

Compendium of Case Histories on Repair of Erosion-Damaged Concrete in Hydraulic Structures

Reported by ACI Committee 210

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This report is a companion document to ACI 210R. It contains a series of case histories on hydraulic structures that have been damaged by erosion from various physical mechanical and chemical actions. Many of these structures have been successfully repaired. There were many examples to select from; however, the committee has selected recent, typical projects, with differing repair techniques, to provide a broad range of current experience. These case histories cover only damage to the hydraulic surfaces due to the action of water, waterborne material or chemical attack of concrete from fluids conveyed along the hydraulic passages. In addition to repairs of the damaged concrete, remedial work frequently includes design modifications that are intended to eliminate or minimize the action that produced the damage. This report does not cover repair of concrete damaged by other environmental factors such as freeze-thaw, expansive aggregate, or corroding reinforcement.

Keywords: abrasion; abrasion resistance; aeration; cavitation; chemical attack; concrete dams; concrete pipes; corrosion; corrosion resistance; deterioration; erosion; grinding (material removal); high-strength concrete hydraulic structures; maintenance; outlet works; penstocks; pipe linings; pipes (tubes); pittings; polymer concrete; renovating; repairs; sewers; spillways; tolerances (mechanics); wear.

CONTENTS

Chapter 1-Introduction, p. 210.1R-1

Chapter 2-Cavitation-erosion case histories, p.210.1R-2

Dworshak Dam
Glen Canyon Dam
Lower Monumental Dam
Lucky Peak Dam
Terzaghi Dam
Yellowtail Afterbay Dam
Yellowtail Dam
Keenleyside Dam

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

Chapter 3-Abrasion-erosion case histories, p. 210.1R-13

Espinosa Irrigation Diversion Dam
Kinzua Dam
Los Angeles River Channel
Nolin Lake Dam
Pine River Watershed, Structure No. 41
Pomona Dam
Providence-Millville Diversion Structure
Red Rock Dam
Sheldon Gulch Siphon

Chapter 4-Chemical attack-erosion case histories, p. 210.1R-25

Barceloneta Trunk Sewer
Dworshak National Fish Hatchery
Los Angeles Sanitary Sewer System and
Hyperion Sewage Treatment Facility
Pecos Arroyo Watershed, Site 1

Chapter 5-Project reference List, p.210.1R-32

CHAPTER 1-INTRODUCTION

This compendium of case histories provides information on damage that has occurred to hydraulic structures and the various methods of repair that have been used. ACI Committee 210 has prepared this report to help others experiencing similar problems in existing work. Knowledge gained from these experiences may help

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avoid oversights in design and construction of hydraulic structures and provide guidance in the treatment of future problems.

Erosion of concrete in hydraulic structures may occur as a result of abrasive action, cavitation, or chemical attack. Damage may develop rapidly after some unusual event such as a flood or it may develop gradually during normal continuous operation or use. In most cases where damage has occurred, simply replacing the eroded concrete will ensure immediate serviceability, but may not ensure long-term performance of the structure. Therefore, repair work usually includes replacing eroded concrete with a more resistant concrete and additional surface treatment, modifying the design or operation of the structure to eliminate the mechanism that produced the damage, or both. A detailed discussion of mechanisms causing erosion in hydraulic structures, and recommendations on maintenance and repair, is contained in ACI 210R.

When damage does occur to hydraulic structures, repair work poses some unique problems and is often very costly. Direct access to the damaged area may not be possible, or may be limited by time, or other constraints. In some cases, such as repair to spillway stilling basin floors, expensive bulkheads and dewatering are required. It may not be possible to completely dry the area to be repaired or maintain the most desirable temperature. A great deal of planning and scheduling for repair work are normally required, not only for the repairs and access, but also for control of water releases and reservoir levels. If time permits, extensive investigation usually precedes planning and scheduling to determine the nature and extent of damage. Hydraulic model studies may also be necessary to evaluate possible modifications in the design or operation of the facility.

This compendium provides the history on 21 projects with hydraulic erosion damage. They vary in size and cover a variety of problems: 8 with cavitation damage, 9 with abrasion-erosion damage, and 4 with erosion damage from chemical attack. [Table 1.1](#) summarizes the projects. Each repair was slightly different. Each history includes background information on the project or facility, the problem of erosion, the selected solution to the problem, and the performance of the corrective action. Histories also contain references and owner information if further details are needed.

CHAPTER 2-CAVITATION-EROSION CASE HISTORIES

DWORSHAK DAM

North Fork, Clearwater River, Idaho

BACKGROUND

Dworshak Dam, operational in 1973, is a straight-axis concrete gravity dam, 717 ft high, 3287 ft long at the

crest, and contains 6,500,000 cubic yards of concrete. In addition to two gated overflow spillways, three regulating outlets, 12 ft wide by 17 ft high, are located in the spillway monoliths. The inlet elevation for each regulating outlet is 250 ft below the maximum reservoir elevation. Each outlet jet is capable of a maximum discharge of 14,000 ft³/s.

Outlet surfaces are reinforced structural concrete placed concurrently with adjacent lean, large aggregate concrete. Coatings to the outlet surfaces were applied during the original construction. In Outlet 1, the wall and invert surfaces from the tainter gate to a point 50 ft downstream are coated with an epoxy mortar having an average thickness of $\frac{3}{8}$ in. The same area of Outlet 2 was coated using an epoxy resin, approximately .05 in. in thickness. Outlet 3 was untreated.

The outlets were operated intermittently at various gate openings for a period of 4 years between 1971 and 1975, resulting in a cumulative discharge duration of approximately 10 months. The three outlets were not operated symmetrically; outlets 1 and 2 were used primarily.

PROBLEM

Inspection in 1973 showed minor concrete scaling of the concrete wall surfaces of Outlets 1 and 2. One year later, in 1974, serious erosion had occurred at wall surfaces of both outlets immediately downstream of the wall coatings, 50 ft from the tainter gate. Part of this wall area had eroded to a depth of 22 in., exposing and even removing some No. 9 reinforcing bars. In the wall surfaces downstream of Outlet 1 medium damage, up to 1 in. depth of erosion, also occurred in over 60 square yards of surface, bordered by lighter erosion. Every horizontal lift joint (construction joint) along the path of the jet, showed additional cavitation erosion.

SOLUTION

Repairs were categorized as three types:

- Areas with heavy damage, with erosion greater than 2 to 3 in., were delineated by a 3-in. saw cut and the interior concrete excavated to a minimum depth of 15 in. ([Fig. 2.1](#) and [2.2](#)). Reinforcement was reestablished and steel fiber-reinforced concrete (FRC) was used as the replacement material.
- Areas with medium damage, where the depth of erosion was less than 1 in., were bush-hammered to a depth of $\frac{3}{8}$ to 1 in. and dry-packed with mortar. The mortar, if left untreated, would easily have failed when subjected to the high velocity discharge.
- Areas with minor damage, surfaces showing a sand-blast texture, were not separately treated prior to polymer impregnation. The entire wall surfaces of Outlet 1 were then treated by polymer impregnation from the downstream edge of the existing epoxy mortar coating to a distance 200 ft downstream.

TABLE 1.1 — SUMMARY TABLE OF PROJECTS COMPRISING THIS REPORT

Project name	Year completed	Type	Location	Owner	Problem	Repair type	Reference page
Dworshak Dam	1974	Gravity dam	Idaho	Corps of Engineers	Cavitation	Polymer impregnation	210.1R-2
Glen Canyon Dam	1964	Arch dam	Arizona	Bureau of Reclamation	Cavitation	Aeration	210.1R-5
Lower Monumental Dam	1969	Navigation lock	washington	Corps of Engineers	Cavitation	Epoxy	210.1R-6
Lucky Peak Dam	1956	Outlet structure	Idaho	Corps of Engineers	Cavitation	various	210.1R-8
Terzaghi Dam	1960	Outlet structure	British Columbia	B.C. Hydro Authority	Cavitation	Hydraulic redesign	210.1R-9
Yellowtail Afterbay Dam	1966	Stilling basin	Montana	Bureau of Reclamation	Cavitation	Various overlays	210.1R-11
Yellowtail Dam	1966	Stilling basin	Montana	Bureau of Reclamation	Cavitation	Aeration and overlays	210.1R-11
Keenleyside Dam	1968	Outlet structure	British Columbia	B.C. Hydro Authority	Cavitation	High-strength concrete	210.1R-12
Espinosa Irrigation Diversion Dam	1984	Diversion dam	New Mexico	Soil Conservation Service	Abrasion	Steel plate armor	210.1R-13
Kinzua Dam	1965	Stilling basin	Pennsylvania	Corps of Engineers	Abrasion	Silica fume concrete	210.1R-15
Los Angeles River Channel	1940s	Channel	California	Corps of Engineers	Abrasion	Silica fume concrete	210.1R-17
Nolin Lake Dam	1963	Stilling basin	Kentucky	Corps of Engineers	Abrasion	Hydraulic redesign	210.1R-18
Pine River Watershed, Structure No. 41	Proposed	Channel	Colorado	Soil Conservation Service	Abrasion	High-strength concrete	210.1R-19
Pomona Dam	1963	Stilling basin	Kansas	Corps of Engineers	Abrasion	various	210.1R-20
Providence-Millville Diversion Structure	1986	Diversion dam	Utah	Soil Conservation Service	Abrasion	Surface hardener	210.1R-22
Red Rock Dam	1969	Stilling basin	Iowa	Corps of Engineers	Abrasion	Underwater concrete	210.1R-23
Sheldon Gulch Siphon	1991	Syphon outlet	Wyoming	Soil Conservation Service	Abrasion	Polymer-modified mortar	210.1R-25
Barceloneta Trunk Sewer	1976	Pipeline	Puerto Rico	Puerto Rico Aqueduct & Sewer Authority	Chemical attack	PVC lining	210.1R-25
Dworshak National Fish Hatchery	1960s	Concrete tanks	Idaho	Corps of Engineers	Chemical attack	Linings	210.1R-26
Los Angeles Sanitary Sewer System and Hyperion Sewage Treatment Facility	Varies	Sewerage structures	California	City of Los Angeles	Chemical attack	Shotcrete and PVC liners	210.1R-27
Pecos Arroyo Watershed, Site 1	1988	Outlet conduit	New Mexico	Soil Conservation Service	Chemical attack	HDPE liner and hydraulic redesign	210.1R-30

Damage to the epoxy mortar was minimal and located near the outlet gate. This area was repaired with new epoxy.

The polymer impregnation process involved drying all the surfaces to a temperature up to 300 F to drive off water and then allowing the surface to cool to 230 F. Monomer was then applied to the surface using a vertical soaking chamber. Excessive monomer was drained and the surface was polymerized by the application of approximately 150 F water.

PERFORMANCE

Operation of the outlets from the time of repair in 1975 until 1982 has been minimal averaging 1400 ft³/s per outlet with peak discharges of 3600 ft³/s per outlet. Durations of usage are not known. After 1982 outlet discharges increased, with durations exceeding 50 days.

Inspections performed in 1976, the year after the repairs, showed no additional concrete damage except for some minor surface spalling adjacent to a major pre-existing crack in an area of dry-packed mortar. The

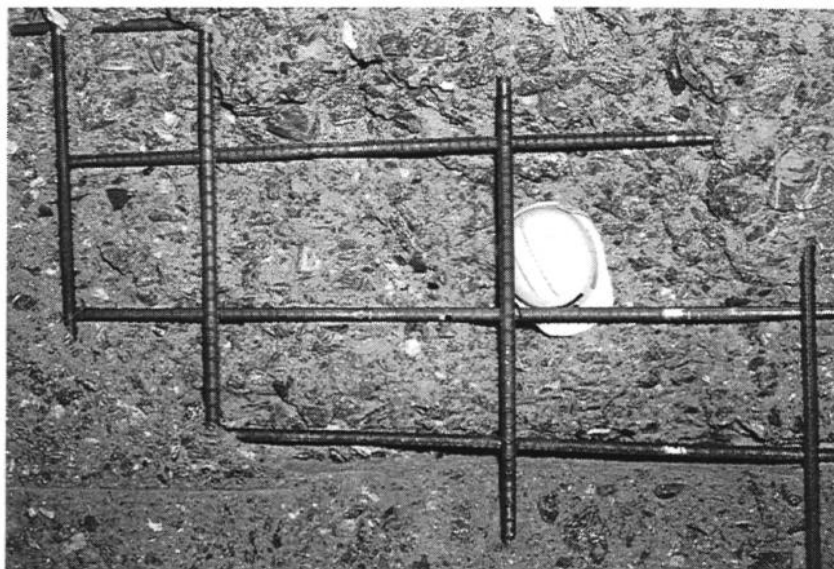


Fig. 2.1-Dworshak Dam. Detail showing depth of erosion behind reinforcing steel



Fig. 2.2-Dworshak Dam. Extent of outlet surface preparation prior to concrete and mortar placements

spalled area was patched with epoxy paste, except that the epoxy paste did not bridge the crack this time. Epoxy resin coating repairs applied to Outlet 2 showed some failures,

Inspections in 1983 and 1988 showed that epoxy mortar coatings in Outlet 1 continued to perform well. Small areas of damage, typically spalls, are periodically repaired with a paste epoxy. Epoxy resin coatings in Outlet 2 are repaired more frequently but are performing adequately. Surfaces repaired with FRC and mortar and subsequently polymer-impregnated showed negligible damage. Polymer-impregnated parent concrete shows a typical matrix erosion around the coarse aggregate to a depth of $\frac{1}{4}$ -in., and lift joints exhibit pitting up to $\frac{3}{8}$ -in. deep. Surfaces

along lift joints not polymer-impregnated show erosion up to $\frac{3}{4}$ -in. in depth and a general surface pitting greater than the companion polymer-impregnated surfaces,

DISCUSSION

Because of variation in the operation of these outlets, both in flow rate and duration, exact time-rate erosion conclusions are difficult to make. Recent outlet discharge has fluctuated annually from moderate flows to none. In general, surfaces that received replacement materials and were subsequently polymer-impregnated have performed well. Original concrete and new polymer impregnated concrete is showing evidence of deterioration at a rate that is less than the non-impregnated surfaces. The

best performance was by the original epoxy mortar coating. The epoxy mortar in Outlet 1 continues to display an excellent surface condition, with no cavitation-generated pitting. The epoxy resin coating in Outlet 2 displays good performance.

In 1988, outlets were modified by adding aeration deflectors, wedges 27 in. wide by 1.5 in. high, to the sides and bottom of each outlet. These deflectors were designed to increase the aeration of the discharge jet and further reduce the cavitation erosion of the outlet surfaces. Subsequent deterioration of the outlet surfaces has not been observed.

The polymer impregnating of the concrete surfaces of the outlets was a very complex system of operations. Success requires continual evaluation of application conditions and flexibility to react to changes in those conditions. Issues relating to safety, cost, and field engineering add significant challenges to a polymer impregnation project. It is doubtful that this process would be attempted today under similar circumstances. It is more likely that the aeration deflectors would be the first remedy considered since they provide a positive solution to the problem without the higher risks of a failure inherent in the polymer impregnation process.

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GLEN CANYON DAM

Colorado River, Northeast Arizona

BACKGROUND

Glen Canyon Dam, operational in 1964, is a concrete gravity, arch structure, 710 ft high with a crest length of 1560 ft. The dam is flanked on both sides by high-head tunnel spillways, each including an intake structure with two 40- by 55-ft radial gates. Each tunnel consists of a 41-ft diameter section inclined at 55 percent, a vertical

bend (elbow), and 985 ft of near horizontal length followed by a deflector bucket. Water first flowed through the spillways in 1980, 16 years after completion of the dam.

PROBLEM

In late May 1983, runoff in the upper reaches of the Colorado River was steadily increasing due to snowmelt from an extremely heavy snowpack. On June 2, 1983, the left tunnel spillway gates were opened to release 10,000 ft³/s. On June 5 the release was increased to 20,000 ft³/s. On June 6 officials heard loud rumbling noises coming from the left spillway. Engineers examined the tunnel and found several large holes in the invert of the elbow. This damage was initiated by cavitation, triggered by discontinuities formed by calcite deposits on the tunnel invert at the upstream end of the elbow. In spite of this damage, continued high runoff required increasing the discharge in the left spillway tunnel to 23,000 ft³/s. by June 23. Flows in the right spillway tunnel were held at 6000 ft³/s. to minimize damage from cavitation. Spillway gates were finally closed July 23, and engineers made a thorough inspection of the tunnels.

Extensive damage had occurred in and near the left tunnel elbow (Fig. 23). Immediately downstream from the elbow, a hole (35 ft deep, 134 ft long, and 50 ft wide) had been eroded in the concrete lining and underlying sandstone foundation. Other smaller holes had been eroded in the lining in leapfrog fashion upstream from the elbow.

SOLUTION

The repair work was accomplished in six phases: 1) removing loose and defective concrete lining and foundation rock; 2) backfilling large cavities in sandstone foundation with concrete; 3) reconstructing tunnel lining; 4) grinding and patching of small defective areas; 5) removing about 500 cubic yards of debris from lower reaches of tunnel and flip bucket; and 6) constructing an aeration device in the lining upstream of the vertical elbow.

Sandstone cavities were filled with tremie concrete before the lining was replaced. About 2000 cubic yards of replacement concrete was used. The aeration slot was modeled in the Bureau of Reclamation Hydraulic Laboratory to ensure that its design would provide the performance required.

The aeration slot was constructed on the inclined portion of the tunnel approximately 150 ft upstream from the start of the elbow. A small 7-in-high ramp was constructed immediately upstream of the slot. The slot was 4 by 4 ft in cross section and extended around the lower three-fourths of the tunnel circumference (Fig. 2.4). All repairs and the slot were completed in the summer of 1983.

PERFORMANCE

Because of the moderate runoff in the Colorado River since completion of the tunnel repairs, it has not been



Fig. 2.3-Glen Canyon Dam. Erosion of spillway tunnel invert and sandstone foundation rock downstream of the elbow

necessary to use the large spillway tunnels. However, shortly after completion of the work, another high runoff period permitted performance of a field verification test. This test lasted 72 hr with a maximum flow during that time of 50,000 ft³/s. The test was conducted in two phases with several interruptions in each for examination of the tunnel. Offsets were intentionally left in place to evaluate whether the aeration slot would indeed preclude cavitation and attendant concrete damage. The tunnel repairs and air slot performed as designed. No sign of cavitation damage was evident anywhere in the tunnel. Aeration has decreased the flow capacity of the spillway tunnels by approximately 20 percent of the original flow capacity.

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CONTACT

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LOWER MONUMENTAL DAM

Snake River, Near Kaloutus, Washington

BACKGROUND

Lower Monumental Dam, operational in 1979, consists of a concrete gravity spillway and dam, earthfill em-

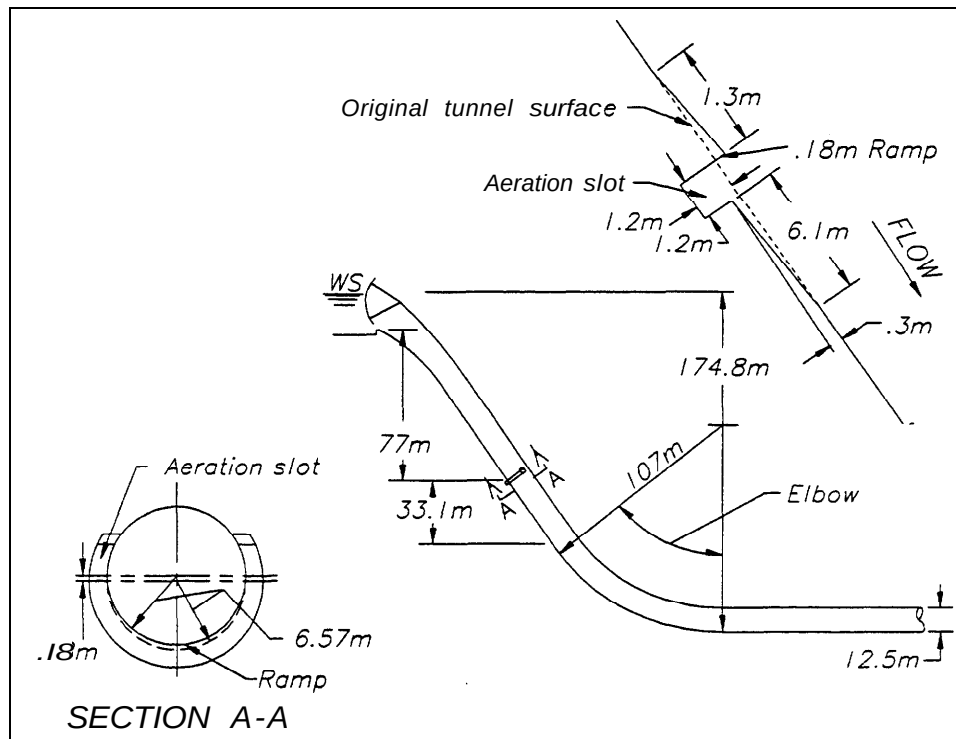


Fig. 2.4-Glen Canyon Dam. Diagram of new tunnel spillway air slot

bankments, a navigation lock, and a six-unit powerhouse.

The 86-ft wide by 675-ft long navigation lock chamber, with a rise of 100 ft, is filled and emptied by two galleries or culverts, landside and riverside of the lock structure. The landside culvert, which supplies five downstream laterals, crosses under the navigation lock to discharge into the river. The riverside culvert supplies and discharges water to the upstream five laterals. Each lateral consists of 10 portal entrances approximately 1.5 ft wide by 3 ft high. Flow velocities in excess of 120 ft/s occur in several of the portals entrances. A tie-in gallery exists between the two main culverts, near the downstream gates, that equalizes the pressure between the two culverts.

PROBLEM

Inspections as early as 1975 revealed that the ceiling concrete of the landslide culvert was spalled at some monolith joints to depths of 9 in. This may have been initiated by differential movement of adjacent monoliths when the lock chamber was filled and emptied. Damage to the invert, at several locations, was irregular, with erosion a maximum of 18 in. deep at the monolith joint, decreasing to 1 in. at a point 10 ft upstream of the joint. Reinforcing steel was exposed. Other areas of erosion in the invert and on wall surfaces were observed, measuring 2 ft square and 2 in. deep.

Later inspections revealed that portal surfaces nearest the culverts of the most downstream laterals were showing signs of concrete erosion (Fig. 2.5). By 1978, the portal walls, ceiling, and invert had eroded as deep as 3 in.

over an area of 5 square ft, exposing reinforcing steel.

All four corners of the tie-in gallery experienced obvious cavitation damage. The damage varied from minor pitting to exposure and undercutting of the 1½-in. aggregate.

SOLUTION

In 1978, the navigation lock system was shut down for two weeks for repairs. The major erosion damage to the landslide culvert was repaired by mechanically anchored steel fiber-reinforced concrete. The smaller areas of damage received a trowel application of a paste epoxy product. Ceiling damage was backfilled with dry-mix shotcrete. Portal and tie-in gallery surfaces received application of a paste epoxy, troweled to a feather edge around the perimeter.

PERFORMANCE

The mechanically anchored fiber-reinforced concrete has performed well to date. No additional erosion has been observed. Shotcrete patches to the ceiling adjacent to the joints show continued spalling, but to a lesser extent than prior to repairs.

The repairs to the portal surfaces and tie-in gallery surfaces performed poorly. After 1 year of service, approximately 40 percent of the epoxy paste had failed; and after 3 years, nearly 100 percent has failed. Concrete erosion in these areas has subsequently increased to depths of 6 to 8 in. in the tie-in gallery and up to 5 to 6 in. on the two most downstream portal surfaces.



Fig. 2.5—Lower Monumental Dam. Cavitation erosion of navigation lock portal surfaces

DISCUSSION

Recent inspections have shown that the rate of erosion has decreased. The accumulated erosion of concrete from certain surfaces is significant; however, subsequent erosion is almost negligible. Consequently, repair schedules are not critical.

Paste epoxy was applied to the concrete surfaces transitioning to feather edges along the perimeter of the patches. Cavitation eroded the concrete adjacent to the feather edges as well as eroding the thin epoxy edges (Fig. 2.5). These new voids undermined the new, thicker epoxy, and at some point caused another failure of the leading edge. As the leading edge void increased in size, the failure progressed until little epoxy was left in the repaired area. After erosion of the epoxy patch material, no further concrete erosion has occurred. It appears that the eroded configuration of the surface is hydraulically stable.

Patch-type repair procedures are not sufficient for this structure because erosion is initiated at the edge of the new patch. Eventual repairs will replace larger areas of the concrete flow surfaces and will include substantial anchoring of new materials.

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LUCKY PEAK DAM

Boise River, Near Boise, Idaho

BACKGROUND

Lucky Peak Dam, operational in 1955, is 340 ft high with a crest length of 2340 ft. The dam is an earth and rockfill structure with a silt core, graded filters, and rock shells. The ungated spillway is a 6000-ft-long ogee weir discharging into an unlined channel. The outlet works consists of a 23-ft-diameter steel conduit that delivers water to a manifold structure with six outlets. Each outlet is controlled by a 5.25-ft by 10-ft slide gate. Individual flip lips were constructed downstream from each slide gate. Downstream of the flip lips is the plunge pool, excavated into the basalt rock, with bottom areal dimensions of 150 by 150 ft. The outlet alignment and design were determined by hydraulic modeling. The six outlets operated under a maximum head of 228 ft with a design discharge of 30,500 ft³/s and a maximum discharge velocity ranging between 88 ft/s and 124 ft/s.

PROBLEM

The steel manifold gates have a long history of cavitation erosion problems. The original bronze gate seals were seriously damaged by cavitation after initial use. Flow rates across the manifold gate frames in excess of 150 ft/s for many days were common. The gate seals were replaced with new seals made of stainless steel and aluminum-bronze. The cast-steel gate frames required continual repair of cavitated areas. In 1975 alone, over 2000 pounds of stainless steel welding rod was manually welded into the eroded areas and ground smooth. Neat cement grout was pumped behind the gate frames to re-establish full bearing of the gate frames with the concrete structure.

The concrete invert and side piers, which separate each of the six flip lips suffered extensive erosion soon after the start of operations in 1955 (Fig. 2.6). 3/4-in.-thick steel plates were anchored to the piers and invert areas just downstream of the manifold gates. These steel wall plates became severely pitted, as did the downstream concrete flip lip invert surfaces. In 1968, the damaged plates were again repaired by filling the eroded areas with stainless steel welding, and grouting behind the plates. Deteriorated concrete on the flip lips was removed and additional steel plates were installed over those areas. This also failed and repairs commenced again. Deep areas of cavitation damage in the invert and piers were filled with concrete. New 1/2-in.-thick plates were installed. These were stiffened with steel beams welded on 5-ft centers in each direction. Deep anchor

bars were welded to the plate material to hold them in place. Again, the voids under the plates were grouted. But during the next two years, these repairs also failed.

In 1974, it was recommended that the outlet be restudied hydraulically. That year, remaining plate material was removed. Cavities were found penetrating the invert and through the piers and into the adjacent outlet invert. These voids were crudely filled with FRC in a “field expedient” manner. Much of this FRC was placed in standing water with little quality control, while adjacent bays were discharging.

SOLUTION

The side piers were redesigned and replaced to provide vents that would introduce air to the underside of the jet just downstream of the gates. This modification was intended to prevent additional invert erosion. However, major modifications to the gates and gate frames were necessary if cavitation erosion was to be eliminated. These modifications were not made since future powerhouse construction would reduce and nearly eliminate the need to use the outlet, reserving the structure for emergency and special operations use only. Steel lining on the piers was strengthened and replaced. Stiffened steel plates, 1 $\frac{1}{4}$ -in. thick, were installed on the piers and invert. Mortar backfill was pumped behind the invert plates and new concrete placed between pier plates.

PERFORMANCE

After one year of above average usage on bays 3 and 4, cavitation was again observed. The side piers just downstream of the gates showed areas of 1 to 2 square ft that had eroded through the steel plate and into the concrete about 6 in. No erosion of the invert plates or the “field expedient” FRC occurred. Use of these bays has almost stopped since the new powerhouse became operational.

DISCUSSION

The introduction of air beneath the jet appears to have cushioned the effects of cavitation on the flip lip invert. However, pier walls continue to erode at an extraordinary rate. The cause lies with the design of the gates and gate frame. It is evident that satisfactory performance of the structure can never be achieved until the gates and frames are redesigned and reconstructed to eliminate the conditions that cause cavitation.

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Fig. 2.6-Lucky Peak Dam. Cavitation erosion of flip lip surface

Inspection Report No. 7, Lucky Peak Lake,” Walla Walla, WA, Jan. 1989.

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TERZAGHI DAM

Bridge River Near Lillooet, British Columbia, Canada

BACKGROUND

Terzaghi Dam, operational in 1960, is 197 ft high with a crest length of 1200 ft. The earth and rockfill embankment consisting of an upstream impervious fill, clay blanket, sheet pile cutoff, and multiline grout curtain, is founded on sands and gravels infilling a deep river channel. The dam impounds Bridge River flow to form the Carpenter Lake reservoir, from which water is drawn through two tunnels to Bridge River generating stations 1 and 2, located at Shalalth, B.C., on Seton Lake.

Terzaghi Dam discharge facilities are composed of a surface spillway consisting of a 345 ft long free overflow section; and a gated section with two 25 ft wide by 35 ft high gates. Two rectangular low level outlets (LLO), each

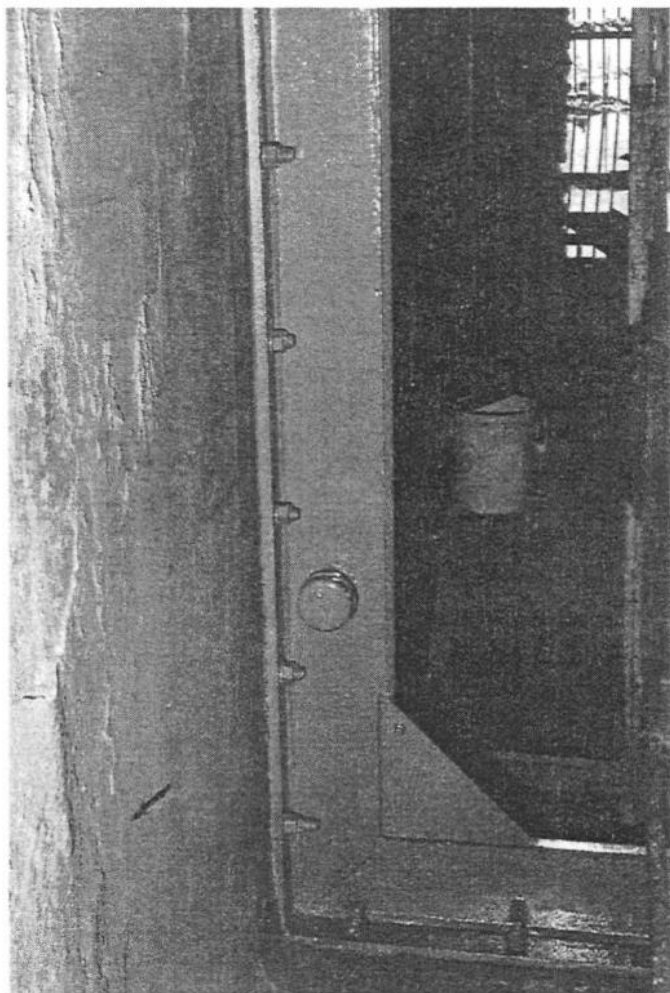


Fig. 2.7-Terzaghi Dam. Downstream detail of constrictor ring

8 ft wide by 16 ft high are subject to a maximum head of 169 ft. These outlets were constructed in the top half of the concrete plug in the 32 ft, horseshoe-shaped diversion tunnel.

PROBLEM

The LLOs were operated in 1963 for about 23 days to draw down Carpenter Lake to permit low-level embankment repairs. Severe cavitation erosion of the concrete wall and ceiling surfaces downstream of bulkhead gate slots was observed in the north LLO after the water release.

Dam safety investigations in 1985 identified that the LLOs were required to permit emergency drawdown of Carpenter Lake for dam inspection and repair, and to provide additional discharge capacity during large floods.

SOLUTION

The repair consisted of three main categories of work — repair of damage, improvement to reduce cavitation potential, and refurbishing gates and equipment.

Repair of cavitation damage in the north LLO in-

cluded repair of the walls, crown, and gate slots.

Improvements to reduce cavitation potential included 1) installing 9-in. deep rectangular constrictor frames (Fig. 2.7) immediately downstream of the operating gates to increase pressures in the previously cavitated area, 2) backfilling old bulkhead gate slots and streamlining the existing LLO invert entrances, and 3) installing piezometers in the north LLO to provide information on flow characteristics of the streamlined LLO during discharge testing.

Refurbishing gates and equipment included 1) replacing leaking gate seals on closure gates; 2) sand-blasting and repainting gates, guides, head covers, and air shafts; 3) cleaning gate lifting rods and replacing bonnet packings; 4) replacing ballast concrete in north LLO gates and installing ballast cover plates on all gates; and 5) refurbishing hydraulic lifting mechanisms of gates.

Repair concrete was designed to fully bond with existing concrete. Surface preparation included; saw cutting around the perimeter of the damage, chipping to expose rebar, and installation of grouted dowels. Latex-modified concrete was used for all repair work, with steel fiber reinforcement for the cavitation-damaged areas.

A total of 26 cubic yards of 3000 psi ready-mixed concrete was placed by pumping. Maximum aggregate sizes of $\frac{3}{8}$ -in. and $\frac{3}{4}$ -in. were used for general repair and invert entrance backfill, respectively.

The constrictor frames were made from $\frac{1}{2}$ -in. and $\frac{3}{4}$ -in. steel plate. They were installed in the LLOs by means of the following: 1) bolting the constrictor frame to the existing concrete with a double row of 1-in. diameter adhesive anchors at 12-in. spacing 2) keying the constrictor infill concrete into the existing concrete; 3) welding the constrictor frame to the existing gate metalwork in the walls and soffit; and 4) embedding the constrictor sill shear bar into the existing concrete invert (Fig. 2.7).

PERFORMANCE

A test with a full reservoir and a peak discharge of 7000 ft³/s, with both gates opened 7 ft, verified that the constrictor frames and concrete repairs, downstream of the closure gates, performed as designed. No cavitation erosion of the wall and ceiling surfaces was observed.

DISCUSSION

Piezometer readings confirmed that the constrictor frames in the LLOs helped maintain pressures above atmospheric, indicating that cavitation should not be a problem in the future.

REFERENCES

B.C. Hydro, "Terzaghi Dam, Low Level Outlet Repairs-Memorandum on Construction," Report No. EP6, Vancouver, B.C., Dec. 1986.

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