the penetration of the epoxy into the joints by drilling 10-in.-deep injection ports at a 45-deg incline to shaft face. These ports were located above each joint such that they intersected the horizontal portion of the joint above the assumed blockout (Figure 52).

219. After the epoxy had cured long enough to preclude its draining out from the injected voids, the pipe nipples were removed and the ports patched with Waterplug. The ports and sealed surfaces were then ground flush with the adjacent concrete surfaces.

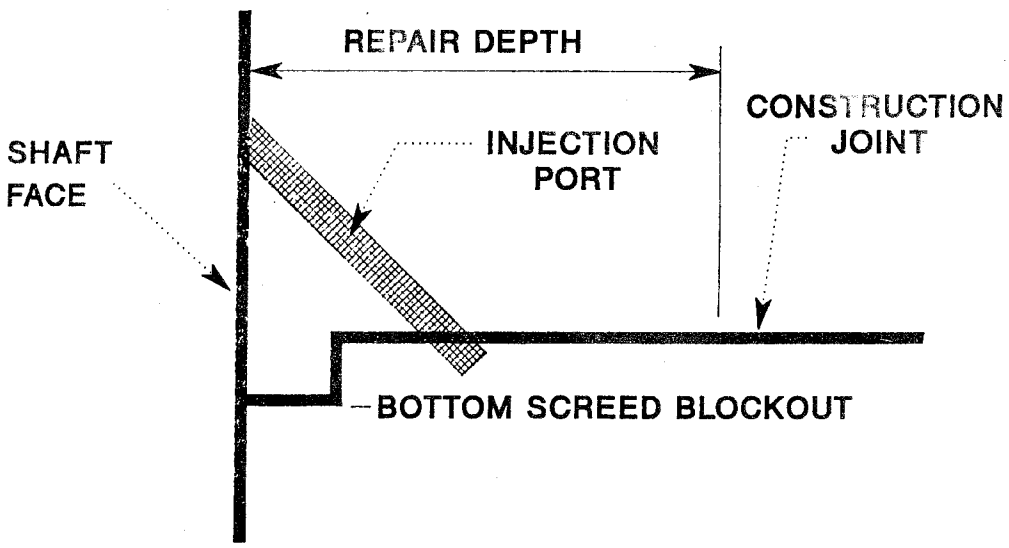

Figure 52. Sketch of epoxy injection joint detail, Warm Springs Dam

220. To evaluate the repair, 4-in.-diameter cores were taken to depths of 36 to 48 in. into the repaired concrete. Aggregate was preplaced into the core holes and injected with epoxy. Visual inspection of the cores indicated a good bond between the epoxy and the concrete. The visual inspection also showed that the required penetration depth of 36 in. had been achieved, however, not all of the joint in this reach contained epoxy. This indicated that the coverage in the joint was not uniform and contained pockets or islands that had no epoxy penetration.

221. Overall 3,500 linear ft of joints and 1,450 linear ft of cracks were injected to a depth of 36 in. at a cost of $600,000. The repair was estimated to be 60 percent effective at the time of the second periodic

inspection by Sacramento District personnel. The quality control documentation indicated that in those areas known to be leaking, the frequency of successful port to port communication during the repair was comparatively low. It was speculated that, in these areas, the travel of injected epoxy material was restricted by the presence of heavy mineral deposits on the interfacing joint surfaces.

222. One factor that has been detrimental to the performance of the repair is the increase in water head. The repair was done when the pool elevation was below the bottom of structure (elevation 212 ft) and the structure subjected to hydrostatic loads due to groundwater only. Leakage has since increased as the pool elevation increased. The elevation of the pool was at 347 ft at the time of the second periodic inspection when the repair was considered 60 percent effective. At that time over 100 ft of water head was needed to bring the pool level up to conservation pool (elevation 451 ft). Since then the effectiveness of the repair has continued to decrease as the pool level approaches normal operating levels.

PART III: DISCUSSION

Case Histories

223. The most common type of deficiency requiring repair in intake structures and conduits was leakage from cracks and joints. Reported methods employed to stop leakage from cracks included cementitious and chemical grouting, epoxy injection, and surface treatment. Methods employed to stop leakage at joints were limited to either cementitious or chemical grouting. The most successful method documented for reducing leakage through both cracks and joints was chemical (urethane) grouting. Projects in which cementitious grouting of joints and cracks was performed required regrouting(s) to maintain leakage at an acceptable level. Repairs using epoxy injection systems to stop leakage from cracks were reported as not successful or not fully successful. Surface treatments employed to stop leakage from cracks were reported as not fully successful. Surface treatment applied to the water side or positive face of concrete performed better than surface treatment applied to dry side or negative face of concrete.

224. The second most common type of deficiency being repaired was cavitation-erosion damage to passageways in the vicinity of gates. The most successful method reported for repairing cavitation damage to concrete surfaces was resurfacing of damaged area using Belzona Magma Quartz. However, before a recommendation can be made, laboratory testing is needed to further substantiate the quartz product's potential for cavitation repair. Also, because this product is extremely expensive, it may be more economical to repeat the resurfacing on a routine basis using an inexpensive patching material rather than to make the repair using the Magma Quartz. None of the materials used to repair cavitation damage in steel gates and liners were considered successful in preventing the reoccurrence of the erosion damage. The performance of some of the other reported repair materials in preventing the reoccurrence of erosion damage was inconclusive due to the operating conditions after the repair not being as severe as those that produced the damage.

225. The performance of repair methods and materials employed to correct other types of deficiencies were considered inconclusive because the operating conditions after repair were not as severe as those that produced

the damage or the environment in which the repair was made and exists is not as severe as that which would be normally encountered at other projects.

226. It is acknowledged that the performance of the materials reported in these case histories might differ under circumstances and conditions other than those reported. For example, the performance of a repair material that was reported as failed might perform successfully under more favorable application and exposure conditions and, conversely, the performance of a repair material that was reported as successful might fail under harsher conditions.

Repair Plan

227. The cause of damage should be determined and an evaluation made to determine its effect on the performance of proposed remedies. Considerations include: (a) whether the cause still exists and if so, can it be eliminated?, and (b) if the cause still exists and cannot be eliminated, what is the mechanism causing the distress and what action can be taken to minimize its effect on the performance of remedies being considered? Such considerations are the foundation for developing a sound plan to achieve a permanent repair.

228. In planning a repair it is essential to provide as compatible an environment as feasible for the success of the repair, such as, controlling the temperature and moisture levels in the repair area, diverting water flow and leakage away from the surfaces to be repaired, and removing contaminates and damaged concrete that would reduce the bond between the repair material and the concrete. Details of the repair should be directed at improving the chances of success of the repair, such as, drilling the extra holes and performing the necessary caulking and sealing of openings to control and reduce flows through an injection plane. Such action would lengthen the time in which the repair material is in the repair plane and, thereby, improve the chances of the material setting before exiting. The temptation of a quick-fix solution should be avoided. Only that portion of the work that can be accomplished within the allotted time restraints and still produce a successful repair should be repaired. The remaining damage should be repaired as the opportunity presents itself. Where practical, selected materials and techniques should be experimented with using small test sections of repair to

identify problems with the repair and to estimate performance. Such extra measures require additional time and cost initially, but will generally result in savings with time due to the increase in repair performance gained.

229. In reviewing a proposed repair plan, problems that might result in poor performance should be sought out and the necessary steps taken to eliminate them. For example, in situations where leakage is involved, partial depth injection repairs made from the downstream end (negative side) of joints and cracks leave the unbonded portions near the upstream end (positive side) subject to the changing hydrostatic pressures of the water source. These changes in pressure produce moving wedges within the unbonded portions of joints and cracks that result in movements across the bonded portions. If the movements are of sufficient magnitude and frequency of occurrence, they will result in overstraining and cracking of either the concrete or repair material whichever is the weaker and a reoccurrence of the leakage. If full-depth repairs are made, the water will be forced out of the joints and cracks, thereby eliminating the source causing the movement and the potential for failure.

230. If the problem cannot be eliminated, plans should be made to minimize the problem. For example, repairs to active or moving joints and cracks, where the cause of movement cannot be eliminated, should be made when conditions are such as to produce at or near maximum widths. This requirement would eliminate or reduce tensile strains and stresses in the repair material and surrounding concrete resulting from the bond created at joint or crack by the repair material. By imposing a mostly compressive strain and stress environment the performance of the repair will be improved.

231. There are situations where no lasting repair is likely. In such situations only temporary repairs can be made. For example, surface sealers and injection of cracks can be used to reduce leakage in concrete structures with alkali-silica reaction potential. However, cracking and consequential leakage is likely to continue as long as reactive aggregate, alkali, and moisture are available in the concrete. In the case of intake structures and conduits, it is unlikely that these structures could be sealed sufficiently to prevent the intrusion of moisture necessary for alkali-aggregate reaction to continue. Therefore, any repair of cracks resulting from alkali-aggregate reaction may not result in a lasting repair unless the reaction is dormant for the remaining life of the structure.

PART IV: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

232. Repairs to intake structures and conduits did not always perform as desired as supported by the findings from the WES damage and repair data base (McDonald and Campbell 1985) and the case histories reported herein. In some instances, a number of repairs were attempted at a project before a satisfactory (if any) repair was achieved. Many of the repair products used had been evaluated (including laboratory analysis) by their manufacturer and in some instances by third parties to determine their suitability for use as a repair material for the type repair being considered. Documentation of these evaluations, where appropriate, was considered in planning the repairs. However, the evaluations generally did not take into account the full influence of the repair environment, and, therefore, did not give a true indication of the performance of the product in the hostile environment that exist in intake structures and conduits.

233. It is, therefore, evident that research is needed to better define the performance of such materials and techniques and to look for ways to improve such. It is also evident that in order to better predict repair performances, materials and techniques should be tested in an environment that closely simulates the field environment.

Recommendations

234. It is recommended that a laboratory-field investigation be designed and implemented to evaluate materials and techniques used in the repair of intake structures and conduits. Repair performance criterion should be established as a part of this study. Laboratory tests employed should closely simulate repair environments and conditions. Materials and techniques that have performed successfully in laboratory testing should be further tested in the field. Field tests should be performed as a part of a maintenance and repair contract for scheduled repair work at Corps projects with cost shared by both the district and REMR Research Program. Results of the investigation should be documented in technical reports, The REMR Bulletin, and the REMR Material Data Base.

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