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DOCUMENTATION OF OPERATION, DAMAGE, REPAIR, AND TESTING OF YELLOWTAIL DAM SPILLWAY

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Bureau of Reclamation

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Prototype operation of the Yellowtail Dam tunnel spillway in 1967 severely damaged the tunnel. Spillway operation and damage, and subsequent model tests, repairs, modifications, and prototype tests are described. Cavitation initiated by surface irregularities in the tunnel lining caused the damage. Heaviest damage was concentrated in the vertical bend of the tunnel and the near horizontal reach just downstream from the bend. Repairs consisted of backfill concrete in the heavily damaged areas and application of an epoxy-bonded epoxy-mortar veneer to cover minor surface irregularities. Modification consisted of constructing an air slot to introduce air along the flow surfaces of the vertical bend. Model tests of the tunnel showed that the optimum location of the air slot was just upstream from the vertical bend. The air slot provided positive air induction along the flow surfaces and worked satisfactorily for all ranges of discharge. Tests of the prototype tunnel after repair and modification confirmed that a sufficient volume of air was being introduced to prevent cavitation erosion.

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PREFACE

During the spring of 1967, heavy rains in the watershed area of the Bighorn River resulted in high inflows in Bighorn Lake behind Yellowtail Dam. To control the reservoir, releases from the spillway were initiated on June 26, 1967. Releases through the spillway varied during the period June 26 until July 28, but never exceeded 18,000 cfs. The total spill lasted more than 30 days. During that time approximately 650,000 acre-feet of floodwaters were discharged exclusively through the spillway.

The spillway is an open-channel flow, tunnel-type, constructed through the left abutment rock, and consists of a gated intake structure, an incline section of varying diameter, a vertical bend, a near-horizontal section, and a combination stilling basin flip bucket structure. The tunnel diameter varies uniformly from 40 feet 6 inches at Station 6+50 to 32 feet 0 inch at Station 7+95.079. Thereafter, the diameter is a constant 32 feet 0 inch to the outlet portal at Station 22+33.

During the spill, severe damage occurred to the concrete lining and underlying rock between Station 9+00 and Station 11+65. Lesser damage occurred upstream from Station 9+00 and downstream from Station 11+65. All of the damage downstream from Station 7+79 was traceable to cavitation erosion initiated by surface irregularities in the tunnel lining.

Corrective procedures were established and a major design modification made. The design modification was for the construction of an aeration slot at Station 7+79. The corrective procedures involved repair to the damaged areas with backfill concrete, epoxy-bonded concrete, and epoxy bonded epoxy mortar.

The documentation that follows describes the use of the near-horizontal tunnel section in the diversion plan, the resulting erosion, the operation of the spillway in 1967, and the ensuing damage and subsequent repair, modification, and prototype testing.

Various problems and difficulties that occurred during construction of the aeration slot, and the repair of both severely and minor damaged areas are described in detail. The experiences gained and the observations made should benefit other Bureau offices and personnel.

PART 1

TUNNEL SPILLWAY OPERATION IN 1967

A. Condition of Tunnel Prior to Making Releases

The spillway stilling basin and the 32-foot-diameter spillway section from tunnel Station 10+31.47 to tunnel Station 22+33, together with the 28-foot-diameter diversion tunnel that transitioned into the spillway at Station 10+31.47, were used to divert the river through the left abutment of the construction area. These structures were completed early in January 1963. On January 22, 1963, the river was diverted into the tunnel and remained in that state until closure on November 3, 1965. During that period, river sediments eroded the invert in varying amounts from the diversion tunnel intake to the end of the spillway stilling basin. Since the portion of the diversion works upstream from tunnel Station 10+31.47 was to be abandoned after diversion, the nature and extent of erosion was of concern only downstream from Station 10+31.47.

The invert surface from tunnel Station 10+31.47 to the outlet of the spillway tunnel, Station 22+23, was eroded down to exposed aggregate. The erosion was deepest at the centerline of the tunnel, decreased uniformly, and ended approximately 10 feet (measured horizontally) either side of the tunnel centerline. Tunnel cross sections from Station 10+40 to Station 21+65, Table 1-1, show a maximum erosion of about 1-1/2 inches.

In general, the invert concrete had eroded uniformly to a fairly smooth surface with most of the exposed aggregate eroded level with the grout matrix, see Figure 1-1. However, depressions due to aggregate popout, local scouring, and gouges in the concrete existed at random locations throughout the invert. The reach between tunnel Station 19+95 and tunnel Station 20+65 was particularly affected with the most damage concentrated at construction joints in the invert.

The portion of the tunnel lining above the invert concrete was placed utilizing a continuous circular form. The lining surface above the invert and extending to the spring line (the area curving upward and away from the form surface) had been affected by the formation of air bubbles during placement, leaving small pits or "bug holes" in the surface. "Bug holes" occurred throughout the tunnel downstream from

Figure 1-1. Typical erosion pattern in the invert lining. Photo P459-640-3667 NA

Station 10+32±. In general, the diversion flows had given the "bug holes" a teardrop shape. Most of these surface irregularities were very shallow and about 3/4-inch long in the upstream-downstream direction with only occasional "bug holes" greater than 1/4-inch deep and 1-inch long. Figure 1-2 is representative of "bug holes" in the lining.

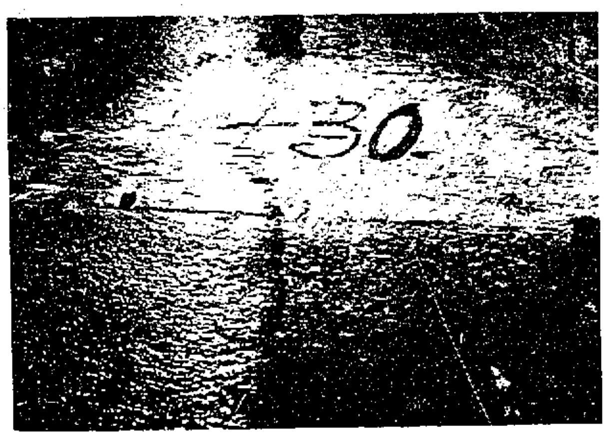


Figure 1-2. View of surface pits in the tunnel lining. These pits or "vugs" resulted from air bubbles formed during placement of this portion of the lining. Photo P459-640-3671 NA

The condition of the stilling basin invert was excellent considering the amount of silt-laden materials passed during diversion. The surface was sufficiently eroded to expose aggregate, see Figure 1-3, but the texture of the eroded surfaces was generally smoother than similarly affected areas in the spillway tunnel. The depth of

Table 1-1

TABULATION OF DEPTH OF EROSION IN SPILLWAY TUNNEL (FEET)
YELLOWTAIL DAM

Tunnel Station	Distance from template to concrete surface*						
	10 feet left	6 feet left	3 feet left	At ℄	3 feet right**	6 feet right	10 feet right
10+40	0.00	0.04	0.10	0.10	0.10	0.08	0.00
10+50	.00	.04	.09	.12	.10	.05	.00
10+70	.00	.06	.11	.13	.12	.06	.00
10+80	.00	.04	.08	.10	.09	.05	.00
10+90	.00	.05	.11	.11	.10	.05	.00
11+00	.00	.03	.07	.09	.07	.03	.00
11+10	.00	.05	.10	.11	.10	.05	.00
11+20	.00	.05	.11	.10	.10	.07	.00
11+30	.00	.02	.06	.10	.07	.03	.00
11+40	.00	.06	.04	.01	.05	.04	.00
11+50	.00	.02	.03	.05	.04	.02	.00
11+60	.00	.04	.05	.05	.06	.06	.00
11+70	.00	.02	.05	.07	.10	.08	.00
11+80	.00	.02	.04	.07	.06	.05	.00
11+90	.00	.02	.05	.05	.06	.03	.00
12+00	.00	.04	.05	.05	.06	.05	.00
12+25	.00	.02	.09	.09	.07	.02	.00
12+50	.00	.03	.06	.04	.06	.05	.00
12+75	.00	.02	.07	.08	.04	.02	.00
13+00	.00	.02	.10	.07	.10	.05	.00
13+50	.00	.05	.07	.04	.05	.03	.00
13+75	.00	.01	.05	.06	.05	.03	.00
14+00	.00	.01	.05	.04	.05	.03	.00
14+25	.00	.01	.05	.06	.05	.03	.00
14+50	.00	.02	.04	.05	.04	.02	.00
14+75	.00	.02	.06	.05	.06	.02	.00
15+00	.00	.02	.05	.10	.08	.03	.00
15+25	.00	.02	.11	.13	.11	.02	.00
15+50	.00	.03	.08	.09	.09	.04	.00
15+75	.00	.04	.11	.12	.11	.06	.00
16+00	.00	.06	.09	.08	.07	.04	.00
16+25	.00	.04	.09	.13	.10	.05	.00
16+50	.00	.05	.08	.11	.07	.04	.00
16+75	.00	.04	.08	.10	.08	.06	00
17+00	.00	.04	.08	.09	.06	.04	.00
17+25	.00	.04	.08	.10	.08	.03	00
17+50	.00	.02	.06	.10	.08	.03	.00
17+75	.00	.03	.05	.05	.05	.02	.00
18+00	.00	.03	.08	.05	.05	.01	.00
18+25	.00	.03	.06	.06	.06	.04	.00
18+50	.00	.04	.08	.09	.08	.05	.00
18+75	.00	.02	.05	.08	.09	.04	.00
19+00	.00	.04	.08	.09	.09	.03	.00
19+25	.00	.04	.08	.10	.09	.05	.00
19+50	.00	.03	.07	.08	.07	.04	.00
19+75	.00	.03	.07	.10	.10	.04	.00
20+00	.00	.04	.10	.10	.10	.06	.00
20+25	.00	.06	.10	.15	.10	.05	.00
20+50	.00	.02	.08	.09	.08	.03	.00
20+77.97	.00	.03	.07	.10	.09	.04	.00
21+00	.00	.04	.09	.10	.07	.04	.00
21+25	.00	.05	.09	.11	.07	.05	.00
21+50	.00	.05	.10	.09	.11	.07	.00
21+65	.00	.04	.08	.08	.09	.04	.00

*The distances shown in the tabulation were taken along and normal to the template and existing concrete. The template was resting on concrete at its extremities; chord and radius lengths of the template were 18.65 and 16 feet, respectively.

**Distance shown is measured horizontally from tunnel centerline. Left or right refers to direction from centerline looking downstream.

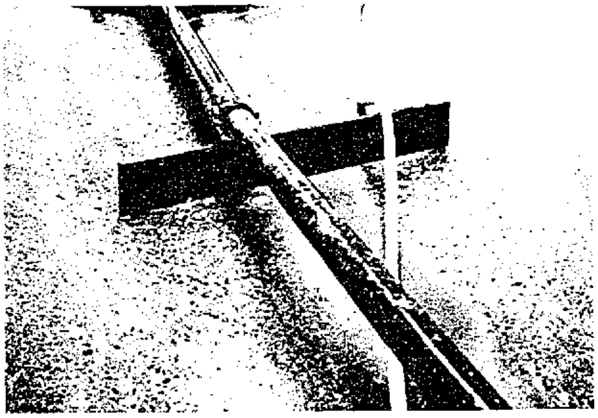


Figure 1-3. Typical erosion pattern in invert of stilling basin. Photo P459-D-55582 NA

erosion was greatest at the basin centerline and decreased uniformly to the original surface in a distance of 6 to 8 feet from the centerline. All construction joints were in good condition and free of spalling or excessive erosion except for a small pocket of spalled concrete at a joint between the curved flip bucket and the right sidewall of the basin. Several small nicks in the lip of the flip bucket had occurred, and erosion had exposed aggregate the full width of the basin, but both nicks and erosion were insignificant. Refer to Figures 1-4 and 1-5 for the lip erosion and a general view of the stilling basin.

The invert of the vertical curve between the tunnel outlet portal and the stilling basin was generally eroded to a rougher texture than the basin invert. Vertical curve alinement appeared excellent with only a small hump noted at the construction joint at Station 22+33.

Since the texture of the invert of the spillway tunnel had been affected enough by diversion flows to present a cavitation problem during high spillway discharges, remedial measures were formulated to restore the invert surfaces to an acceptable condition. Restoration of the tunnel surfaces downstream from Station 10+31.47 were made according to the following guidelines:

1. The contact line (below the spring line) between the new elbow concrete (at Station 10+31.47) and the existing concrete was ground as smooth as possible with all offsets and adjacent gradual irregularities eliminated on a 1:100 bevel.

2. All of the eroded invert concrete from Station 10+31.47 to Station 10+40 was bushhammered and ground smooth to a texture similar to that in the

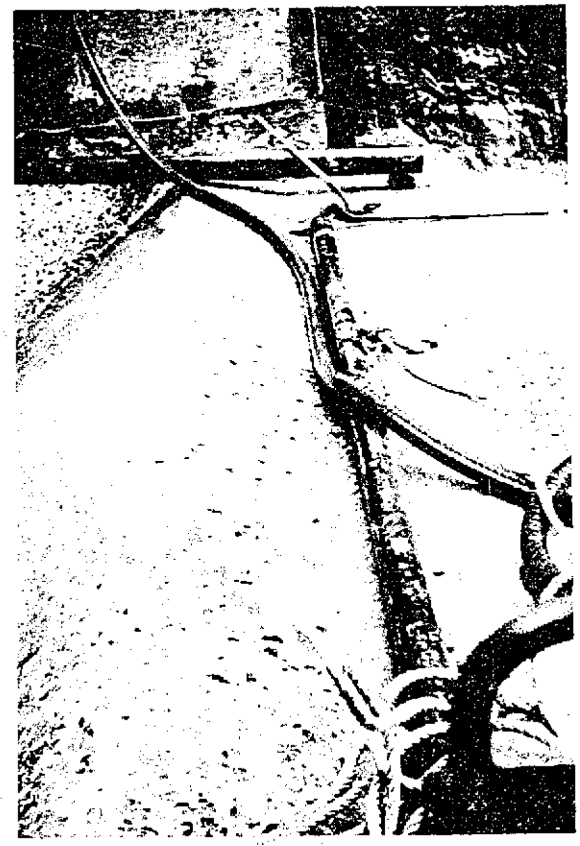


Figure 1-4. Eroded concrete surfaces downstream of bucket lip. Stoplog seal plate is undamaged. Photo P459-D-55583 NA

elbow concrete. All gradual or abrupt irregularities in this reach, parallel and not parallel to the flow, were eliminated on a 1:100 bevel.

3. All of the eroded invert concrete from Station 10+40 to Station 10+55 was bushhammered and ground such that the texture of the treated area transitioned from a smooth surface (similar to elbow surface concrete) at Station 10+40 to a texture left by bushhammering at Station 10+55. All gradual or abrupt irregularities in this reach, parallel and not parallel to the flow, were eliminated on a bevel of 1:100.

4. All of the eroded invert concrete from Station 10+55 to Station 10+80 was bushhammered to transition to the existing eroded surface at Station 10+80. All gradual or abrupt irregularities in this reach, not parallel to the flow, were eliminated on a bevel of 1:100 at Station 10+55 transitioning to a bevel of 1:50 at Station 10+80. Gradual or abrupt

Figure 1-5. View of spillway stilling basin. Photo P459-D-55581 NA

irregularities, parallel to the flow, were eliminated on a bevel of 1:100 at Station 10+55 transitioning to a bevel of 1:25 at Station 10+80. Large depressions or gouges in the invert were filled with epoxy-bonded epoxy mortar or by concrete patches keyed into the original concrete.

5. From Station 10+80 to Station 11+30 eroded surfaces were not treated except that all gradual or abrupt irregularities, not paralled to the flow, were eliminated by bushhammering on a bevel of 1:50 at Station 10+80 transitioning to a bevel of 1:30 at Station 11+30. Gradual or abrupt irregularities, paralled to the flow, were eliminated by bushhammering on a bevel of 1:25 at Station 10+80 transitioning to a bevel of 1:15 at Station 11:30. Depressions or gouges in the invert that could not be eliminated by bushhammering were filled with epoxy-bonded epoxy mortar or concrete patches keyed into the original concrete.

6. Downstream from Station 11+30 and extending to Station 20+77.97 (P.C. of vertical curve), the eroded surfaces were not treated except that gradual or abrupt irregularities, not parallel to the flow, *were eliminated* by *bushhammering* on a bevel of 1:30. Gradual or abrupt irregularities, parallel to the flow, were eliminated by bushhammering on a bevel of 1:15. Depressions or gouges in the invert that could not be eliminated by bushhammering, and the deteriorated concrete at construction joints were repaired with epoxy-bonded epoxy mortar or concrete patches keyed into the original concrete.

7. Downstream from Station 20+77.97 to the downstream end of the stilling basin, the eroded surfaces were not treated except that all gradual or abrupt irregularities not paralled to the flow were eliminated on a bevel of 1:50 and those parallel to the flow were eliminated on a bevel of 1:25. Gouges and depressions in the invert that could not be eliminated by bushhammering, and deteriorated concrete at construction joints were repaired with epoxy-bonded epoxy mortar or concrete patches keyed into the original concrete.

8. From Station 10+31.47 to Station 11+30 and below the springline, all "bug holes" 1/4-inch and greater in depth, and 3/4-inch and greater in length (measured in the upstream-downstream direction) were routed and backfilled with dry-pack mortar.

9. From Station 11+30 to the downstream end of the tunnel and below the springline, all "bug holes" 1/4-inch and greater in depth and 1-1/2 inches in length (measured in the upstream-downstream direction) were routed and backfilled with dry-pack mortar.

In order to evaluate surface irregularities subjected to high-velocity flows with a fully developed boundary layer, the following irregularities were not repaired as described in the preceding paragraphs:

1. A gouged area, 2.6 feet right of tunnel centerline, at Station 11+60. The area was about 6 inches wide and 3/4-inch deep.

2. A group of "bug holes," 12 feet right of tunnel centerline, at Station 11+95.

3. A large "bug hole," 13.3 feet left of tunnel centerline, at Station 13+67.

4. A gouged area, 4 feet right of tunnel centerline, at Station 17+96. The area was about 4 inches wide and 1/4-inch deep.

5. A gouged area, tunnel centerline along a construction joint, at Station 18+94.

6. Deteriorated concrete around the end of a steel form tie, 8.6 feet right of tunnel centerline, at Station 20+53.

7. Painted sections in the invert, extending 2 feet each side of the centerline and 3 to 5 feet wide, at Stations 20+69, 16+90, 14+90, and 12+90.

In addition to the repairs in the spillway tunnel, Station 10+31.47 to Station 22+33, and stilling basin, the inclined portion of the spillway from the intake transition to Station 7+95.08 (P.C. of the tunnel elbow) also required repairs to correct rock pockets caused by insufficient vibration of concrete, holes resulting from form ties, and grouting and drainage operations, spalls, and "bug holes."

Repairs throughout the tunnel were made by the dry-pack method, concrete patches keyed into the original concrete, and epoxy-bonded epoxy mortar. The epoxy resins used were Probond ET-150 and Concresive 1063-1. All repairs in the tunnel and stilling basin were completed by December 16, 1966.

B. Description of Spillway Operation during the June-July Flood of 1967

During the period from June 15 through June 29, releases from the reservoir were controlled to help reduce the flooding Yellowstone River at Miles City. At midnight, June 15, the reservoir at Yellowtail Dam was approximately at elevation 3620, 20 feet below the top of joint-use space. On June 23, the reservoir reached elevation 3640, the bottom of the exclusive flood-control space. Releases at that time were being made only through the powerplant and averaged about 7,600 cfs. By June 26, the reservoir was almost 9 feet into the exclusive flood-control space and some concern was felt for the rapid decrease of space. Releases past the dam were increased to 10,645 cfs, including an initial spillway release of 3,000 cfs. Spillway discharges were increased gradually until a momentary peak of 12,000 cfs was reached on June 27, after which the discharge was adjusted to 10,500 cfs on June 28. Because a rise above flood stage at Miles City was forecast, total releases past the dam were reduced to 17,000 cfs (7,000 cfs through spillway) on June 28 and to 15,000 cfs (5,000 cfs through spillway) on the morning of June 29. When additional data on reservoir inflows became available later on June 29, it was decided that the remaining flood-control space was the critical factor rather than the flooding on the Yellowstone River. Accordingly, the total releases were increased to 20,000 cfs at 12:30 p.m. on June 29, with the spillway allocation adjusted to an average discharge of about 10,000 cfs. The spillway was operated at this discharge until the middle of July 3. Inflows into the reservoir continued high on July 3 and with a rapidly decreasing flood-control space, the total releases were increased to 24,000 cfs (13,000 cfs through the spillway) on July 3, and to 25,000 cfs (14,500 cfs through the spillway) on July 4. Until the 14,500 cfs discharge on July 4, the energy from all previous spillway releases was dissipated in the spillway stilling basin through a hydraulic jump. However, at 14,500 cfs and for the existing tailwater, the jump swept out of the basin and was flipped into the river. The flow remained in the flipped attitude with the discharge averaging about 16,000 cfs from the middle of July 5, to the evening of July 7. Peak discharges during that period fluctuated for several hours from 16,000 cfs to a maximum of 18,000 cfs. The fluctuations occurred when releases from the river outlets were exchanged to the spillway to permit temporary shutdown and inspection of the outlet works stilling basin. On the evening of July 7, the spillway discharge was stabilized at 15,000 cfs and remained in that position until July 13. For 6 hours on July 13, spillway discharges were increased to 18,000 cfs when releases from the river outlets were again exchanged to the spillway. The outlets shutdown was required to permit maintenance work on the louvers for the air intake to the outlet hollow-jet valves. After completion of the work, the outlets were reactivated and the spillway discharge reduced to 15,000 cfs.

At 5:30 p.m., Friday evening, July 14, the 15,000 cfs discharge from the spillway basin suddenly quit sweeping out. Several attempts involving increased spillway flows to 15,500 cfs and a decreased tailwater, were made to again flip the flow from the basin. None of the attempts were successful and the spillway discharge was reduced to 9,000 cfs. At 10:10 p.m. the discharge was increased to 15,000 cfs and held until 11:00 p.m. The spillway basin would not sweep out and the spillway discharge was again reduced to 9,000 cfs, and reduced further to 8,300 cfs on July 18. Since a complete spillway shutdown was impossible due to the need to further evacuate the inviolate flood-control space, the spillway was operated at 8,300 cfs until July 23. On that day, the spillway releases were decreased gradually until complete shutdown was achieved at 8:00 a.m., July 25. Complete spillway shutdown was maintained for about 10 hours to permit a scuba diver examination of the tunnel lining. Major damage was found in the near horizontal section and in the elbow. Because additional reservoir evacuation was desired, the spillway was again activated and by the beginning of July 26, was operating at 3,000 cfs discharge. The

3,000 cfs discharge was maintained until the morning of July 28 when the flood danger had passed and *complete shutdown* could be made.

Except for the 10-hour shutdown July 25, the spillway was operated continuously from June 26 through July 28. In that period, approximately 650,000 acre-feet of floodwaters were passed at discharges that varied from zero to a maximum of 18,000 cfs. Maximum total releases through all waterways were 27,000 cfs on July 6. The reservoir peaked at elevation 3656.44 at 4:00 a.m. on July 6. Elevation 3656.44 is 0.56 foot below the top of the *exclusive flood-control space* and 3.56 feet below the crest of the dam. Figure 1-6 shows the operation of all waterways during the June 26 through July 28 period.

C. Condition of Spillway after 1967 Operation

As stated in previous paragraphs, the tunnel spillway was examined on July 25, 1967, by scuba divers. They reported the following:

1. Station 11+26±.—A hole 8 to 10 feet long in the upstream-downstream direction, about 35 feet wide in the transverse direction and about 6 to 8 feet deep.

2. Station 9+50±.—A hole 10 feet long, 5 feet wide, and 2 feet deep.

3. Station 9+00±.—A hole 12 feet long, 4 feet wide, and 1 foot deep.

4. The stilling basin was reported in good condition. Various amounts of broken concrete and rock were reported in the basin with the largest single piece of concrete (6 by 8 by 10 feet) located just at the beginning of the basin.

The area adjacent to Station 9+00 was examined also by Bureau personnel who noted that the concrete surfaces were badly pocked by minor cavitation and erosion damage in addition to the large area reported under Item 3.

After complete shutdown of the tunnel on July 28, arrangements were made to dewater the tunnel for a closeup examination of the concrete lining.

The examination was conducted August 14 through August 16, and described below:

Stilling basin: The basin was essentially in good condition. The large 6- by 8- by 10-foot block of concrete reported during the scuba diver examination was located at the upstream end of the horizontal invert of the basin. Immediately upstream from this block 10 to 12 cubic yards of rock and concrete fragments were located. These fragments ranged from a few inches to 1 foot in diameter.

The vertical curve invert from the stilling basin to the outlet of the tunnel, Station 22+33, was heavily eroded to a maximum depth of 3 inches. Nowhere was the erosion sufficiently deep to expose *reinforcement. Several gouged* areas within the eroded matrix were 1-1/2 inches deep and about 5 inches across.

The invert surfaces upstream from the tunnel portal to about *Station 20+70 appeared very similar in* texture to that observed before spillway operation. The depth of erosion was considerably less than that noted in the invert downstream from the portal. Epoxy-mortar patches, placed to fill in gouged areas and deteriorated concrete at construction joints that resulted during diversion flows, generally were unsatisfactory. The epoxy mortar was rubbery and poorly bonded to the matrix.

The tunnel from Station 20+70 to Station 17+90 appeared to be in the same condition as before spillway operation. Surface irregularities at Stations 20+53, 18+94, and 17+96 that were not repaired after diversion was completed were unchanged in appearance nor did they initiate cavitation. Areas that had been bushhammered in this reach to remove high points in the invert remained intact and were not contributory to any damage. The painted section at Station 20+69 was also unchanged. The paint was in good condition except for minor scratches incurred by material moving along the section during spillway operation.

Upstream from Station 17+90 and extending to Station 12+90, the tunnel surfaces were undamaged and similar in appearance to the surface observed after diversion. Painted sections at Stations 16+90, 14+90, and 12+90 remained in good condition.

The largest damaged area began at Station 10+40 (8 feet plus or minus downstream from the P.T. of the tunnel elbow), ended at Station 11+65, and was centered approximately on the tunnel centerline. The cause of cavitation damage was failure of an epoxy patch, 1/4 inch deep and 6 inches (width) by 10 inches (length) in area, placed to fill in an eroded area in the invert.

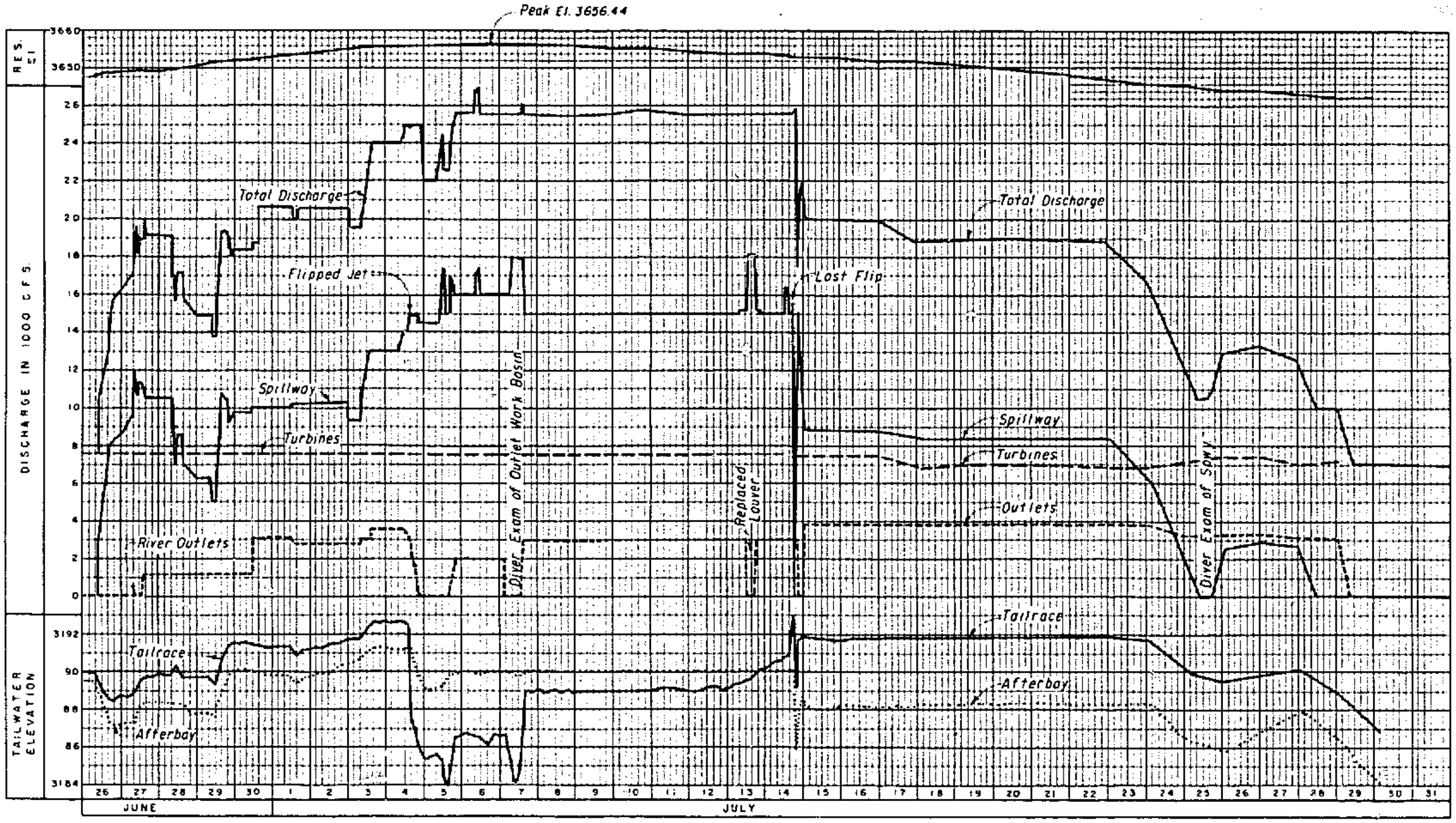


Figure 1-6. Operation of waterways during June-July flood of 1967.

The remains of epoxy mortar could be seen around the perimeter of the depression. Cavitation damage started immediately downstream from the depression in the form of a small cavitated area about 3 feet long, followed by a large tear-drop shaped area about 20 feet long. Damage in the latter area was sufficiently deep to expose reinforcement. Immediately downstream from Station 10+80± to Station 11+26± the cavitation damage extended through the concrete lining and into the limestone foundation. The depth of damage was almost 7 feet maximum and extended 19.5 feet in the transverse direction. Portions of the concrete lining along the sides and in the downstream direction were undercut as much as 6 feet. Downstream from Station 11+26± the damage necked down for a short distance then widened to 16 feet at Station 11+45. Downstream from Station 11+45, the damaged area again constricted, widened to 5 feet at Station 11+58±, and terminated at Station 11+65±. Maximum depths of damage at Stations 11+45 and 11+58± were 3.5 and 1.5 feet, respectively. The tunnel between Stations 11+65 and 12+90 was undamaged except for minor gouges left and right of the tunnel centerline at Station 12+88. The gouges were probably caused by the movement of large blocks of concrete and rock from the heavily damaged area upstream from Station 11+65. Figures 1-7, 1-8, and 1-9 are representative of the damage between Stations 10+39.5 and 11+65. Figure 1-10 shows the depression resulting from the failed epoxy-mortar patch that initiated the damage.

Two major areas of damage occurred in the tunnel elbow. One originated at Station 9+45± and terminated at Station 9+75±, and was centered on a line 3 feet right of the tunnel centerline.

Cavitation damage in this instance was initiated when an epoxy-mortar patch within an 18-inch-square dry-pack mortar patch failed. The resulting depression caused damage that took the form of five distinct tear-drop shaped holes downstream from the failure. The farthest upstream hole was about 1 foot wide and 6 inches deep. Progressing downstream the second hole was 2 feet across, the third 3 feet across and 1 foot deep, the fourth 6 feet across and 2 feet deep, and the last approximately 5 feet across.

The second area of major cavitation damage in the elbow began at Station 9+00± and terminated at Station 9+20±. The damage was centered along the tunnel centerline, and consisted of three concave holes about 3 feet in diameter, 6 to 12 inches deep, interconnected by shallow longitudinal cavitated areas. Damage was induced by failure of an epoxy-mortar patch within a dry-pack mortar patch. Figure 1-11 shows the damage in the elbow.

In addition to the major damaged zones in the elbow, many smaller areas of cavitation damage occurred in the elbow from the P.C. Station 7+95.08 to the P.T. Station 10+31.47. The cavitation causing these damaged areas was initiated by buildup of calcium carbonate deposits, failure of mortar applied in thin layers to bring the original surface up to grade, failure of small epoxy-mortar repairs, and loss of aggregate that had been heavily bushhammered and ground to eliminate high spots in the surface. In many cases bushhammering and grinding had either loosened the aggregate from the matrix or resulted in a very thin layer of aggregate that became loose when subjected to high velocity flow. The ensuing depressions in most cases initiated minor cavitation damage. Cavitation damage did not occur in every instance of aggregate and mortar loss or of calcium carbonate buildup. Although size and location of the surface irregularities were influencing characteristics, a more controlling influence (in the case of depressions formed by mortar or aggregate loss) was probably the time at which these failures occurred, i.e., they could have occurred late in spillway operation.

The tunnel reach upstream from the elbow was free of cavitation damage; however, at random locations from Station 6+50 to Station 7+75, numerous surface irregularities occurred when poorly bonded epoxy-mortar patches failed or when concrete spalled or eroded due to water action or passage of debris through the tunnel. None of these surface irregularities, many of which were 1/2 to 3/4 inch deep and about 12 inches in diameter, initiated cavitation damage.

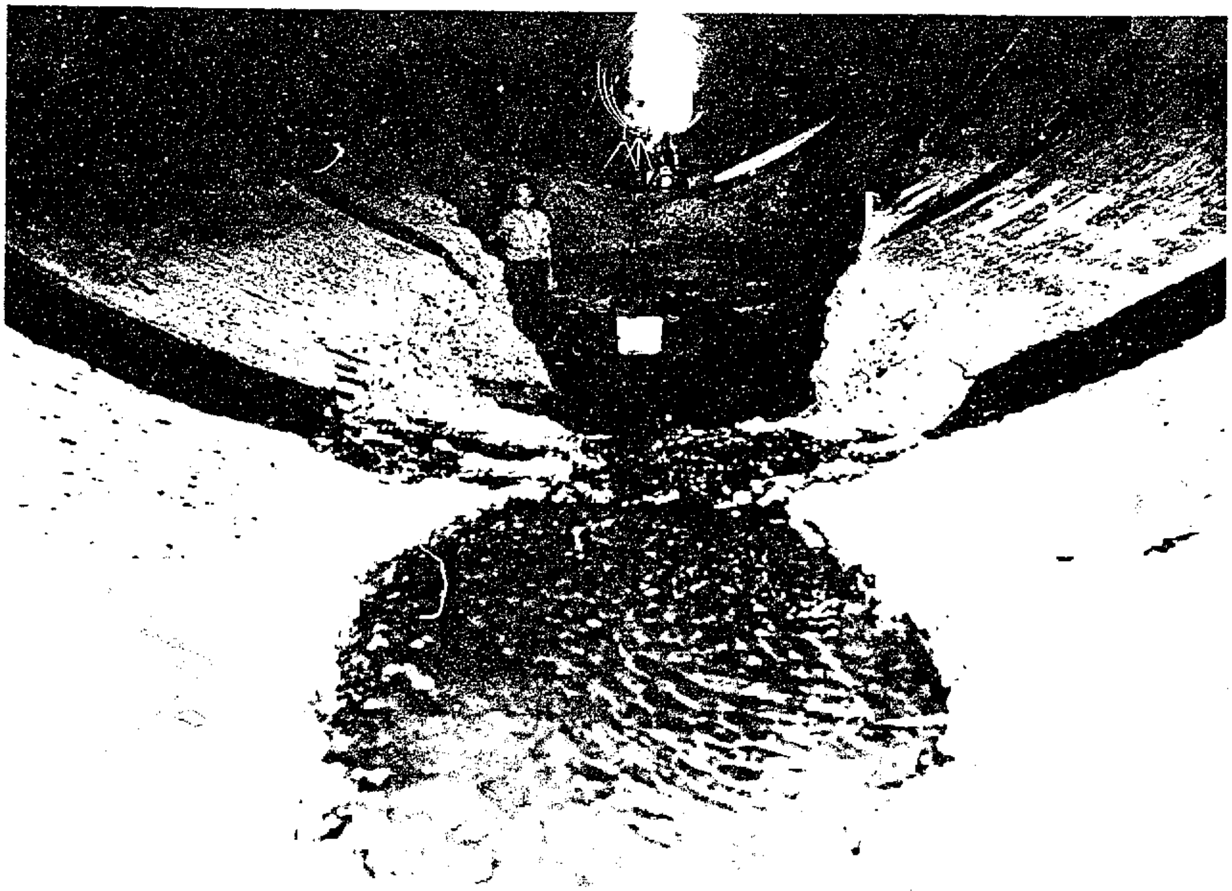


Figure 1-7. Looking downstream in major damaged area in horizontal reach of tunnel. Photo P459-640-4042

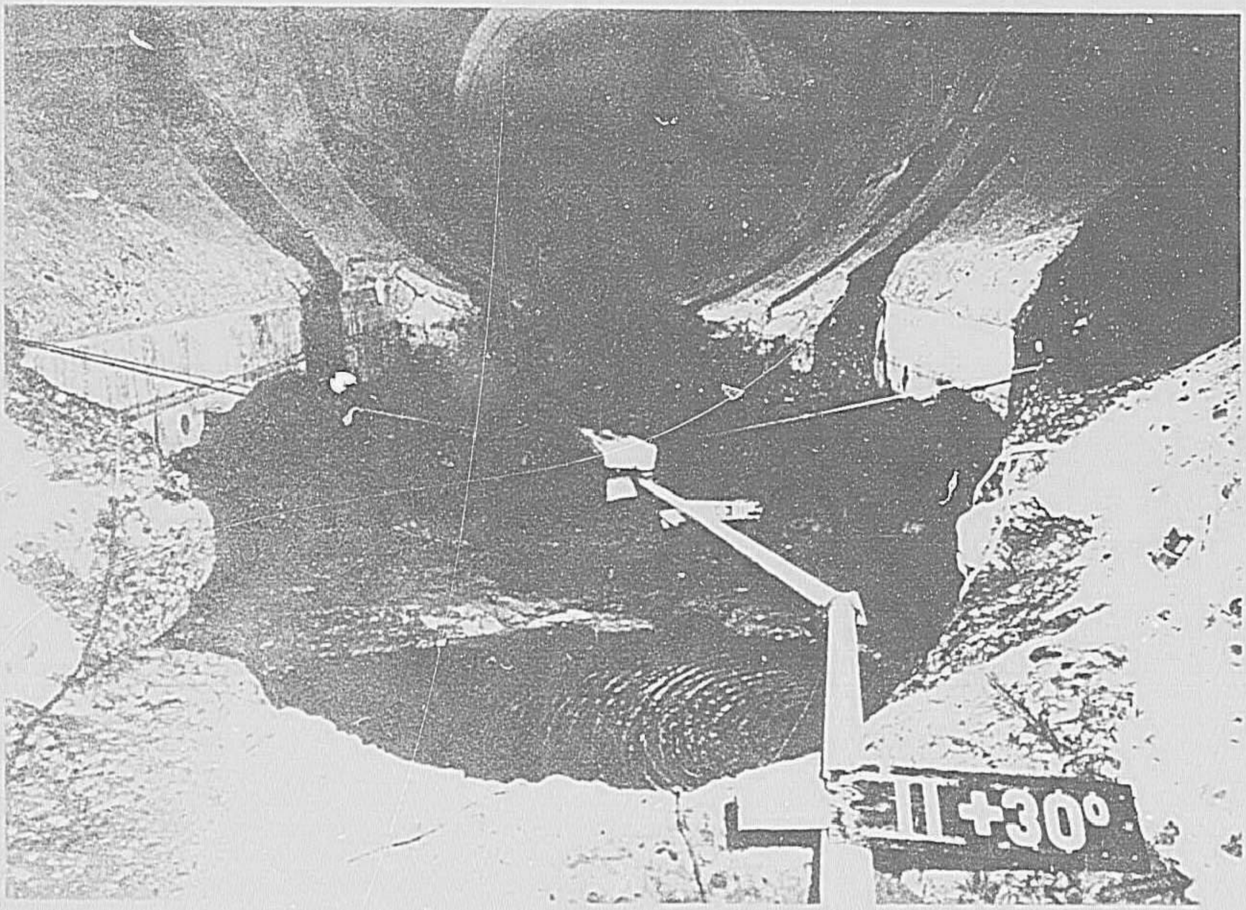


Figure 1-8. Major damaged area in horizontal reach of tunnel. Photo P459-640-4169

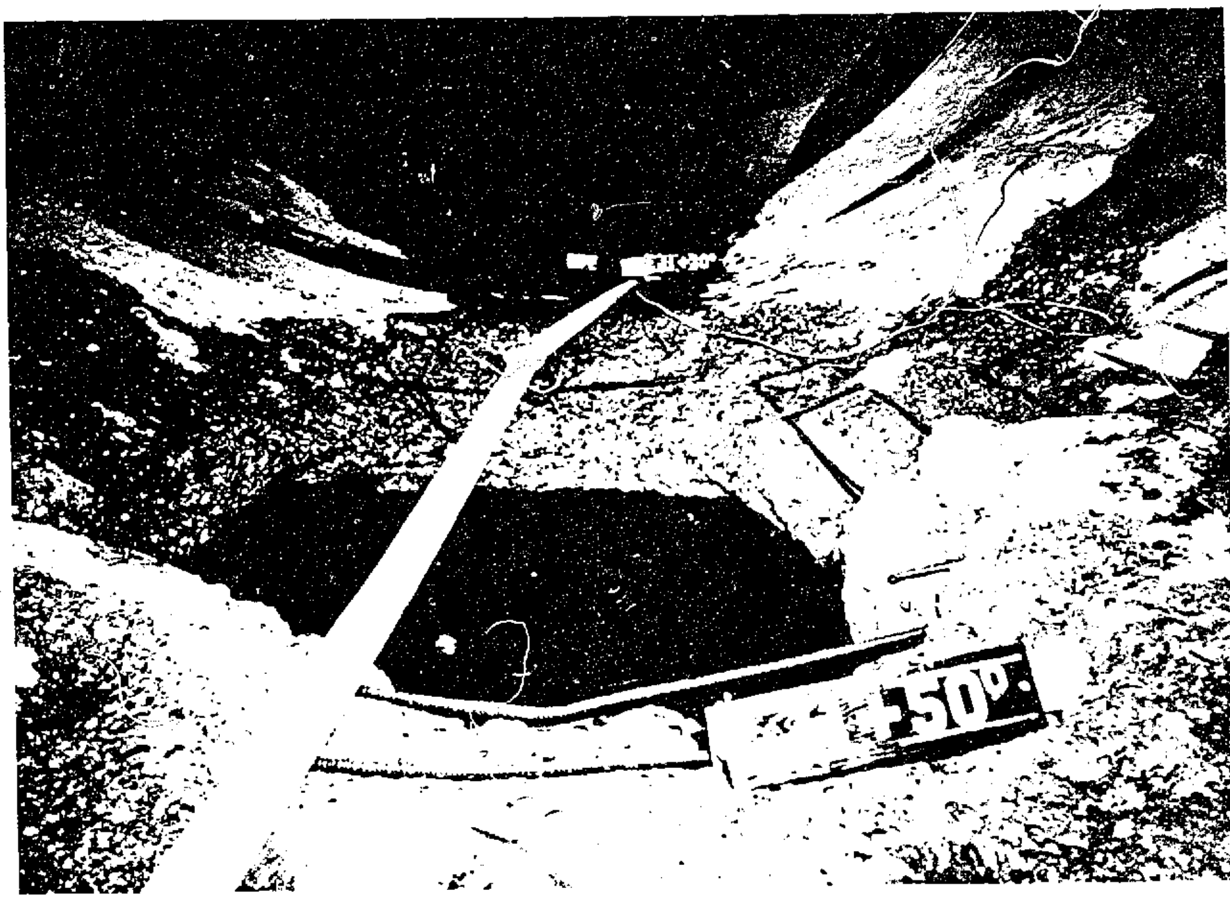


Figure 1-9. Major damaged area in horizontal reach of tunnel. Photo P459-640-4170

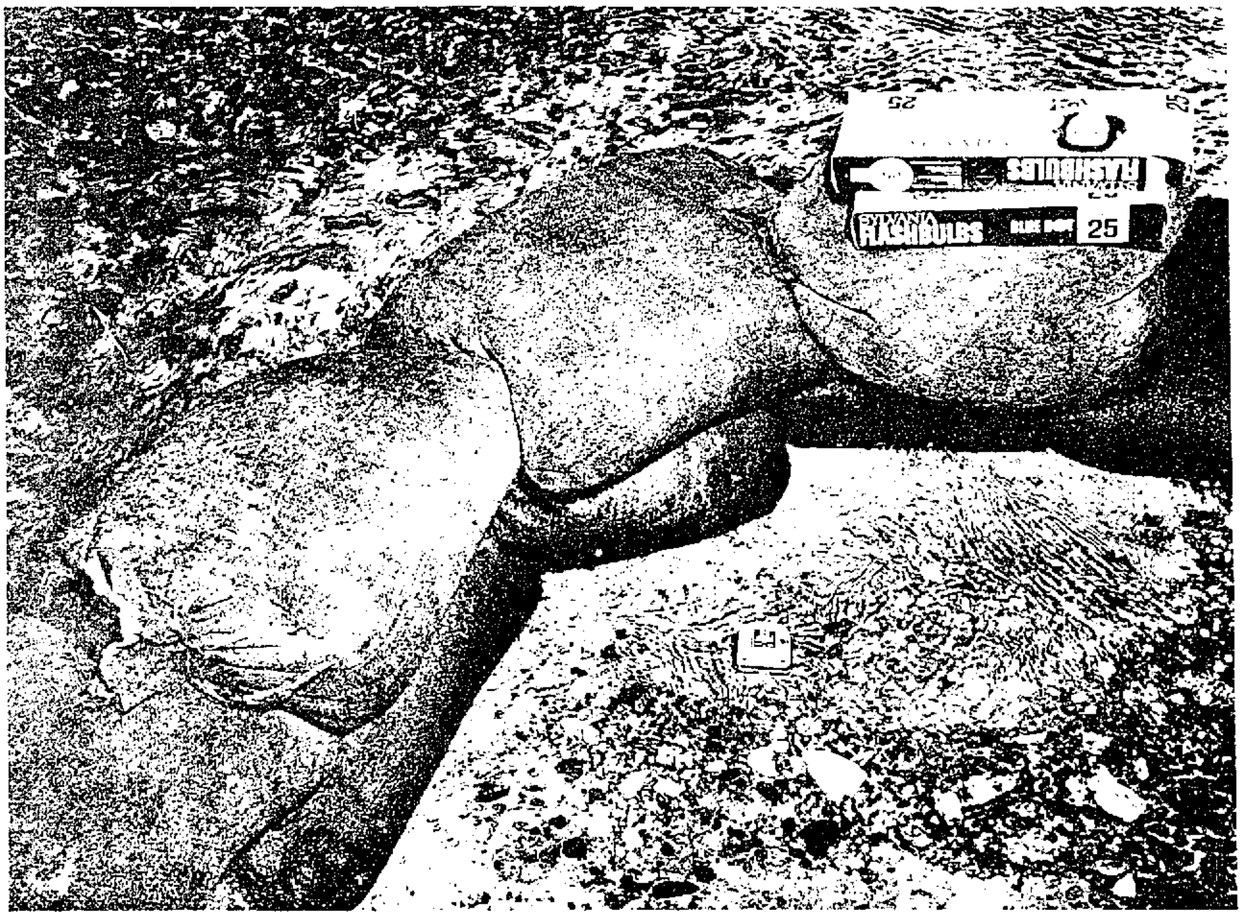


Figure 1-10. Depression that initiated the damage in horizontal reach of tunnel. Photo P459-640-4051 NA

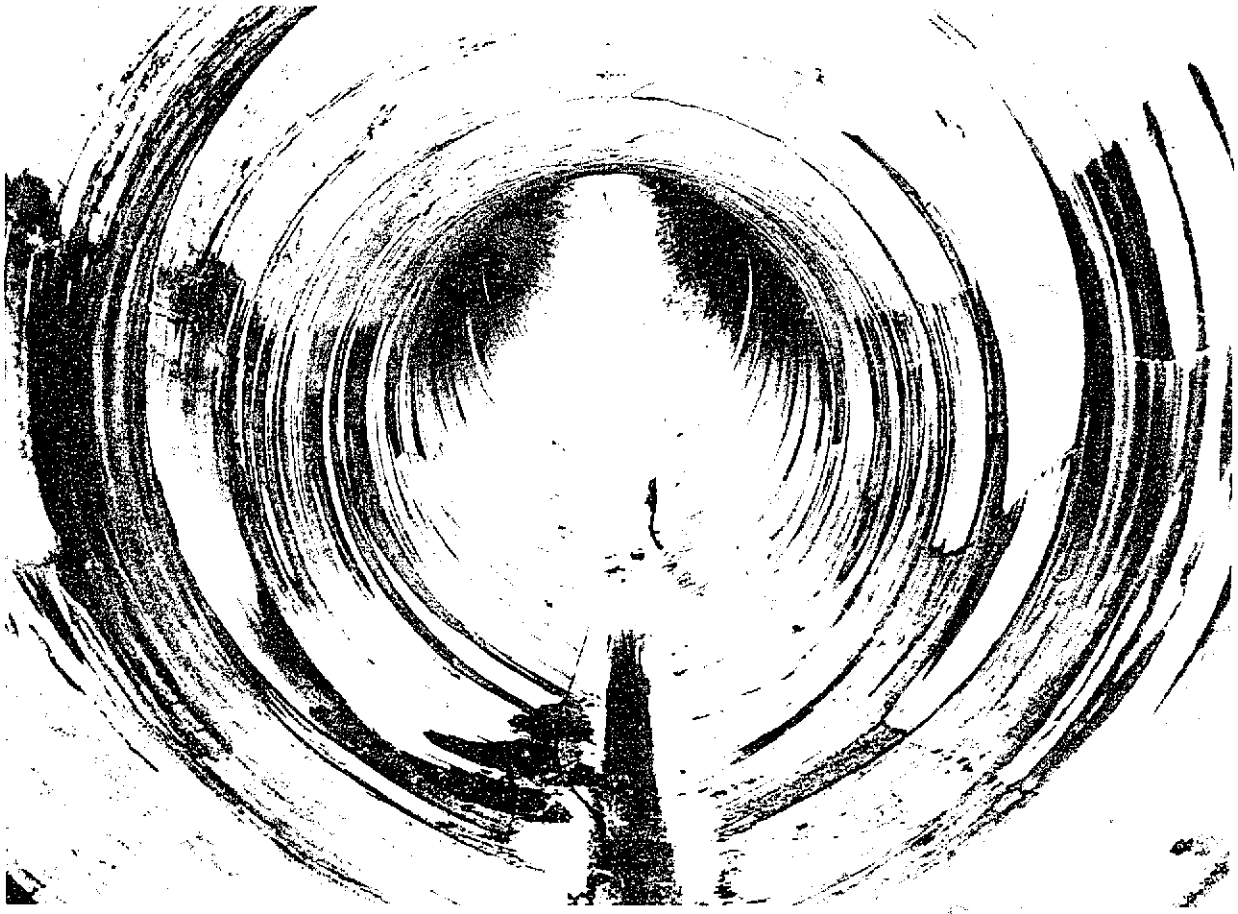


Figure 1-11. Looking upstream at damaged areas in tunnel elbow. Photo P459-640-4028 NA

PART 2

REPAIRS AND MODIFICATIONS TO TUNNEL (DESIGN)

A. General

Repairs and modifications to the tunnel were made under Specifications No. DC-6599. Award for the work was given to A and B Construction Company—COP Construction Company and included the following items of work:

(a) Repair of large damaged areas with bonded concrete.

(b) Repair of shallower damaged areas with epoxy-bonded concrete, and epoxy-bonded epoxy mortar.

(c) Replacement of existing epoxy mortar patches where defective.

(d) Grinding of surfaces to meet the tolerances required in the specifications.

(e) Painting of the tunnel surfaces below the spring line with an epoxy-phenolic paint from Station 6+50* to Station 12+15.

(f) Construction of aeration slots at Stations 7+79 and 6+64.*

(g) Hydrophone installation in the aeration slot at Station 7+79.

(h) Removal of calcium carbonate deposits below the spring line from Station 6+50 to Station 11+65.

B. Repair of Major Damaged Areas

Damaged areas having a depth of repair 6 inches and greater and with an appreciable continuous area were repaired with bonded concrete. The major damaged areas occurred in the elbow in the reaches from Station 9+00 to Station 9+20 and from Station 9+45 to Station 9+75, and in the near horizontal tunnel reach from Station 10+[illegible]0 to Station 11+65, as described previously and shown on Figures 2-1, 2-2, 2-3, and 2-4. In addition, random areas throughout the tunnel were likewise repaired with bonded concrete.

Bonded concrete repairs were made by sawcutting around the perimeter of the damaged area to a depth of 1-1/2 inches. Damaged concrete, including undercut concrete, was removed. Damaged reinforcement was also removed and replaced either by welding or lapping new reinforcement in place. All concrete or rock surfaces to receive the bonded concrete were then sandblasted, washed clean, and dried before placement of the repair concrete.

C. Repair of Shallow Damaged Areas

Where the depth of damage was between 2 and 6 inches, repairs were made with epoxy-bonded concrete. The repairs were made by sawcutting a groove 1-1/2 inches deep around the perimeter of the damaged area, removing all damaged concrete, and preparing the surface to receive the epoxy-bond coat. The epoxy-bonding agent was then applied to clean, dry surfaces, and while in a tacky state, overlain with the repair concrete.

When the depth of repair was less than 2 inches to featheredge, repairs were made by preparing the damaged areas to receive the epoxy-bonding agent and applying a veneer of epoxy mortar to the area.

The epoxy-bonded concrete, as well as the bonded concrete, generally presented no unusual problems during construction, except on construction joints which were formed or damp. Epoxy-bonding agents could not be used in these circumstances. Considerable difficulty was encountered with the epoxy-bonded epoxy mortar and is discussed in Part 3 of this documentation.

D. Grinding and Painting of Flow Surfaces

After the repairs, whether bonded concrete, epoxy-bonded concrete, or epoxy-bonded epoxy mortar, had cured properly, all repaired surfaces not meeting the required specifications for finishes and tolerances were corrected by grinding as permitted in the specifications.

When the flow surfaces were judged to meet all the requirements as to finishes and tolerances, the tunnel reach from Station 7+64 to Station 12+15 extending from springline to springline was painted with a two-coat epoxy phenolic paint treatment. The purpose of the paint treatment was twofold: (1) to provide a smooth uniform texture to flow surfaces, and (2) to cement particles of the materials in the epoxy mortar or concrete to each other. A white pigmented epoxy-phenolic paint was used as the second coat to provide a high degree of visibility to flow surfaces.

E. Aeration Slot

Cavitation damage to concrete surfaces subjected to high-velocity flows has been recognized by engineers for many years. Cavitation occurs when a surface

*Revised by supplemental notice.

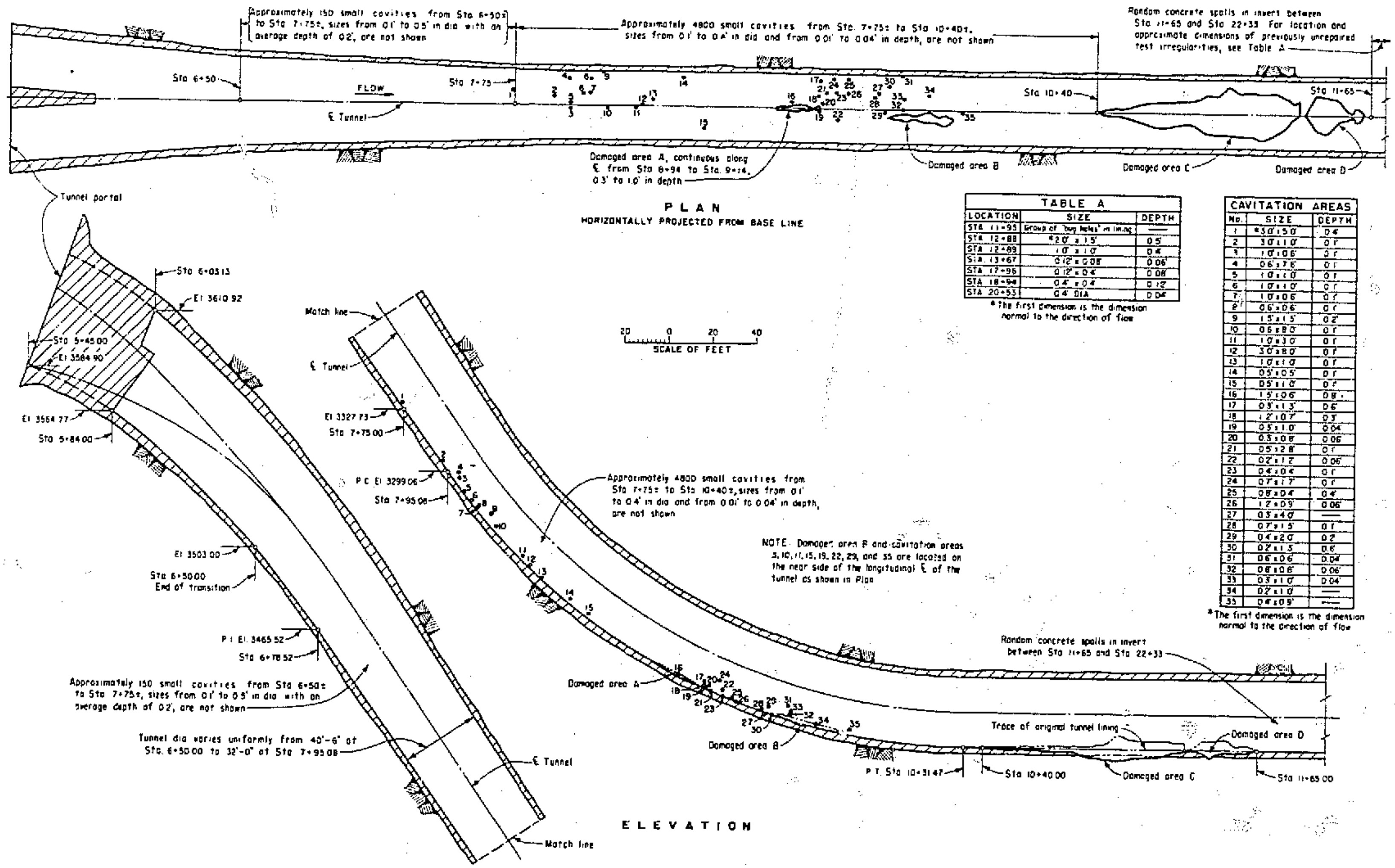


TABLE A

LOCATION	SIZE	DEPTH
STA 11+95	Group of "bug holes" in lining	—
STA 12+88	*2.0' x 1.5'	0.5'
STA 12+89	1.0' x 1.0'	0.4'
STA 13+67	0.12' x 0.08'	0.06'
STA 17+96	0.12' x 0.4'	0.08'
STA 18+94	0.4' x 0.4'	0.12'
STA 20+53	0.4' DIA	0.04'

* The first dimension is the dimension normal to the direction of flow

CAVITATION AREAS

No.	SIZE	DEPTH
1	*3.0' x 5.0'	0.4'
2	3.0' x 1.0'	0.1'
3	1.0' x 0.6'	0.1'
4	0.6' x 7.6'	0.1'
5	1.0' x 1.0'	0.1'
6	1.0' x 1.0'	0.1'
7	1.0' x 0.6'	0.1'
8	0.6' x 0.6'	0.1'
9	1.5' x 1.5'	0.2'
10	0.6' x 8.0'	0.1'
11	1.0' x 3.0'	0.1'
12	3.0' x 8.0'	0.1'
13	1.0' x 1.0'	0.1'
14	0.5' x 0.5'	0.1'
15	0.5' x 1.0'	0.1'
16	1.5' x 0.6'	0.8'
17	0.3' x 1.3'	0.6'
18	1.2' x 0.7'	0.3'
19	0.5' x 1.0'	0.04'
20	0.5' x 0.8'	0.06'
21	0.5' x 2.8'	0.1'
22	0.2' x 1.2'	0.06'
23	0.4' x 0.4'	0.1'
24	0.7' x 1.7'	0.1'
25	0.8' x 0.4'	0.4'
26	1.2' x 0.9'	0.06'
27	0.3' x 4.0'	—
28	0.7' x 1.5'	0.1'
29	0.4' x 2.0'	0.2'
30	0.2' x 1.5'	0.6'
31	0.6' x 0.6'	0.04'
32	0.8' x 0.6'	0.06'
33	0.3' x 1.0'	0.04'
34	0.2' x 1.0'	—
35	0.4' x 0.9'	—

* The first dimension is the dimension normal to the direction of flow

Figure 2-1. Damaged areas in tunnel.

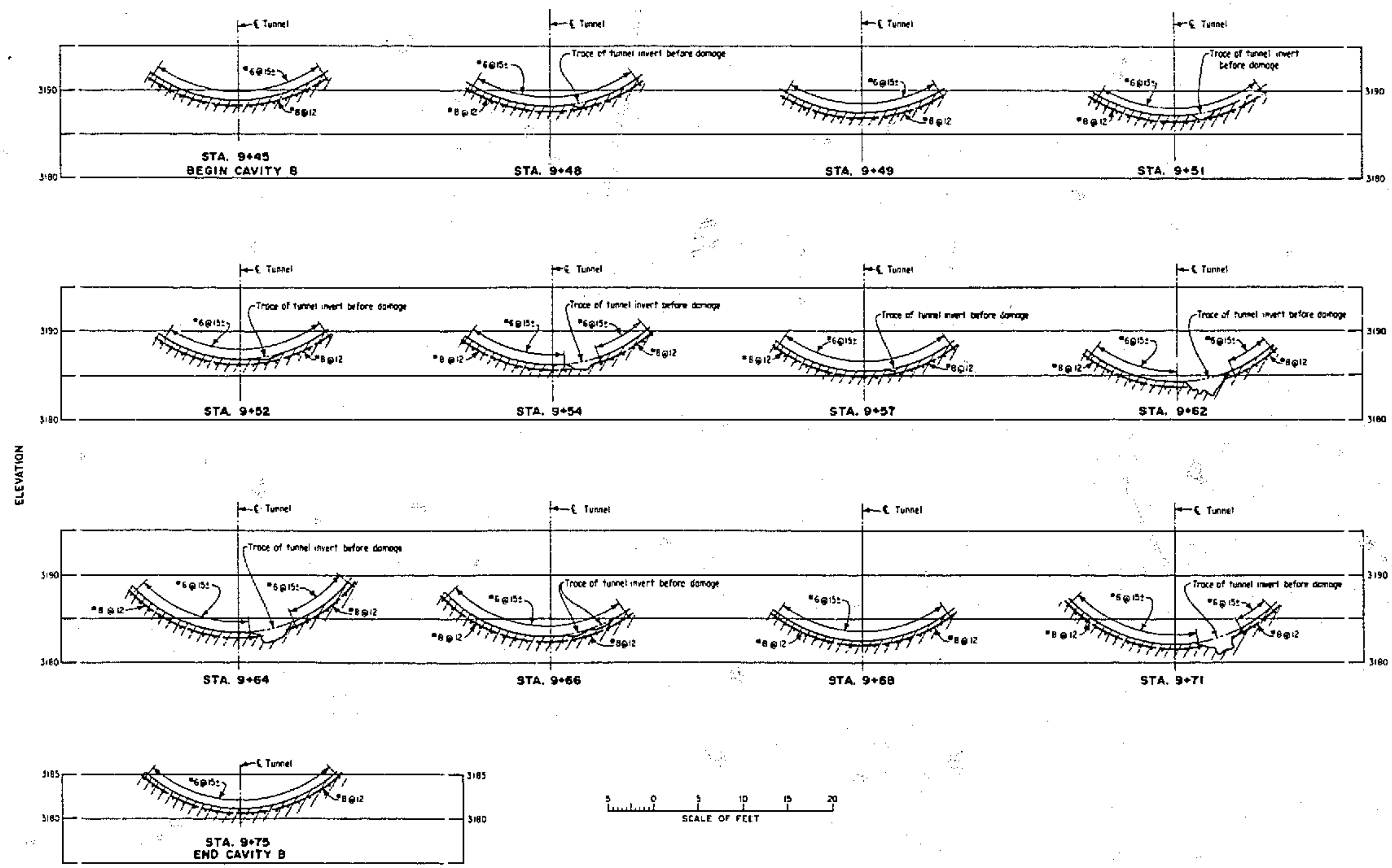


Figure 2-2. Sections through damaged Area B.

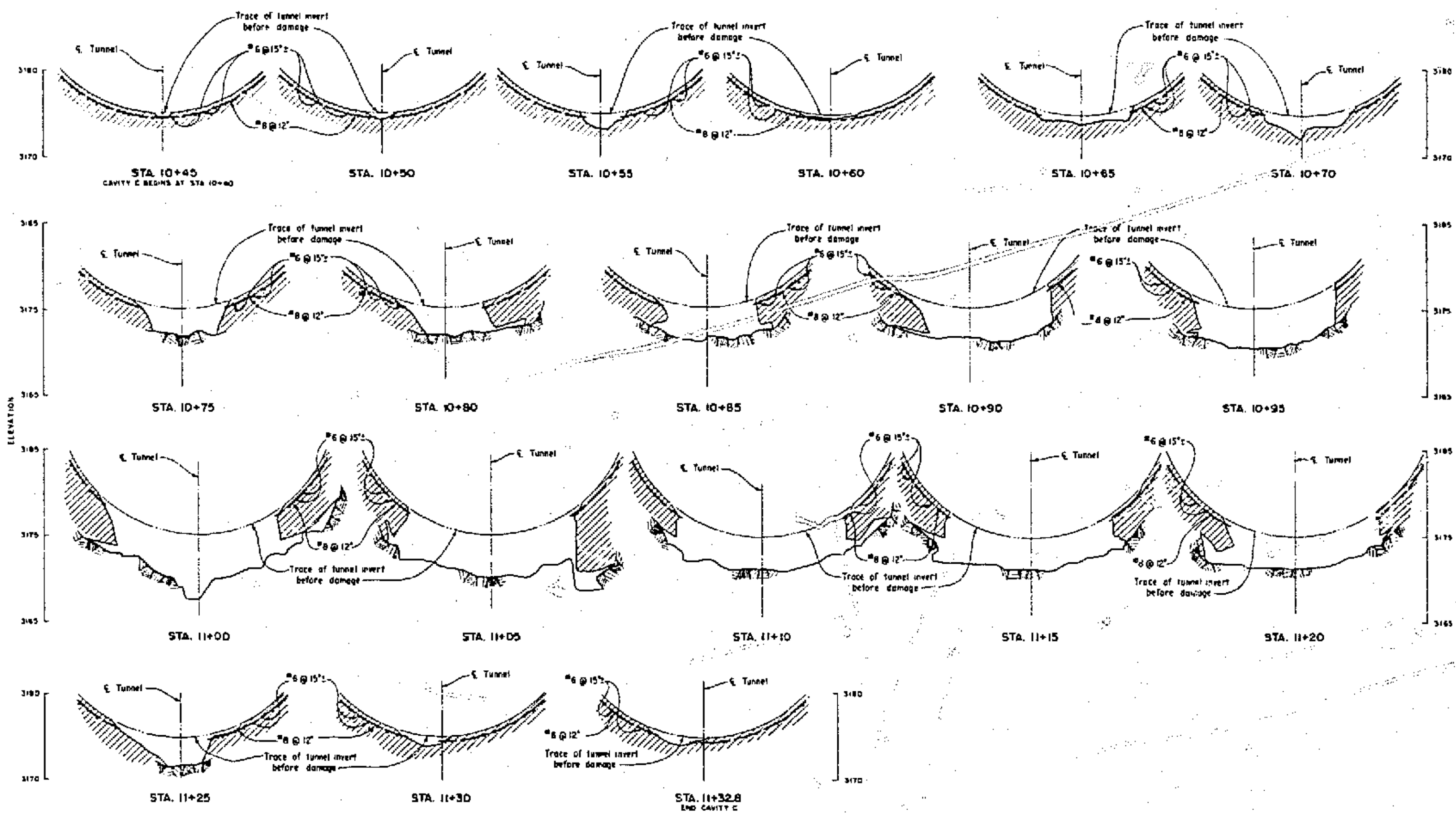


Figure 2-3. Sections through damaged Area C.

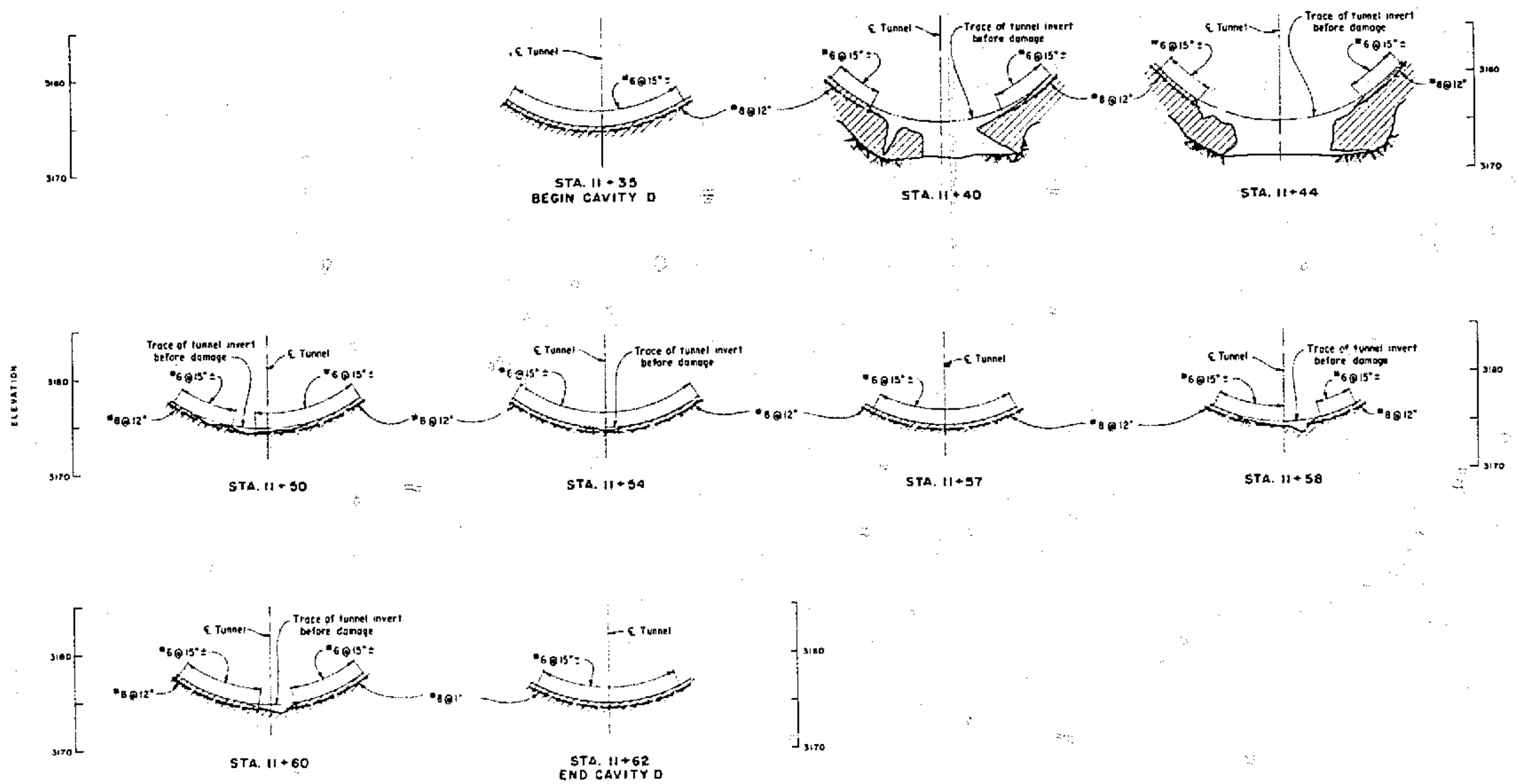


Figure 2-4. Sections through damaged Area D.

irregularity can create a condition whereby the pressure in a fluid becomes so severely negative that the fluid contiguous to the flow surface vaporizes in the form of water vapor bubbles. These bubbles are then transported by the fluid into areas of high-positive pressure where the bubbles collapse suddenly. The result is instantaneous pressures of thousands of pounds per square inch. These inordinately high pressures produce the spongelike appearance of concrete damaged by cavitation.

To mitigate cavitation potential in critical areas, an aeration slot was located in the tunnel as shown on Figure 2-5 (Drawing No. 459-D-2301). The upstream lip of the slot was designed to flip tunnel discharges over the slot. In doing so, air will be entrained in the exposed surfaces of the free jet and carried along in the flowing water to dampen or cushion the high pressures which would result from the collapse of air-free water vapor.

The laboratory model studies made to determine the optimum location and configuration for the aeration slot are described in the following paragraphs:

(a) General.—A previously used and successful concept of a device for entraining air in water flowing in a tunnel consisted of a slot in the tunnel lining to admit air to the periphery of the jet as it passed over the slot.

Two such slots were suggested for the Yellowtail Spillway, one at Station 6+64 to protect the sloping conical portion of the tunnel, and one at Station 7+79 to protect the vertical bend. A 1:49.5 scale model of the Yellowtail Spillway, with aeration slots at the two suggested stations, was constructed in the Bureau of Reclamation's Hydraulics laboratory (Figures 2-6 and 2-7).

(b) Aeration slot at Station 6+64.—Initial model operation indicated that air was drawn into the flowing water at the upstream slot (Station 6+64) for all discharges smaller than 55,000 cfs. The air entrained at the slot, however, appeared to leave the boundary rather rapidly as the water passed down the inclined tunnel, and at the PC of the vertical bend the water was practically free of air. For discharges greater than 55,000 cfs the flow impinged on the downstream edge of the slot filling it with water thus preventing air from entering the slot.

Concurrently with the model study, examinations were continuing at Yellowtail Dam. Upon close inspection and reevaluation it was determined that cavitation had not been a contributing factor in the surface distress in the conical section of the tunnel. Careful surface preparation and repair in this reach of tunnel would be adequate to prevent further surface distress. Model studies concerning the slot at Station 6+64 were terminated, and the slot was physically removed from the model.

(c) Aeration Slot at Station 7+79.—The proper operation of an aeration slot at Station 7+79 was considered to be essential for the protection of the flow surfaces in the vertical bend. The aeration slot would be required to furnish air to the flowing water for all discharges up to the maximum, 92,000 cfs. In addition, the configuration of the slot and all adjacent areas would necessarily be such that the flow would be hydraulically acceptable for all discharges.

An aeration slot with the preliminary shape, shown in Figure 2-7, was installed at Station 7+79. The slot was uniform in cross section for the entire circumference of the tunnel. Although air was entrained in the flowing water for low flows, the edges of the jet near the water surface impinged on the downstream edge of the slot for discharges greater than about 5,000 cfs. As the discharge increased more of the jet impinged on the downstream edge of the slot and greater amounts of water entered the slot. As more water entered the slot the amount of air entrained in the flowing water decreased and stopped entirely at about 50,000 cfs.

Figure 2-8 shows the preliminary slot with 92,000 cfs flowing in the tunnel. Note the absence of entrained air downstream from the slot.

The slot was narrowed to 6 inches, with a 6-inch away-from-the-flow offset downstream, in an attempt to prevent water from entering and filling the slot, (Figure 2-9A). Although some air was entrained in the jet at low discharges, water entered the slot near the water surface for all spillway discharges. For discharges less than 20,000 cfs water drained down the slot and flowed out into the jet at the tunnel invert, reducing the amount of entrained air as the spillway discharge increased. At a spillway discharge of 20,000 cfs the water surface in the slot was at the same elevation as the spillway water surface, and no air entered the jet. For spillway discharges greater than 20,000 cfs the water was forced into the slot at a sufficiently high head to cause it to boil up the slot above the spillway water surface and spill out onto the surface of the jet.

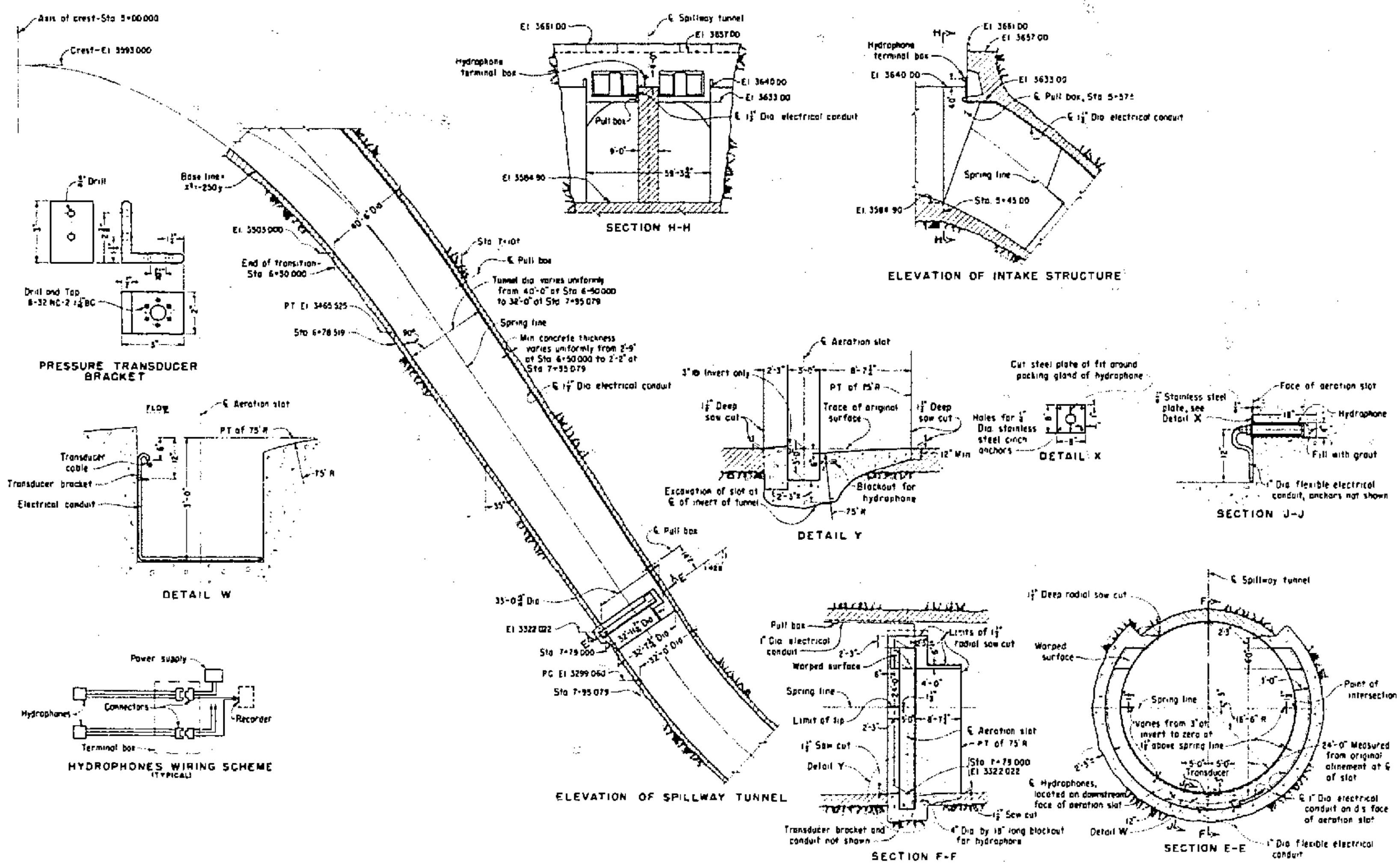


Figure 2-5. Tunnel aeration and instrumentation—Elevation and sections.

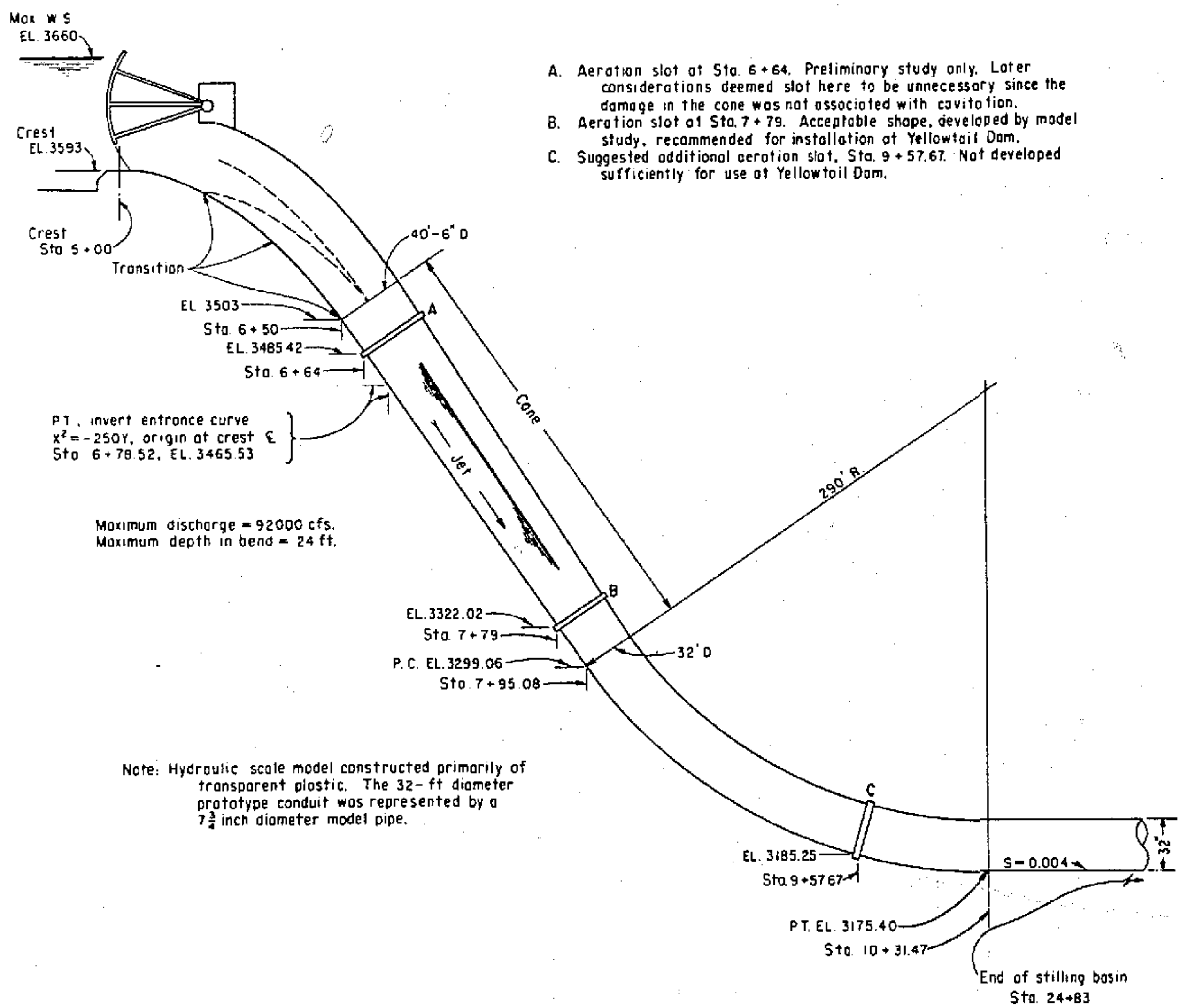


Figure 2-6. Yellowtail Dam Tunnel Spillway. The laboratory installation was a true scale model to Station 12+75, but did not include the radial control gates.

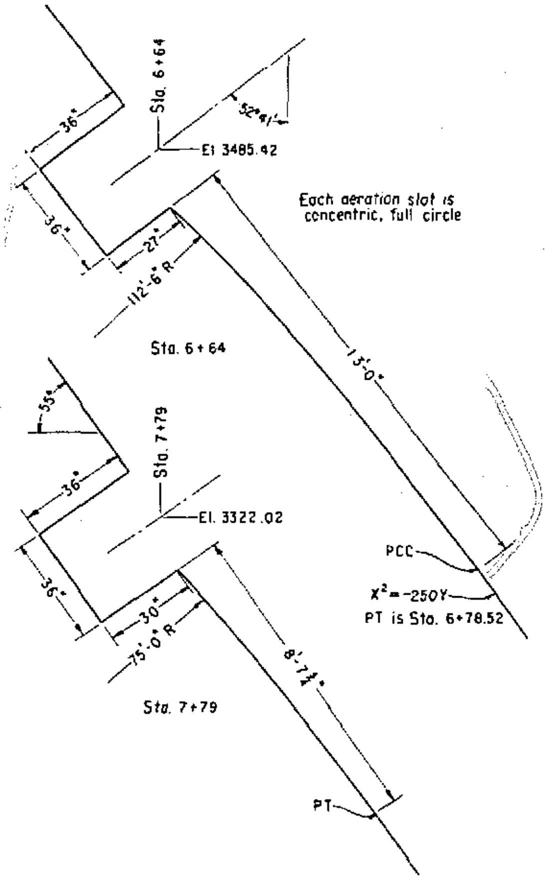


Figure 2-7. Preliminary aeration slots.

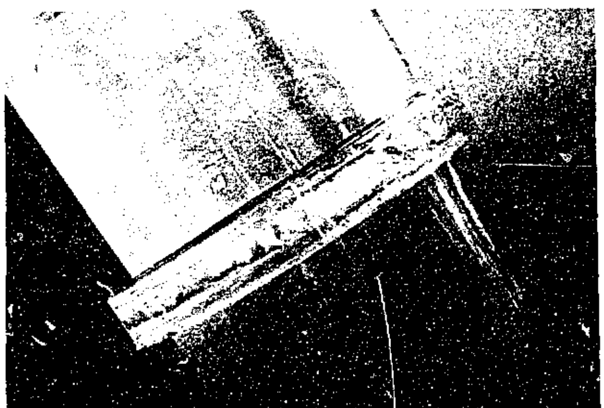


Figure 2-8. Preliminary aeration slot—Station 7+79—Q = 92,000 cfs—The slot has filled with water and air cannot enter the jet. Photo P459-D-68807

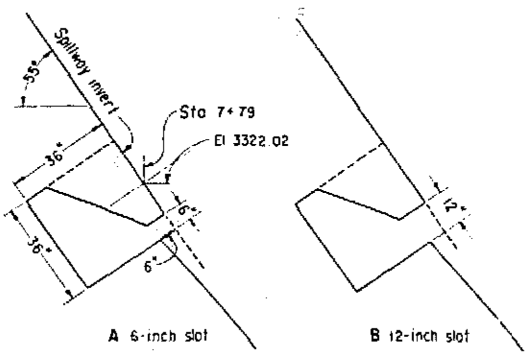


Figure 2-9. Narrow aeration slots.

The slot was opened up to 12 inches wide (Figure 2-9B). The flow with this configuration was similar to that with the 6-inch slot. The discharge at which the water surface in the slot was equal to that of the spillway water surface was 30,000 cfs.

The preceeding tests indicated that some type of lift or elevated spring point would be required to raise the jet over the slot for all spillway discharges so the slot would remain open and furnish air to the jet.

A conical nozzle was installed in the tunnel at the upstream face of a 3- by 3-foot aeration slot at Station 7+79. The nozzle exit was 32 feet in diameter, and the cone extended 5 feet upstream from the slot (Figure 2-10). Air was entrained in the jet, and the slot remained free of water, for all discharges. A large fin of water formed on either side of the tunnel where the jet from the nozzle impinged in the bend. The fins became progressively larger and arched higher with increasing discharge until, at a discharge of 6,000 cfs, the side fins extended to the crown of the tunnel. For discharges greater than 40,000 cfs, the water folded over the top of the jet and choked the tunnel. The above test indicated that the nozzle shape, or a ramp type of lift, upstream from an air slot would permit the jet to entrain air for all discharges, but modifications were needed to prevent side fins from choking the tunnel at the higher discharges. Several transitional ramp shapes were evaluated in the model before a configuration was perfected which would operate satisfactorily for all discharges up to the design maximum of 92,000 cfs.

The recommended design consisted of a 3- by 3-foot aeration slot at Station 7+79. A ramp, 27 inches long in the direction of flow, raised the upstream face of the slot 3 inches at the tunnel invert. The trace of the spring point of the raised lip was the

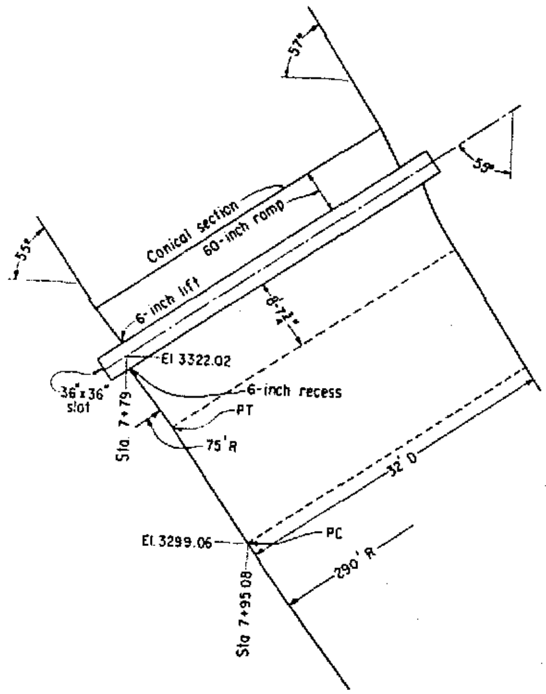


Figure 2-10. Conical nozzle—6-inch lift, 60-inch ramp.

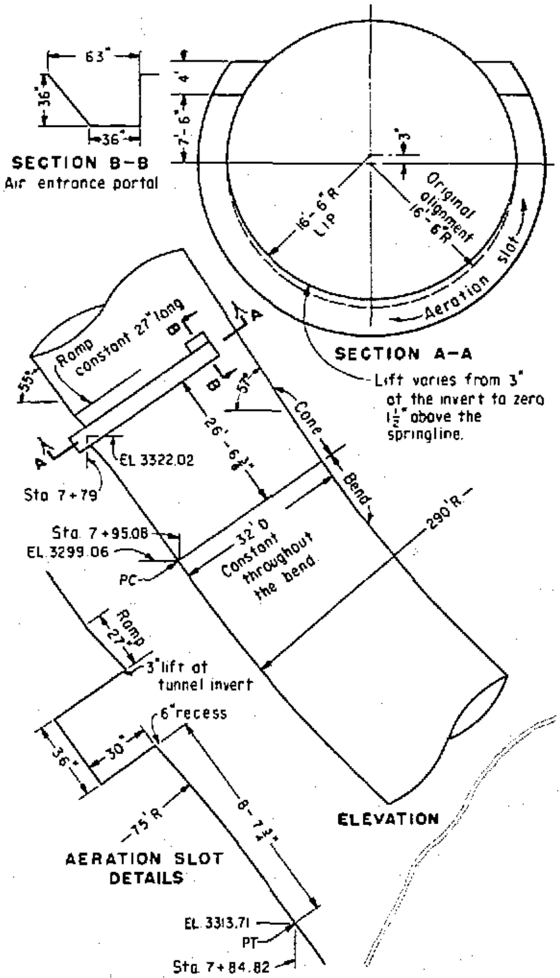


Figure 2-11. Recommended aeration slot.

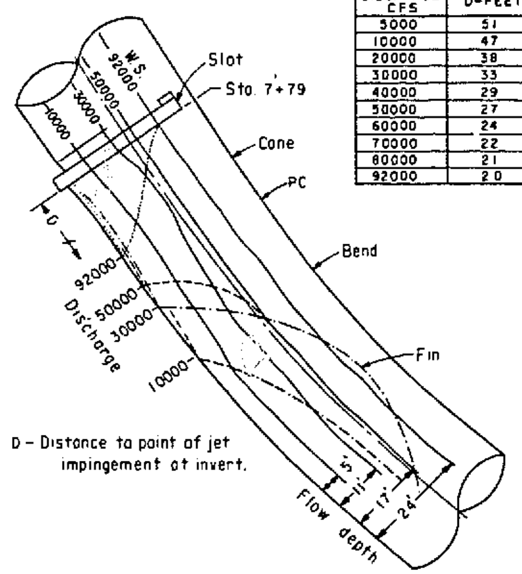
same diameter as the tunnel, with a 3-inch eccentricity in a plane perpendicular to the tunnel centerline. Thus the lift varied from 3 inches at the tunnel invert to zero at a point 1-1/2 inches above the tunnel spring line. The ramp upstream from the spring point was a constant 27 inches long (Figure 2-11).

The lift, or ramp, forced the bottom of the jet away from the tunnel flow surface, over the aeration slot, and the jet remained free for a considerable distance downstream before it impinged on the tunnel invert. The distance to the point of jet impingement on the tunnel invert reached a maximum of 52 feet downstream from the aeration slot at a discharge of 4,000 cfs. This distance decreased as the flow depth and discharge increased, and was 20 feet for a discharge of 92,000 cfs (Figure 2-12). This type of impingement will not damage smooth concrete surfaces.

The impingement of the jet on the tunnel invert downstream from the aeration slot caused side fins to form. For discharges less than 30,000 cfs these side fins swept uninterrupted up the walls of the tunnel past the contracted jet. The maximum side fins, which occurred at a discharge of 30,000 cfs, were not objectionable and did not reach the top of the tunnel (Figure 2-13). Since the lift diminished as it neared the tunnel spring line, the upper portion of the jet was subjected to less contraction than the lower portion. Consequently, the upper elements of the jet impinged on the walls of the tunnel further upstream, and at a much smaller angle, as the discharge and flow depth increased. For discharges greater than 30,000 cfs the upper portion of the jet interferred with and reduced the side fins, and for discharges greater than 50,000 cfs the side fins were entirely contained by the upper portion of the jet (Figure 2-14).

The aeration slot remained free of water, and air was drawn into the jet, for all discharges.

Air was visible in the model jet starting at the air slot and continuing well downstream from the PT of



POINT OF IMPINGEMENT	
DISCHARGE CFS	D-FEET
5000	51
10000	47
20000	38
30000	33
40000	29
50000	27
60000	24
70000	22
80000	21
92000	20

Figure 2-12. Trace of jet impingement and side fins—Recommended aeration slot.

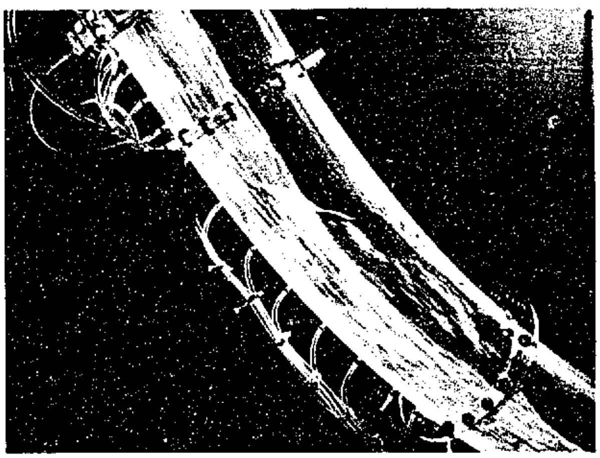


Figure 2-13. Recommended aeration slot—Station 7+79—Q = 30,000 cfs—Maximum side fins. The side fins, one each side of the jet, do not reach the crown of the tunnel. Photo P459-D-68809

the bend (Figure 2-15). However, it was not known whether the amount of air remaining adjacent to the tunnel flow surfaces in the downstream portion of the bend would be sufficient to prevent cavitation damage; the relationship between model and prototype with respect to entrained air is at best poorly understood. As a further precaution against cavitation damage, it appeared that air should be reintroduced into the jet at some station upstream from the PT of the bend.

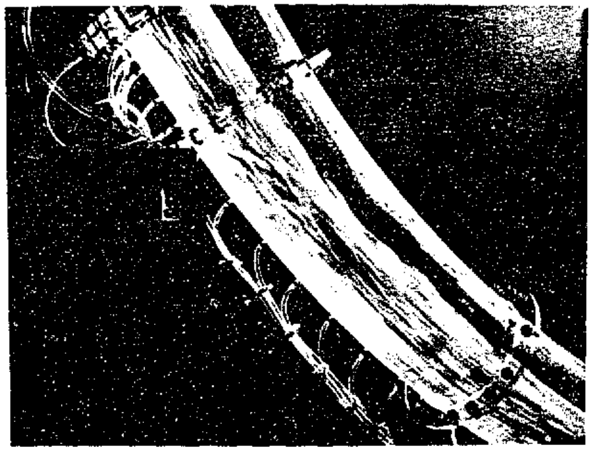


Figure 2-14. Recommended aeration slot—Station 7+79—Q = 50,000 cfs—Suppressed side fins. The upper portion of the jet has spread to the tunnel walls and suppressed the side fins. Photo P459-D-68810

(d) Aeration slot at Station 9+57.67.—Model studies were continued to evaluate an aeration slot in the downstream portion of the vertical bend at Station 9+57.67 (Figure 2-16).

The centrifugal force of the water in the bend made this location quite different hydraulically from the location in the conical portion of the tunnel. The initial study was made with no lift upstream from the slot, and a 12-inch away-from-the-flow offset downstream. The slot partially or completely filled with water for all discharges. A small amount of air entered the jet for discharges less than 15,000 cfs, but for discharges greater than 15,000 cfs water filled the slot and no air was entrained in the jet.

Various ramps and lifts were installed in the tunnel upstream from the slot. Each design tested produced satisfactory air entrainment and hydraulic operation for a limited range of discharges; however, none would operate satisfactorily over the full range of discharges (Figure 2-17).

These limited tests indicated that, for a satisfactory aeration device to be installed in the vertical bend of an existing structure, the necessary repairs and modifications appear to be too extensive to be practical. An aeration device to be installed in a bend during initial construction of a spillway tunnel could probably be developed.

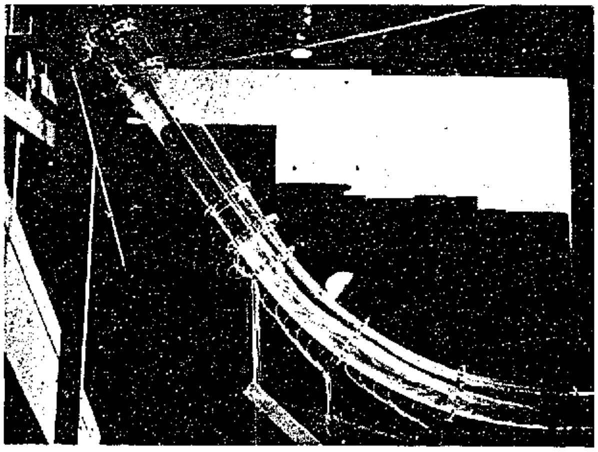


Figure 2-15. Overall spillway model—Recommended slot, Station 7+79—Q = 50,000 cfs—Air entrained at the aeration slot continues throughout the bend. Mixing tends to reduce the air near the flow surfaces in the downstream portion of the bend. (Note: Slot at Station 6+64, upper left, has been closed.) Photo P459-D-68808

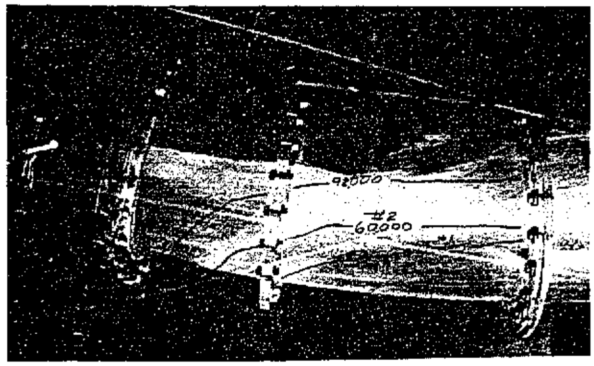
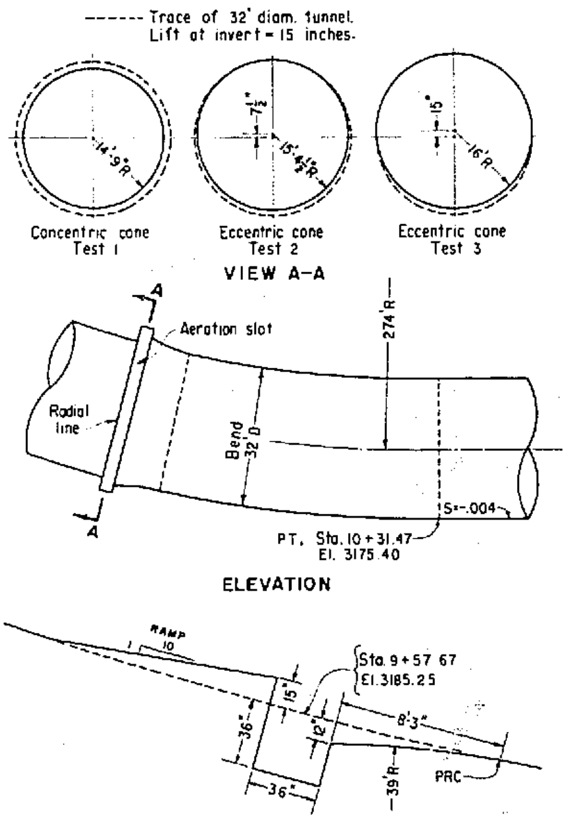


Figure 2-17. Aeration slot in the bend—Station 9+57.67—Q = 60,000 cfs—Side fins choking the tunnel downstream. Photo P459-D-68811



AERATION SLOT DETAILS AT INVERT

Figure 2-16. Aeration slot in the bend—Station 9+57.67 (Not recommended).

PART 3

REPAIRS AND MODIFICATIONS TO TUNNEL (CONSTRUCTION)

A. General

Preliminary to any construction repairs to the near horizontal reach of the spillway, access to the downstream portal (outlet) was necessary. Initially, it was contemplated that a bridge would have to be built across the narrow portion of the channel opposite the spillway tunnel stilling basin. This, it was believed, would be the first operation for the repair.

The contractor, however, obtained permission to use the draft tube bulkhead gate deck of the powerplant for his bridge crossing. This necessitated a timber ramp on the north side of the deck that connected to an existing access road to the spillway stilling basin. The use of this deck provided access for light trucks and pickups. All equipment and materials for work in the downstream reach of the tunnel were transported over this route.

Previous to actual work in the tunnel, a stiffleg crane was set up on the right side of the spillway stilling basin. All equipment and materials were lowered by means of this crane, Figure 3-1, to the portal below. Access for workmen was provided by a timber stairway extending from the top to the bottom of the basin, Figure 3-2.

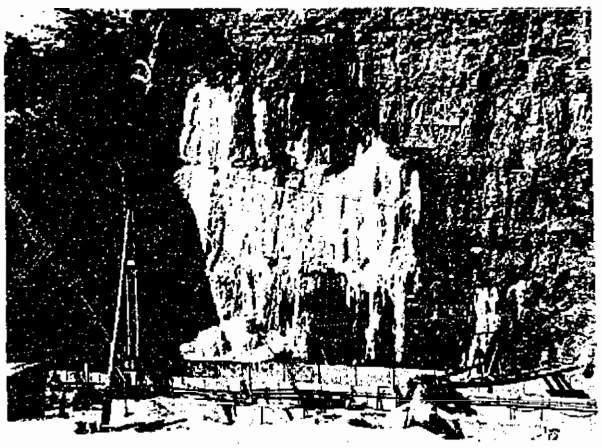


Figure 3-1. View of spillway stilling basin and stiffleg crane used to lower materials and concrete into spillway tunnel outlet portal. Photo P459-640-4428 NA

Once the stiffleg and stairway were operative, the contractor constructed a full tunnel section bulkhead,

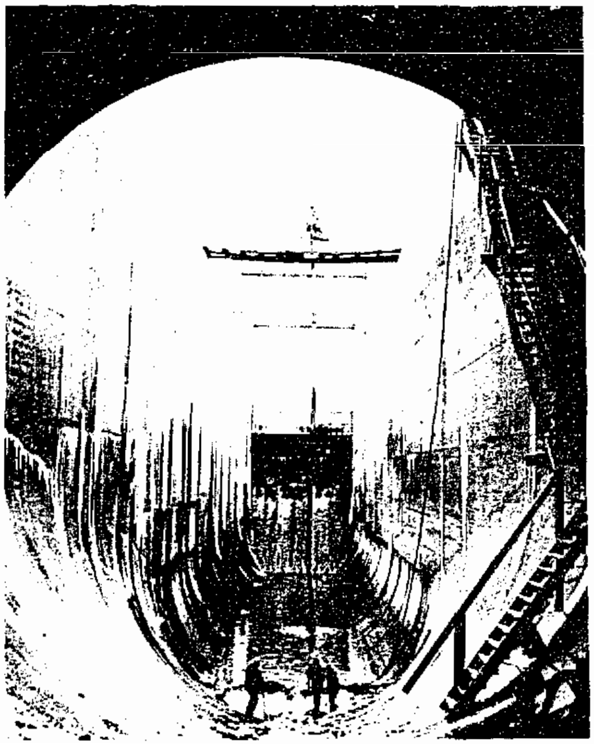


Figure 3-2. View of spillway tunnel stilling basin showing access to downstream spillway tunnel portal by stairway. Photo P459-640-4429 NA

Figure 3-3, just inside of the downstream portal and a horizontal bulkhead over the radial gates at the spillway tunnel entrance.

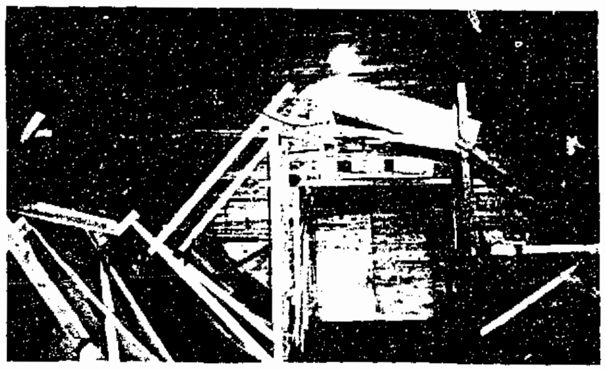


Figure 3-3. View of full tunnel bulkhead just inside of the downstream portal of the spillway tunnel. Photo P459-D-68578

B. Repair of Major Damaged Areas

The major damage to concrete areas were designated as A, B, C, and D, Figures 2-1 through 2-4. All of the damaged areas were marked out so that all damaged concrete would be included in removal and the periphery of the areas would be in sound concrete. The boundary of the areas were saw cut to a depth of 1-1/2 inches, Figure 3-4.

Figure 3-4. View of chipped out damaged Areas A and B showing the saw cut periphery of the damaged area and the partially replaced reinforcement steel mat. Photo P459-D-68576

Line drilling was then performed just inside of the saw cut. The drilled holes were 1-1/2 inches in diameter and were on 6-inch centers and extended to a depth of 12 inches or through the concrete lining in damaged Areas C and D. These holes were lightly loaded and shot. The resultant excavation was neat with half drill holes visible along the entire boundary of the excavations. Loose rock damaged by cavitation erosion was removed to sound rock.

By excavating in this manner, a sufficient stub of reinforcement steel extended beyond the excavated surface to permit an adequate lap for welding the new bars to the undamaged reinforcement, Figure 3-4.

The specifications provided that the major damaged areas would be backfilled with concrete to within 3 feet of the finished invert grade at tunnel centerline, Figure 3-5. From this point the repair area would be hand finished and unformed when within a segment defined by a 55° arc ($27\text{-}1/2^{\circ}$ each side of the tunnel centerline). Concrete repairs beyond the 55° segment would be formed and placed in conventional fashion.

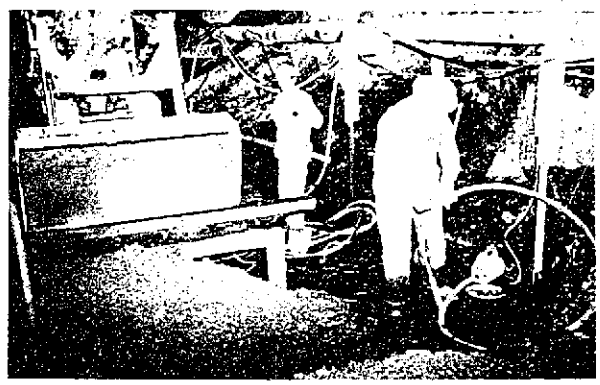


Figure 3-5. Placing concrete in the C and D damaged areas of the spillway invert. This concrete is backfill concrete and was placed to a depth of 3 feet below the finished invert grade of the tunnel at centerline. Photo P459-D-68575

Placement of concrete in the 55° segment of the invert was controlled by wood screeds placed transversely to the tunnel centerline at 10-foot intervals. This, it turned out to be, was not the proper way to finish the surface of the invert as finishing had to be done to the bottom of the transverse screeds and resulted in considerable unevenness of the finished surface. This unevenness was corrected later by extensive epoxy-bonded epoxy-mortar repair and grinding to bring the surface within the 1:100 tolerance required.

Had the work been performed by finishing to the top of transverse preformed steel ribbon screeds and alternate 10-foot sections placed, much of the epoxy-mortar repairs and grinding of the surface would have been unnecessary.

Concrete for the repair of the A, B, C, and D areas was manufactured in a portable batching and mixing plant located on the west side of the Bighorn River about

2,000 feet downstream from Yellowtail Dam. The mixed concrete was discharged into 1/2-cubic yard concrete buckets and hauled in 1/2-ton pickup truck. The trucks traveled a route of about 3-1/2 miles crossing the Bighorn River to the east side of the river and again crossing the Bighorn River via the draft tube bulkhead gate deck to the west side or left abutment of the dam. From this point the concrete was transported another 1,000 feet to the spillway tunnel stilling basin. From the stilling basin location, the stiffleg crane previously mentioned was used to transport the concrete to a small LeRoy Bobcat front end loader with a 1/2-cubic yard capacity bucket, Figure 3-6.

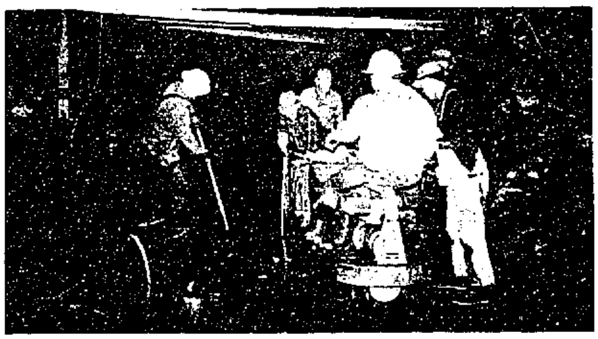


Figure 3-6. A small Leroy Bobcat front-end loader was used to transport concrete from the outlet portal of the tunnel to the point of placement inside the tunnel. Photo P459-D-68574

From the downstream portal the concrete was transported for approximately 1,000 feet in the tunnel to the point of placement. The total concrete placement for C and D areas amounted to about 361 cubic yards. Obviously this method of placing was slow. Concrete was placed on the day shift only.

A 3/4-inch-maximum-size aggregate concrete mix was used for repairing the damage in Areas A, B, C, and D. The mix contained 7 sacks of cement, 38 to 40 percent sand, and 60 to 62 percent minus 3/4-inch aggregate. The aggregates were obtained from existing stockpiles in the proximity of the portable concrete batching and mixing plant. The slump of the concrete at the point of placement varied from 1/2 to 1 inch. Internal air-operated vibrators were used to consolidate the concrete.

C. Repair of Shallow Damaged Areas

(a) Introduction.–The repair techniques included conventional-bonded concrete, epoxy-bonded concrete, and epoxy-bonded epoxy mortar. An epoxy-phenolic paint system was specified for a portion of the invert area to improve the surface character and reinforce the surface concrete.

Since the project personnel were not familiar with epoxy resins, a preconstruction training program was conducted at the Engineering and Research Center in Denver. Two men from the Yellowtail Construction Office attended this 1-month training program. Following this training, construction commenced with technical assistance by personnel from the Engineering and Research Center. Technical assistance was given by various people as unusual problems occurred. Construction supervision personnel maintained close liaison with the project as the work progressed.

Various problems and difficulties did occur and some unique techniques were developed during the repair program. A field report evaluating the repair program was requested.[1] Portions of this report are incorporated into this documentation.

In general, repair of damaged areas less than 1-1/2 to 2 inches in depth was accomplished by using epoxy-bonded epoxy mortar. Damaged areas 2 to 6 inches in depth were repaired with epoxy-bonded concrete, and repairs over 6 inches were completed by using bonded concrete. However, if an area of repair was relatively small in comparison to its depth, epoxy-bonded epoxy mortar was used in lieu of the epoxy-bonded concrete because of the convenience in preparation and application compared to the concrete. These repairs were usually less than 2 cubic feet in volume. After repairs were completed the surface was cleaned and painted with an epoxy-phenolic paint from Station 7+70 to Station 12+15.

The major use of the epoxy mortar was the application of an epoxy "skin" or "veneer" over practically the entire invert surface from Station 7+70 to Station 10+50 approximately 11.5 feet left and right of the tunnel centerline. This veneer was applied to correct the surface texture and to treat "large" areas of aggregate popout and cavitation damage caused by calcium buildup, etc. It was much easier to completely cover larger areas rather than to patch random damaged areas due to the difficulty in obtaining a smooth featheredge on the individual patches.

[1]The Use of Epoxy and Epoxy Mortars in the Repair of Yellowtail Dam Spillway Tunnel, Yellowtail Unit, by Leonard Rohrer, October 1969.

(b) Materials.—

1. Epoxy resin.—The specifications called for an epoxy-bonding agent to conform to the requirements of Federal Specification MMM-B-350a, for binder, adhesive, epoxy resin, flexible, Type I or II. The contractor selected Concresive No. 1161 binder, Type II, manufactured by the Adhesive Engineering Company, San Carlos, California. The two components were mixed in a 5:3 proportion by volume to form the bonding agent and the binder for the epoxy mortar.

2. Epoxy mortar.—The epoxy mortar was prepared by mixing a clean, dry, well-graded sand with the epoxy resin. Initial mix tests were made using a minimum No. 16 sand obtained by screening stockpiled concrete sand. Several mix proportions were tried in an effort to find the most workable mix. A mix of one part mixed resin to three parts sand by volume was established. This was a 1:4 resin to sand mix by weight.

3. Epoxy-phenolic paint.—The specifications called for the paint to be as manufactured by the Wisconsin Protective Coating Corporation, Green Bay, Wisconsin. This was the paint purchased by the contractor. The first coat was Plasite No. 7122H Clear, and the second coat was Plasite No. 7122B White.

(c) Procedures and methods.—

1. Drying the Surface.—Approximately 50 percent of the shrinkage cracks and construction joints in the area to be repaired and painted were seeping water in varying amounts. The specifications required that these seeps be stopped before any patching or any painting could be done. Several methods were tried in an effort to stop the water, some very successful and some unsuccessful.

Before any work began on the cracks themselves, a metal eaves trough approximately 4 inches deep and 4 inches wide was attached to the concrete just above the tunnel spring line with a ram-set tool. The upper edge of the trough next to the concrete was then calked, diverting all water into the trough which delivered the water downstream of the work area.

Water from drain holes was diverted by screwing a 3-inch nipple into the drain. A firehose was attached to the nipple and the water was then directed into an 8-inch thin-walled pipe which ran through the work area. This 8-inch pipe also carried water running down the invert which was leaking through the gate seals at the inlet and draining from drain holes upstream from the work area. The water in the invert was collected immediately upstream of the work area by sandbag dams lined with polyethylene or bentonite.

An overall view of these details is shown in Figure 3-7.

After the water had been collected from outside the work area the cracks making water within the area had to be calked. The first major effort along these lines was chipping or routing of the cracks and tamping lead wool in the bottom of the groove. The cracks were chipped approximately 1-1/2 inches deep. After calking with lead wool, the routing was then repaired with epoxy mortar.

This method did not prove successful. It was impossible to tamp the lead wool in tight enough to stop the water. In many cases the water was "chased" from one area to another. When a damp or seeping crack was dried the water would begin showing up in a crack which had previously been dry and most of the time would reappear in the repaired crack. Also, the amount of repair work necessary after the crack was calked proved to be extensive. Occasionally a hairline crack would develop into a groove 4 to 6 inches deep and would be recalked two to three times.

After approximately a month, this method was abandoned. During this period several modifications were attempted. A French drain was built into the bottom of the chipped groove in an attempt to keep the top portion of the crack dry while the epoxy-mortar patch was being placed. When an attempt was made to grout the drains with a cement grout it was learned they could not be successfully grouted. Also, after the grouting attempt in which the outlet was plugged, the water would seep through the epoxy mortar. The drains were removed and replaced with epoxy-bonded concrete.

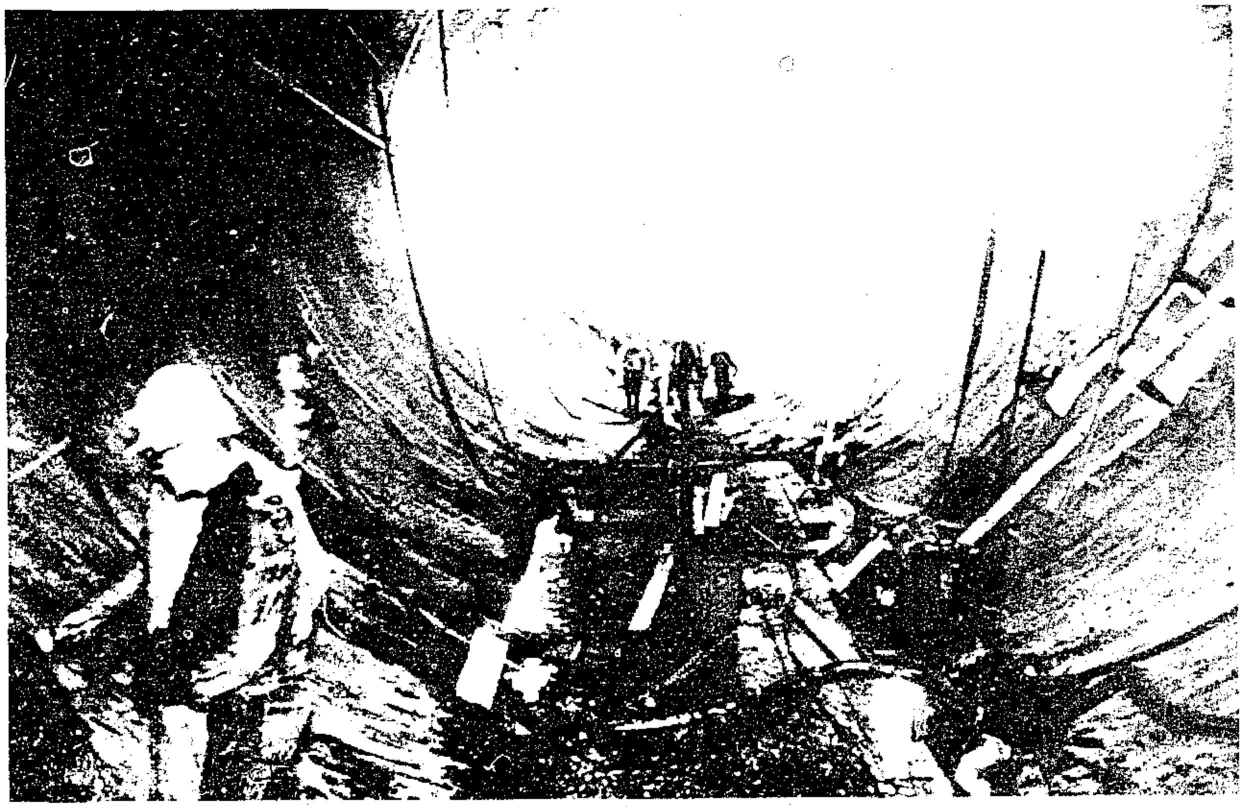


Figure 3-7. Overall view is shown of work area in spillway tunnel looking downstream from approximately the P.T. station of the elbow. Bonded concrete repairs have been completed; epoxy repairs are underway. Photo P459-D-68814

Plastic pipe, one-eighth inch in diameter, was embedded below the lead wool in an attempt to carry off the seepage water. The pipe was perforated in some trials and cut in half with the open side down in other trials. Success was limited in this case because of the difficulty in keeping the plastic pipe from collapsing.

After abandoning the chipping and calking a chemical grouting contractor was employed, Penetryn System of Salt Lake City, Utah. The trade name of the grout was Q-Seal as distributed by Community Utility Equipment and Supply, Inc., Orlando, Florida. This product is similar to AM-9 as manufactured by American Cyanamid Company.

The procedures used in grouting the construction joints and shrinkage cracks were as follows:

A series of 1/2-inch holes about 1 foot deep were drilled at an angle to the surface of the concrete so that a hole beginning 3 inches from the crack at the surface would intercept the crack about 8 inches below the surface. These holes were drilled at about 2-foot centers with alternating holes originating on opposite sides of the crack.

Drilling was accomplished with a diamond core bit driven by a 3/8-inch Black and Decker electric drill with a speed of 1,000 rpm. A waterhead was attached to the drill in place of the chuck. A 1-inch diamond bit was tried at one point in an effort to intercept a larger portion of the cracks. This was abandoned when it was found that no advantages were apparent.

The grout was a two-component compound, mechanically mixed and injected into the cracks through the drilled holes. Each component had its individual manually operated pump, connected by a common handle for pumping equal volumes. The components were mixed by a mixing valve which was attached to a short piece of 3/8-inch pipe. At the end of the pipe was attached a tapered coarse-threaded nozzle arrangement. This tapered nozzle was first screwed into a piece of 3/8-inch soft plastic pipe about 3 inches long. The nozzle and plastic pipe were then inserted in the hole and the tapered nozzle turned. The resulting pressure on the plastic pipe served as a seal.

The pumps were capable of pumping pressures up to 1,000 psi. Normally, pressures of 300 to 600 psi were used.

Set time is controllable with chemical grout and was varied to meet conditions such as amount of travel, amount of seepage water, and quantity of take. Generally, a set time of 2 to 3 minutes was used. This controlled set time coupled with the interception of the drill holes with the cracks minimized the injection of grout behind the concrete lining.

The chemical grouting proved very successful. One time over an area as described above would stop 90 to 95 percent of the water. Some areas required as many as five or six injections with an average of three required to completely dry up an area. This would indicate higher than normal hydrostatic pressures behind the concrete lining.

The drilling crew consisted of three drillers working on a two-shift basis. The grout crew included two experienced men supplied by Penetryn with two helpers working on a one-shift basis.

Leakage in some local areas proved to be difficult to stop even with the chemical grout. In these individual leaks it was determined that by chipping 3 to 4 inches into the concrete while following the water, a 1/4-inch copper pipe could be calked into the crack or hole making water. The chipped area was then repaired with epoxy mortar, Figure 3-8. If the repair was in a critical area the copper pipe was ground flush with the surface, then driven full

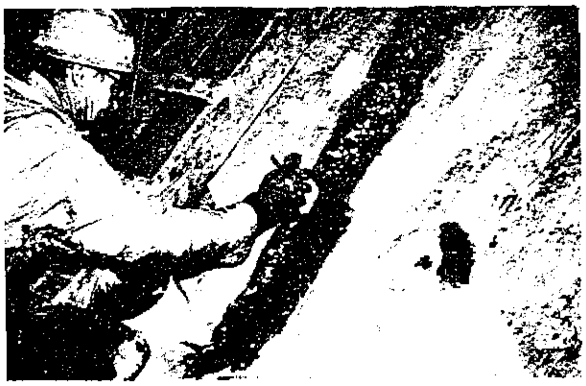


Figure 3-8. Epoxy-mortar repair being made. Finisher is compacting epoxy mortar with wooden tamper. Photo P459-D-68812

of lead wool. If the repair was not in an area subject to cavitation, the pipe was ground flush and left to weep.

A commercial fast-set cement called Sika Plug was used extensively in drying up small local areas. This cement was used to build small temporary dams to divert water around the area to be patched.

2. Heating the tunnel.—The specifications required that the epoxy mortar be cured at the recommended application temperature for 24 hours immediately following application. The manufacturer of the Concresive 1161, Type II, recommends a 40° F minimum application temperature.

The temperatures specified for the epoxy-phenolic paint were a concrete and air temperature of 60° F at the time of application, and a 60° F minimum air temperature between coats and for 48 hours after the last coat.

The average temperature in the tunnel before heating was approximately 45° F. The average temperature maintained during the work was 65° F. The 20° F temperature rise was accomplished with a Janitrol propane-fired heater manufactured by Midland-Ross Corporation. The heater was rated at 2,500,000 Btu per hour with a blower driven by a 25-horsepower electric motor and capable of moving 25,000 cfm. The heater was mounted in the downstream bulkhead located near the downstream portal. The upstream end of the tunnel was also covered.

3. Surface preparation.—Surface preparation for the epoxy-bonded repairs and the epoxy paint was accomplished by sandblasting. For the epoxy-bonded mortar applied to small areas, an air-suction-type gun was used. Cleanup for the paint was performed with a small commercial sandblaster. After blasting, the area was cleaned thoroughly with an air jet.

4. Epoxy resins.—The epoxy resin was stored in the tunnel in lots of about 5 gallons. Storage was in 3- by 3- by 3-foot plywood box with an access door on the side. The box was heated with one 250-watt heat lamp. This maintained a temperature of 75° to 80° F in the storage box, Figure 3-9.

Figure 3-9. Wooden storage compartment for warm storage of epoxy components. Material temperatures of 75° to 80° F were maintained in this simple setup. Photo P459-640-1004 NA

The two components of the resin system were measured by volume, as directed by the manufacturer, in a plastic container. Mixing was simply by stirring or beating thoroughly with a wooden paddle. Batch size was 1 quart or less depending on the size of the area being worked. The mixed material was then used as a bond-coat or as the binder in an epoxy mortar. The mortar was prepared as discussed in Subparagraph C.(b)2.

Immediately prior to the application of the bond coat the concrete surface was heated with a propane torch until warm to the touch. The bond coat was then applied using a stiff bristle bathroom-type brush. In the case where the bond coat was being applied to concrete, all pits and holes were carefully covered. Where the epoxy-bonded mortar was being applied to a previously placed mortar patch or mortar "veneer," which occurred occasionally to make alinement corrections, extreme care was used to insure a liquid cover of bond coat on the surface of the in-place epoxy mortar. The epoxy mortar absorbed the bond coat readily in some areas, depending on the density of the mortar. Several applications were sometimes necessary with the last coat applied immediately before the epoxy mortar was placed.

As the veneer repairs developed, the project reported that finishing became more of a problem. By adding more sand it was found that the mortar became less sticky and easier to trowel. A mix of one part mixed resin to six parts sand by volume was used on most of the veneer application.

The sand was dried in small quantities with the use of a small mixer and a propane torch. The mixer had a capacity of about 3 gallons of sand. As the sand was mixed, a propane torch was held by hand on the surface of the mixing sand and on the outside of the mixer.

The epoxy mortar was then mixed by adding the sand to a known volume of epoxy resin which had been thoroughly mixed in the proportions described above. Mixing was accomplished with a paddle made from a 3/8- or 1/2-inch rod driven by an electric drill. Batch size was usually about 1 gallon.

In application of the epoxy mortar there were at least four problems:

a. Density.—The density was obtained by tamping the mortar with a 2- by 4-inch board in the case of a large deep patch. Where the patch was small in area and 1/2 inch to 2 inches deep, the finisher usually tamped the mortar with the handle of his trowel. Epoxy-mortar veneer was compacted with heavy pressure on the trowel as the mortar was placed. The veneer was normally around one-fourth inch thick. Surface texture was very rough if the density was not obtained.

b. Finishing.—Finishing of the mortar requires a technique different from that of finishing concrete. The troweling motion must be much slower. A fast motion will tend to pull the mortar with tearing of the surface and the resulting low density and rough texture. The finishers found that the heating of their trowel slightly with a low-burning propane torch resulted in much easier troweling and less sticking to the trowel. At no time was solvent used by the finishers except in cleaning their tools.

c. Joints and edges.—The application of epoxy-mortar veneer over large areas resulted in joints between successive areas of work. It was found that one source of trouble was the tendency of the finishers to carry the epoxy mortar beyond the limits of the bond coat. This occurred at the joints when placing the epoxy mortar and also on individual patches of small areas. To overcome this, the bond coat was extended beyond the limits of the epoxy mortar when the patch was complete or beyond the epoxy mortar at the joints in placing the epoxy. This was accomplished in the placing of the veneer by placing the mortar to the limits of the bond coat and then trimming the mortar back with a small trowel. A straight, vertical edge resulted.

When the joint was left overnight or if the bond coat became set during the day before the adjoining section was applied, the exposed bond coat was completely removed by sandblasting.

The finishers usually worked in a pattern consisting of strips 2 to 3 feet wide when applying the epoxy mortar. The length of the strips was governed by the tackiness of the extension of bond coat beyond the placed mortar. When this extension began to become tacky, the finisher would drop back and start a new strip. The result was that only the ends of the strips and the last extension of the bond coat in the day would need to be sandblasted. These strips usually ran from 10 to 15 feet in length.

When applying bond coat to the joint, the bond coat was carried beyond the joint in order to insure bond at the joint.

d. Cure.—One of the major problems in the use of the epoxy mortar was obtaining a satisfactory cure. The humidity in the tunnel was relatively high, averaging around 75 percent. Also contributing to failures in the work was the unpredictable nature of the seepage water. Seepage cracks could be dried up and kept dry for several days. Then, for no apparent reason, they began seeping again.

The air temperature in the tunnel was maintained at about 65° F. The surface temperature of concrete was 65° F. This surface temperature was increased with a propane torch before applying the bond coat as described previously. Sand temperatures were 60° F. The temperature of the epoxy resin was 75° to 80° F before mixing.

5. Grinding.—The finishers were never able to apply the epoxy-mortar veneer to the 100:1 tolerances prescribed in the specifications. Consequently, all of the epoxy-mortar veneer was ground to obtain the desired tolerances.

Grinding was accomplished with air and electric grinders using a No. 16 grit concrete stone. Grinding of the epoxy mortars was fairly fast, when grinding was done dry. This presented a dust problem and the grinder operators were required to use respirators at all times. Some gumming of the grinding stones was encountered. However, this situation likely resulted from attempting to grind incompletely cured epoxy and/or allowing the area being ground to become too hot.

Wet grinding was tried, but the stones gummed up and grinding was very slow.[2]

6. Painting.—The epoxy-phenolic paint was applied in two coats. The first coat was clear and the second coat white pigmented. Specifications called for a minimum thickness of 1-1/2 mils per coat with a maximum thickness of 6 mils for the two coats. The first coat was required to be thinned with 1-1/2 parts paint thinner.

The paint was applied with double 7-inch rollers fitted with a long handle. This gave an effective roller width of 14 inches. The application of the paint was very easy. In the horizontal portion of the tunnel, the paint was applied to spring line from the invert by extending the handles of the rollers.

Because the tunnel surface was not 100 percent dry when painted, those areas which were seeping were dried with a propane torch immediately before applying the paint. This method was used before applying both the first and second coat. Extreme caution was required when using an open flame around this paint because the solvent is inflammable.

When the clear or first coat was applied over an area that had been torch dried, the water when reappearing would come through the paint and flow over it. However, when the water appeared after painting the white or second coat, a blister would form in the paint. If enough water was seeping, the water would collect under the surface and cause long blisters in the painted surface.

It was determined during the first application that it would be impossible to obtain the desired 1-1/2-mil thickness with a single coat over the epoxy mortar due to the absorption of the paint by the mortar. In order to obtain a dry-film thickness of 3 mils the first coat was applied with a heavily loaded roller. The white coat was then applied to achieve a 6-mil wet-film thickness. This was to be accomplished by multiple applications where necessary. In some areas it was never possible to achieve the 6-mil wet-film thickness due to the rapid absorption of the paint by the epoxy mortar. Up to 10 applications were tried in an effort to obtain the minimum thickness.[3] It was found when removing some of the painted epoxy mortar that the white paint had penetrated up to one-fourth inch.

The coverage of both the first and second coats averaged approximately 200 square feet to the gallon. The coverage should have been approximately 400 square feet as computed. This indicated the degree of absorption.

Soon after the painting was started there were some blisters which developed in the epoxy-mortar veneer. These blisters were approximately 1 inch to 1 foot in diameter. Originally, it was suspected that the blistering was caused by an expansion in the epoxy mortar caused by the solvents in the paint. Thereafter, the first coat of the paint over the epoxy mortar was applied without thinning. A longer time between coats was specified to allow the solvents time to dissipate. The time was increased to 4 hours from the approximate 2 hours which was needed for the first coat to become tack free as required by the specifications.[4]

Laps over previously painted surfaces were held to a minimum, usually under 2 inches. In all

[2] Other contractors have successfully wet ground epoxy mortar. On this job it is believed that some of the early attempts at wet grinding were made before full cure occurred in the applied epoxy mortar, with resultant difficulties.

[3] The absorption of the paint was due to the porosity of the epoxy mortar. Various porosities were present because of the nonuniformity of the epoxy-mortar mixes as discussed later. The 1:6 mixes were extremely porous resulting in high absorption rates.

[4] It was later found that the solvents in the paint actually softened the epoxy mortar and if topcoated too soon were entrapped resulting in the blistering that was experienced.

cases, the previously placed paint at the lap was roughened using a power grinder with a No. 16 grit stone.

After curing, the paint appeared to be very tough and, in cases where patchwork was done adjacent to painted areas, it was found the paint could be removed by sandblasting only with persistent effort.

(d) June 1968 examination.—

1. Inspection.—Early in June 1968, a detailed inspection of the condition of concrete and repair work and contractor's operations in the horizontal and inclined sections of the 32-foot-diameter spillway tunnel was made. The following observations were noted:

a. Horizontal section of tunnel downstream from Station 10+00 was relatively free from seeping moisture; 16 to 20 small damp to wet areas were visible between Stations 10+00 and 12+00. Tunnel upstream of Station 10+00 was considerably wet, partly from seepage through cracks, etc., and partly from flowing pressure relief holes.

b. A crew from Penetryn Grouting Company was injecting chemical grout into cracks in concrete between Stations 9+00 and 9+50. Prior to this inspection, the same crew had chemically grouted seeping cracks downstream from Station 9+50.

c. Many cracks downstream from Station 10+00 had been routed and calked to prevent seepage but some were still leaking. Cracks (originally hairline) at approximate Stations 11+80, 11+90, 12+05, and 12+15 had been routed as many as six times and were now about 6 to 12 inches wide and 6 to 12 inches deep.

d. Some of the epoxy-bonded epoxy-mortar patches were soft and some had had water seep through them, but the majority were hard and satisfactory.

e. Contractor was working on last stages of excavation for aeration slot.

f. All of concrete repair work in the horizontal section of the tunnel was complete except for the aforementioned routed cracks.

g. A considerable amount of epoxy-mortar patchwork remained to be done.

h. Surface and air temperatures were 65° F.

2. In-place epoxy-mortar patches.—Three conditions of in-place epoxy-mortar patches existed. The first condition was where patches had cured hard and were completely satisfactory, except possibly for some that needed grinding to smooth the surfaces. Ninety to ninety-five percent of all the epoxy-mortar patches were in this category. Those that required grinding were known to the inspectors and the grinding scheduled for some future date.

The second condition of epoxy-mortar patches in place was where the epoxy-mortar patches were soft, but dry, after considerable time in place. About 5 to 8 percent of the applied epoxy-mortar patches were in this condition. A piece from one of these patches that had been chipped out the preceding day cured hard overnight in the glove compartment of a car. This indicated that the softness was due to incomplete cure of the material, probably due to the relatively low prevailing temperature in the tunnel.

A test was set up to determine if heating would accelerate the cure of the soft in-place epoxy-mortar patches. A test patch at Station 11+01 on the right side was selected. Surface and air temperatures were 65° F. Heat was applied with a portable propane-fired heater placed about 2 feet from the surface. A time/surface temperature log was kept by the inspector. In 2 hours and 10 minutes the surface temperature rose to 101° F. An additional 2 hours of heating only resulted in a 1° rise, to 102° F. At this time the heating was discontinued. One hour and thirty-five minutes later the surface temperature of the patch and surrounding concrete was still 83° F, indicating that heating in depth had been achieved. The epoxy-mortar patch was now hard and well cured.

A five-lamp infrared heater approximately 5 feet long, was built for use in heating long epoxy-mortar patches. This lamp heater and the propane heater were then used to cure a second long patch at Station 10+31 on the right side, as shown in Figure 3-10. Temperature under the infrared lights reached approximately 90° F in about 4 hours; quite sufficient to post cure the in-place epoxy mortar. It was recommended at that time that all new patches be post cured with the heaters during the swing shift after application, and that all relatively new in-place patches be post cured also. It was suggested that the heaters be used on one patch or another continuously until all were post cured.

The third condition of the in-place epoxy-mortar patches was where water had seeped through

Figure 3-10. Bank of infrared lamps and a propane-fired radiant heater being used to post cure epoxy-mortar patches. Photo P459-D-68813

before a cure-hard condition was achieved. They were soft and damp or wet. Raising the temperature of these patches by transmission of heat from the heaters improved the condition somewhat, but a full hard cure was not obtained. A try to salvage one such patch was made at Station 10+19, but without success. This patch was replaced. Later prolonged heating of another soft wet patch in the same vicinity was tried, but again without success. It was then directed that all wet patches that were soft when tested with a knife be removed and replaced. It was apparent that water in contact with the uncured epoxy mortar seriously degraded the cure and final hardness.

3. New concrete repairs.—Concrete repairs made prior to this inspection appeared to be satisfactory except for a few areas which had to be ground to bring them within the specified tolerance. Unlike the procedures followed in the mixing and transporting of the concrete placed in the large repair area, concrete placed in the small areas was batched and mixed in the tunnel. Concrete materials for 1-cubic-foot batches were weighed on a small platform scale and mixed in a small rotating drum mixer. The concrete looked and worked very good. The following is the concrete mix used and the compressive strengths of control cylinders:

Cement: Type II, low alkali—7 sacks per cubic yard
Sand: 34 percent by weight of total aggregate
Coarse aggregate: 3/4-inch maximum size
Water-reducing agent (WRA): Plastiment
Air-entraining agent: Protex single strength
Air content: 3 percent plus or minus
Water-cement ratio: 0.38

Control cylinder compressive strengths:

7 days' age: 6,120 to 6,920 psi
28 days' age: 8,190 to 9,020 psi
90 days' age: 9,270 to 9,850 psi

Concrete repairs being made encompassed about four routed out cracks (6 to 12 inches wide by 6 to 12 inches deep) between Stations 11+80 and 12+15. Prior to the placing of concrete, the areas to be repaired were sandblasted, cleaned, and dried. Epoxy-bond coat was applied to the periphery of all of the deeper repair areas (the 6 inches plus or minus nearest the top of excavation) and the entire surface area in shallow excavations. Low slump concrete was then placed. This entire operation was conducted very smoothly and very adequately.

The only aspect of the concrete repair operation that was considered less than desirable was the slow setting caused by the WRA. Set-retarding characteristics of the WRA was required when the bulk of the concrete was previously placed, since mixing was done out of the tunnel and elapsed time from mixing to placing was 1 to 1-1/2 hours. However, the small batches prepared for the smaller patches were placed within 5 to 10 minutes after mixing and as a result the curing of the epoxy-bond coat tended to get somewhat ahead of the setting of the concrete. This situation was discussed at the jobsite; however, no change was instigated since placing of the last batch was underway. A water-reducing agent that is a set-retarding agent should not be used in such cases.

4. New epoxy-mortar repairs.—The entire operation regarding preparation and application of the epoxy-bonded epoxy mortar was

satisfactory. It was suggested that the two components of the epoxy be mixed for a longer period of time than was being done just to insure that a thorough mixture was obtained. A typical epoxy-mortar application went as follows:

a. Concresive No. 1161 epoxy material, mixed five parts A to three parts B by volume, was prepared and applied with a brush to the previously sandblasted, cleaned, and dried concrete area. It should be noted here that little or no warming of the concrete was done since the specifications indicated it was not necessary. The concrete surface at time of placement was about 65° F.

b. As soon as the application of the bond coat was started additional resin was prepared as described in "a" and project sand was added, three parts sand to one part resin by volume (4 to 1 by weight). This material after a thorough mixing was tamped into the area to be repaired then steel troweled. The epoxy mortar mixed in the aforementioned proportions provided essentially a no-slump mix; however, it was quite porous.

Subsequent to the laboratory-mix tests discussed herein in Section 7, it was recommended that the minus No. 200-size sand be used in conjunction with the project sand. About 200 pounds of minus No. 200 material was obtained for this use.

Aside from the additional fines added to the sand mix, it was directed that additional heat be applied to the epoxy-mortar repairs to accelerate the cure. This was particularly important where water seepage was a potential problem. Epoxy-mortar repairs that were heat cured (110° to 115° F) were very hard the day following placement. As many mortar in-place patches as possible were to be cured with heat just to insure their adequacy. Such curing was to be accomplished on the swing and graveyard shifts since placing of patches was being done only on the day shift. This arrangement was also desirable from the standpoint that patches placed on day shift would have time to take an initial set before heat would be applied on swing shift.

5. APCO 436.—An experimental epoxy-patching material, APCO 436, furnished by Applied Plastics Company, Inc., El Segundo, California, was sent to the job. This was an especially batched material formulated to meet the special job conditions. Supposedly, it was a fast-cure concrete repair system that was particularly suited for use under the prevailing tunnel conditions. Recommended application time was 20 minutes after mixing. Curing times were stated to vary from 10 minutes at 180° F to 2 hours at 70° F. A cure time for 65° F, the actual-use temperature, was not given, but would be expected to be in excess of 2 hours based on information that was given.

Three tests were made with this material. The first was an attempt to stop slowly flowing water in a crack where a bonded concrete repair was to be made. The second was a trial application as a surfacer on clean, dry, eroded concrete. The third test was a trial application of a surfacer on eroded concrete that had leakage water flowing over the surface.

Approximately 18 hours after the test materials were applied, that applied to the dry, cool surface was still soft and not cured. That applied to the other surface, dry and warm, was hard where the material had remained dry, but was relatively soft where wetted with water flowing over it. Twenty-four hours later (approximately 42 hours after application) material in the "wet" area was in about the same condition, whereas that in the cool, "dry" area was still soft; however, a surface skin was forming and the thinner areas were beginning to firm up. The low temperature was retarding the cure, and that cure which was occurring was happening from the surface inward.

6. Chemical grouting.—The chemical grouting operations underway in the spillway tunnel were originally proposed by the contractor as a method for eliminating seepage from cracks in the tunnel lining. However, its use was not instigated until the method of routing cracks and calking with lead wool had been tried and proven unsuccessful. The injection of chemical grout into the weeping and flowing cracks was a slow operation, but its use was successful in drying of the tunnel surfaces.

The chemical grout used for this particular application was "Q-Seal" which is more commonly known as "AM-9." This material is a mixture of two organic monomers dissolved in water producing an aqueous solution which then produces very stiff gels when properly catalyzed. The characteristics of this particular grouting material lends itself very well to applications such as encountered on this job. Controlled gel times can be varied by proportioning of components, temperature, and other factors.

Grouting operations consisted of essentially the following:

a. Locations where it appeared grout injections would be most suitable were spotted by the Superintendent for Penetryn Grouting Company. COP Construction Company personnel then drilled 1/2-inch-diameter by 12-inch-deep holes, the collars of which were located about 3 to 5 inches from the leaking crack, and the direction of drilling oriented so that it hopefully would intersect the crack.

b. A proportioning system was used to inject the 14 percent strength, two-solution grouting material. The two solutions were pumped with independent, single-piston, positive-displacement, hand-operated pumps through independent lines to a mixing nozzle at the injection point.

c. Pressures used to inject the material varied, but probably averaged 300 to 600 psi with 600 psi as a maximum. No grouting materials were injected behind the concrete liner. Gel times were set at about 3 minutes; however, this varied somewhat also.

It was evident that the grouting was well in hand, although three aspects were questioned, namely:

a. No special effort was being made to flush or jet out grout holes after drilling and before grouting. It was suggested that more grout might be injected and under lower pressures if the grout holes were thoroughly cleaned, prior to grouting, with a water jet; however, this was never done. Thereafter, however, the holes were drilled with a higher pressure stream of water flowing through the bit. The benefits gained from this are unknown.

b. Only 1/2-inch-diameter holes were being used for grout injection. One reason for their using such small holes was that electric drill-type equipment could be used to achieve portability. By having 2 and 3 men drilling 3 shifts, 30 to 40 holes per day were made available for grouting. It was suggested that a 1-inch-diameter drill bit be used on a trial basis. Two ways it could improve the operation were:

(1) Provide twice as much crack length through which the grout could be pumped.

(2) Considerably better chance of intersecting crack to be grouted.

(In a subsequent report from the field on the trial with a 1-inch bit, it was reported that drill time was about doubled. Drilling was done normal to the surface over the crack. In many cases the crack was lost in a few inches depth. The grouters reported no noticeable increase in grout take. Based on this brief evaluation, the use of the larger bit appeared to offer no advantages and was terminated.)

c. It was suspected that possibly some of the chemicals in the cracks of the concrete such as carbonates, etc., were reacting with the chemical grout, thus reducing its gel time. A small laboratory investigation was conducted to clear up this point. Test results indicated that any effects on the grout from chemicals in the grout area were insignificant.

7. Mix tests.—It was apparent that although the specified epoxy mortar being used was adequate it was not an optimum mix. Therefore, some mix tests were undertaken in the field laboratory. These tests also showed that proper cure was obtainable with the specified materials that were furnished.

To prepare the epoxy binder, five parts by volume of A (resin) were mixed with three parts by volume of B (hardener). This resulted in a 14:10 ratio of A:B by weight.

Gradation of the sandpile sand was as follows:

Retained on No. 16	3 percent
Retained on No. 30	18 percent
Retained on No. 50	46 percent
Retained on No. 100	28 percent
Retained on No. 200	4 percent
Retained on Pan	1 percent

This resulted in a filler gradation passing the No. 16 screen as follows as compared to an optimum filler gradation as determined in previous laboratory work (Bureau of Reclamation General Report No. 28, Epoxy Resins for Concrete Construction and Repair).

	Filler as used	Optimum gradation
Retained on No. 30	18.5	31
Retained on No. 50	47.4	23
Retained on No. 100	29.0	16
Retained on No. 200	4.1	–
Retained on Pan	1.0	30

It was readily apparent that a prime deficiency in the filler being used was in the percentage of fines, that passing No. 100. This was also evident in the coarseness and porosity of the epoxy-mortar mix that was being used.

Mix tests were started with the specified suggested starting mix, one part mixed resin to four parts sandpile sand by weight. This resulted in a resin-rich mortar that flowed slightly when spread on a flat surface. Such a mix would not be suitable for use in the tunnel and would have certain undesirable properties.

A mix was then prepared that represented a useable mix by adding sandpile sand to mixed resin until a proper consistency was reached. The mix calculated out to one part mixed resin to six parts sand by weight. This mix had good application properties and showed no tendency to slump, but had a coarse surface appearance.

The mix that was actually being used, 1 part of mixed resin to 6-1/4 parts of sandpile sand by weight was then tried. Application properties were good and no slump was evident. However, a coarse, dry appearance with apparent porosity resulted.

Modification of the filler by the addition of minus No. 200 sand to a basic 1:4 mix was tried. The resulting mix was in the proportions of one part, by weight, mixed resin to five parts of modified sand. This mix was judged to be too resin-rich, but did show improved density and surface appearance.

A final mix was made consisting of one part, by weight, of mixed resin to six parts of modified sand (63 percent sandpile sand, 37 percent minus 100 sand). This resulted in a dense, trowelable, no-slump mix. This mix was the basis on which the recommended changes were made.

It was decided that the most reasonable modification would be one that could be readily worked into the procedures being used; that is, proportioning in parts by volume. Some minus No. 200 silica sand was obtained from a supplier in Billings. Minus No. 200 was suggested because the finer particle size enhances the thixotropy of the mix. It was further suggested that the filler be prepared by blending two parts by volume of minus No. 16 sandpile sand with one part of the minus No. 200 sand, and that this blend then be used to prepare the epoxy mortar in the same manner being used. This resulted in a mix of one part by volume of mixed resin and three parts by volume of blended sand.

The approximate gradation of the modified filler as compared to the optimum gradation for filler was as follows:

	Modified filler	Optimum filler
Retained on No. 30	12	31
Retained on No. 50	32	23
Retained on No. 100	20	16
Retained on Pan	36	30

As can be seen, the percentage of fines was much more reasonable in the blended, modified filler.

This new mix was tried later in the tunnel. No mixing or application problems resulted from the changes and the applied epoxy mortar was considerably denser than that previously used. All mortar was thereafter prepared with the modified filler.

As previously indicated, no unusual curing characteristics were evident in this brief laboratory study. Room temperature was 76° F and all mixes cured relatively hard in about 4 hours and full hard overnight. Exotherm in one mix raised the measured temperature to about 81° F. This indicates that a proper cure of the specified materials is practical at a reasonable temperature level. At 65° F, the prevailing tunnel temperature, the cure of the epoxy mortar was very slow. It was suggested that heat be used to accelerate the cure of all epoxy-mortar patches.

8. Conclusions—June 1968 examination.—On the basis of observations and other considerations at the jobsite, the following conclusions were made:

a. Major problem with the soft epoxy-mortar patches which were placed prior to this examination was the extremely slow cure due to relatively low temperatures.

b. With heating, the soft, dry epoxy-mortar patches were cured to a very hard material, and the cure of newly applied epoxy-mortar patches accelerated.

c. Epoxy-mortar patches were damaged in areas that became wet before the cure had advanced far enough. (These were subsequently removed.)

d. Seeping water was definitely a major problem in making the repairs; however, the chemical grouting substantially reduced this problem.

e. Application techniques used by personnel in making the epoxy repairs were good.

f. The total work under this contract could have been speeded up with additional personnel.

g. Water-reducing agents that are also set-retarding agents should not be used in concrete to be epoxy bonded unless the delay in placing of the concrete is sufficient to require it.

h. The addition of minus No. 200 silica (sand) to the project sandpile sand for use in the epoxy mortar provided a considerably denser mortar patch without sacrificing workability or slump characteristics.

i. Specifications for future repair jobs should be written to provide for a specific epoxy-mortar mix design for the specified job. The effects of temperature on the epoxy materials should be more clearly delineated.

9. Recommendations.—The following recommendations resulted from this examination and constituted changes from the specifications:

a. Further routing of cracks for sealing with epoxy-bonded epoxy mortar be discontinued and that chemical grouting be used to stop seepage through the cracks.

b. The epoxy-mortar mix be modified by the addition of minus No. 200 fines.

c. Heat be used to accelerate the cure of epoxy-mortar patches.

(e) Epoxy-mortar veneer.—

Application of the epoxy-mortar veneer proceeded, thereafter, with procedures modified by previous recommendations. Late in 1968 and early 1969, the presence of blisters in the veneer was noted by project personnel and construction supervision personnel from the Engineering and Research Center.

It was decided that further detailed examination by research personnel to find the causes and to initiate remedial measures was warranted. Two trips were made; the first to accomplish the above; the second, to further evaluate the veneer lining after unusual results were found following recommended procedures.

(f) January—February 1969 examination, modification, and results.—

1. Inspection.—Initially the problem areas were examined which primarily consisted of scattered blisters in applied epoxy-mortar veneer. All operations in mixing, applying, and finishing epoxy mortar as well as other finishing operations were critically observed.

The underside of epoxy mortar from virtually all of the blisters examined showed smooth resin around the sand particles. The surface from which the blister lifted was also smooth. There were no fracture zones in the surfaces of the exposed resin. This indicated that the separation of mortar from the surface occurred before the resin had set. These characteristics were later verified under high magnification in the laboratory in comparison with fractured mortar that showed distinct fracture zones in the resin surface. One blister had a gel material in it (later identified by IR absorption as the chemical grout) and virtually all had water in them. Later during the application of veneer such blisters were observed to form in the freshly applied epoxy mortar when heating was started to accelerate the cure. The underside of a large blister so formed had the same appearance as the other blisters. This gave fairly conclusive evidence that the blisters were formed during the early stages by shallow separation within the wet bond coat by expansion of air and/or water vapor in the underlying concrete surface as a result of heating. Large blisters formed later when sufficient pressure built up within the separated area by entrapped air, water, or in the one case by chemical grout.

Two important deviations from previous recommendations came to light during this examination. The first was that the filler gradation had changed. Virtually no fines were present. The sand supplier in Billings was not furnishing minus 200-mesh sand as ordered, and it is probable that the gradation of sand delivered from Billings increased in coarseness from shipment to shipment throughout the course of work as evidenced by the nonuniformity of applied mortar. Epoxy-mortar application was stopped until fines could be included in the mix. Minus 100 pan grade sand was requested from the Engineering and Research Center laboratories for delivery to the job.

The second deviation was in the mix proportions of the mortar being applied. A mortar of one part mixed resin to six parts of filler by volume was being used, primarily to facilitate the application. The 1:6 mix was very lean and considerably different from the richer recommended mix. As a result the applied mortar veneer was very porous.

2. Mix modifications.—When the pan sand was received, a new mix was designed for the remaining epoxy work. For the filler 2 parts by volume of −30 project sandpile sand were blended with 1 part of the −100 pan sand. This was then mixed in a 1:3 by volume mortar which proved to be too rich. A 1:4 mix was then prepared and tried. This gave a dense, workable mix that greatly improved the character of the epoxy mortar, and, although not a carefully designed mix, would suffice for the remaining work.

3. Remedial measures.—Remedial measures to reduce the possibility of separation of mortar from bond coat and subsequent blistering were initiated. These included prolonged preheating of prepared surfaces, preparation, and application of mortar at higher temperatures (70° to 80° F), initial cure at ambient conditions, and post curing at elevated temperature. Although these measures, and the new mix, greatly improved the situation, they were not 100 percent effective, particularly where severe water problems were present. However, many successful repairs were made. Specific written instructions were given to continue with these procedures.

4. Unusual results.—During the preheating operation, softening of epoxy mortar around a repair area was noted. In this particular case the soft, hot mortar smelled strongly of paint solvent and polysulfide. This peculiar behavior was discussed and a series of heat tests on applied lining suggested. These heat tests were conducted by project personnel during the week of February 10, 1969, with telephone instructions from personnel at the Engineering and Research Center.

The intent of the heat test was to show up areas that would soften under heating and to perhaps force the blistering where disbonded areas existed but were undetected.

A second trip was made to evaluate the results of the heat test, when it was reported that virtually all of the mortar heated turned soft. Some of the original tests were repeated and additional tests conducted. The reported results were verified and additional data obtained. No blistering was forced. Using a modified hand, concrete penetrometer and an arbitrary scale ranging from (1) very hard, to (5) very soft, the overall condition of the lining was evaluated.

Except for one spot that was relatively soft at the tunnel temperature, which was later removed and replaced, the lining was found to be hard to very hard in the cool conditions at tunnel temperature which changed to condition varying between hard, slightly soft, soft, and very soft, when heated to 120° F as determined by spot tests. In all of the soft states, the lining smelled strongly of paint solvent and polysulfide. An epoxy-mortar patch several months old in a downstream location was partly painted with the specified paint system and new patches made with the improved 1:4 mortar painted with the white only. The old mortar softened in depth under the paint when heated whereas there was only some surface softening in the new patches. In all cases the softened mortar returned to a hard state when cooled back to tunnel temperature, 60° to 65° F. When hard, the lining was hard clear through and when soft the lining was soft clear through, but to different degrees as previously noted.

It was apparent that the softening was the result of increased activity by entrapped solvent from the paint system when the mass was heated. Twenty-four-hour heating did not drive off the entrapped solvent, although very long-term heating might. Because of the molecular structure of the epoxy system being used, it was probable that no chemical decomposition had occurred, only absorption and entrapment of solvent in the molecular latticework. At the low tunnel temperature there was little activity by the solvent, thus the mortar was hard. At increasing temperatures the activity increased and softening was apparent. The entrapped solvent should diffuse out of the lining in time without permanent after effects.

5. Conclusions.—It was concluded that the majority of the applied epoxy mortar contained entrapped solvent due to several contributing factors; such as, use of a lean epoxy mortar, absence of fines in the filler (both contributing to excessive porosity), and unexpected effects by solvent entrapment as a result of subsequent painting. The denser the mortar, the less the overall effect. Although the lining is not the best quality because of the above factors, it should be serviceable. Some additional blistering and lining repair should be expected.

6. Recommendations.—Because of anticipated additional latent blistering, it was recommended that an inspection be made by project personnel just prior to the forthcoming hydraulic test, and any defects repaired.

(g) Final examinations of repairs—March 1969.—In March 1969, a team from the Engineering and Research Center visited the project to conduct a near end-of-construction examination. Following are the results, with brief statements of work through various reaches:

1. Tunnel reach from Station 12+15 to Station 22+33 (downstream portal).—The major item of work consisted of repairing large irregularities in the tunnel lining with epoxy-mortar patches. All of these patches were examined and found to be hard and when tapped with a hammer gave every indication of being well bonded to the concrete.

2. Tunnel reach from Station 7+69 to Station 12+15.—The major items of work consisted of construction of an aeration slot at Station 7+79, application of epoxy-mortar veneer to repair small irregularities, placement of concrete to repair the major damaged areas, placement of epoxy-mortar patches as required, sack rubbing mortar to fill "bug holes" below the spring line, grinding of surfaces to obtain the specifications requirements for finishes, modification to the aeration slot to obtain proper lift at the lip, installation of monitoring equipment in the slot, and application of epoxy phenolic paint to the flow surfaces.

All items of work had been completed with the exception of minor patching in the lining at random locations between Stations 11+10± and 8+84±, minor grinding in the area immediately downstream from the aeration slot, and application of epoxy phenolic paint over newly repaired or ground areas.

The epoxy-mortar veneer and paint application were examined and judged acceptable. Two small blisters were found at Stations 8+88± and 9+06±. These appeared not to be newly formed and were detected after tunnel cleanup in the area.

Modifications to the aeration slot were essentially complete. Measurements made on the lip indicated that acceptable lift existed everywhere except in a small area 18 inches below the spring line (right side) where no lift was noted. Since this deficiency was near the spring line where no lift was required and adequate lift exists 24 inches below the spring line, corrective measures were not considered necessary. Repairs to the area adjacent to and extending about 15 feet upstream of the slot were completed except for application of a reinforcing coat of epoxy resin on the veneer and painting of the lip. Finishes and alinement at this area were satisfactory. During the examination the contractor was engaged in applying epoxy-mortar veneer to ground areas in the concrete on both sides of the tunnel several feet downstream from the slot.

It was noted during the examination that the contractor's activities had gouged the epoxy-mortar veneer in some areas and that in other areas epoxy mortar "slobber" had adhered to the surface. All such irregularities, in addition to any calcium carbonate buildup, were to be removed prior to final cleanup of the tunnel. Cleanup was to include removal of temporary "dams" used to divert tunnel leakage around work areas, removal of flumes and repair of anchor boltholes that support the flumes, and repair of any other surface deficiencies caused by the contractor's activities.

3. Tunnel reach from Station 6+00 to Station 7+69.—Major items of work in this area consisted of epoxy-mortar patches in areas where previous patches had failed or where deficient patches had been replaced, and removal of calcium carbonate buildup. Random epoxy-mortar patches were tested and, although varying degrees of hardness were noted, the work was generally acceptable. Two patches were found deficient and were to be

replaced. Although most of the carbonate deposits had been removed, additional removal was required at random locations. Several holes, up to 3 inches in diameter and with varying depths, were noted and were to be repaired.

The appearance of the spillway tunnel at this time is shown in Figure 3-11.

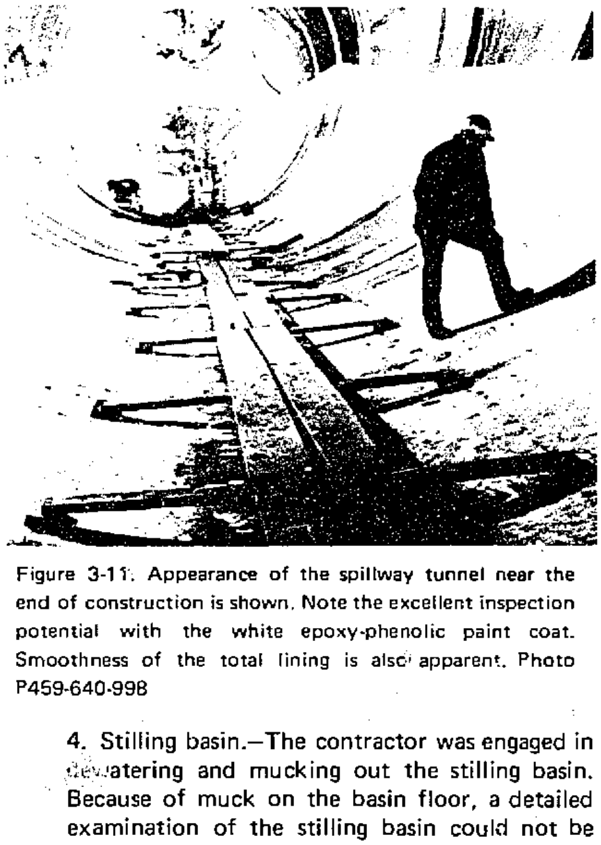


Figure 3-11. Appearance of the spillway tunnel near the end of construction is shown. Note the excellent inspection potential with the white epoxy-phenolic paint coat. Smoothness of the total lining is also apparent. Photo P459-640-998

4. Stilling basin.—The contractor was engaged in dewatering and mucking out the stilling basin. Because of muck on the basin floor, a detailed examination of the stilling basin could not be made. Except for two small spalled areas immediately downstream from the tunnel portal, the unwatered portions of the basin were satisfactory. Instructions were given to repair these spalled areas and any other deficiencies found after complete basin cleanup had been made.

D. Aeration Slot

Two working platforms or jumbos were constructed to facilitate work in the tunnel, one was located in the vicinity of the aeration slot and the other below the slot. The latter jumbo was used for the repair work in the elbow of the tunnel, Figure 3-12. These well

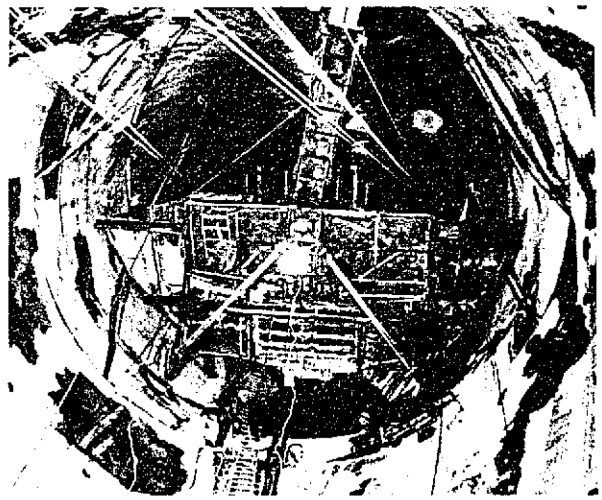


Figure 3-12. View of upper jumbo and nearly completed aeration slot. Dark areas below aeration slot are epoxy-bonded epoxy-mortar repair areas. Photo P459-640-4432 A

constructed jumbos were capable of up and down movement by operation of an electric motor-driven hoist from the jumbo decks.

Access up and down the 55° sloping section of the spillway was accomplished by use of a small rubber-tired cart steered by hand, Figure 3-13. The cart was attached to a 1-inch cable connected to an electric motor-powered single drum hoist located at the intake to the spillway tunnel. Through a system of pulleys, snatch blocks, and a power cable remotely controlled by the cart operator, egress and ingress up and down the slope were effectively accomplished.

Excavation on the aeration slot consisted of removing concrete, rock, and reinforcement steel from an area about 6 feet deep by 10 feet wide for about three-fourths of the tunnel periphery.

Initially, an attempt was made to drill through the No. 9 reinforcing mat by drilling along the perimeter of the aeration slot. An NX diameter core bit and portable air drill mounted on the upper jumbo were employed. Progress of the drilling was slow and impeded the progress of the work, Figure 3-14. Consequently, it was decided to eliminate the NX drilling and commence pattern drilling with a 1-1/2-inch drill bit and jackhammer and blast the concrete and rock in small quantities. The amount of concrete and rock removed at any one time could not exceed 3 cubic yards as this was considered the maximum weight that could be

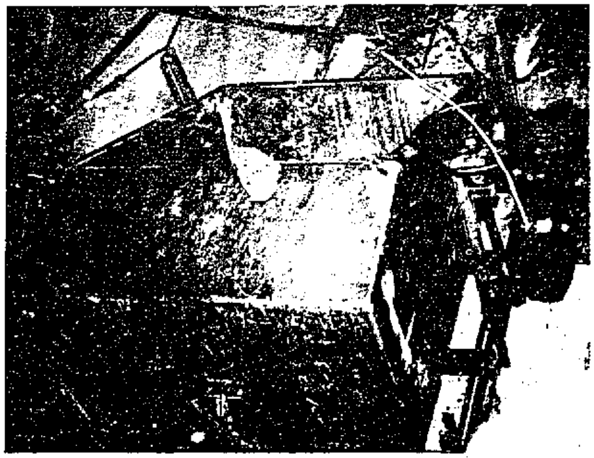


Figure 3-13. Cart used for ingress and egress up and down the 55° slope of the tunnel. The cart was connected by a 1-inch cable to a single drum electric motor-powered hoist controlled by the operator of the cart. Photo P459-D-68568

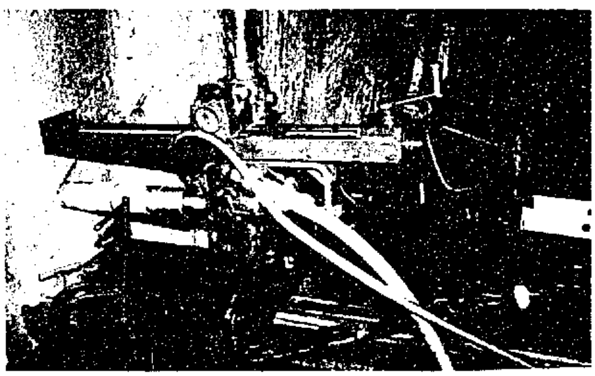


Figure 3-14. Diamond drill working off the upper jumbo and drilling reinforcement steel on periphery of the aeration slot excavation. This tool was later abandoned because it was too slow and impeded progress. Photo P459-D-68569

handled on the jumbo at one time. Excavated material, totaling 245 cubic yards, was loaded in the cart by hand labor, hauled to the upstream portal of the tunnel, and hoisted out by crane to the waiting trucks.

The drilling pattern for each shot in the keyway consisted of three to four rows of 1-1/2-inch holes perpendicular to the surface of the tunnel and 5 feet deep. These holes were spaced in alternating rows of four and three holes spaced on a grid of 12 inches. About three rows of holes were all that were required to loosen about 3 cubic yards of the materials. All loaded holes were decked with small quantities of 45 percent strength stick powder not exceeding a total load of 4 pounds. All blasts were blanketed with several layers of wire mesh to prevent fly rock. No electric caps were permitted because of the transmission of electric power in the area and the danger of stray electrical currents which could prematurely set off electric caps. As a result, mine delays and cap and fuse were used to detonate the charge. This manner of blasting presented a safety problem in that the fuse had to be ignited manually and the powderman then had to exit the area by the cart to the upstream portal. Fuse length was set for 8 minutes. Later ladders were constructed downward from the upper jumbo to permit powder personnel to exit to the downstream portal before the blast occurred. From a safety standpoint this was the more positive way of protecting personnel from injury.

The construction of the forms and the forming and placement of concrete in the aeration slot was an extremely difficult construction process.

Forms were constructed from 2- by 12-inch lumber and 3/4-inch plywood panels. The 2 by 12's were used as studs placed 16 inches apart. The studs were about 14 feet long and spanned the gap over the slot excavation. The plywood panels were used for form sheathing. These panels were cut to developed surfaces to satisfy the compound surface required to shape the slot lips. The entire form was constructed in six segments and prototype faired up on a fairing floor. The prototype was checked separately by contractor and Government personnel, Figures 3-15 and 3-16.

Each section of the form had to be lowered into place by the use of the cart and jumbo. The forms were extremely difficult to set because of the out-of-roundness of the existing tunnel concrete. This required the forms to be forced down with rock bolts at their upstream extremes to fit the tunnel concrete. Even though the forms were distorted at the form extremes, upstream and downstream, where they were bolted to the concrete, dimensional requirements of the upstream and downstream lips of the aeration slot were maintained.

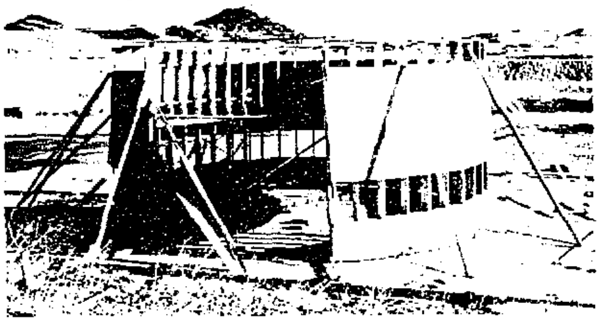


Figure 3-15. Prototype of the aeration slot forms. The assembled form at this stage is ready for disassembly and final installation at the excavated slot area of the lining in the spillway tunnel. Photo P459-D-68570

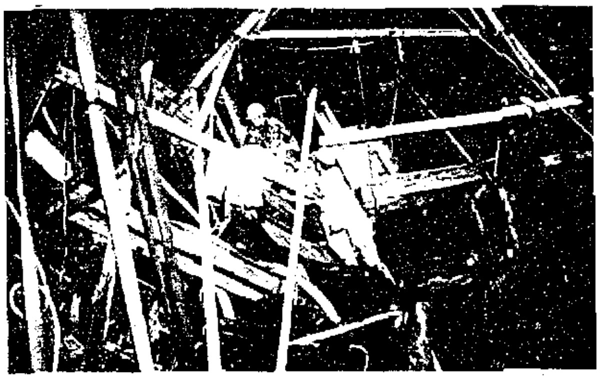


Figure 3-16. Aeration slot forms installed on the 55° sloping section of the spillway tunnel. Workmen are changing location of the concrete pump discharge line in the forms. Access cart can be seen in the photograph. Note unwatering hoses and difficult formwork and bracing. Photo P459-D-68572

A Model I-35 Case-McCarr double piston pump hydraulically operated concrete placer was placed at the entrance of the spillway at approximate elevation 3593, Figure 3-17. The machine was capable of supplying about 30 cubic yards of concrete per hour with a maximum pump pressure of 1,000 psi. Concrete from this machine was pumped through a 6-inch siamese reducer into a 4-inch-diameter pipeline. The 4-inch pipeline extended from this connection down the 55° tunnel slope for a distance of about 400 feet.

The location of the pump above the point of placement was contrary to good concrete placement practices because of the difficulty of pumping concrete downhill. However, pumping concrete from below the aeration slot into the slot forms presented difficult problems of supplying concrete to the pump from the

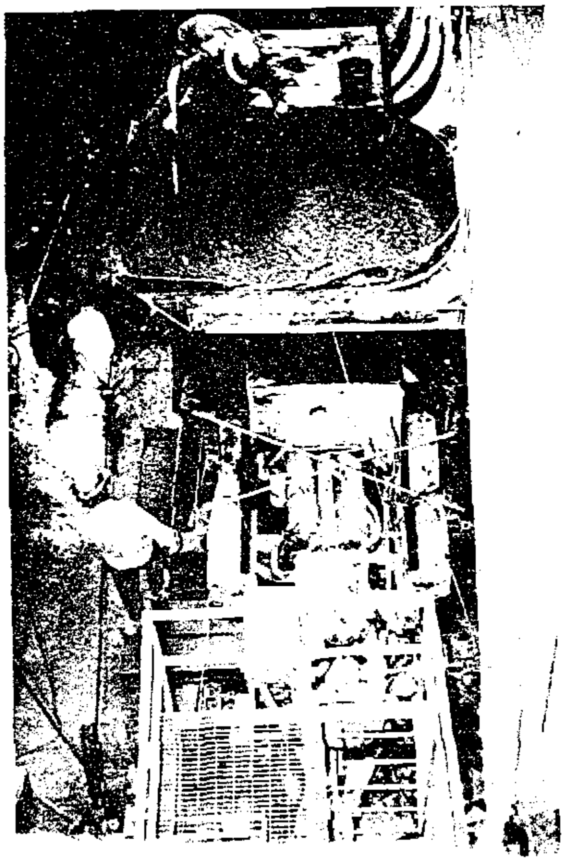


Figure 3-17. Case-MaCarr hydraulically operated concrete pump used to place the concrete in the aeration slot. Photo P459-D-68571

downstream portal. This problem materialized when Areas "C" and "D" were repaired with concrete.

To assure workability of the concrete and in an attempt to overcome anticipated difficulties of pumping concrete downhill, a starting concrete mix of the following proportion and characteristics was used:

Maximum size agregate	= 3/4 inch
Cement content	= 7 sacks per cubic yard
Sand	= 55 percent
3/16- to 3/4-inch rock	= 45 percent
Slump limits	= 5-1/2 to 7-1/4 inches
Average slump	= 6 inches
Water-cement ratio	= 0.46 to 0.53
Air-entraining agent	= 4 to 5 percent
WRA	= 20 ounces

Five truck mixers were used to mix and transport the concrete to the upstream portal of the spillway. At this point the concrete which was discharged into a 1-cubic yard bucket was hoisted by crane to the hopper of the pump below.

For safety reasons, when concrete placement started all other work in the tunnel was stopped because the placement was above the tunnel work in the bend and the near horizontal section of the tunnel.

The concrete placement was plagued with plugged lines, segregation of concrete, and difficult access to the portions of the forms. The placement consisting of 180 cubic yards of concrete took 52 hours of continuous operation to complete. The last several yards of concrete were placed in the forms by use of 5-gallon buckets lowered down the incline in the access cart.

During the concrete placement it became obvious that the concrete could not be pumped downhill through the 4-inch discharge line. Consequently, the sand content was increased to 58 percent and the 3/4-inch rock reduced to 42 percent. The cement content was increased to 7-1/2 sacks of cement per cubic yard of concrete. This improved the flow of the concrete in the line to some extent.

Concrete compressive strengths as determined from 6- by 12-inch concrete test cylinders for 28-day cylinders ranged from 5,900 to 6,000 psi.

After the forms for the slot concrete had been stripped, the general appearance of the work was excellent. There were, however, some areas along the upstream and downstream lips that were repaired, Figure 3-18. These areas were saw cut and the porous concrete excavated below the reinforcement steel and replaced with sound concrete. Minor areas were repaired with epoxy-bonded epoxy mortar. Chemical grout was used to dry up hairline cracks in the slot previous to making the repairs and the coating of the concrete with epoxy-phenolic paint.

Measurements along the arc of the upstream lip of the slot revealed that the lip was low in relation to the existing concrete. This was corrected by bushhammering and grinding on a slope of 1:100 followed by a thin skin of epoxy-bonded epoxy mortar. This treatment was successful in meeting the dimensional requirements.

Airholes or "bugholes" in the concrete, a normal phenomena, at and below about 3 feet below spring line of the tunnel were filled with sack rubbed mortar. The "bugholes" appeared after the specially finished surfaces were sandblasted prior to application of epoxy-phenolic paint. The "bugholes" ranged in size from about one-fourth inch in diameter and less than one-fourth inch deep to pinholes.

Specifications required that the sack rubbed "bugholes" be water cured for 7 days. To water cure

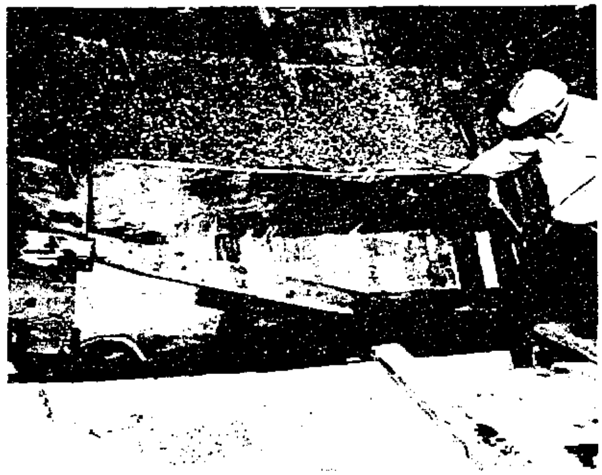


Figure 3-18. Aeration slot at invert showing pressure transducer installation and repairs by grinding of slot lip for grade corrections. Platform of jumbo visible in lower left of photograph. Photo P459-640-4435 NA

the repair would have significantly delayed the epoxy-mortar repair work. In view of the anticipated delay of the work if water curing was employed, no water curing of the "bughole" repair mortar was performed. It was determined that water curing was not necessary because of the high-relative humidity (75 percent) in the tunnel.

The work in repairing the major damaged areas, applying the epoxy-mortar veneer, construction of the aeration slot, and application of the epoxy-phenolic paint was accomplished by working two 10-hour shifts per working day. The work was among the most difficult repair jobs contracted by the Bureau. Figure 3-19 shows the completed tunnel repair.

Figure 3-19. Completed repair work. View looking upstream into bend of tunnel. White slash in center of photograph is the completed aeration slot. Photo P459-D-68573

PART 4

PROTOTYPE TESTS

A. June 1969 Test

Although laboratory tests indicated that the aeration slot was introducing air along the tunnel invert, the amount of air and its tendency to rise rapidly towards the free water surface away from the invert where it was required, were highly speculative questions. The adequacy of the epoxy veneer surface beneficiation in and immediately downstream from the elbow was also questionable. These questions could be resolved only through controlled prototype tests that approximated spillway operation during the summer of 1967.

Plans for a two-phase test were developed early in 1969. Phase 1 was to be a 5-day, 5,000 cfs continuous discharge test, followed by an examination. If no damage was incurred, Phase 2, sustained discharge at 15,000 cfs was to be implemented. Water conditions in the summer of 1969 were such that only Phase 1 could be accomplished satisfactorily.

The spillway was readied for testing the week of June 16. On June 16, the entire tunnel from intake to outlet portal was examined in the dry. The tunnel surfaces were in excellent condition except for calcium carbonate buildup throughout the tunnel and the formation of "blisters" in the epoxy-mortar veneer at the upper end of the elbow. These surface irregularities had occurred since the Government accepted the contractual work as complete on April 28, 1969. Because carbonate buildup had induced cavitation in 1967, and since the efficacy of the air slot was as yet an unknown quantity, calcium carbonate deposits were removed from Station 7+75 to Station 12+15 for a distance of 10 feet each side of the tunnel centerline. Project personnel completed the calcium carbonate removal on June 17. Blisters in the epoxy veneer were not removed since their location and configuration were thought to be less critical to cavitation. Additionally, repairs of blisters would have been time consuming and could have meant postponement of the test.

For electronic monitoring of the tests, two hydrophones were embedded in the downstream face of the aeration slot, and one pressure transducer suspended against the upstream face of the slot. The output of the pressure transducer was monitored and recorded on a direct writing recorder, and the hydrophones were connected to a two-channel oscilloscope for visual readout. The output of the pressure transducer and one hydrophone could be recorded simultaneously on magnetic tape. These electronic devices were capable of indicating any change in the flow characteristics in the aeration slot or in the vertical bend downstream during a test run.

Instrument installation and checkout for monitoring the test were completed the afternoon of June 16. At 8:30 a.m. on June 18, the spillway gates were opened incrementally until the 5,000 cfs discharge was reached about 2-1/2 hours later. Gate openings were adjusted as required during the next 5 days to obtain a constant discharge of 5,000 cfs plus or minus. The oscilloscope and direct-writing recorder were monitored continuously for the first 15 minutes of the test and at 1/2-hour intervals thereafter. Magnetic tape recorder readings were made at 3-hour intervals of a hydrophone signal and the pressure drop in the air slot. The spillway was operated continuously until 9:00 a.m., June 23, when complete shutdown was made.

The tunnel was examined June 23 and 24. The inclined portion of the tunnel, aeration slot, and elbow were examined from an inspection vehicle operated from the tunnel intake. The horizontal tunnel reach was examined from a boat. The following surface conditions and irregularities were observed; none of the irregularities caused cavitation damage to the adjacent flow surfaces:

1. None of the individual epoxy-mortar patches in the tunnel incline failed.

2. Two bonding failures occurred in the epoxy-mortar veneer above the aeration slot. The failures were in the invert and were about 6 inches long by 4 inches wide by 1/4 inch deep.

3. Three small bonding failures occurred in the epoxy-mortar veneer installed to bring the aeration slot lip up to grade. These areas were grouped 1 inch above the lip and 4.5 feet to the right of the tunnel centerline (looking downstream). The largest of the failed areas was 1 inch long by 2 inches wide by 1/8 inch deep. Figure 4-1 shows the lip and failed areas.

4. Blisters that were noted previous to the test had not increased in size nor had they adversely affected the flow surfaces.

5. Calcium carbonate deposits in the elbow had not been removed completely previous to the test.

6. Station 9+05± (elbow).—Two epoxy-mortar veneer failures, one along the invert, 2 inches long by 1 inch wide by 1/16 inch deep; the second about

Figure 4-1. Looking down on upstream side of slot at failed epoxy veneer. Photo C459-D-68458 NA

Figure 4-2. Paint failure in tunnel elbow at Station 9+56±. Photo C459-D-68459 NA.

6 inches long by 3 inches wide by 1/8 inch deep and 24 inches from the tunnel centerline.

7. Station 9+11± (elbow).—A gouge in the epoxy-mortar veneer about 3 inches long by 1 inch wide by 1/8 inch deep along the invert near the tunnel centerline.

8. Station 9+16± (elbow).—A gouge in the epoxy-mortar veneer about 2 inches long by 1/2 inch wide by 1/16 inch deep near the tunnel centerline.

9. Station 9+45± (elbow).—An area about 18 inches square with 20 small pits, the largest of which was 1 inch long by 3/4 inch wide by 1/8 inch deep. The pits were centered around the tunnel centerline.

10. Station 9+50± (elbow).—An epoxy-mortar veneer failure about 6 inches long by 7 inches wide by 1/4 inch deep, located about 9 feet to the right of the tunnel centerline (looking downstream).

11. Station 9+56± (elbow).—Paint failure occurred in two areas. The first area, about 24 inches downstream from a calcium carbonate deposit, was approximately 18 by 18 inches and irregularly shaped. The second area, about 4 feet farther downstream, was 9 inches long by 24 inches wide. Both areas were located to the left of the tunnel centerline, Figure 4-2. There was no damage to the concrete lining itself and it was concluded that paint failure occurred because of poor bonding to the lining surface.

12. Station 9+63± (elbow).—An epoxy-mortar veneer failure 3 inches long by 1 inch wide by 1/4 inch deep, located 2 feet left of the tunnel centerline.

13. Station 9+80± (elbow).—An epoxy veneer failure about 3 inches long by 1 inch wide by 1/4 inch deep, located near the tunnel centerline.

14. Station 9+90± (elbow).—Paint was removed in two areas. The first area was about 4 feet long by 18 inches at its widest, the second area, about 18 inches long, was located 3 feet downstream from the first. Both areas were located left of the tunnel centerline. The tunnel lining adjacent to the paint failures was undamaged.

15. The horizontal reach of the tunnel was examined by boat. Areas of minor epoxy-mortar veneer and paint failures, similar in scope and appearance to those seen in the elbow, had occurred.

Of the total area of epoxy veneer surfaces involved in the test, it was estimated that over 99 percent of these surfaces did not exhibit any failure. Furthermore, minor disbonding that did occur did not produce cavitational damage normally associated with these types of failures. Although highly encouraging, positive conclusions as to the efficacy of the air slot and the surface beneficiation could not be made because of the modest flow involved in the test, and the realization that many of the veneer failures that occurred might have occurred late in the test before cavitation damage could be initiated. The logical conclusion was to schedule an additional test at higher discharges. The recommendation was made to personnel in the region and the Engineering and Research Center, and agreement was reached that another test would be scheduled when water conditions were appropriate.

On June 26, 1969, inflows into the reservoir were such that a 1-day test at a higher discharge was feasible. Accordingly, the test commenced at 2:05 p.m. on June

27. Instrumentation to monitor the test was similar to that developed for the first test. Discharges were restricted to 15,000 cfs for approximately 24 hours with complete shutdown accomplished at 3:30 p.m. the following day. It should be emphasized that this test was conducted with existing surface irregularities that occurred during the 5,000 cfs test.

The tunnel surfaces were examined in the late afternoon of June 28 and the morning of June 29. The following was noted:

1. Existing veneer and paint bonding failures in the elbow that resulted from the 5,000 cfs test were not affected by the 15,000 cfs test.

2. A new epoxy-mortar failure occurred in the elbow about 100 feet downstream from the aeration slot. The area was 10 feet right of the tunnel centerline (looking downstream) and was approximately 9 inches long by 12 inches wide by 1 inch deep.

3. Additional paint and epoxy veneer failures occurred in the horizontal reach of the tunnel (examined by boat) with one large paint failure extending from Station 12+04± to Station 12+15. The failure was centered along the invert and was approximately 18 inches wide, maximum. Epoxy veneer failures appeared small in area and relatively shallow with a maximum depth estimated at 1/4 inch.

The results of the 15,000 cfs test were particularly important in that a discharge, approaching the maximum discharge in the 1967 flood, did not initiate cavitation downstream from existing surface irregularities whose location, frequency of occurrence, area, and depth were more critical than similar irregularities causing cavitation damage in 1967. It was generally concluded that the 15,000 cfs discharge indicated the aeration slot was in fact supplying air to the invert surfaces in sufficient volume to mitigate cavitation damage; however, a sustained 15,000 cfs test was considered as essential before final recommendations could be made relative to future reservoir and spillway operations.

B. Special Repairs after 1969 Test

Following the June 1969 test and subsequent inspection, special repairs were undertaken to correct the minor damage to the epoxy veneer caused by latent blistering. Personnel from the Division of General Research traveled to Yellowtail Dam for this purpose. Their objectives were twofold: first, to make repairs and get the work completed; and second, to train field personnel in the use and handling of a special epoxy. Because of the known adverse conditions of temperature and moisture existing in the tunnel, an investigation to determine the most effective epoxy-type material under these conditions was undertaken in the Engineering and Research Center laboratories. Of six epoxy damp- or wet-curing compounds tested, one proved to be most effective and was selected for this repair work.

(a) Initial inspection.—On August 18, 1969, a detailed inspection of the tunnel was made. The following major observations resulted:

1. Concrete surfaces in the horizontal reach of the tunnel between Stations 10+31 and 12±15 were muddy, and for the most part damp, with flowing water in a few areas. Concrete surfaces in the inclined reach, Station 7+76 to Station 10+31, were generally wet from water entering the area through drainholes. Most of the water flowing in the invert originated from leakage around the spillway gates at the upstream portal.

2. A 3-foot-wide stairway had been installed from the horizontal reach of tunnel nearly to the aeration slot, and brackets were being installed thereon to support portable wing scaffolds which were also under construction, Figure 4-3. This scaffolding system was constructed for permanent use, and it proved to be a very satisfactory arrangement. Other equipment was being installed, such as: additional lighting, dry warm cabinet for housing epoxy materials and sand, etc.

3. Air temperature in the tunnel varied from about 60° F in the morning to about 70° F in the afternoon. Water temperature averaged about 55° F.

4. No attempt had been made to divert the flowing water, although methods for doing so had been thought out and the necessary materials and equipment required had been obtained, such as sections of 8-inch-diameter by 30-foot-long aluminum pipe, Sika Plug quick-setting sealing compound, etc.

5. Essentially all of the required equipment and materials had been procured and were available for use.

(b) Surface preparation.—The damaged areas were cleaned and water was diverted for patching. A

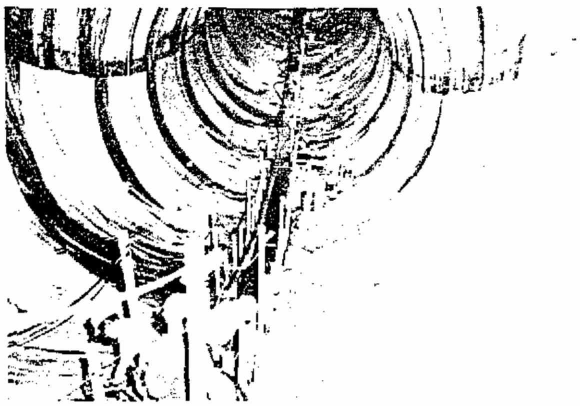


Figure 4-3. Three-foot-wide access stairway built from the horizontal reach of tunnel to aeration slot. Note the movable wing scaffolds use for transverse access. Photo P459-D-64771 NA

putty knife was used to excavate all loose and foreign material from the damaged areas. Blisters were broken and loose material removed. In damaged areas where required, Sika Plug dikes, sandbags, box dikes, etc., were used to divert water, Figure 4-4. Paper towels were also used to dry up patch areas. When all damaged spots within a given accessible area were located, dewatered, and freed of loose material, they were sandblasted with an aspirator-type nozzle blaster. The blasting agent was Del Monte, white sand, EI-16.

(c) Materials.—Principal materials used in these repairs of the Yellowtail spillway tunnel were Sea Go-In Poxy Putty, graded aggregate, and Sika Plug fast-setting sealing compound. Sika Plug, manufactured by Sika Chemical Corporation, Lyndhurst, New Jersey, was used to build dikes to divert the water from the patch areas, Figure 4-4.

Graded sand for making the mortar was preweighed in Denver and shipped to Yellowtail Dam. The graded sand was weighed in 500- and 1,000-gram batches and packaged in plastic bags. This gave a clean, dry, graded sand for use in multiples of 500 grams. The graded aggregate was weighed as shown in the following chart below.

Figure 4-4. Typical applications of Sika Plug, rapid setting sealing compound, and plastic sheathing to divert water ***around repair areas. These photographs show completed*** **Sea Go-In Poxy Putty mortar repairs made in August 1969. Photos P459-D-64767 NA top, and P459-D-64766 NA, bottom**

Sea Go-In Poxy Putty No. 1324, manufactured by Permalite Plastics Corporation, Costa Mesa, *California, was used for the bond coat and mixed* with aggregate to make the mortar. The mix ratio was 2.5 parts of aggregate, by weight, to 1 part epoxy. Equal weights of Parts A and B were preweighed into separate widemouth fruit jars and capped. Only the daily epoxy requirement was preweighed at one time; then it was stored in a heated cabinet in the tunnel until used.

Sand size	Percent retained	Accumulated percent retained	Accumulated weight (gms) (500-gm batch)	Accumulated weight (gms) (1,000-gm batch)
No. 30	31	31	155	310
No. 50	23	54	270	540
No. 100	16	70	350	700
Pan	30	100	500	1,000

Sand Sizes 30, 50, and 100 were from Clear Creek, and the pan was a manufactured material.

1. Bond coat and pure epoxy patches.—The preweighed Sea Go-In Poxy Putty was mixed and applied as a bond coat for the mortar. In the case of small patches (1/2-inch-diameter or less and 1/8 inch or less in depth), straight epoxy was used to make the patch.

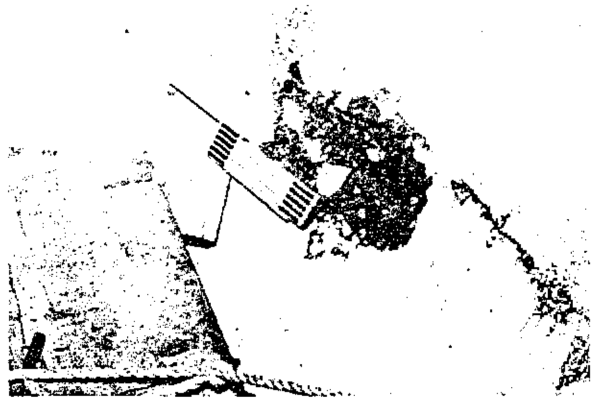
Equal weights of the two epoxy components (100 grams of each, Part A (white) and Part B (green)) were mixed to a uniform putty without streaks of green or white.

After the Poxy Putty bond coat was thoroughly mixed, a putty knife was used to apply it to large patch areas (2 inches in diameter or larger). Then it was scrubbed into the surface with the thumb or finger to force out free moisture and effect a good bond to the patch area. For small mortar patches, a finger was used in place of the aforementioned putty knife. For the straight epoxy patches, a small amount was rubbed in by hand and then additional epoxy was placed and leveled with a putty knife to bring the patch to grade. Pot life of the material was 30 to 45 minutes.

2. Epoxy mortar.—The Sea Go-In Poxy Putty was mixed together as for the bond coat and then mixed with the preweighed, graded sand. Most batches mixed consisted of 500 grams of sand and 200 grams of epoxy. A few larger (1,500 grams of sand and 600 grams of epoxy) batches were also mixed.

After Parts A and B were thoroughly mixed, a plastic bag of preweighed graded sand was poured on a 3- by 3-foot sheet of polyethylene. The epoxy was transferred from the jar onto the sand. Sand and epoxy were mixed together using a kneeding process similar to making bread. After the mortar was thoroughly mixed, it was placed in a 5-quart plastic bucket for handling during the patching operation.

Mortar was placed into the repair area and leveled with a pointed trowel. The mortar was then pounded into the bond coat with a 1-1/2-inch-diameter plastic-headed hammer. Excess mortar was then removed with the trowel and the cycle repeated. Most of the large patches required some grinding for final finishing to grade. Pot life to place the mortar was about 30 minutes. Figures 4-5, 4-6, and 4-7 show typical areas repaired with the Sea Go—In Poxy Putty mortar.



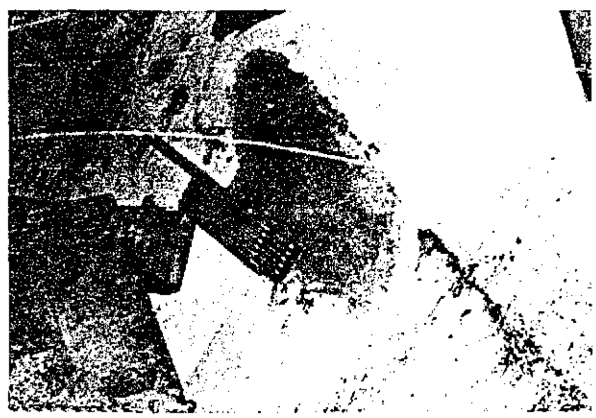


Figure 4-5. This damaged area (upper photograph) located approximately at Station 8+45, 11 feet right of the centerline, was the largest area repaired with the Sea Go-In Poxy Putty in August 1969. Lower photograph shows completed epoxy-mortar repair. Photos P459-D-64764 NA, top, and P459-D-64770 NA, bottom

(d) Curing and finishing.—New patches were placed on old concrete and existing epoxy under several different moisture and substrata conditions. Patching material was placed on surfaces that were dry, damp (dull sheen), and wet (shiny), and a few underwent curing with water running over them. The epoxy and epoxy mortar cured hard in about 3 hours under dry conditions and hard overnight under water. Temperature of the concrete surface averaged 55° F with the daily air temperature rising to about 70° F.

Due to the limited pot life of the mortar, the amount of time spent working the larger patches to an exact grade was restricted, and hence, some of them required grinding as previously mentioned.

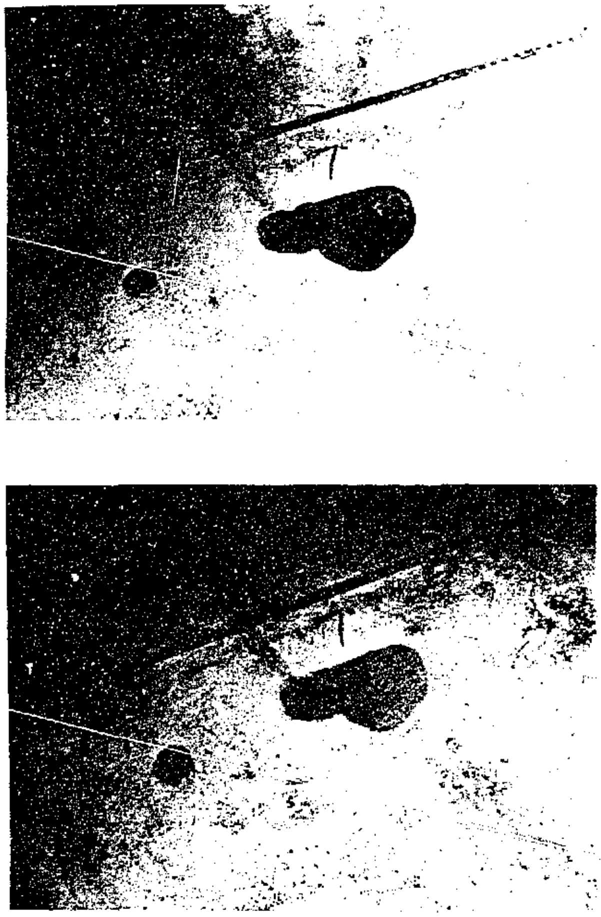


Figure 4-6. Typical areas (top photograph) requiring repairs with the Sea Go-In Poxy Putty in August 1969. In many of the areas, such as this one, the new repair material was placed over previously placed epoxy mortar. Lower photograph shows completed epoxy-mortar repairs. Photos P459-D-64769 NA, top, and P459-D-64768 NA, bottom

(e) Repair evaluation.—Several techniques were used which permitted an evaluation of the repairs a few hours and a few days subsequent to the placement. Also, preliminary work was done which would be of help later when the repair work was scrutinized and evaluated during the pre- and post-spill inspections.

(1) Immediate post placing evaluation.—The earliest test that could be performed on the repair materials aside from visual observations consisted of trying to penetrate the newly placed epoxy and epoxy mortar with a semisharp device, such as a putty spatula, pocket knife, or screwdriver. These tests indicated that the epoxy (mortar in particular) was quite firm (hard) about 3 hours after placement in the tunnel and very hard after 24 hours' curing. The tests also indicated that mortar, which became submerged shortly after placement, firmed up and cured more slowly than those cured in air. Apparently this was principally due to the 55° F water temperature as compared to the 70° F air temperature.

2. Grinding.—Several patches were ground about 24 hours after placement to evaluate the grindability of the hardened mortar. This in itself was an evaluation of the cured mortar. In all cases, grinding confirmed that the patch material was very hard and dense.

3. Impact hammer evaluations.—Impact hammer, Model No. N-15 (red hammer), was used on randomly selected patches to evaluate their quality. At each particular location 14 impact readings were taken randomly around the new patch on the original surface, be it concrete or epoxy mortar. Then 14 impact readings were taken randomly on the new patch. Out of each set of 14 readings, the 2 high and 2 low readings were discarded and the remaining 10 were averaged. Table 4-1 presents impact averages as well as other data pertinent to the evaluation of the readings. In all but a few cases, readings taken on the new patches averaged greater than those taken on the original adjacent material. This is indicative that the quality of the Sea Go-In Poxy Putty mortar patches is equal to or higher than that of the original surface. Since most of the patches checked were less than 1/2 inch deep, the quality of both the material upon which they were placed, and the bond between the two were reflected in the impact readings. This indicates that the new patches were satisfactorily bonded to the substrata.

4. Compressive strength evaluation.—Eight 2-inch-diameter by 4-inch-long cylinders were cast in the tunnel using Sea Go-In Poxy Putty mortar. Sets of these specimens were cured under two tunnel conditions; in air and underwater. Two cylinders having each type of cure were tested at 7 and 14 days' age to determine their compressive strengths, Table 4-2. Also presented in Table 4-2 are some data on laboratory-prepared specimens using similar materials. Several observations are worth noting:

a. Most of the compressive strength gain of this epoxy occurred in the first 2 or 3 days.

b. Mortar cured underwater had a somewhat lower compressive strength than that cured in air.

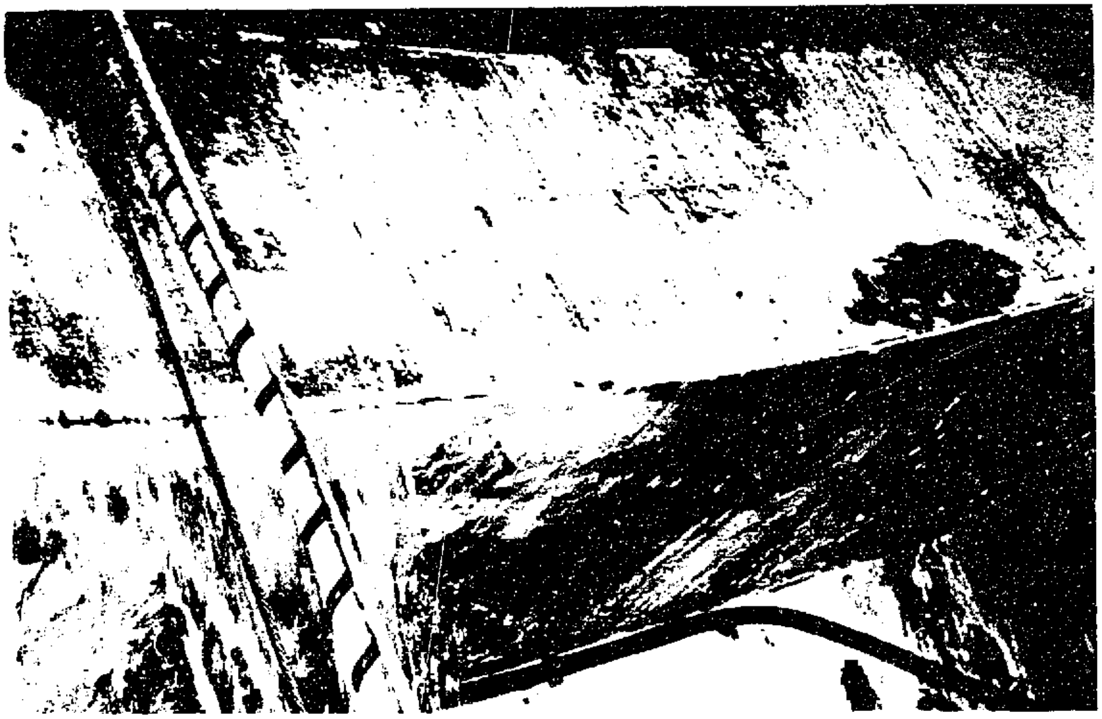


Figure 4-7. April 1970 view of upper edge of aeration slot showing Sea Go-In Poxy Putty mortar repairs made to area about 4 feet right of tunnel centerline 8 months earlier in August 1969. (See Figure 4-1 for view of damaged area before it was repaired.) Photo C-8272-17 NA

c. Field dry-cured mortar developed somewhat less compressive strength than similar mortar laboratory dry cured. One hypothesis for this phenomenon is that the higher tunnel humidity caused a slower curing rate. Another hypothesis is that the lower strength resulted from use of the manufactured sand (pan size) used in the field but not in the laboratory.

d. One-day shear bond strength of the laboratory mixed and cured mortar averaged well above the minimum required; and the 7-day strength was over twice the minimum required. Based on these results and other data, the indication is that high-quality patches have been obtained in the field.

5. Logging of new repairs.—Nearly all new patches, larger than about 1 inch in diameter, were logged in detail for aid in their subsequent evaluations prior and subsequent to the next spill. Patches were identified as to location in tunnel, depth, size, substrata, moisture condition when placed, material new patch placed on, etc., Table 4-3. Considerably more repairs were made than were logged; however, they were for the most part quite small in size and depth. Included on the log sheets is the identification of repaired areas that were blisters.

(f) Summary.—A total of about 425 repairs utilizing a total of approximately 25 pounds of epoxy and epoxy mortar were made between Stations 7+76 and 12+15, Table 4-4. Of that group, 137 were of significant size to warrant logging. Twenty-four logged patches were placed between Stations 7+76 and 7+95, 88 between Stations 7+95 (PC) and 10+31 (PT), and 25 between Stations 10+31 (PT) and 12+15 (lower end of repair area).

Epoxy and epoxy-mortar repairs cured satisfactorily under the existing tunnel conditions thereby requiring no supplemental heating, etc. In most cases, water was not allowed on the newly placed patches until they had cured at least 24 hours.

Patches that were submerged prior to hardening apparently were not adversely affected aside from the slower curing rate.

(g) Subsequent evaluation.—

Table 4-1

IMPACT HAMMER READINGS
AUGUST 1969
Spillway Tunnel Repairs—Yellowtail Dam

Patch No.	Patch age at test (days)	Patch thickness (inches)	Moisture condition at placing	Average impact readings on patches*	Average impact readings on material around patches*	Remarks
3	1	1/8–3/8	Dry	42.3	44.2	
7	1	1/8–3/16	Dry	42.4	32.3	
7	1	1/8–3/16	Dry	42.4	32.3	After grinding
7	5	1/8–3/16	Dry	41.5	32.3	Days later
8	1	1/16–5/8	Wet	33.4	35.7	
9	1	1/8–1/4	Damp	40.6	33.7	
14	1	1/16–1/8	Wet	39.1	36.8	
41	2	1/8	Wet	37.9	28.5	
42	3	1/4–3/8	Damp	47.9	34.8	
50B	2	1/8–3/16	Wet	26.8	24.7	
50B	2	1/8–3/16	Wet	26.3	24.7	After grinding
50C1	3	1/8	Damp	45.1	25.7	
52	2	1/16–1-1/4	Damp	44.7	42.4	Largest patch made
59	2	1/8–3/8	Wet	35.8	25.9	
71	1	1/8	Wet	42.0	33.2	
75	1	1/8	Wet	37.7	35.3	
76	1	1/16–1/8	Wet	32.9	35.3	
Average				39.2	33.5	

*Fourteen readings taken at each location. High 2 and low 2 were discarded and remaining 10 readings were averaged.

1. Inspection.—During an inspection by a design engineer of other repairs on the afterbay dam, a general inspection of the spillway tunnel repair was made. One defective patch (Sea Go-In Poxy Putty material placed in August 1969), was found and several others looked questionable but were inaccessible. A subsequent meeting was held at the Engineering and Research Center to discuss these findings. The consensus from that meeting was that a detailed inspection and evaluation of the Sea Go-In Poxy Putty patches placed in the tunnel was warranted and should be made as soon as possible. Since additional work in the tunnel was not going to be done until spring 1970, an evaluation at this time would allow sufficient time to study the test data and develop some new application procedures if necessary.

A research engineer traveled to the dam for further examination. Access to the repair area in the tunnel was by motorboat. Due to the existing power requirements and a sandbar downstream of the spillway stilling basin, the water level could not be lowered below about elevation 3185. All Sea Go-In Poxy Putty patches between Stations 9+60 and 12+15 were inundated and inaccessible for testing. However, nearly all of the patches between Stations 7+76 and 9+60 that were previously logged were evaluated during this inspection.

2. Evaluation techniques.—The Sea Go-In Poxy Putty patches were evaluated visually as to overall appearance, color, wet or dry condition, presence of carbonates, etc., Table 4-5. The apparent color of the material varied somewhat due to surface moisture condition, lighting, roughness, stains, and material, whether epoxy bond coat or epoxy mortar. The presence of a little pinnacle of carbonate over one edge of a patch usually indicated a water leak through the epoxy.

Table 4-2

SEA GO-IN POXY PUTTY MORTAR STRENGTHS
Compressive and Shear Bond
Spillway Tunnel Repairs—Yellowtail Dam

Compressive Strength		
Field Mix (2- by 4-inch cylinder specimens)		
1. Dry cure at 55° to 60° F:		
	7 days' age—	8,300 psi
	14 days' age—	8,640 psi
2. Underwater cure at 55° to 70° F:		
(55° F for 4 days)	7 days' age—	5,800 psi
(70° F for 10 days)	14 days' age—	4,880 psi
Laboratory mix (2- by 4-inch cylinder specimens)		
1. Dry cure at 45° to 50° F:		
	2 days' age—	10,300 psi
	7 days' age—	11,000 psi
Shear Bond Strength		
Field mix—No tests		
Laboratory mix (2- by 4-inch cylinder specimens)		
1. Damp to wet surface—cured at 45° to 50° F:		
	1 day's age—	482 psi
	2 days' age—	449 psi
	7 days' age—	859 psi

Penetrometer readings were taken on randomly selected patches. The penetrometer instrument is constructed to indicate a maximum penetration resistance of 7,000 psi. None of the patches checked showed the slightest penetration at the maximum load. Some of the pure epoxy patches showed a very slight indentation but no penetration. These tests indicate that the epoxy material in the patches was not soft or defective.

Gouging or picking checks with the sharp point of a knife blade were made on every accessible patch as well as on the original material adjacent to the patches. Prior to initiating these tests, a code was established as to patch condition, i.e., "A" meant patch was hard, "B" meant patch was slightly soft, and "C" meant patch was very soft. However, except for two cases, all patches were rated A or Hard, Table 4-5. Little or no difference could be detected between patches or between the Sea Go—In Poxy Putty patches and the original material adjacent to the patches.

Impact hammer readings were obtained as before during the actual repair program. In most cases, readings taken on the patches averaged higher than those taken on the surrounding material, Table 4-6. These high readings indicated that the patches were still sound and serviceable. Some of the patches tested during this inspection were ones also tested in August 1969. Readings obtained during this evaluation corresponded very closely with those obtained in August 1969. This indicated that the patches had not undergone a general deterioration to moisture or any other cause. The magnitude of these readings would probably indicate compressive strengths in the 3,000- to 6,000-psi range.

3. Conclusions.—The Sea Go-In Poxy Putty patches made in August 1969 were, on the basis of this evaluation, considered generally as good as or better than the surrounding repairs. Some of the patches need to be ground or reground to bring them closer to the proper grade. It is apparent from the few defective patches that water can be detrimental to this material if it is allowed to penetrate the patch prior to its being cured.

Patches placed above the aeration slot seem to be of slightly lower quality than these placed below the slot. However, this difference is evident only in the impact hammer test results.

Patches both above and below the slot that were chipped out for samples indicated that a satisfactory bond existed between the new epoxy and the old base material. This chipping also indicated that the Sea Go-In Poxy Putty mortar was not as brittle hard as some materials used, but it is very tough. It was very resistant to removal by a cold chisel.

A timber ladder that was used in making the aforementioned repairs was permitted to remain in the tunnel during the winter. Early in 1970,

Table 4-3

LOG SHEETS*
August 1969
Spillway Tunnel Repairs—Yellowtail Dam

Patch No.	Station**	Location from ℄ Distance (feet)	Direction	Date placed	Size (inches)	Depth (inches)	Moisture condition	Placed on	Remarks
1	12+00.0	4.0	L	8-20-69	1.5 x 1.5	3/16	D	C	
2	11+92.0	1.0	R	8-22-69	0.5 x 1.0	1/8	W	C	Cured under water
3	11+91.0	6.0	R	8-20-69	3.0 x 5.0	1/8–3/8	D	C	
4	11+81.0	2.0	R	8-21-69	3.0 x 2.5	1/8	W	C	
5	11+69.4	6.0	R	8-20-69	1.5 x 1.5	1/16–1/8	D	E	
6	11+68.6	6.0	R	8-20-69	1.5 x 1.0	1/8	D	E	Patch high for grinding test
7	11+68	7.0	R	8-20-69	5.0 x 10.0	1/8–3/16	D	E	Patch high for grinding test
8	11+42.3	8.8	L	8-20-69	3.0 x 7.0	1/16–5/8	W	C	Flowing water
9	11+35.3	7.5	R	8-20-69	5.0 x 9.0	1/8–1/4	Dp-W	E	
10	11+21.7	1.9	R&L	8-21-69	0.5 diameter	1/16	Dp-W	E	Several spots
10A	11+23.4	10.7	L	8-21-69	3 diameter	1/8	Dp	E	
11	11+05.6	5.7	L	8-20-69	4.0 x 7.0	1/8	D-Dp	E	Several small spots
11A	10+93.2	12.4	R	8-22-69	1.0 x 1.5	1/8	D	E	
12	10+85.2	6.0	L	8-20-69	1.0 x 6.0	1/16	Dp-D	E	
13	10+81.5	8.4	L	8-22-69	3.0 x 4.0	1/4–1/8	Dp-W	E	
14	10+80.3	5.8	R	8-20-69	2.5 diameter	1/16–1/8	W	E	
14A	10+77.9	18.9	L	8-22-69	2.5 x 4.5	1/8	D	E	
15	10+76.7	4.8	R	8-20-69	3.0 x 5.0	1/8	Dp	E	
16	10+64.5	7.5	R	8-20-69	1.5 x 4.0	1/32–1/16	D	E	
16A	10+61.4	8.5	R	8-21-69	3 diameter	1/16–1/4	Dp	E	Blister area
17	10+60.0	1.0	L	8-20-69	1.5 x 3.0	1/16–1/8	W	E	
18	10+51.2	5.7	L	8-20-69	1.5 x 1.5	1/16–1/4	D	C	
19	10+50.8	5.7	L	8-21-69	1.5 x 5.0	1/16–3/16	W	E	Blister area
19A	10+35.2	7.7	L	8-21-69	1.0 diameter	1/16–3/16	Dp-D	E	Blister area
19B	10+30.8	7.7	L	8-21-69	1.0 diameter	1/16–3/16	Dp-D	E	Blister area
19C	10+29.8	7.7	L	8-21-69	1.0 diameter	1/16–3/16	Dp-D	E	Blister area
20	10+32.2	8.0	R	8-20-69	1.0 diameter	1/8	W	E	
21	10+26.0	8.8	R	8-20-69	2.0 x 4.0	1/8	D	E	
22	10+19.5	9.8	L	8-21-69	0.5 diameter	1/8–3/16	D-Dp	E	Blister area
23A	10+16.6	4.7	L	8-21-69	1.0 diameter	1/16	D	E	
23B	10+16.6	4.9	L	8-21-69	1.0 diameter	3/4	D	E	
23C	10+16.6	5.7	L	8-21-69	1.0 diameter	1/2	D	E	
23D	10+10.0	2.3	L	8-22-69	1.0 x 2.0	1/16–1/8	Dp	E	Blister area
23E	10+12.9	10.1	R	8-21-69	1.5 x 2.0	1/8–3/16	Dp-D	E	Blister area
23F	9+92.9	8.0	L	8-21-69	1.0 diameter	1/8–3/16	D-Dp	E	Blister area
24	9+81.0	6.4	L	8-20-69	1.0 diameter	1/8–1/4	D	C	Several small spots
24A	9+77.5	12.4	L	8-22-69	0.5 x 2.0	1/8–3/8	Dp-D	C	
25	9+84.0	5.0	R	8-20-69	1.5 x 5.0	1/8	D	E	
26	9+79.5	9.0	R	8-20-69	1.0 x 2.5	1/8	D	E	

Table 4-3—Continued

Patch No.	Station**	Location from ℄ Distance (feet)	Direction	Date placed	Size (inches)	Depth (inches)	Moisture condition	Placed on	Remarks
27	9+79.0	9.9	R	8-20-69	2.5 x 3.0	1/8	D	E	
28	9+72.9	9.8	R	8-21-69	4.0 x 7.0	1/8–5/16	W	E	Blister area
29	9+70.0	9.8	R	8-20-69	2.5 x 9.0	1/8	W	E	
29A	9+61.8	12.2	L	8-22-69	1.5 x 2.0	1/16–3/8	D	E	Blister area
30	9+60.9	10.5	R	8-21-69	2.5 diameter	1/8–1/4	Dp	E	Blister area
31	9+60.4	9.5	R	8-20-69	1.5 x 8.0	1/8–1/4	Dp	E	
32	9+58.2	1.8	L	8-20-69	2.0 x 4.0	1/8	Dp	C	
33	9+51.4	10.0	R	8-21-69	4.0 x 6.0	1/8–1/2	Dp-D	C	
34	9+41.7	12.0	R	8-22-69	3.0 diameter	1/8	Dp-W	E	
35	9+42.4	10.0	L	8-20-69	1.5 x 2.0	1/8–1/4	D	E	
36	9+35.9	1.5	R		1.0 diameter			C	Rebar anchor
37	9+35.9	1.5	L		1.0 diameter			C	Rebar anchor
38	9+22.5	0.5	R	8-23-69	1.0 x 2.0	1/16–1/8	Dp	C	
39	9+22.5	12.2	L	8-22-69	1.5 x 3.0	1/8–1/4	Dp-D	C	
39A	9+13.0	13.0	L	8-22-69	1.5 diameter	1/8–3/16	Dp-D	E	Blister area
40	9+10.4	0.3	R	8-22-69	3.0 x 4.0	1/8	W	E	
41	9+09.7	1.7	L	8-23-69	5.0 x 8.0	1/8	W-Dp	E	
41A	9+05.3	7.2	R	8-22-69	1.0 x 1.0	1/16	Dp-D	C	
41B	9+04.0	0.0	-	8-23-69	1.0 diameter	1/16–3/16	W	E	
42	9+01.9	6.0	L	8-22-69	5.0 x 6.0	1/4–3/8	Dp-W	E	Blister area
42A	9+01.3	2.0	R	8-22-69	1.0 x 1.5	1/16–1/4	Dp	C	
43A	9+00.0	2.3	L	8-22-69	1.5 x 2.5	1/8–1/4	Dp	C	
43B	9+00.0	2.3	L	8-22-69	1.5 x 1.5	1/8–1/4	Dp	C	
44	8+95.1	7.0	R	8-23-69	1.0 diameter	1/4	Dp-D	E	Blister area
44A	8+92.0	9.4	R	8-23-69	1.5 x 2.0	1/4–3/8	Dp	C	Blister area
45	8+91.5	6.0	R	8-23-69	0.2 to 1.0	1/8	Dp-D	C	Many small blisters
46	8+89.7	9.4	R	8-23-69	2.0 diameter	1/8–3/16	Dp-D	C	
47	8+86.3	0.5	L	8-23-69	1.0 diameter	1/16–1/8	W	–	
48	8+89.0	16.8	L	8-23-69	2.5 diameter	1/4	Dp	E	Blister area
49	8+83.5	9.8	R	8-23-69	1.5 diameter	1/8	Dp	C	Blister area
50	8+76.3	10.4	R	8-25-69	1.5 diameter	1/8	Dp-W	E	Blister area
50A	8+72.6	9.2	L	8-23-69	1.0 diameter	1/16–1/8	Dp-D	E	Blister area
50B	8+71.6	9.6	L	8-23-69	4.0 x 6.0	1/8–3/16	W	E	Blister area
50C	8+64.5	10.3	R	8-25-69	3.0 x 8.0	1/8–1/4	D	E	Blister area
50C1	8+64.7	8.3	L	8-25-69	6.5 x 8.0	1/8	Dp	E	Blister area
50D	8+61.4	10.5	L	8-25-69	2.0 x 6.0	1/16–1/2	D	E	Blister area
51	8+59.5	8.5	R	8-25-69	1.5 diameter	1/8	Dp-W	C	Blister area
51A	8+47.7	4.5	L	8-25-69	2.0 diameter	1/8	W	C	Blister area
51B	8+47.0	2.0	R	8-25-69	1.0 diameter	3/16	Dp	C	Blister area
52	8+45.9	11.6	R	8-26-69	7.0 x 14.0	1/16–1-1/4	Dp	C	Largest repair
53	8+34.8	9.3	R	8-26-69	1.5 x 6.0	1/8–3/16	D-Dp	E	
53A	8+30.7	8.3	R	8-26-69	1.5 x 2.0	1/8	Dp-W	E	Blister area
54	8+27.8	0.0	–	8-26-69	2.0 x 3.0	1/32–1/4	W	C	
55	8+25.0	2.3	L	8-26-69	1.5 x 4.0	1/8–1/4	W	E	
56	8+24.0	0.0	–	8-27-69	2.0 diameter	1/2	W	E	
56A	8+25.0	9.0	L	8-26-69	2.0 x 2.5	1/8–1/4	W	E	Blister area
57	8+25.5	9.2	R	8-27-69	1.0 x 1.0	1/16–1/8	W	E	

Table 4-3—Continued

Patch No.	Station**	Location from ℄		Date placed	Size (inches)	Depth (inches)	Moisture condition	Placed on	Remarks
		Distance (feet)	Direction						
57A	8+23.5	8.0	L	8-27-69	1.0 diameter	1/8	Dp-W	E	Blister area
57B	8+25.0	9.6	R	8-27-69	1.0 x 2.0	1/16	Dp	E	Blister area
57C	8+22.5	11.0	L	8-27-69	0.75 x 2.5	1/8	W-Dp	E	Blister area
57D	8+18.3	3.3	L	8-27-69	1.0 x 2.0	1/8–1/4	W	E	Blister area
57E	8+18.0	2.2	L	8-27-69	0.75 x 1.5	1/4	W	E	Blister area
57F	8+23.7	15.8	L	8-27-69	1.0 x 1.0	1/16–1/4	W	C	Blister area
58	8+16.5	0.5	L	8-27-69	6.0 x 7.0	1/3–1/4	W	E	
58A	8+19.7	10.7	L	8-27-69	0.75 x 1.25	1/8	W	E	Blister area
58B	8+10.0	8.7	L	8-27-69	3.0 x 4.0	1/16–1/8	W	E	Blister area
58C	8+09.0	8.7	L	8-27-69	3.0 diameter	1/16–1/8	W	E	Blister area
59	8+06.4	4.0	R	8-27-69	4.5 diameter	1/8–3/8	W	E	Blister area
59A	8+02.5	3.4	L	8-27-69	1.5 diameter	1/8	W	E	Blister area
59B	8+06.0	8.6	L	8-27-69	1.0 diameter	1/16–1/8	W	E	Blister area
59C	8+06.0	11.0	L	8-27-69	1.0 diameter	1/16	W	E	Blister area
60	8+04.1	5.7	R	8-27-69	1.0 x 2.0	1/8	W	E	Group of 4-6 patches
60A	8+03.9	5.3	R	8-27-69	2.0 diameter	1/8	W	E	Blister area
61	8+03.0	6.0	R	8-27-69	1.5 to 3.5 diameter	1/8	W	E	Three blister areas
61A	8+01.3	7.9	R	8-27-69	3.0 x 4.0	1/4	W	E	Blister area
61B	8+01.1	6.8	R	8-27-69	2.5 diameter	3/16	W	E	Blister area
62	8+04.5	1.2	L	8-28-69	1.0 x 6.0	1/2	W	—	
63	8+01.5	4.1	L	8-28-69	2.0 diameter	1/16–1/8	W	E	Blister area
64	8+02.0	9.0	L	8-28-69	2.0 diameter	1/8	W	E	Blister area
65	8+01.9	9.0	L	8-28-69	3.0 x 4.0	1/8–3/8	W	E	Blister area
66	8+01.4	10.0	L	8-28-69	1.5 x 2.0	1/16–1/4	W	E	Blister area
67	8+01.0	10.3	L	8-28-69	1.2 x 2.0	1/16–1/2	W	C	Blister area
68	7+92.5	7.0	R	8-28-69	1.5 x 2.0	1/8	W	E&C	Blister area
69	7+92.3	6.4	R	8-28-69	2.5 diameter	1/8	W	E	Blister area
70	7+92.2	7.0	R	8-28-69	1.5 diameter	1/16–1/8	W	E	Blister area
71	7+90.3	5.6	L	8-28-69	2.5 x 7.0	1/8	W	C	Blister area
72	7+90.3	7.5	L	8-28-69	2.0 x 4.0	1/8	W	C	Blister area
73	7+86.3	2.1	R	8-28-69	1.5 diameter	1/16–1/8	W	E	Blister area
74	7+80.1	4.0	L	8-28-69	1.5 x 2.0	1/16–1/8	W-Dp	C	Blister area
75	7+80.0	3.3	L	8-28-69	4.0 x 5.0	1/8	W-Dp	C	Blister area
76	7+79.7	4.0	L	8-28-69	3.0 x 6.0	1/16–1/8	W-Dp	C	Blister area
77	7+78.0	4.2	R	8-29-69	1.5 diameter	1/16–1/8	W	C	
78	7+76.1	0.0	-	8-29-69	1.0 diameter	1/8	W	C	Blister area
79	7+74.1	1.0	R	8-29-69	4.0 x 7.0	3/8	W	E&C	Blister area
80	7+72.3	4.7	R	8-29-69	3.0 diameter	1/4	W	E	Blister area
81	7+72.0	4.7	R	8-29-69	3.0 diameter	1/4	W	E	Blister area
82	7+70.4	2.3	R	8-29-69	5.0 x 9.0	3/8	W	E	Blister area
83	7+75.0	3.5	L	8-29-69	1.0 diameter	1/4	W	E	Blister area
84	7+74.5	3.5	L	8-29-69	1.5 diameter	1/4	W	E	Blister area
85	7+74.5	7.5	L	8-29-69	1.0 diameter	3/16	W	E	Blister area
86	7+70.9	1.0	L	8-29-69	1.5 diameter	3/16	W	E	Blister area
87	7+70.9	1.5	L	8-29-69	3.0 diameter	1/4	W	E	Blister area
88	7+70.4	6.5	L	8-29-69	2.5 x 5.0	3/8	W	E	Blister area

Table 4-3—Continued

Patch No.	Station**	Location from ℄ Distance (feet)	Location from ℄ Direction	Date placed	Size (inches)	Depth (inches)	Moisture condition	Placed on	Remarks
89	7+70.4	6.5	L	8-24-69	1.5 diameter	3/16	W	E	Blister area
90	7+70.9	7.5	R	8-29-69	4.5 diameter	3/8	W	E	Blister area
91	7+76.4	8.5	R	8-29-69	1.5 diameter	3/16	W	E	Blister area

*Explanatory notes:

L—Left of tunnel ℄ as looking downstream
R—Right of tunnel ℄ as looking downstream
D—Area dry when patch placed
Dp—Area damp when patch placed
W—Area wet when patch placed
Dp-W—Area somewhat wetter than normal damp condition
Dp-D—Area somewhat dryer than normal damp condition
C—Patch placed on concrete
E—Patch placed on old epoxy mortar
Patching material—Sea Go-In Poxy Putty with and without filler
Tunnel air temperature—60° to 70° F
Tunnel water temperature—55° F
Distance—This is the distance from ℄ of tunnel to patch along arc rather than chord

**Stations are approximate.

the ladder failed due to ice load buildup, and skidded along the elbow invert. The resulting surface irregularities are tabulated in Table 4-7.

C. July 1970 Test

Inflows into the reservoir in late June of 1970 were of sufficient magnitude and duration to permit scheduling a sustained 15,000 cfs discharge test. Surface irregularities that occurred due to ladder failure had not been repaired. Since the irregularities were comparatively minor and because a better evaluation of aeration slot efficiency could be made with known surface deficiencies located in critical areas of the elbow, it was decided to test the tunnel without repairing the lining irregularities.

Testing was scheduled to commence at 7:00 a.m., July 2. However, at 7:00 p.m., July 1, the operation of the river outlets at 5,000 cfs discharge had started to undermine the powerplant access road. Continued operation of the outlets would have washed out enough of the road to prevent vehicular access to the powerplant. Since tunnel operation was imminent in any event, it was decided to exchange the outlet discharge to the spillway tunnel. The exchange was completed at 9:25 p.m., and the spillway operated at 5,000 cfs until the morning of July 2, 1970. As in the 1969 test, tunnel operation was monitored by observing and recording the responses from the hydrophones and pressure transducer.

At 7:00 a.m., July 2, the spillway discharge was increased incrementally until a discharge of 14,000 cfs was reached at 8:53 a.m. Flipping action of the discharge from the basin into the river appeared somewhat unstable at 14,000 cfs; the discharge was dropped to 10,000 cfs, then increased incrementally to 16,000 cfs in an effort to improve flipping characteristics. No discernible improvement could be seen. Rather than risk any damage to the lining, it was decided to terminate operation and make a quick inspection of the tunnel. The inspection indicated no damage whatsoever. The spillway gates were opened at 2:41 p.m., and the test flow of 14,000 cfs was reached at 3:40 p.m. The 14,000 cfs flow was maintained continuously from 3:40 p.m., July 2, until complete spillway shutdown at 2:30 p.m., July 6. A short time after shutdown, Corps of Engineers personnel requested that Yellowtail Dam maintain a release of

Table 4-4

SUMMARY SHEET
Spillway Tunnel Repairs—Yellowtail Dam

Day	Total patches made	Total mortar patches	Total logged patches made	Locations (approximate) (stations)	Total blisters repaired	Patches placed on: Concrete	Patches placed on: Epoxy	Moisture condition: Dry	Moisture condition: Damp	Moisture condition: Wet
8-20 Wednesday	46	40 (est)	25	10+31 to 12+15	0	6	19	14	6	5
8-21 Thursday	25	20 (est)	17	10+31 to 12+15	11	1	16	5	8	4
8-22 Friday	35	30 (est)	17	10+31 to 12+15	7	6	8	2	10	2
8-23 Saturday	52	40	12	8+70 to 10+31	7	5	7	0	8	4
8-25 Monday	92	52	7	8+35 to 8+70	7	3	4	2	4	1
8-26 Tuesday	64	30	7	8+20 to 8+36	2	2	5	2	2	3
8-27 Wednesday	59	39	23	8+00 to 8+20	19	1	22	0	2	21
8-28 Thursday	34	22	15	7+79 to 8+00	14	5	10	0	0	15
8-29 Friday	18	17	17	Above slot	14	3	14	0	0	17
TOTALS	425	290	137		81	32	105	25	40	72

Notes:

1. Logged patches were usually at least 1 inch in diameter and 1/16 inch or greater deep.
2. Figures listed under "Moisture condition" refer to logged patches only.
3. Figures shown under "Total blisters repaired" do not necessarily reflect all blisters located, only those logged.
4. Repairs made August 20 through August 29, 1969.

12,000 cfs for a 24-hour period. Since repairs to the powerplant access road adjacent to the outlet works stilling basin had not been completed, the spillway was again activated for a release of 4,000 cfs. The 4,000 cfs release was maintained from 2:56 p.m., July 6, until 8:15 a.m., July 7, when final shutdown was accomplished.

The tunnel was examined on July 8, with the following results:

(a) Surface irregularities existing in the tunnel lining prior to testing neither initiated cavitation nor did the irregularities become larger.

(b) Calcium carbonate buildup on tunnel surfaces did not initiate cavitation.

(c) Four new areas of minor epoxy disbondments in the elbow resulted from the test:

(1) Station 9+52, 18 inches right of centerline—An area 3 by 3 inches by 1/4 inch deep.

(2) Station 9+00, 4 feet right of centerline—An area 3 inches by 1/2 inch by 1/4 inch deep.

(3) Station 8+50, 4 feet right of centerline—An area 3 inches by 1/2 inch by 1/4 inch deep.

(4) 50 feet downstream from aeration slot, 5 feet right of centerline—An area 4 by 4 inches by 1/8 inch deep.

(d) An area of minor disbondment in the epoxy veneer occurred at the lip of the aeration slot. The area is located 4.5 feet right of the tunnel centerline and is approximately 3 inches (transverse direction) by 1/2 inch by 1/8 inch deep. This area appeared to be a disbondment of the same area reported by personnel from the Engineering and Research Center during an April 1970 visit to the project. At that time the estimate of the deteriorated area was 1 by 1/2 inch by 1/8 inch deep.

(e) The repair work performed August 17 through August 30, 1969, remained in good condition except for: (1) Station 8+72, where a

Table 4-5

SUMMARY OF EVALUATION OF SPECIAL EPOXY PATCHES*
Spillway Tunnel Repairs—Yellowtail Dam

Patch No.	Station	Penetrometer reading	Relative hardness	Moisture condition	General appearance	Remarks
30	9+60.9	7+	A	Wet	LG, GRD, CAR	Air temperature 50° F.
31	9+60.4	7+	A	Damp	LG, GRD	
32	9+58.2	—	A	Wet	R	
33	9+51.4	7+	A	Wet	LG, GRD	
34	9+41.7	7+	A	Dry	LG, GRD	
35	9+42.4	7+	A	Dry	LG, GRD	
38	9+22.5	7+	A	Wet	LG, R	
39	9+22.5	7+	A	Dry	LG, S	
39A	9+13.0	7+	A	Dry	LG, S	
40	9+10.4	7+	A	Wet	LG, R	
41	9+09.7	7+	A	Wet	LG, S	
41A	9+05.3	7+	A	Damp	R	
41B	—	—	—	—	—	Could not identify.
42	9+01.9	7+	A	Wet	LG, GRD, S	
42A	9+01.3	—	A	Wet	LG, R	
44	8+95.1	7+	A	Wet	LG, S	
44A	8+92.0	—	A	Damp	LG, S	
45	8+91.5	7+	A	Wet	LG, S, SY	
46	8+89.7	7+	A	Dry	LG, R, SY	
47	8+86.3	7+	A	Wet	LG, R	
48	8+89.0	7+	A	Wet	LG, R	
49	8+83.5	7+	A	Dry	LG, R	
50	8+76.3	7+	A	Dry	LG, S, SY	Bond coat looks yellowish.
50A	8+72.6	7+	A	Wet	LG, S	
50B	8+71.6	7+	A	Dry	LG, GRD, S	Three plastic weep holes. Colors due to some mortar and other bond coat. Impact hammer caused slight indent in bond coat material.
50C	8+64.5	7+	A	Wet	LG, S	
51C1	8+64.7	7+	A	Wet	LG, S, SY	Border has yellowish tint.
50D	8+61.4	7+	A	Wet	LG, S, SY	
51	B+59.5	7+	A	Wet	LG, S	
51A	8+47.7	7+	A	Wet	LG, S, SY	

Table 4-5—Continued

Patch No.	Station	Penetrometer reading	Relative hardness	Moisture condition	General appearance	Remarks
51B	8+47.0	–	A	Dry	LG, S	
52	8+45.9	7+	A	Wet	LG, S	
53	8+34.8	7+	A	Dry	LG, GRD, R	
53A	8+30.7	7+	A	Dry	LG, S	
54	8+27.8	7+	A	Wet	LG, S	
55	8+25.0	–	A	Wet	LG, GRD, S	
56	8+24.0	7+	A	Wet	LG, S	
56A	8+25.0	7+	A	Wet	LG, GRD, S	
57	8+25.5	7+	A	Wet	LG, S	
57A	–	–	–	–	–	No evaluation.
57B	8+25.5	–	A	Damp	LG, S	
57C	8+22.5	7+	A	Wet	LG, GRD, S	
57D	8+18.3	–	A	Wet	LG, GRD, S	
57E	8+18.0	–	A	Wet	LG, GRD, S	
57F	8+23.7	–	A	Wet	LG, GRD, S	
58	8+16.5	7+	A	Wet	LG, S	
58A	8+19.7	–	A	Wet	LG, GRD, S, SY	
58B	8+10.0	7+	A	Wet	LG, GRD, S	
58C	8+09.0	7+	A	Wet	LG, GRD, S, CAR	
59	8+06.4	7+	A	Wet	LG, GRD, S	
59A	8+02.5	–	A	Wet	LG, S	
59B	8+06.6	–	–	–	–	No evaluation.
60	8+04.1	7+	A	Wet	LG, S	Carbonate from above.
60A	8+03.9	7+	A	Wet	LG, S	
61	8+03.0	–	A	Wet	LG, R	
61A	8+01.3	–	A	Wet	LG, S	Carbonate from above.
61B	8+01.1	–	A	Wet	LG, S	
62	8+04.5	7+	A	Wet	R, CAR	
63	8+01.5	7+	A	Wet	LG, S	
64	8+02.0	–	A	Wet	LG, GRD, S	Carbonate from one corner of patch. Patch removed for sample.
65	8+01.9	7+	A	Wet	LG, GRD, S	
66	8+01.4	7+	A	Wet	LG, GRD, S	Carbonate from one corner of patch. "C" hardness in that small area.
67	8+01.0	7+	A	Wet	LG, GRD, S	
68	7+92.5	–	A	Wet	LG, GRD, S	Sizable quantity of water from over-head drain falling Patches No. 68, 69, and 70.
69	7+92.3	–	A	Wet	LG, GRD, S	

Table 4-5—Continued

Patch No.	Station	Penetrometer reading	Relative hardness	Moisture condition	General appearance	Remarks
70	7+92.2	–	A	Wet	LG, GRD, S	
71	7+90.3	7+	A	Wet	LG, GRD, S	
72	7+90.3	7+	A	Wet	LG, GRD, S	
73	7+86.3	7+	A	Wet	LG, GRD, S	
74	7+80.1	–	A	Wet	LG, GRD, S	Carbonate on part of patch. Epoxy bond coat patch 4" away rated A–.
75	7+80.0	7+	A	Wet	LG, GRD, S	
76	7+79.7	–	–	Wet	Patch removed	Material previously removed by LMC. Carbonate and water seeping from patch area. Patch removed for sample.
77	7+78.0	–	A	Wet	LG, GRD, S	Upper lip of slot.
78	7+76.1	–	A	Wet	LG, GRD, S, SY	
79	7+74.1	–	A	Wet	LG, GRD, S	Appears sand not thoroughly mixed with epoxy.
80	7+72.3	–	A	Wet	LG, GRD, S, SY	3/4" area in one corner of patch soft, rated C. Patch removed for sample.
81	7+72.0	–	A–	Wet	LG, GRD, S	Impact hammer made very slight indent on Patches No. 81 and 82.
82	7+70.4	–	A–	Wet	LG, GRD, S	
83	–	–	–	–	–	Not evaluated.
84	–	–	–	–	–	Not evaluated.
85	–	–	–	–	–	Not evaluated.
86	–	–	–	–	–	Not evaluated.
87	7+70.9	–	A	Wet	LG, GRD, S	
88	7+70.4	–	A	Wet	LG, GRD, S	
89	–	–	–	–	–	Not evaluated.
90	–	–	–	–	–	Not evaluated.
91	–	–	–	–	–	Not evaluated.

Explanatory notes:

7+: Penetrometer readings would have been greater than the maximum instrument reading of 7. These data indicate values of penetration resistance greater than 7,000 psi.

Relative hardness:

A—Hard
B—Slightly soft
C—Very soft

Table 4-5—Continued

Explanatory notes—Continued

General appearance:

LG—Patch looked good
GRD—Patch had been ground
R—Patch was rough
S—Very smooth patch
CAR.—Some carbonates present
SY—Slightly yellowish or rusty in color

1-inch-diameter depression occurred within a 6- by 3-inch repair patch; and (2) a 5-inch-diameter epoxy failure within a 12-inch-diameter repair patch located 175 feet downstream from the aeration slot and 6 feet right of centerline.

It is highly significant that there was no evidence of cavitation damage anywhere in the tunnel after the test with the surface irregularities noted in Table 4-7, and at a discharge approaching the average discharge maintained during the 1967 operation. The inference is that the aeration slot is providing sufficient air to dampen the damaging effects of cavitation thereby preventing destruction of the flow surfaces.

D. Future Operation and Maintenance Procedures

As a result of the June 1969 and July 1970 tests, all restrictions previously adopted for operation of the reservoir and spillway were removed.

Future inspection and maintenance procedures for the tunnel were subsequently developed and transmitted to the region by the Director of Design and Construction's letter dated September 30, 1970. For purposes of documentation these procedures are repeated in the following paragraphs:

Maintenance.—Repairs requiring correction of concrete spalls, epoxy veneer failures, blisters in the epoxy veneer, calcium carbonate removal, etc., should be made in advance of anticipated spills.

The flow surfaces starting at the lip of the aeration slot and extending 18 feet upstream, and from springline to springline, shall be maintained free of any surface irregularities, including complete removal of calcium carbonate deposits.

The tunnel surfaces from the intake portal to Station 7+63± are less prone to cavitation and some relaxation to flow surface maintenance can be tolerated. In this area 1/4 inch calcium carbonate buildup is permissible. Random concrete spalls or epoxy repair failures approximately 12 inches in diameter and with a maximum depth up to 1/2 inch are also permissible.

The tunnel surfaces downstream from the slot to Station 12+15 and from springline to springline should be maintained such that calcium carbonate buildup does not exceed 1/4 inch in height, parallel or transverse to the flow. Random concrete spalls or epoxy veneer failures approximately 6 inches in diameter and with a maximum depth up to 1/4 inch are also permissible.

The tunnel surfaces from Station 12+15 to Station 16+00± and from springline to springline shall be maintained such that calcium carbonate buildup does not exceed 1/4 inch in height, parallel or transverse to the flow. Random concrete spalls or epoxy repair failures approximately 12 inches in diameter and with maximum depth up to 1/2 inch are also permissible.

The tunnel surfaces downstream from Station 16+00± should not require special criteria to mitigate cavitation. Repairs may, however, be required for other reasons and can be initiated, as determined, after an inspection is made.

Those portions of the tunnel that do not require complete removal of calcium carbonate will eventually be covered with sheets of the deposit. Since removal of extensive carbonate buildup would involve a considerable effort, consideration should be given to advertising specifications for complete removal of calcium carbonate throughout the tunnel.

Inspection.—The entire tunnel shall be inspected each spring in advance of anticipated releases. Particular care should be given to the aeration slot areas and the area downstream from the slot

Table 4-6

IMPACT HAMMER READINGS
November 1969
Spillway Tunnel Repairs—Yellowtail Dam

Patch No.	Date patch placed	Station	Patch age at test (days)	Average impact readings on material around patches	Average impact readings on patches	Average August 1969 impact readings on patches
33	8-21-69	9+51.4	75	33.9	30.8	–
34	8-22-69	9+41.7	74	40.4	34.7	–
40	8-22-69	9+10.4	74	25.1	30.9	–
*41	8-25-69	9+09.7	71	30.6	37.9	37.9
*42	8-25-69	9+01.9	71	37.3	46.8	47.9
*50B	8-25-69	8+71.6	71	25.2	31.2	26.8
50C	8-25-69	8+64.5	71	33.1	42.4	–
*50C1	8-28-69	8+64.7	68	28.8	39.0	45.1
50D	8-25-69	8+61.4	71	26.1	38.3	–
*52	8-28-69	8+45.9	68	40.8	43.6	44.7
53	8-26-69	8+34.8	70	25.8	43.1	–
55	8-26-69	8+25.0	70	31.6	39.3	–
58	8-27-69	8+16.5	69	23.8	31.2	–
58B	8-27-69	8+10.0	69	39.8	39.1	–
58C	8-27-69	8+09.0	69	43.3	43.4	–
*59	8-27-69	8+06.4	69	25.9	38.6	35.8
61B	8-27-69	8+01.1	69	37.1	33.9	–
64	8-28-69	8+02.0	68	40.0	35.3	–
*71	8-28-69	7+90.3	68	35.6	40.3	42.0
*75	8-28-69	7+80.0	68	37.6	39.4	37.7
*76	8-28-69	7+79.7	68	37.2	33.8	32.9
79	8-29-69	7+74.1	67	38.6	29.8	–
80	8-29-69	7+70.4	67	39.2	29.5	–
81	8-29-69	7+72.0	67	39.2	30.3	–
87	8-29-69	7+70.9	67	34.5	34.9	–
Average of all patches				34.9	36.7	39.0
*Average of patches checked in August 1969				33.2	39.0	39.0

extending to Station 12+15. Any surface irregularities including epoxy veneer failure, calcium carbonate buildup, concrete spalls, etc., should be noted as to location and size.

After any spillway discharge, the tunnel should again be inspected noting any new surface irregularities or any change to existing irregularities. Reports of all inspections should be forwarded to Denver, Attention: Code 210.

Table 4-7

TABULATION OF SURFACE IRREGULARITIES
Spillway Tunnel Repairs—Yellowtail Dam

Irregu-larity No.	Station	Location from ℄ (feet)	Size (inches unless otherwise shown)	Depth (inches)
1	9+95	*4L	2.5 x 2.5 feet	Paint scour
2	9+90	*4R	2.5 x 2.0 feet	Paint scour
3	9+81	℄	9 x 1/4	1/8
4	9+82	1L	8 x 1/4	1/8
5	9+82	1L	4 x 1/8	1/8
6	9+82	0.75L	2 x 1/8	1/8
7	9+78	0.50L	3 x 1/16	1/8
8	9+77	0.50L	3 x 1/16	1/8
9	9+69	2.5L	4 x 1/4	1/4
10	9+65	3R	9 x 1/4	1/16
11	9+64	1R	5 x 1/4	1/16
12	9+63	3.5R	15 x 1/4	1/8
13	9+59	3L	5 x 1/8	1/16
14	9+58	2R	9 x 1/8	1/8
15	9+58	3R	4 x 1/8	1/16
16	9+54	2R	15 x 1/4	1/8
17	9+54	1L	2.5 x 3/8	1/4
18	9+54	1L	10 x 1/8	1/4
19	9+47	6L	2 x 2 feet	Paint scour
20	9+47	1.5L	10 x 1/4	1/16
21	9+44	2R	4 x 1/4	1/4
22	9+41	3R	8 x 6	Paint scour
23	9+42	10R	15 x 1/4	1/8
24	9+40	11R	6 x 1/8	1/16
25	9+41	7R	4 x 1/8	1/16
26	9+38	3R	5 x 1/8	1/16
27	9+37	3R	5 x 1/8	1/16
28	9+41	6L	10 x 1/8	1/16
29	9+40	6L	4 x 1/8	1/8
30	9+39	3L	5 x 1/16	1/16
31	9+40	3L	4 x 3/8	5/16
32	9+37	3L	11 x 1/8	1/4
33	9+36	1L	3 x 3	12
34	9+40	0.5R	10 x 1/8	1/16
35	9+36	1R	3 x 3	13
36	9+35	1R	4 x 1/8	1/16
37	9+36	6L	5 x 1/8	1/8
38	9+31	5L	6 x 6	Paint scour
39	9+25	℄	18 x 1/4	1/16
40	9+23	1R	10 x 1/8	1/16
41	9+24	2L	4 x 1/16	1/16
42	9+22	2.5L	6 x 6	Paint scour
43	9+24	6L	10 x 1/8	1/8
44	9+23	8L	4 x 1/16	1/16
45	9+20	11L	10 x 3/8	5/16
46	9+15	3R	16 x 3/8	5/16

Table 4-7—Continued

Irregularity No.	Station	Location from ℄ (feet)	Size (inches unless otherwise shown)	Depth (inches)
47	9+13	2R	10 x 1/4	1/4
48	9+13	3R	5 x 1/8	1/16
49	9+10	3R	19 x 1/4	1/16
50	9+08	5R	11 x 1/4	1/4
51	9+07	4R	10 x 1/8	1/16
52	9+07	3R	32 x 1/8	1/8
53	9+10	5L	10 x 1/16	1/8
54	9+10	9L	16 x 1/8	1/16
55	9+09	4L	21 x 1/8	1/8
56	9+07	4L	10 x 1/8	1/8
57	9+06	6L	4 x 1/8	1/8
58	9+02	2R	10 x 1/4	1/8
59	9+03	8R	11 x 1/8	1/8
60	8+92	3R	4 x 1/8	1/16
61	8+89	2R	10 x 1/8	1/16
62	8+99	7R	10 x 1/16	1/16
63	8+96	3R	9 x 1/8	1/8
64	8+96	4L	6 x 1/4	5/16
65	8+96	1L	4 x 1/8	1/16
66	8+92	1R	10 x 1/4	5/16
67	8+92	6R	9 x 1/8	1/8
68	8+85	0.5R	10 x 3/8	5/16
69	8+80	1R	10 x 1/2	5/16
70	8+80	5R	10 x 1/8	1/16
71	8+79	5R	11 x 1/8	1/16
72	8+79	6R	16 x 1/8	1/16
73	8+80	7R	14 x 1/16	1/16
74	8+79	4R	5 x 1/8	1/8
75	8+79	3R	4 x 1/8	1/16
76	8+80	2R	10 x 1/8	1/8
77	8+77	10R	11 x 1/8	1/8
78	8+75	10R	10 x 3/8	1/4
79	8+74	2L	10 x 1/8	1/8
80	8+70	2L	6 x 1/8	1/4
81	8+65	2R	10 x 1/8	1/4
82	8+64	2R	6 x 1/8	1/4
83	8+60	2.5R	10 x 1/8	1/8
84	8+60	1L	4 x 3/8	1/4
85	8+58	1L	5 x 3/8	1/4
86	8+56	3L	10 x 1/4	1/4
87	8+52	5R	11 x 1/8	1/16
88	8+51	2L	16 x 1/8	1/16
89	8+51	2R	11 x 3/16	1/4
90	8+50	5R	10 x 1/8	1/8
91	8+49	2R	6 x 1/4	1/4
92	8+46	1L	22 x 1/8	1/8
93	8+45	3R	11 x 1/8	1/8
94	8+45	5R	7 x 1/16	1/8

Table 4-7—Continued

Irregularity No.	Station	Location from ℄ (feet)	Size (inches unless otherwise shown)	Depth (inches)
95	8+45	6R	6 x 1/8	1/8
96	8+44	7R	6 x 1/4	5/16
97	8+41	5L	7 x 1/4	5/16
98	8+39	3R	5 x 1/8	1/8
99	8+39	3L	4 x 1/8	1/8
100	6+25	1R	5 x 1/8	1/8
101	8+25	3R	5 x 1/8	1/8
102	8+21	2R	7 x 1/4	1/4
103	8+17	2R	5 x 1/8	1/16
104	8+18	2R	4 x 1/8	1/16

*Left or right from tunnel centerline (looking downstream).

CONVERSION FACTORS–BRITISH TO METRIC UNITS OF MEASUREMENT

The following conversion factors adopted by the Bureau of Reclamation are those published by the American Society for Testing and Materials (ASTM Metric Practice Guide, E 380-68) except that additional factors (*) commonly used in the Bureau have been added. Further discussion of definitions of quantities and units is given in the ASTM Metric Practice Guide.

The metric units and conversion factors adopted by the ASTM are based on the "International System of Units" (designated SI for Systeme International d'Unites), fixed by the International Committee for Weights and Measures; this system is also known as the Giorgi or MKSA (meter-kilogram (mass)-second-ampere) system. This system has been adopted by the International Organization for Standardization in ISO Recommendation R-31.

The metric technical unit of force is the kilogram-force; this is the force which, when applied to a body having a mass of 1 kg, gives it an acceleration of 9.80665 m/sec/sec, the standard acceleration of free fall toward the earth's center for sea level at 45 deg latitude. The metric unit of force in SI units is the newton (N), which is defined as that force which, when applied to a body having a mass of 1 kg, gives it an acceleration of 1 m/sec/sec. These units must be distinguished from the (inconstant) local weight of a body having a mass of 1 kg, that is, the weight of a body is that force with which a body is attracted to the earth and is equal to the mass of a body multiplied by the acceleration due to gravity. However, because it is general practice to use "pound" rather than the technically correct term "pound-force," the term "kilogram" (or derived mass unit) has been used in this guide instead of "kilogram-force" in expressing the conversion factors for forces. The newton unit of force will find increasing use, and is essential in SI units.

Where approximate or nominal English units are used to express a value or range of values, the converted metric units in parentheses are also approximate or nominal. Where precise English units are used, the converted metric units are expressed as equally significant values.

Table I

QUANTITIES AND UNITS OF SPACE

Multiply	By	To obtain
	LENGTH	
Mil	25.4 (exactly)	Micron
Inches	25.4 (exactly)	Millimeters
Inches	2.54 (exactly)*	Centimeters
Feet	30.48 (exactly)	Centimeters
Feet	0.3048 (exactly)*	Meters
Feet	0.0003048 (exactly)*	Kilometers
Yards	0.9144 (exactly)	Meters
Miles (statute)	1,609.344 (exactly)*	Meters
Miles	1.609344 (exactly)	Kilometers
	AREA	
Square inches	6.4516 (exactly)	Square centimeters
Square feet	*929.03	Square centimeters
Square feet	0.092903	Square meters
Square yards	0.836127	Square meters
Acres	*0.40469	Hectares
Acres	*4,046.9	Square meters
Acres	*0.0040469	Square kilometers
Square miles	2.58999	Square kilometers
	VOLUME	
Cubic inches	16.3871	Cubic centimeters
Cubic feet	0.0283168	Cubic meters
Cubic yards	0.764555	Cubic meters
	CAPACITY	
Fluid ounces (U.S.)	29.5737	Cubic centimeters
Fluid ounces (U.S.)	29.5729	Milliliters
Liquid pints (U.S.)	0.473179	Cubic decimeters
Liquid pints (U.S.)	0.473166	Liters
Quarts (U.S.)	*946.358	Cubic centimeters
Quarts (U.S.)	*0.946331	Liters
Gallons (U.S.)	*3,785.43	Cubic centimeters
Gallons (U.S.)	3.78543	Cubic decimeters
Gallons (U.S.)	3.78533	Liters
Gallons (U.S.)	*0.00378543	Cubic meters
Gallons (U.K.)	4.54609	Cubic decimeters
Gallons (U.K.)	4.54596	Liters
Cubic feet	28.3160	Liters
Cubic yards	*764.55	Liters
Acre-feet	*1,233.5	Cubic meters
Acre-feet	*1,233,500	Liters

Table II

QUANTITIES AND UNITS OF MECHANICS

Multiply	By	To obtain
MASS		
Grains (1/7,000 lb)	64.79891 (exactly)	Milligrams
Troy ounces (480 grains)	31.1035	Grams
Ounces (avdp)	28.3495	Grams
Pounds (avdp)	0.45359237 (exactly)	Kilograms
Short tons (2,000 lb)	907.185	Kilograms
Short tons (2,000 lb)	0.907185	Metric tons
Long tons (2,240 lb)	1,016.05	Kilograms
FORCE/AREA		
Pounds per square inch	0.070307	Kilograms per square centimeter
Pounds per square inch	0.689476	Newtons per square centimeter
Pounds per square foot	4.88243	Kilograms per square meter
Pounds per square foot	47.8803	Newtons per square meter
MASS/VOLUME (DENSITY)		
Ounces per cubic inch	1.72999	Grams per cubic centimeter
Pounds per cubic foot	16.0185	Kilograms per cubic meter
Pounds per cubic foot	0.0160185	Grams per cubic centimeter
Tons (long) per cubic yard	1.32894	Grams per cubic centimeter
MASS/CAPACITY		
Ounces per gallon (U.S.)	7.4893	Grams per liter
Ounces per gallon (U.K.)	6.2362	Grams per liter
Pounds per gallon (U.S.)	119.829	Grams per liter
Pounds per gallon (U.K.)	99.779	Grams per liter
BENDING MOMENT OR TORQUE		
Inch-pounds	0.011521	Meter-kilograms
Inch-pounds	1.12985×10^6	Centimeter-dynes
Foot-pounds	0.138255	Meter-kilograms
Foot-pounds	1.35582×10^7	Centimeter-dynes
Foot-pounds per inch	5.4431	Centimeter-kilograms per centimeter
Ounce-inches	72.008	Gram-centimeters
VELOCITY		
Feet per second	30.48 (exactly)	Centimeters per second
Feet per second	0.3048 (exactly)*	Meters per second
Feet per year	$*0.965873 \times 10^{-6}$	Centimeters per second
Miles per hour	1.609344 (exactly)	Kilometers per hour
Miles per hour	0.44704 (exactly)	Meters per second
ACCELERATION*		
Feet per second^2	*0.3048	Meters per second^2
FLOW		
Cubic feet per second (second-feet)	*0.028317	Cubic meters per second
Cubic feet per minute	0.4719	Liters per second
Gallons (U.S.) per minute	0.06309	Liters per second
FORCE*		
Pounds	*0.453592	Kilograms
Pounds	*4.4482	Newtons
Pounds	$*4.4482 \times 10^5$	Dynes

Table II—Continued

Multiply	By	To obtain
WORK AND ENERGY*		
British thermal units (Btu)	*0.252	Kilogram calories
British thermal units (Btu)	1,055.06	Joules
Btu per pound	2.326 (exactly)	Joules per gram
Foot-pounds	*1.35582	Joules
POWER		
Horsepower	745.700	Watts
Btu per hour	0.293071	Watts
Foot-pounds per second	1.35582	Watts
HEAT TRANSFER		
Btu in./hr ft^2 degree F (k, thermal conductivity)	1.442	Milliwatts/cm degree C
Btu in./hr ft^2 degree F (k, thermal conductivity)	0.1240	Kg cal/hr m degree C
Btu ft/hr ft^2 degree F	*1.4880	Kg cal m/hr m^2 degree C
Btu/hr ft^2 degree F (C, thermal conductance)	0.568	Milliwatts/cm^2 degree C
Btu/hr ft^2 degree F (C, thermal conductance)	4.882	Kg cal/hr m^2 degree C
Degree F hr ft^2/Btu (R, thermal resistance)	1.761	Degree C cm^2/milliwatt
Btu/lb degree F (c, heat capacity)	4.1868	J/g degree C
Btu/lb degree F	*1.000	Cal/gram degree C
Ft^2/hr (thermal diffusivity)	0.2581	Cm^2/sec
Ft^2/hr (thermal diffusivity)	*0.09290	M^2/hr
WATER VAPOR TRANSMISSION		
Grains/hr ft^2 (water vapor) transmission)	16.7	Grams/24 hr m^2
Perms (permeance)	0.659	Metric perms
Perm-inches (permeability)	1.67	Metric perm-centimeters

Table III

OTHER QUANTITIES AND UNITS

Multiply	By	To obtain
Cubic feet per square foot per day (seepage)	*304.8	Liters per square meter per day
Pound-seconds per square foot (viscosity)	*4.8824	Kilogram second per square meter
Square feet per second (viscosity)	*0.092903	Square meters per second
Fahrenheit degrees (change)*	5/9 exactly	Celsius or Kelvin degrees (change)*
Volts per mil	0.03937	Kilovolts per millimeter
Lumens per square foot (foot-candles)	10.764	Lumens per square meter
Ohm-circular mils per foot	0.001662	Ohm-square millimeters per meter
Millicuries per cubic foot	*35.3147	Millicuries per cubic meter
Milliamps per square foot	*10.7639	Milliamps per square meter
Gallons per square yard	*4.527219	Liters per square meter
Pounds per inch	*0.17858	Kilograms per centimeter

GPO 837-589

ABSTRACT

Prototype operation of the Yellowtail Dam tunnel spillway in 1967 severely damaged the tunnel. Spillway operation and damage, and subsequent model tests, repairs, modifications, and prototype tests are described. Cavitation initiated by surface irregularities in the tunnel lining caused the damage. Heaviest damage was concentrated in the vertical bend of the tunnel and the near horizontal reach just downstream from the bend. Repairs consisted of backfill concrete in the heavily damaged areas and application of an epoxy-bonded epoxy-mortar veneer to cover minor surface irregularities. Modification consisted of constructing an air slot to introduce air along the flow surfaces of the vertical bend. Model tests of the tunnel showed that the optimum location of the air slot was just upstream from the vertical bend. The air slot provided positive air induction along the flow surfaces and worked satisfactorily for all ranges of discharge. Tests of the prototype tunnel after repair and modification confirmed that a sufficient volume of air was being introduced to prevent cavitation erosion.

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REC-ERC-71-23
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DESCRIPTORS–/ *cavitation/ *cavitation control/ *spillways/ *repairing/ *tunnel linings/ tunnel failure/ *damages/ model tests/ fluid mechanics/ *design modifications/ prototype tests/ air entrainment/ tunnels/ epoxy mortar/ *aeration/ hydraulic models/ hydraulics/ construction methods/ concrete linings/ operation and maintenance/ erosion
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