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Report 2

REPAIR OF EROSION-DAMAGED STRUCTURES

by

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Investigation of maintenance and preservation problems associated with Civil Works concrete structures was initiated in February 1977 at the U. S. Army Engineer Waterways Experiment Station (WES). The primary objectives of the initial phase of this investigation were to (a) identify and evaluate various abrasion-erosion-resistant materials for repairing and improving the durability of concrete stilling basin slabs, (b) develop optimum techniques for

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20. ABSTRACT (Continued).

repair and rehabilitation of stilling basins, and (c) develop guidance for designing stilling basin exit configurations to avoid entrapping abrasive materials within the basin.

A survey of Corps Divisions and District offices identified 52 structures that have experienced concrete damage due to erosion. Depths of erosion ranged from a few inches to approximately 10 ft. In general, this erosion damage resulted from the abrasive effects of waterborne rocks and other debris being circulated over the concrete surface during construction and operation of the structure.

A variety of materials including armored concrete, conventional concrete, epoxy resins, fiber-reinforced concrete, and polymer-impregnated concrete were used with varying degrees of success in the 31 repairs reported. The degree of success generally was inversely proportional to the degree of exposure to those conditions conducive to erosion damage. These materials have been used with various construction procedures, including dewatering and underwater repairs.

It appears that given appropriate flow conditions in the presence of debris, all of the materials are susceptible to some degree of erosion. No one material demonstrated a consistently superior performance advantage over alternate materials. Pending the results of additional laboratory tests and field inspections to evaluate repairs, it is recommended that conventional concrete of the lowest practical water-cement ratio containing the hardest coarse aggregate economically available should be used for repair and in new construction of structures subjected to abrasion erosion damage.

Improvements in materials should reduce the rate of concrete damage due to erosion, but will not solve the problem. Until the adverse hydraulic conditions that caused the original damage are minimized or eliminated, it will be extremely difficult for any of the materials currently being used in repair to perform in the desired manner. Prior to major repairs, model studies of the existing stilling basin and exit channel should be conducted to verify the cause(s) of erosion damage and to evaluate the effectiveness of various modifications in eliminating undesirable hydraulic conditions.

In existing structures, releases should be controlled so as to avoid discharge conditions where flow separation and eddy action are prevalent. Substantial discharges that can create a good hydraulic jump without causing eddy action should be released periodically in an attempt to flush debris from the stilling basin. Guidance as to discharge and tailwater relations required for flushing must be developed through model/prototype tests. Periodic inspections should be required to determine the presence of debris in the stilling basin and the extent of erosion.

PREFACE

This study was performed by the Structures Laboratory (SL) of the U.S. Army Engineer Waterways Experiment Station (WES) under the sponsorship of the Office, Chief of Engineers (OCE), U. S. Army, as a part of Civil Works Investigation Work Unit 31553, Maintenance and Preservation of Civil Works Structures. The study was authorized 16 February 1977 by first indorsement of a WES letter dated 3 January 1977. Mr. James A. Rhodes of the Concrete Branch, Engineering Division, OCE, served as technical monitor.

The study was conducted under the general supervision of Mr. Bryant Mather, Acting Chief, SL, and Mr. John Scanlon, Chief, Engineering Mechanics Division, SL, and under the direct supervision of Mr. James E. McDonald, Chief, Structures Branch, SL. The report was prepared by Mr. McDonald. This is Report 2 in a series of reports related to the maintenance and preservation of Civil Works structures. Report 1, "Annotated Bibliography, 1927-1977," Technical Report C-78-4, was published in September 1978.

Commanders and Directors of the WES during this study and the preparation and publication of this report were COL John L. Cannon, CE, and COL Nelson P. Conover, CE. Technical Director was Mr. Fred R. Brown.

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CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI)
UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
acre-feet	1,233.489	cubic metres
cubic inches	16.38706	cubic centimetres
cubic feet	2.831685	cubic metres
cubic feet per second	2.831685	cubic metres per second
cubic yards	0.7645549	cubic metres
cubic yards per hour	0.7645549	cubic metres per hour
Fahrenheit degrees	5/9	Celsius degrees or Kelvins*
feet	0.3048	metres
feet per second	0.3048	metres per second
feet per minute	0.3048	metres per minute
gallons (U. S. liquid)	3.785412	cubic metres
gallons (U. S. liquid) per square foot	40,745.85	litres per square metre
gallons (U. S. liquid) per square yard	4,527.31	litres per square metre
horsepower (water)-days	746.043	watt-days
inches	25.40000	millimetres
kips (force)	4.448222	kilonewtons
miles (U. S. statute)	1.609344	kilometres
mils	0.0254	millimetres
ounce (U. S. fluid)	29.57353	cubic centimetres
pounds (force) per square inch	6,894.757	pascals
pounds (mass)	0.4535924	kilograms

(Continued)

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

Multiply	By	To Obtain
pounds (mass) per square foot	4.882438	kilograms per square metre
pounds (mass) per square yard	0.54249	kilograms per square metre
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
pounds (mass) per cubic yard	5.93276	kilograms per cubic metre
square feet	0.09290304	square metres
square yards	0.8361274	square metres
tons (2000 lb mass)	907.1847	kilograms

MAINTENANCE AND PRESERVATION OF CONCRETE STRUCTURES

Report 2

REPAIR OF EROSION-DAMAGED STRUCTURES

PART I: INTRODUCTION

1. There are a variety of structures (dams, locks, floodwalls, bridges, etc.) associated with the Corps civil works program. Fifty-nine percent of these structures are more than 20 years old with more than a fourth (26 percent) being over 40 years old. With the decline in new construction starts in recent years, many of these old structures are having to continue in operation well beyond their original design service life; consequently, maintenance and rehabilitation have assumed increased importance.

2. Investigation of maintenance and preservation problems associated with civil works concrete structures was initiated in February 1977 at the U. S. Army Engineer Waterways Experiment Station (WES). The overall objective of this investigation was to develop information necessary to ensure the continued safety of dams and other civil works structures. Specifically, this information includes (a) developing and evaluating materials and techniques for repair and rehabilitation of civil works structures; (b) developing engineering guidance for evaluating and monitoring the safety of structures; and (c) developing design and construction methods for rehabilitating older structures to comply with current structural design criteria.

Purpose

3. The primary objectives of this phase of the overall investigation were to (a) identify and evaluate various abrasion-erosion-resistant materials for repairing and improving the durability of concrete stilling basin slabs; (b) develop optimum techniques for repair and rehabilitation

of stilling basins; and (c) develop guidance for designing stilling basin exit configurations to avoid entrapping abrasive materials within the basin.

Scope

4. The first step in this investigation was to review techniques and materials used in the repair and rehabilitation of stilling basins. Input was solicited by Engineer Circular No. 1110-2-181 (Office, Chief of Engineers (OCE), Department of the Army 1977) from all Corps of Engineers (CE) Divisions and Districts performing Civil works functions. The principal components of the requested input were (a) the project name, location, and date of repair; (b) a description of the stilling basin; (c) a description of repair techniques and materials; (d) the direct costs associated with the repair; (e) a narrative discussion of the effectiveness of the repair including the results of follow-up evaluations, if any; and (f) any plans for future inspections and/or repairs of stilling basins.

5. The information obtained from the various Division and District offices varied widely in its content and in its relationship to specific objectives of this investigation. The information was checked for completeness, and in some cases, follow-up contact was made to obtain missing data or to clarify information.

PART II: DESCRIPTION OF STRUCTURES AND REPAIRS

6. The survey of CE Division and District offices identified 52 structures that have experienced concrete damage due to erosion. Depths of erosion ranged from a few inches to approximately 10 ft.* In the latter case, nearly 2000 cu yd of concrete and bedrock was removed from the stilling basin by erosion at Dworshak Dam. Subsequent to the survey, two additional structures experiencing damage were identified in the South Atlantic Division. The distribution of these structures is shown in Figure 1. Thirty-three of these structures have been repaired and over 70 percent of the repairs have been performed since 1970. A variety of repair materials (Table 1), including armored concrete, conventional concrete, epoxy resins, fiber-reinforced concrete, and polymer-impregnated concrete, has been used with varying degrees of success. These materials have been used with various construction procedures, including dewatering and underwater repairs.

Old River Low Sill Structure

7. The Old River Control Structure is located on the west bank of the Mississippi River about 50 miles northwest of Baton Rouge, Louisiana, and about 35 miles southwest of Natchez, Mississippi. The structure was constructed in the 1950's and 1960's to prevent the impending adoption of the Atchafalaya River's course as the lower Mississippi's main route to the Gulf of Mexico. The principal features of the plan consist of two mechanically operated control structures, inflow and outflow channels, a lock for navigation, an earthen dam closing the natural Old River, levee enlargement, and bank stabilization. The two control structures are designated the low sill and overbank structures.

8. The low sill control structure (Figures 2 and 3) is a reinforced concrete structure with vertical lift gates, founded on steel

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* A table for converting U. S. customary units of measurement to metric (SI) units is given on pages 3 and 4.

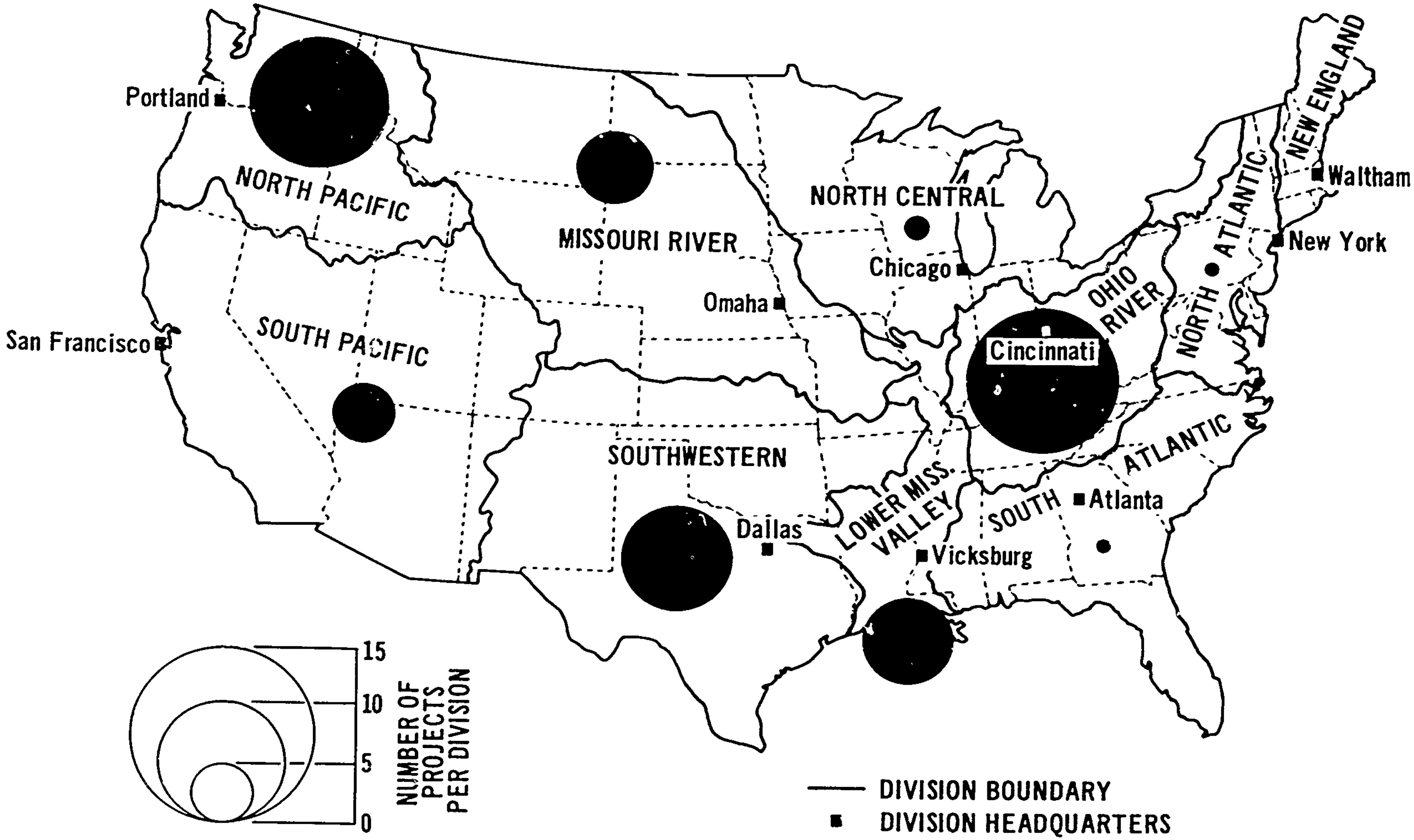

Figure 1. Number and distribution of structures which have experienced concrete damage due to abrasion erosion

Table 1

General Classification of Materials and Methods Used in Repair of Eroded Concrete Surfaces

Structures by District	Armor-Plated Concrete	Epoxy Mortar/Concrete	High-Strength Concrete	Reinforced Concrete/Grout	Revised Configuration	Latex Mortar	Fiber Concrete	Conventional Concrete, Grout	Shotcrete	Tremie Concrete	Polymer-Impregnated Fiber Concrete	Reinforced Fiber Concrete
New Orleans District (LMN)												
Old River	X				X							
Vicksburg District (LMV)												
Arkabutla		X	X									
Enid		X	X									
Grenada		X	X									
Sardis		X	X									
Kansas City District (MRK)												
Pomme De Terre			X									
Pomona		X	X	X	X							
Tuttle Creek				X								
Baltimore District (NAB)												
Curwensville					X							
St. Paul District (NCS)												
Lac Qui Parle				X	X	X						
Upper St. Anthony Falls	X	X					X	X				
Portland District (NPP)												
Bonneville			X	X	X				X			
Seattle District (NPS)												
Chief Joseph										X		
Libby								X			X	X
Walla Walla District (NPW)												
Dworshak		X						X	X		X	X
Ice Harbor										X		
Louisville District (ORL)												
Barren River			X	X								
Nolin			X		X							
Nashville District (ORN)												
Center Hill				X				X				
Pittsburgh District (ORP)												
Kinzua	X	X			X		X	X				
Tionesta								X				
Los Angeles District (SPL)												
San Gabriel		X										
Sacramento District (SPK)												
Folsom		X										
Pine Flat								X				
San Francisco District (SPN)												
Coyote			X	X	X							
Albuquerque District (SWA)												
Conchas					X			X				
Ft. Worth District (SWF)												
Somerville								X				
Waco				X								
Little Rock District (SWL)												
Bull Shoals			X	X	X							
Nimrod								X				
Table Rock		X										
Tulsa District (SWT)												
Oologah		X	X	X	X							
Webbers Falls										X		

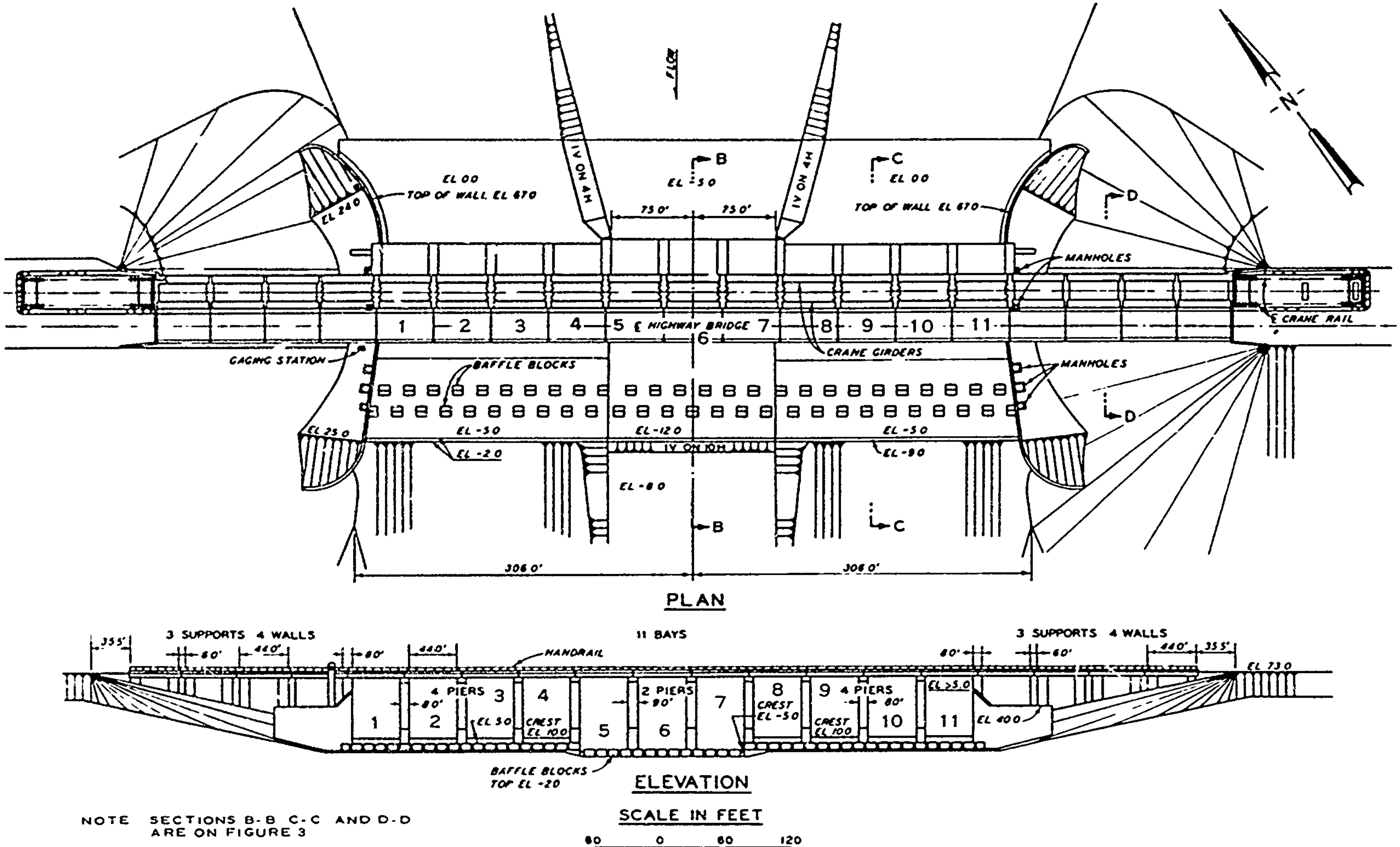

Figure 2. Plan and elevation, Old River Low Sill Structure

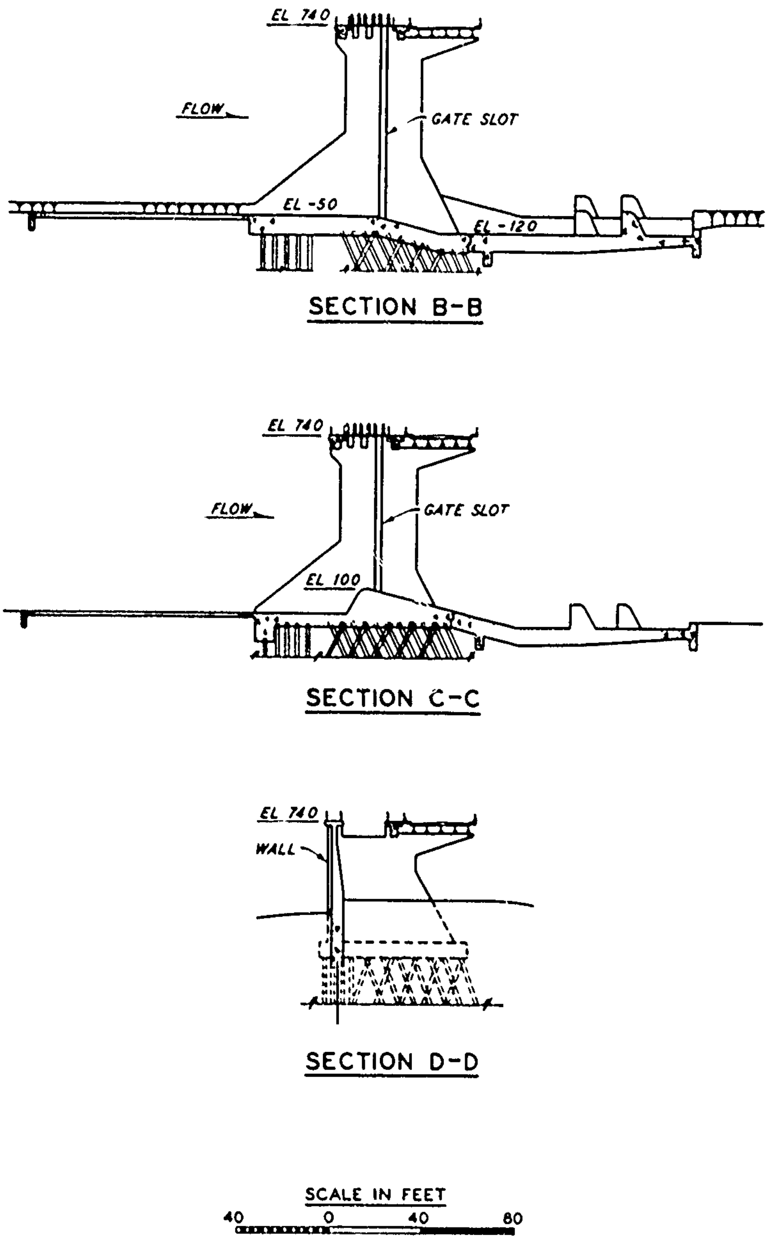

Figure 3. Piers, Old River Low Sill Structure

H-piles. The structure has a gross length of 566 ft between abutments, with 11 gate bays that have a 44-ft clear width between each pier. The three center bays have weir crests at elevation -5.0* and the eight outer bays have a crest at el +10.0. The bays are closed with vertical lift steel gates operated by two traveling gantry cranes. A 26-ft-wide highway bridge is located on top of the piers on the downstream side and is part of Louisiana State Highway 15.

9. The stilling basin is a rectangular concrete channel with a width of 566 ft near the downstream edge of the piers with a 1 on 8 flare to a width of 592 ft at the end sill. The central portion serving the three lowered bays is 150 ft wide with a floor elevation of -12.0 and terminates with an end sill at el -2.0. The basin slab is reinforced in each direction with approximately 0.3 percent of steel equally divided between the top and bottom faces. Two rows of 10-ft-high baffles 12 ft apart extend across the entire stilling basin. The floor elevations and lengths of the two parts of the stilling basin were based on computed theoretical determinations, on the results of model studies of the structure, and on the results of operating analogous structures. Construction was essentially completed in 1963.

10. In August 1976, water stages were low enough to permit systematic underwater inspection of the stilling basin by divers using underwater TV cameras. The stilling basin slab was found to be severely eroded in the area between the end sill wall and downstream row of baffles. The erosion of the slab was most severe in the area directly behind the low-flow gate bays (gates 5-7). The erosion in this area varied generally from depths of 6 in. to 2 ft, with isolated areas having as much as 4 ft of erosion, or 80 percent of the original slab thickness. Both transverse and longitudinal steel reinforcements were exposed over the area. In many areas the reinforcing steel was found to be free on one end and bent upward from the slab. In addition to the damaged stilling basin slab, a considerable amount of riprap was also found in this area. The survey of the stilling basin behind the high gate bays on the

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* All elevations (el) are in feet referred to mean sea level (msl).

south portion of the structure (gates 8-11) also revealed erosion damage to the downstream portion of the stilling basin slab. Although this area was not as severely eroded as the low-bay area, the damage was considered significant. The average depth of erosion varied from 4 in. to 1 ft, with both transverse and longitudinal steel exposed over half of the area. This area was also overlaid by various sized riprap. In addition, a large pile of debris was discovered on the downstream portion of the stilling basin slab. The debris consisted of pieces of the upstream approach apron, segments of the upstream wing wall, and riprap. Twisted throughout this debris were numerous reinforcing bars. The survey of the stilling basin behind the high gate bays on the north portion of the structure (gates 1-4) revealed a few pieces of riprap, general wear of the concrete surface, and only two isolated areas of severe erosion.

11. Since the structure is located only 2000 ft from the Mississippi River, currents in the river would be expected to produce unsymmetrical flow conditions in the approach channel. During an early hydraulic model investigation (WES 1959a) of the structure there was concern as to the effect of such flows on the capacity of the structure. Accordingly, the model was modified to include the entire approach channel to the control structure and a portion of the right side of the river channel. Tests revealed currents on the left side of the approach channel to be higher than those on the right side (Figure 4), but careful discharge measurements were made and no change in capacity could be detected. However, for certain discharge conditions, flow conditions through the adopted design structure (Figure 5) would appear to be more conducive to erosion and/or deposition of debris behind the high gate bays on the south (left side looking downstream) than on the north portion of the structure. Model tests (Rothwell and Grace 1977) were conducted in 1975 to evaluate and develop satisfactory means of regulating the existing structure to achieve the desired flow objectives without creating adverse hydraulic flow conditions.

12. Completion of the stilling basin inspection resulted in two decisions: the stilling basin would have to be repaired in order to

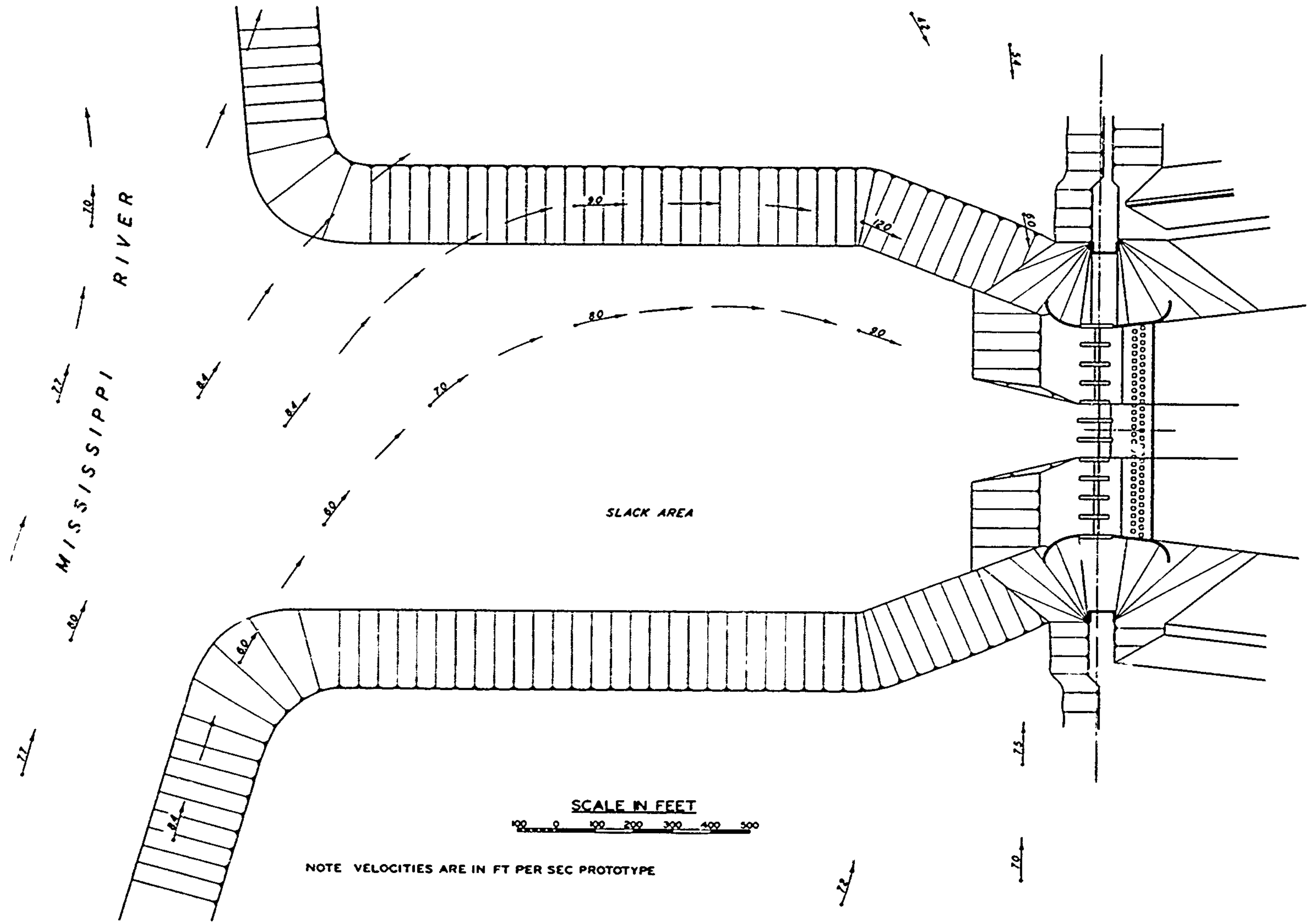
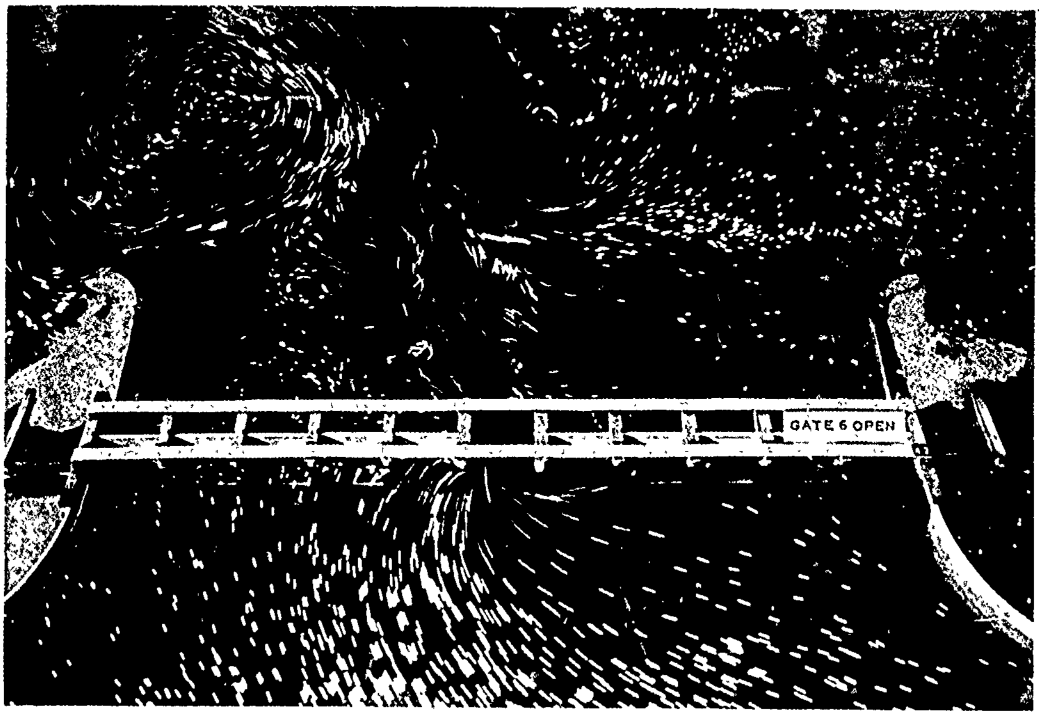
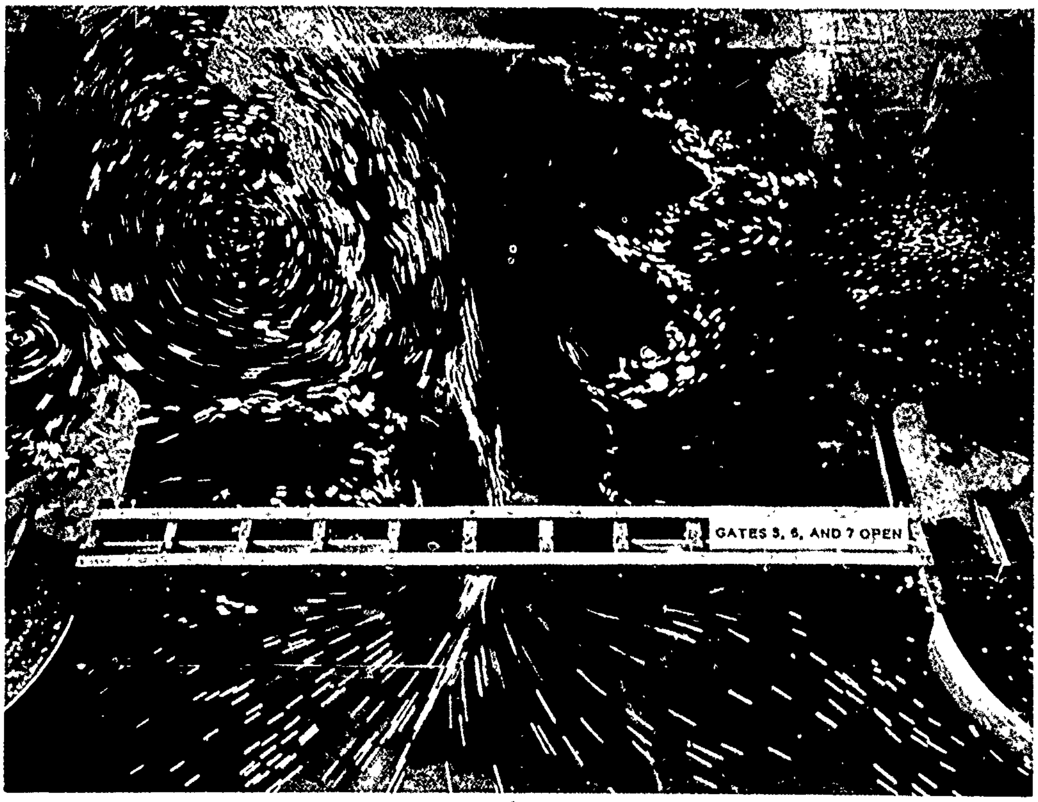


Figure 4. Typical velocity distribution in approach channel, Old River Low Sill Structure (from WES 1959a)



a. Gate 6 open



b. Gates 5, 6, and 7 open

Figure 5. Flow conditions at control structure (comprehensive model) with one and three gates open, pool el 56.50, tailwater el 52.00, 550,000 cfs (from WES 1959a)

protect the integrity of the overall structure and the structure would not be completely closed to facilitate repair. Rather, repairs would have to be made under flow with careful use of partial closure to produce acceptable underwater working conditions. Conceptual schemes for performing the repairs were developed immediately. During the design phase and preparation of plans and specifications, an emergency contract was let to clean all the loose debris from the stilling basin and to prepare it for the repair work. The cleanup started at one end of the stilling basin. A crane equipped with a grapple bucket picked up as much of the broken rocks and concrete as possible within the radius of the boom. A diver was sent down with equipment to cut the reinforcing steel projecting above the slab in this area. After the steel was cut, a drag bucket was used to complete the cleaning of the stilling basin area. This method was repeated as the cleanup operation progressed across the entire stilling basin.

13. Several repair concepts were proposed. Based on an exhaustive investigation and study, a plan to repair the basin with modules of steel plate anchored and grouted to the end sill and to the floor slab directly behind the downstream row of baffles (Figure 6) was selected. Of the plans studied, this plan offered the best combination of cost, permanent construction, and time for construction. The steel plate would provide more protection against scour and riprap damage than concrete alone, and placing the module on the end sill as planned would provide the most predictable final alignment of all the plans. Although this plan appeared to present the most hydraulic change to the stilling basin, detailed model tests at WES* indicated that none of the proposed plans would create adverse hydraulic conditions in the stilling basin and exit channel.

14. Thirty modules 24 ft long and ranging in width from 3 to 22 ft were fabricated from 1/2-in.-thick steel plate (Figure 7). Vertical diaphragm plates were welded to the horizontal plate, both to stiffen

* E. D. Rothwell. "Old Sill Existing Low Sill Control Structure, Rehabilitation of Stilling Basin," (in preparation), U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

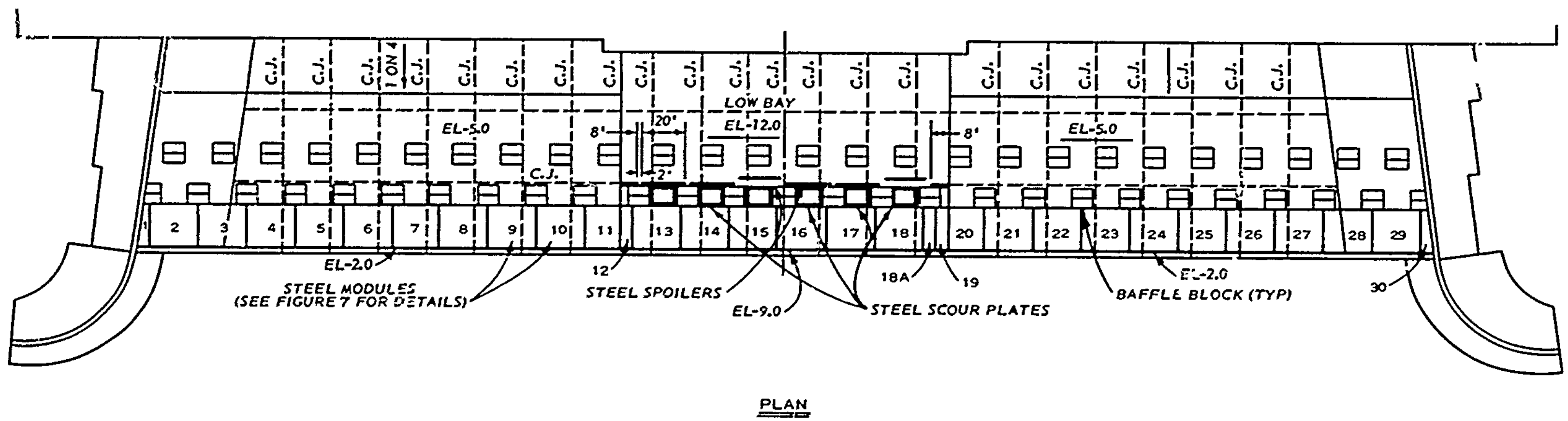
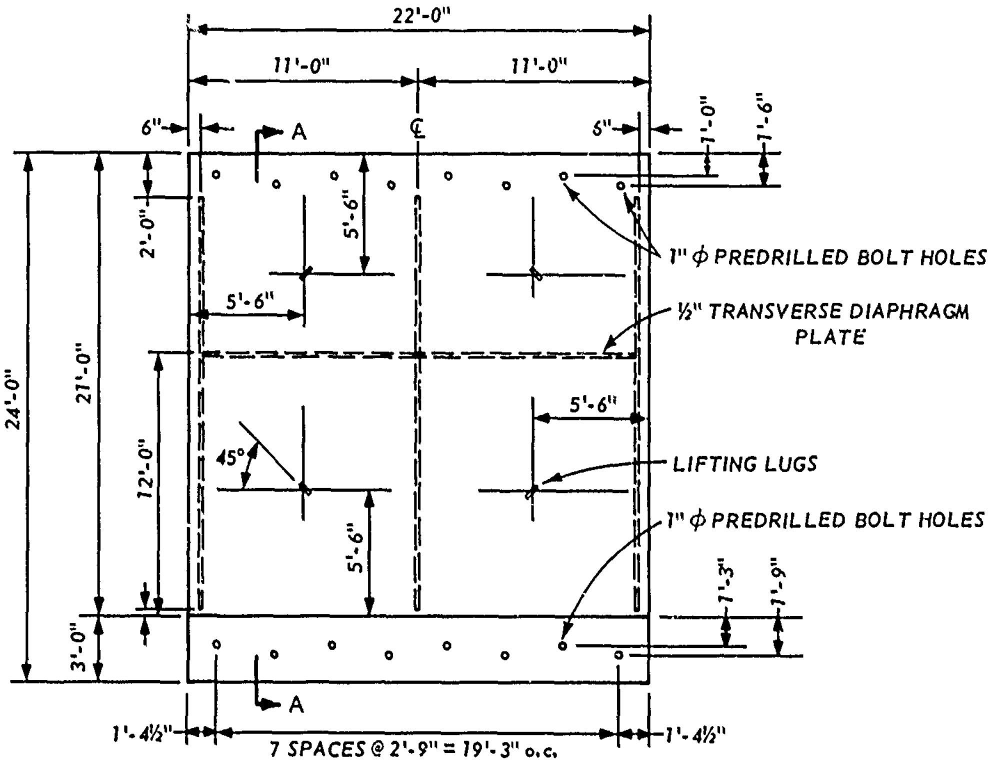
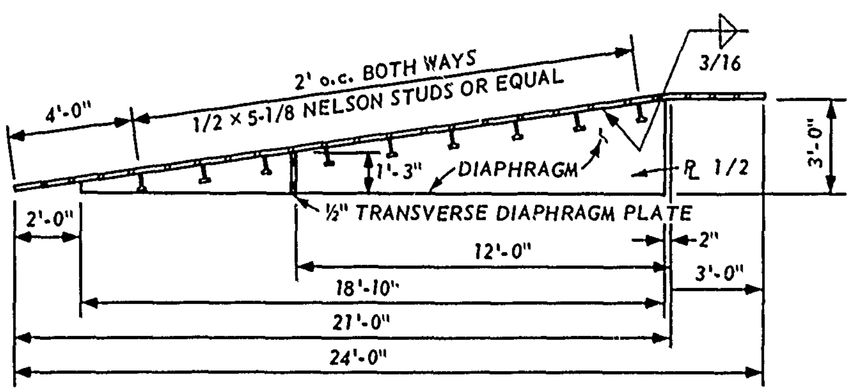


Figure 6. Location of steel modules, Old River Low Sill Structure



PLAN



SECTION A-A

Figure 7. Typical details of prefabricated module, Old River

the plate and to provide a formed void in which to retain the grout. Modules were prefabricated in New Orleans and were shipped to the job site by barge. The first module installed (No. 12), 6 ft wide and 24 ft long, was placed (Figure 8) behind gate bay 5 in the low bays. The

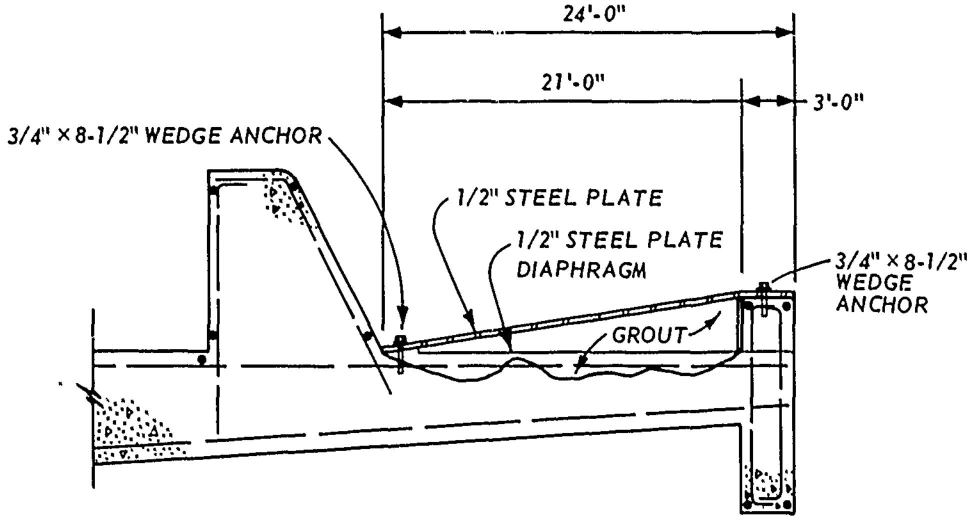


Figure 8. Typical placement of prefabricated module, Old River

required number (4) of 8-in.-long expansion anchor bolts was installed to secure the module until it could be grouted. Upon completion of the first day's work, the low bays were opened to allow as much water through the structure as possible because of the extremely low water stages. The following morning, after the low bay gates were closed to allow work to begin, divers inspected the first module. The module was found to be displaced as a result of uplift forces due to flow over the wedge-shaped module. The bolts on the upstream edge of the installed module had been pulled out of the concrete. The nuts on the bolts placed in the end sill wall had worked loose from vibration, thus stripping the threads and allowing the module to lift up. The module was retrieved and found to be in good condition. As a result of this action, it was apparent that gates immediately upstream of an ungrouted module would have to be closed at night to protect the modules until they could be grouted. Modules 13 and 14 were placed next, and the required number of bolts was increased from 4 to 8-16 per module. In some cases, the void beneath the plate along the bolt line was too deep to get enough bolt penetration into the concrete. Before modules 15 and 16 were installed, it was decided to change the anchor bolts from the expansion anchor bolts to epoxy-grouted bolts 1 by 12 in. long. The epoxy specified in the contract and used at the site was a two-component mixture packaged in a plastic cartridge. The bolt installation procedure was to drill a hole, place the epoxy cartridge in the hole, insert the bolt to break the cartridge, and spin the bolt with an air drill for a minimum of 1 min or until the epoxy set. Within 5 min the nut could be torqued on the bolt and the nut welded to the module to keep it from backing off.

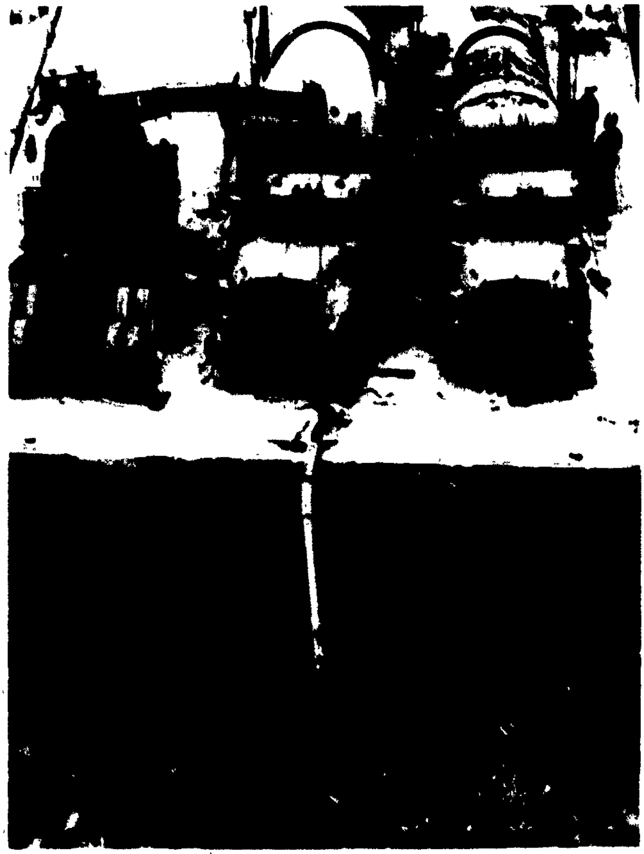
15. The WES personnel designed a fluid, fast-setting grout mixture that could be pumped and would set up underwater in about 2 hr. Mixture proportions were as follows:

Material	Weight
Cement, Type I	1034 lb
Sand	2080 lb
Bentonite Gel	10 lb
1-in. Steel Fibers	100 lb
Water-reducing admixture (P)	11 oz
Air Entraining agent	4 oz
Water	465 lb

Two representatives from WES were at the site during the repair operation to assist in any problems with the grouting operation.

16. An on-site plant provided grout batches to transit mixer trucks, and after it was mixed, the grout was discharged into a displacement-type concrete pump on the structure bridge and pumped directly into the modules. About 12 hr after the first module was grouted, a diver inspection revealed a 6-in. void between the top of the grout and the top plate, and the material was badly segregated. The segregation was believed to result from the grout free falling 95 ft in the vertical grout line into 15 ft of water. The grouting plan was altered to provide a grout pump on the structure bridge and another pump and two ready-mix concrete trucks on a barge over the stilling basin (Figure 9). The grout was pumped from the bridge into the ready-mix trucks located on the barge.

a. Grout mixers and pump on bridge



b. Grout mixers and pump on barge over the stilling basin

Figure 9. Grout pumping operations, Old River

The grout was agitated in the trucks, discharged into the pump on the barge, and pumped into the modules. The time factor was very critical to this type of operation, as the grout could only be left in the conveyor pipe a short time before it would start to set. The modified method proved to be satisfactory with little or no segregation of the materials.

17. Because the grout was placed in the modules beneath the water, a question arose as to the flow characteristics of the grout once it was placed into the module. Therefore, the repair contract was modified to construct a wooden test form of the same size and shape as the actual steel modules. A test hole was excavated and the form placed within. The hole was flooded with water, and after the form was completely submerged, grout was pumped into the form. After the grout was allowed time to set, the water was pumped from the hole and the form was removed. The results of this test indicated that the form was being completely filled and the steel fibers in the mixture were being spread throughout the grout with little or no segregation. The test also indicated that excessive pumping pressure lifted the plate, causing the bolts to pull out of the concrete, and the forms failed.

18. Uplift forces that the modules might experience during operation of the structure were of concern; therefore, WES was requested to conduct model tests* to determine the magnitude of forces that might be expected along the upstream edge of the module. Based on these tests, spoilers (12-in. steel angles) were anchored immediately upstream of the lower row of baffles in the low gate bays (see Figure 6). The tests indicated that such a spoiler would effectively reduce the uplift force about 1.0 kip per foot of width. Results of the model studies also indicated the need for an anchorage system to securely anchor the grouted module to the base slab. Therefore, a line of holes, 4 ft from the upstream edge of the modules, was drilled through the grouted modules and into the basin slab to a depth of 3 ft. A total of 23 anchor bolts 5 ft

* Rothwell, op. cit.

long were grouted in the holes with the same epoxy used to anchor the modules prior to grouting.

19. Modules behind the low gate bays 5, 6, and 7 were placed and grouted first. The operation was then moved to the south side of the stilling basin because damage was more extensive on that side. Six modules were installed and grouted on the south side. Operations were then moved to the north side to place six modules. This was done to keep the stilling basin in symmetry if the river stages changed such that repair operations had to be discontinued. However, the river stages were favorable, thus allowing the modules to be placed across the entire width of the stilling basin.

20. Actual construction work on the repair started on 13 September 1976, and all work was completed on 16 December 1976 at an estimated contract cost of $1,850,000.

21. An underwater inspection of the repairs (New Orleans District 1977) was initiated on 1 August 1977 and completed on 4 August 1977. Inspected features of the structure included the grouted steel modular plates that were placed during 1976, the baffle blocks, and the floor area around each of the baffle blocks.

22. Using video-audio equipment, the diver systematically inspected all the anchor bolts on each plate and the elevation of the upstream and downstream edges of each plate module with respect to stilling basin surfaces, and randomly checked the grout injection holes. An inspection was also performed on the faces of each individual baffle and the floor area adjacent to each face.

23. Damage to the stilling basin repairs was generally confined to loss of some of the steel plating from the modules used to form the voids for placement of the grout (Figure 10). Seven of the 30 module plates and all six of the flat plates sustained varying degrees of damage. All spoilers were missing. In those areas where steel plating was lost, the exposed grout surface showed no evidence of erosion. The loss of the structureward one third of the plate of module 16 and its associated flat plate revealed that no grout had been placed under the flat plate and that grout had not entirely filled the void between the

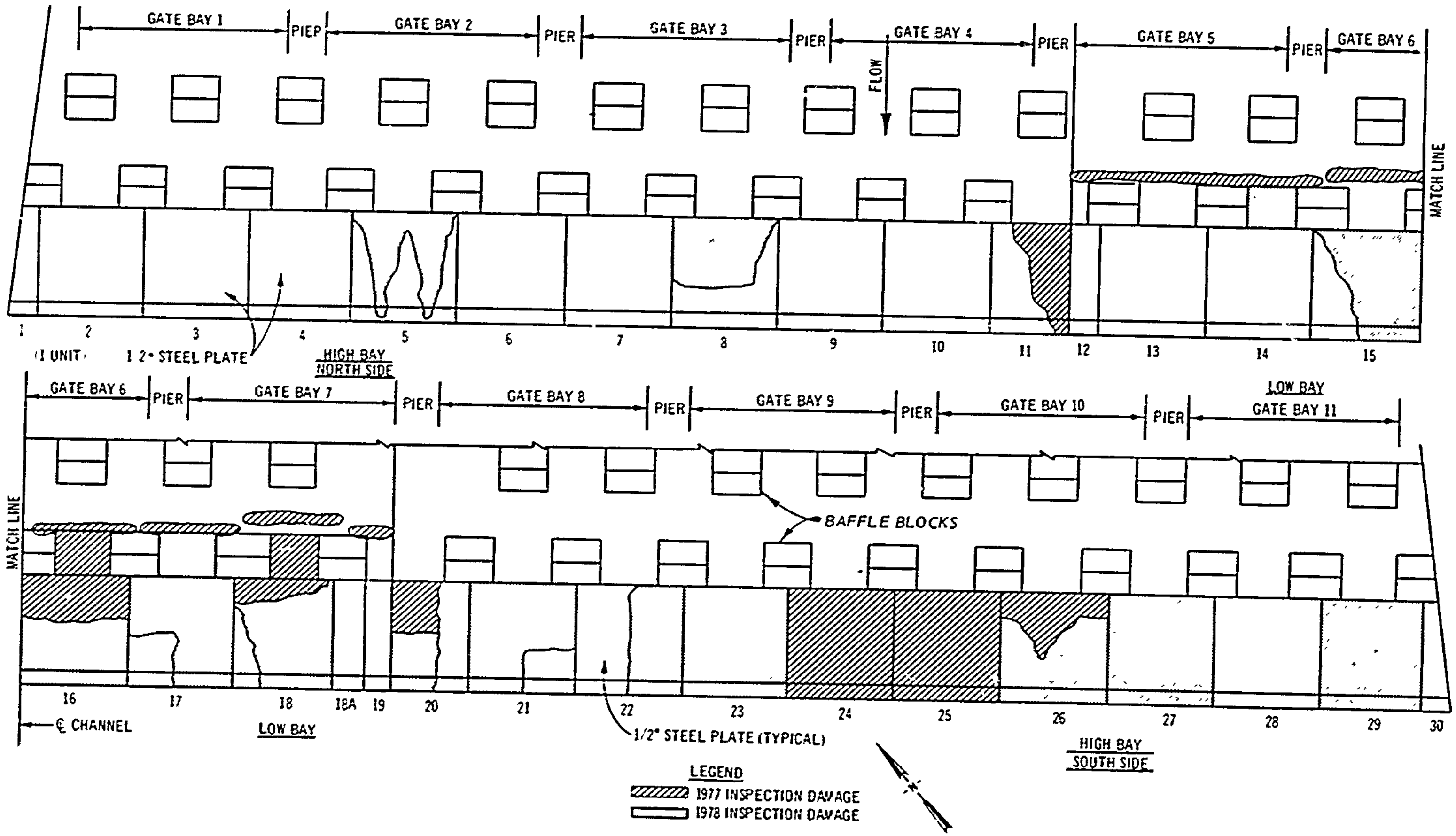


Figure 10. Damage to stilling basin repairs, Old River

top plate of the module and the stilling basin slab. Portions of the plates in modules 11, 18, 20, 24, 25, and 26, ranging from about 20 to 100 percent, were lost. A number of anchor bolts were found broken either flush with the module plate, flush with the grout, or pulled completely out.

24. The inspection of the baffles located behind the four high bays on the north side of the structure revealed minimal distress. Erosion in this area was limited to rounded edges and small chips or spalls on the lower 2 ft of the blocks. The baffles behind the three low bays and the four high bays on the south side of the structure received considerably more damage.

25. The grout within the modules appeared to be firm and in a nondeteriorated state, with the grout completely adhered to the concrete slab. The scour areas were generally dish-shaped with an approximate depth of 6 in. at the deepest point. There was an exposed reinforcing bar grid with approximately a 2-in. clearance between the bottom of the reinforcing bars and stilling basin slab. This was the worst condition and existed only upstream of module 16. The 6-in.-deep scour area, when compared to the approximate 5-ft thickness of the slab, was shallow and did not appear to endanger the integrity of the stilling basin.

26. The underwater inspection of the stilling basin required gate closures, which resulted in a differential head in excess of 13 ft. During these intervals of high differential head, uplift pressures were monitored by periodic reading of piezometers located under both the structure and stilling basin. The piezometer data indicated that the uplift forces beneath the structure and stilling basin were not excessive and posed no threat during the periods of time the structure was subjected to differential heads of over 13 ft. Based on this inspection it was concluded that remedial repairs, except for the spoilers, were effective and were performing in a satisfactory manner. The baffle blocks and the stilling basin floor are generally in good condition. The grout, where exposed, shows no tendency to erode or spall.

27. A second inspection of the repairs was begun on 19 November 1978 with divers working off the top girder of one of the high-bay lift

gates. Gate bays 8-11 were closed and the first two modules (29 and 30) downstream from Bay 11 were inspected. An attempt was made to continue the inspection on 20 November 1978; however, the inspection had to be postponed due to problems with the divers' video-audio equipment.

28. The inspection was resumed on 21 November 1978, but the divers continued to experience problems trying to operate from the top of the lift gates, due to the length of air hose and communication cable required from the gates to the downstream end of the stilling basin. Because of problems with the lines tending to catch on the baffles, the divers were only able to make a cursory inspection of the steel plates and concrete surfaces of modules 24-30.

29. A string of barges from which the divers could work was moved into position for diving on 22 November 1978. Gate bays 4-8 were closed and the divers worked in the low-bay area (modules 12-19). The divers continued to work in the low-bay area and completed the inspection of the modules and the concrete surfaces immediately upstream from the modules, 23 November 1978.

30. The remaining modules (20-23) and the downstream row of baffle blocks of the high-bay stilling basin on the south side of the structure were inspected 24-25 November. Gate bays 6-11 were closed for this inspection. Modules 1-11 of the high-bay stilling basin on the north side of the structure were inspected 26 November, behind closed gate bays 1-6. Due to rising river stages that restricted the divers' ability to move freely and a severe thunderstorm at the structure, the divers were only able to make a cursory inspection of the steel plates and concrete surfaces of these modules. The inspection was terminated on 26 November.

31. In general, the inspection revealed that additional steel plates had been ripped from the modules (Figure 10) and minor erosion had occurred in the stilling basin slab upstream from the modules in the all low bays and from module 23 in the south high bays. No deterioration of the fiber-reinforced concrete was reported. The stilling basin was reported to be free from rock and other debris. Apparently, any rock or debris discharged through the structure is flushed from the

stilling basin over the fillet formed by the modules at the end sill.

32. The eroded areas upstream of the 9 low-bay modules occurred prior to the initial repairs of the stilling basin. An attempt was made to fill these eroded areas during the initial repairs by injecting fiber-reinforced concrete beneath anchored steel plates. All of these plates are now missing and all of the eroded areas under the plates were apparently not filled completely with concrete.

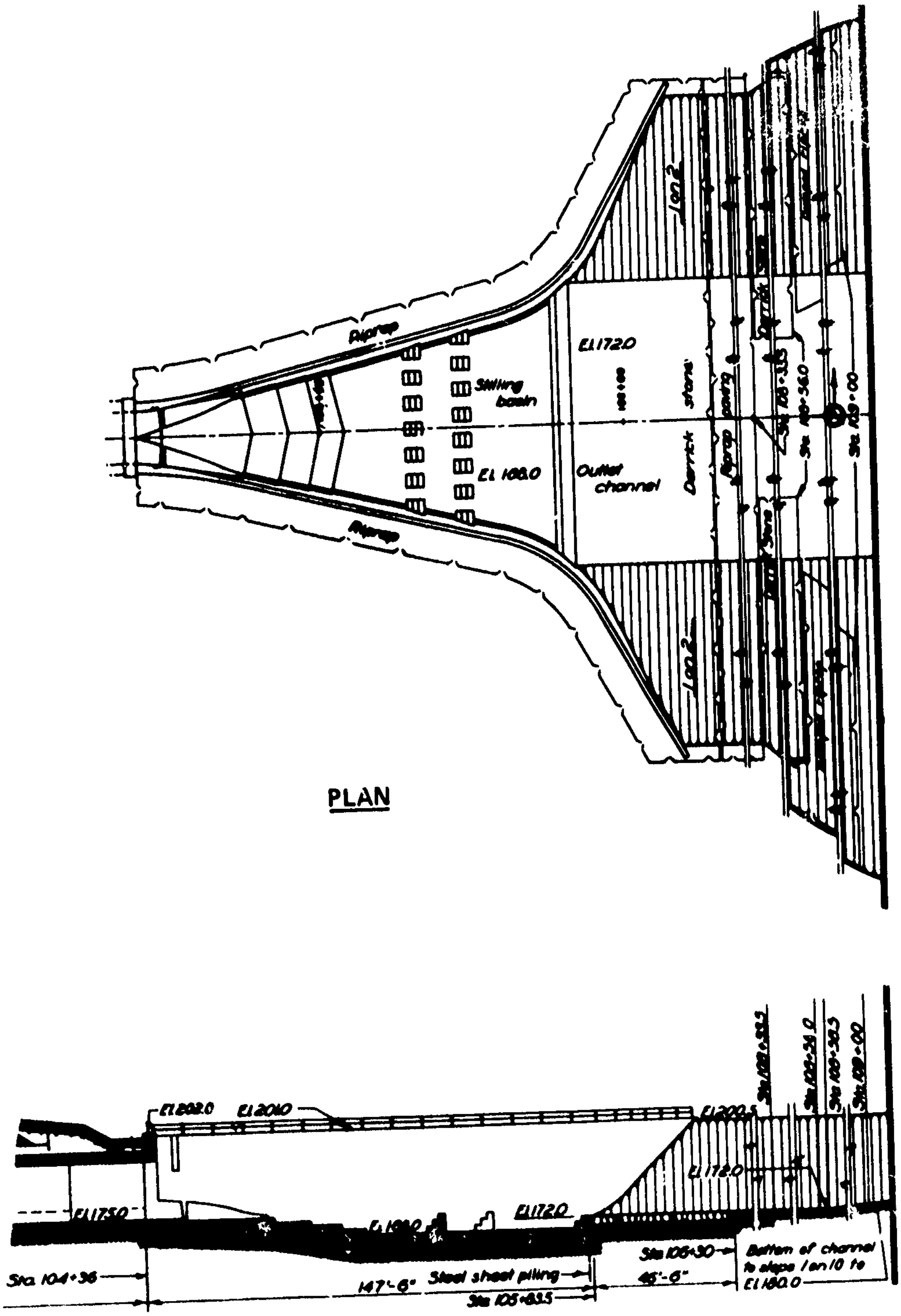
33. The available evidence indicates that the 9- to 12-in.-deep eroded area reported upstream from module 23 probably occurred prior to the initial repairs. However, the 1977 inspection report does not cover this eroded area. The District will review records made during repair operations and the 1977 inspection to determine if the damage is recent or if it occurred prior to the initial repairs. Additional inspections are planned annually, river stages permitting.

Arkabutla Dam

34. Arkabutla Dam is a rolled-fill structure located on the Coldwater River in Tate and Desoto Counties in northwestern Mississippi, approximately 12 miles northwest of the town of Coldwater. The outlet works, located near the south abutment, consist of a concrete approach, three-gated control structure, transition, single-barrel egg-shaped conduit, transition chute, and stilling basin. The design head and discharge are 50 ft and 10,000 cfs, respectively.

35. The reinforced concrete stilling basin (Figure 11) is 143.5 ft long from headwall to end sill. There is a smooth transition from the conduit section at the portal to the trapezoidal basin. The width at the portal is 16 ft and at the end sill it is 88 ft. Essential elements of the hydraulic jump-type basin are (a) a chute that drops 7 ft by steps, (b) horizontal apron 75 ft long, (c) two rows of stepped baffles, (d) stepped end sill, and (e) diverging spray and wing walls.

36. The retaining walls are reinforced concrete cantilevers with the exception of the section between the portal and station 105+08.5, which is designed as a U-frame. The basin slab has a minimum thickness



SUBTRACT 0.7 FOOT FROM ALL ELEVATIONS

Figure 11. Plan and profile outlet works stilling basin, Arkabutla

of 5 ft of concrete proportioned for a 28-day compressive strength of 3400 psi. Approximately 0.6 percent reinforcement was provided by placing 1-1/4-in. square bars spaced 12 in. each way in the top and bottom of the slab.

37. The bottom width of the outlet channel at the end sill is 88 ft, with side slopes 1 on 2 and a bottom elevation of 171.3 ft. This elevation is maintained for approximately 300 ft downstream where the bottom then slopes upward 1 on 10 to el 179.3 ft.

38. An informal inspection of the stilling basin was made in 1945, two years after the project was put into operation. The three gates were closed but the basin was not unwatered. This inspection indicated 12-15 in. of silt and mud on the basin floor. A second inspection in 1967 under similar circumstances revealed an accumulation of about 4 ft of mud, silt, and riprap. In 1969, for the first time the basin was unwatered for the initial periodic inspection (Vicksburg District 1969). Erosion had occurred in the stilling basin, exposing reinforcing steel on the corners of three baffle piers (Figure 12) and at three locations on the floor surface. Depth of erosion was estimated at 5-6 in.

39. Due to the depth of the erosion at the locations where reinforcement was exposed, it was decided at the time of the 1969 inspection to repair these areas with thin-ply patches of a portland cement base material mixed with marble dust and a gaging liquid (wetting agent), consisting primarily of polyvinylacetate with polystyrene. Application of the material was accomplished by cleaning the subject area with water pumps and stable brooms, and excess standing water was removed afterward with a compressor and air hose. The area was then spray-primed with a fine mist of gaging liquid. After the area was primed, a thin coat (1/8 in. to 1/4 in.) of the patch mortar was scrubbed into the primed area with stiff stable brooms. The area was then covered with more mortar and troweled to an even finish slightly above the exposed aggregate and steel in the eroded areas. With these repairs it was concluded that no further repairs would be necessary prior to the next inspection.

40. In 1974, the second periodic inspection (Vicksburg District 1974) indicated continued erosion of the concrete in the stilling basin

a. Exposed reinforcement at base of baffle

b. Temporary repair of erosion

Figure 12 Baffle erosion and temporary repair, Arkabutla Dam since the last inspection. The increase in eroded depth was estimated at approximately 1 in. The number of areas in the basin slab with exposed reinforcing had doubled since the 1969 inspection (Figures 13 and 14). The concrete patches made at the time of the initial inspection (shown at the base of the baffle on the left in Figure 15) appeared in good condition with little effect of erosion noted. Therefore, it was decided to patch the eight eroded areas shown in Figure 13 with the same material. Two baffles were also repaired in a similar manner.

41. In 1976 it was decided to restore the stilling basin floor slab and baffles to original grades and dimensions. After gate closure, a sandbag dike was built across the end sill and the stilling basin was unwatered by pumping. Silt and debris were removed from the area by hosing with water and using shovels. Loose concrete and previous temporary repairs were removed with jackhammers. After all surfaces to be repaired were sandblasted, they were thoroughly cleaned and dried prior to application of the epoxy bonding course.

42. The epoxy resin bonding material was mixed and applied to the receiving surfaces immediately prior to placement of the filler concrete. Stiff stable brooms were used to ensure complete coverage of the receiving surface with bonding material (Figure 16). Filler concrete proportioned with 3/4-in. maximum size aggregate, as shown in the following tabulation for 28-day compressive strength of 4000 psi, was placed (approximately 53 cu yd) while the bonding course was still wet.

Material	Saturated Surface Dry Weight, lb	Solid Volume cu ft
Portland cement, Type III	611	3.11
Fine aggregate	1203	7.36
Coarse aggregate	1932	12.39
Water	258	4.14
Water-reducing admixture	(52 oz)	--
Total	4004	27.00

43. After finishing was completed, the concrete was cured under polyethylene for 3 days. At the end of the curing period, the polyethylene was removed and the surface was lightly sandblasted to remove surface

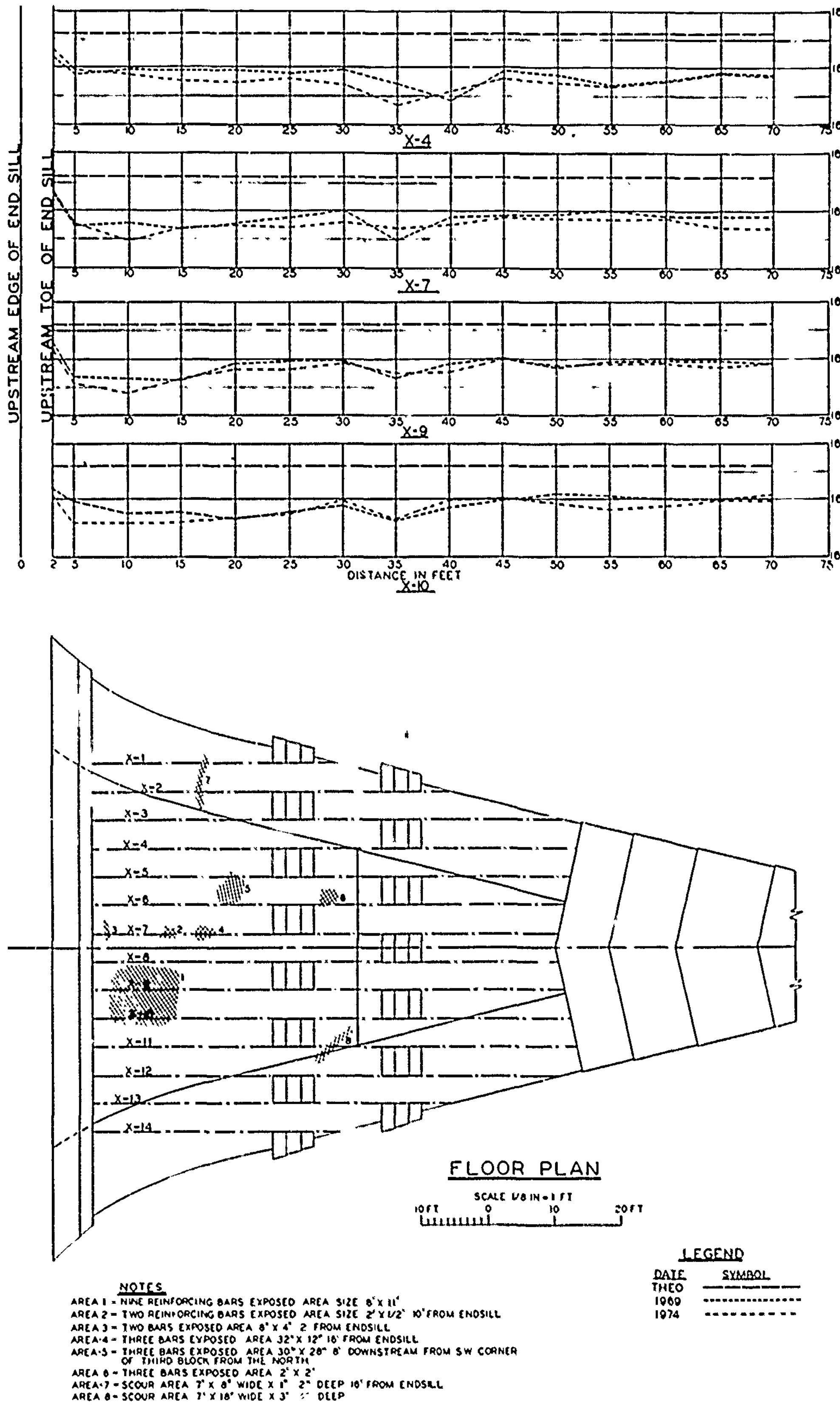


Figure 13. Cross sections of outlet works stilling basin, Arkabutla

Figure 14. Exposed reinforcement in the stilling basin downstream of the baffles, Arkabutla

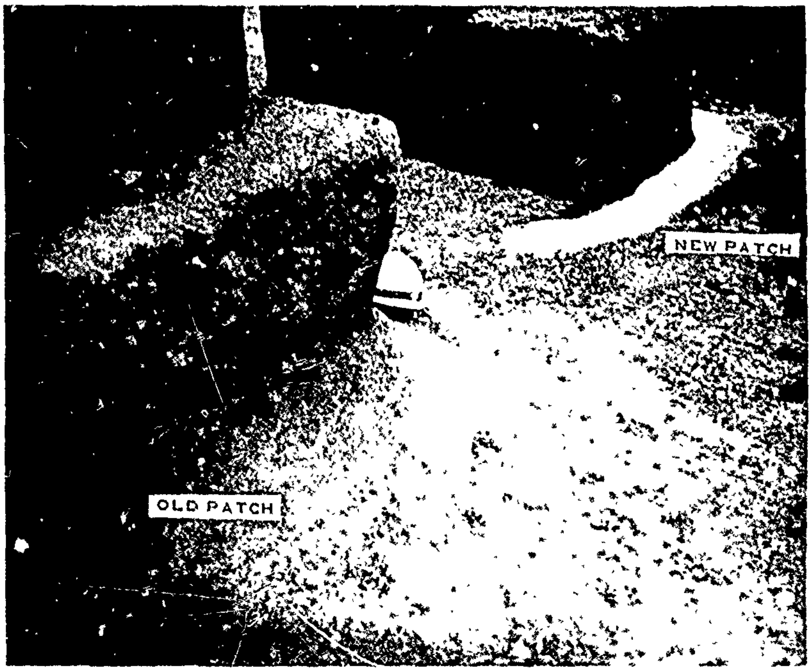


Figure 15. Stilling basin baffles showing old and new concrete patches, Arkabutla

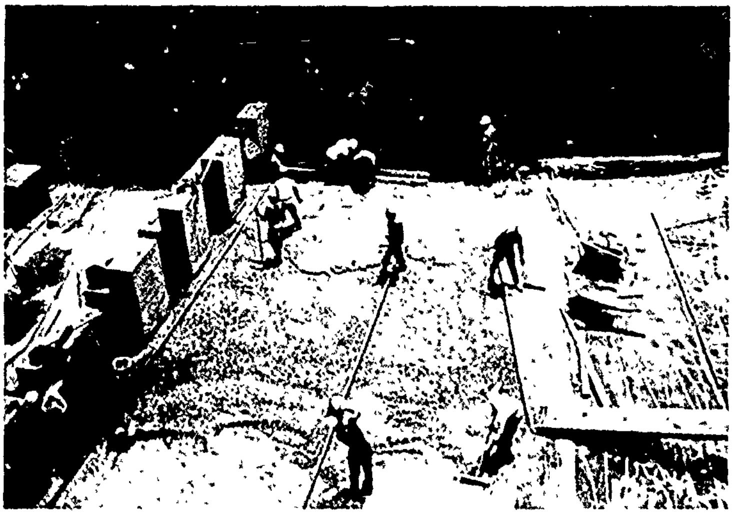


Figure 16. Stilling basin floor slab repairs, Arkabutla Dam

laitance and produced a roughened surface. A light prime coat of neat epoxy was applied to the thoroughly cleaned and dried surface and followed by a 1/4-in. epoxy mortar sealer and wearing course (Figure 17). The mortar, consisting of one part epoxy to three parts silica sand, was mixed in a 4.5-cu-ft mortar mixer. The mortar was finished to desired grade with steel trowels. Existing expansion joints were replaced with premolded, sponge rubber joint filler and extended upward to the new surface.

44. Surface preparation on the baffles was similar to that previously described for the basin floor slabs. Immediately after application of the epoxy bonding material, epoxy mortar was hand-trowelled onto uneroded areas of the baffles (Figure 18) to a minimum thickness of 1/4 in. On eroded areas, the thickness of coating was that necessary to restore the baffles to their original dimensions.

45. Repairs were completed at a contract cost of $36,873 excluding unwatering, which was performed by Vicksburg District personnel. The efficiency of these repairs will be evaluated during the next periodic inspection scheduled for July 1979.

Enid Dam

46. Enid Dam is a rolled-fill structure located in northwestern

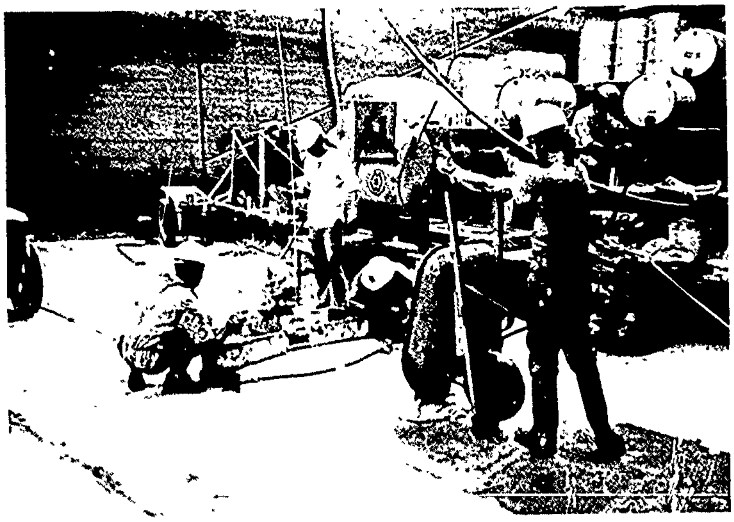


Figure 17. Application of epoxy mortar sealer and wearing course, Arkabutla Dam

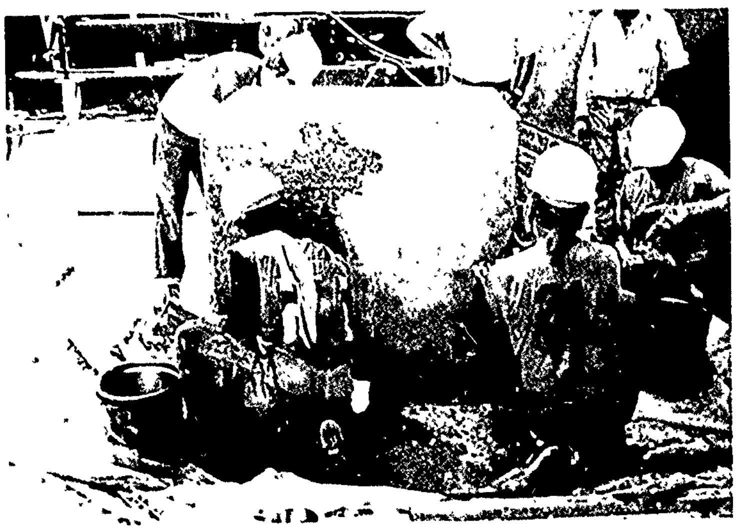


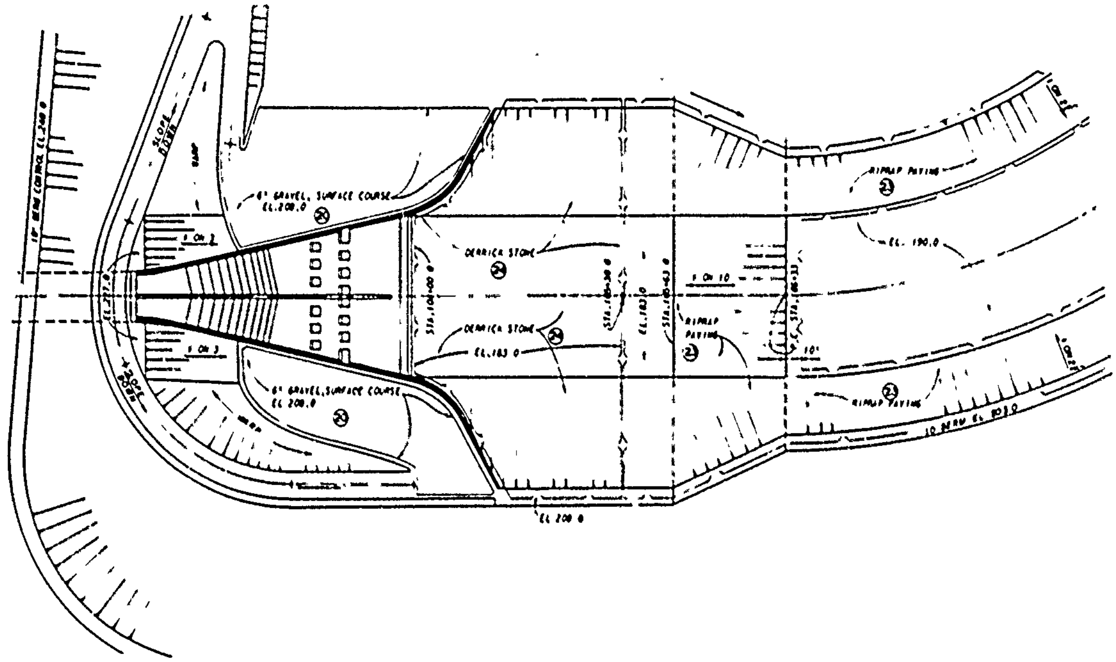
Figure 18. Epoxy mortar repair of baffle, Arkabutla Dam

Mississippi, approximately 12 miles south of Batesville on the Yocona River. The outlet works consist of a concrete approach, two-gated control structure, transition, two-barreled conduit, chute, and stilling basin. The design of discharge capacity is 9400 cfs.

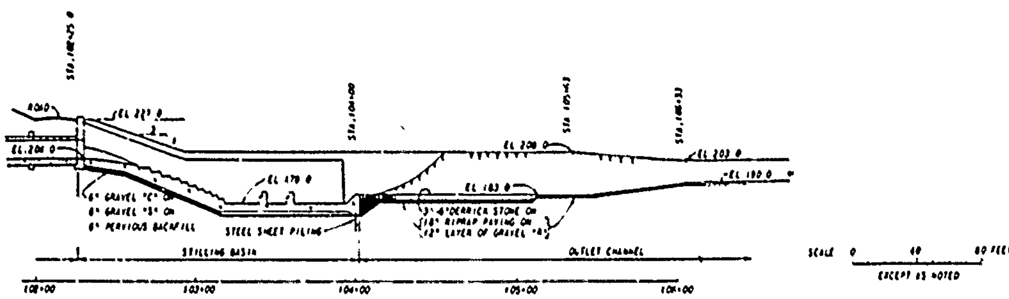
47. The stilling basin (Figure 19) is 72 ft long from headwall to end sill. A longitudinal splitter wall in the center of the structure, running from headwall to within 10 ft of the end sill, divides the basin into two symmetrical halves. The shape of the chute at the portal coincides with the inverts of the twin circular (11-ft diameter) conduit section, and the chute has a overall width of 30 ft. From el 204, the chute drops by steps to the stilling basin floor (el 178); it is 42 ft wide at the stilling basin. In each half of the basin, a smooth transition from semicircular to trapezoidal section is provided by means of constant radius fillets. The length of this transition is about 38 ft. The stilling basin is 95 ft wide at the sloping face end sill, which is 5 ft high. The stilling basin floor is surmounted by two rows of baffles 6 ft high and 5 ft wide spaced 10 ft on center.

48. The reinforced concrete stilling basin walls and base slab were designed as a framed structure. The base slab was investigated by the theory of an elastic beam resting on an elastic foundation. Wall stems were designed as cantilevers. Minimum thickness of the basin slab is 5 ft of 3000-psi design compressive strength concrete. The dam and appurtenances were placed in operation for flood control in December 1952. Major features of construction were completed in August 1955.

49. An informal inspection of the stilling basin in 1959 revealed a heavy deposit of silt, which was removed from the basin. The initial periodic inspection in 1968 (Vicksburg District 1968) revealed eroded and pitted surfaces of the stilling basin floor, steps, and baffle piers. The deepest penetration into the concrete was about 5-1/2 to 6 in. in the stilling basin floor in an area approximately 3-1/2 to 4-1/2 ft north and south, respectively, of the splitter wall and approximately midway between the lower step and first line of baffles (Figure 20). In the south passage seven reinforcing steel bars were exposed from 1 to 11 in. in length. In the north passage, four bars were exposed from



PLAN



PROFILE

Figure 19. Plan and profile of outlet works stilling basin, Enid Dam

Figure 20. Erosion in north passage of stilling basin, Enid Dam

1 to 7 in. All joints appeared to be in good condition and no deep cracks were observed. There was no visible deflection in wing walls or center wall.

50. Areas of most advanced erosion were repaired (Figure 21) at the time of the 1968 inspection with a temporary protective coating. The material and method of application was the same as previously described for the Arkabutla stilling basin. Although erosion on the steps was not as extensive as that on the stilling basin floor, two test patches were made on the seventh step, el 192, in both the north and south passages. These patches were exposed each time the gates were closed so that durability could be monitored.

51. In 1973, the second periodic inspection of the stilling basin (Vicksburg District 1973) indicated continued erosion of the concrete at an increasing rate. The temporary repairs to the basin floor in 1968 had eroded except for a few small areas.

52. In both the north and south portions of the stilling basin

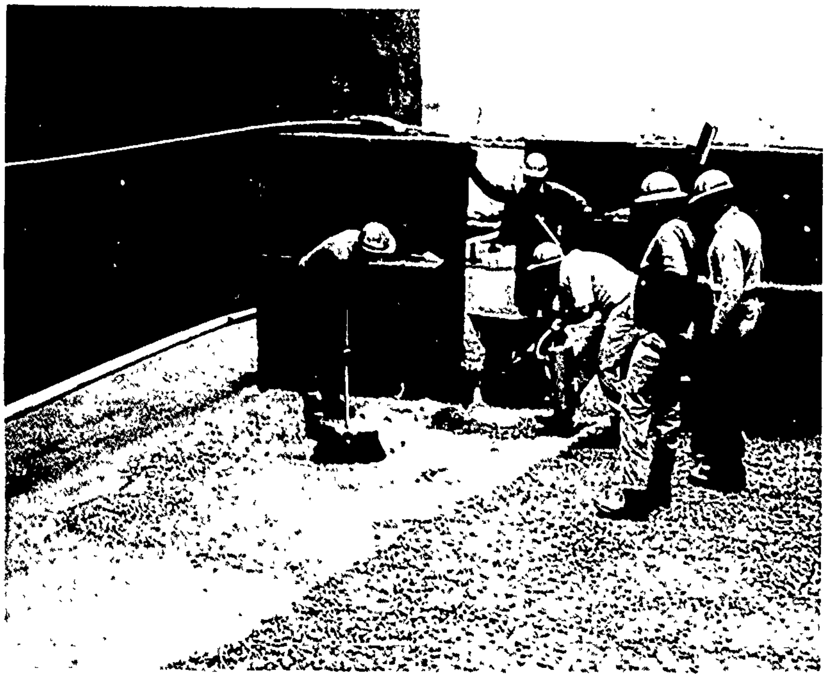


Figure 21. Temporary repairs to stilling basin, Enid Dam

the major areas of erosion were located approximately 5 to 6 ft from the splitter wall and midway between the lower step and first row of baffle piers (Figure 22). In addition, there were two deeply eroded areas in each passage, approximately 3 ft from the splitter wall and midway between the two rows of baffles. Maximum depth of erosion was approaching 1 ft. Nine pieces of reinforcement were exposed in the south passage, ranging from 2 to 14 in. in length. Also, in the north passage, six reinforcing bars were exposed, ranging in length from 2 to 12 in. Until permanent repairs could be made, these areas with exposed reinforcement were temporarily repaired with the protective coating previously used.

53. In 1976, permanent repairs were made to the stilling basin floor slab and baffles. This work was accomplished under the same contract with essentially the same materials and techniques previously described for Arkabutla. The stilling basin slab was restored to original grade with an epoxy-bonded unreinforced concrete overlay (approximately 93 cu yd) and epoxy mortar sealer and wearing course (Figure 23). Strength gain characteristics of the filler concrete are shown in

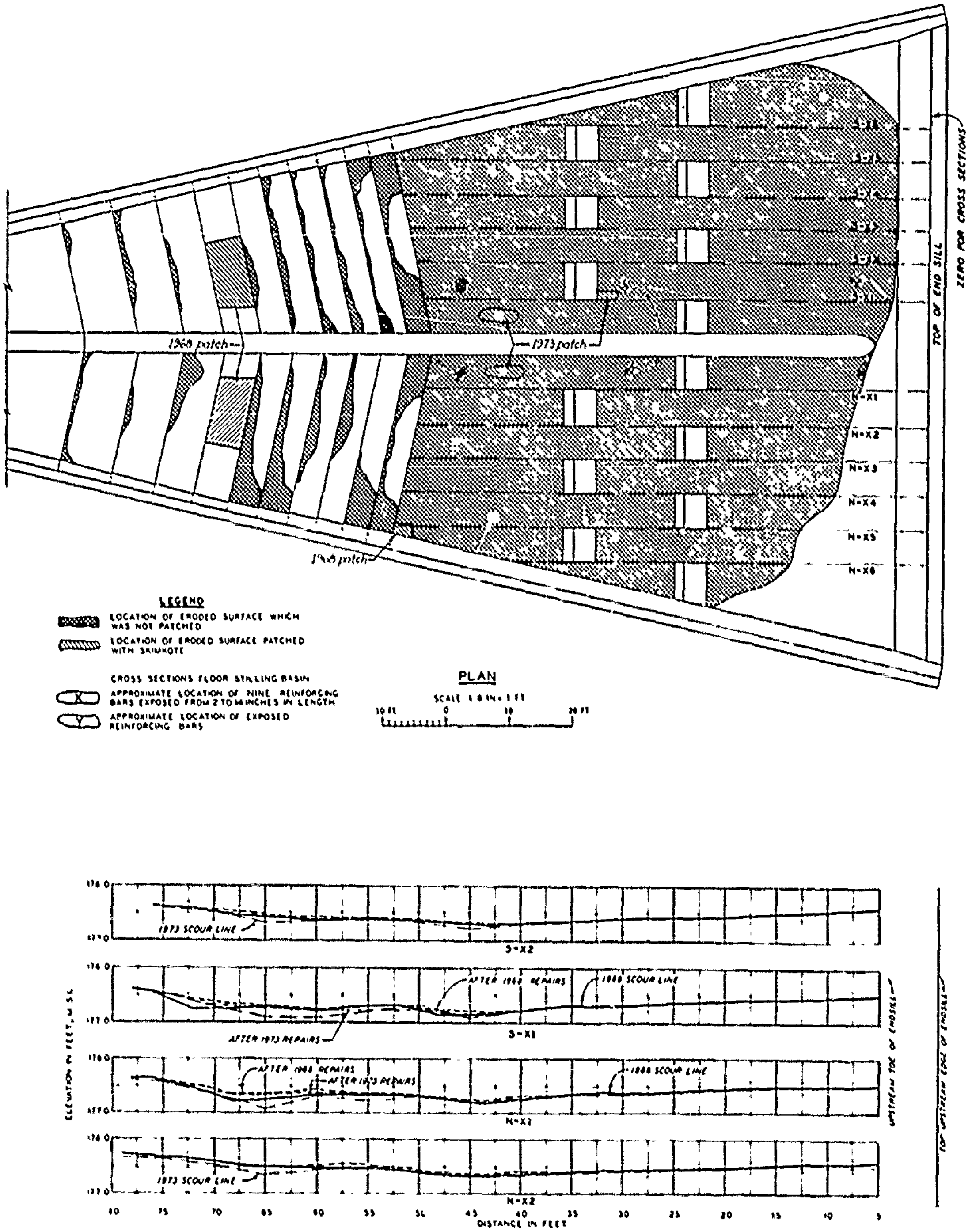


Figure 22. Cross sections, outlet works stilling basin, Enid Dam

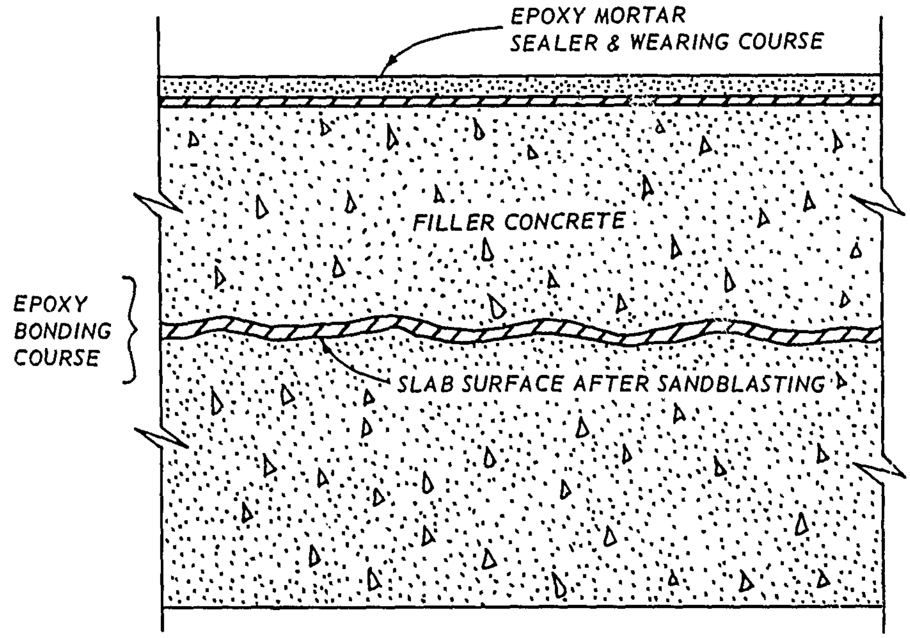


Figure 23. Stilling basin slab repair, Enid Dam

Figure 24. Contract cost of repairs, excluding unwatering accomplished by personnel of the Vicksburg District, was $48,810.

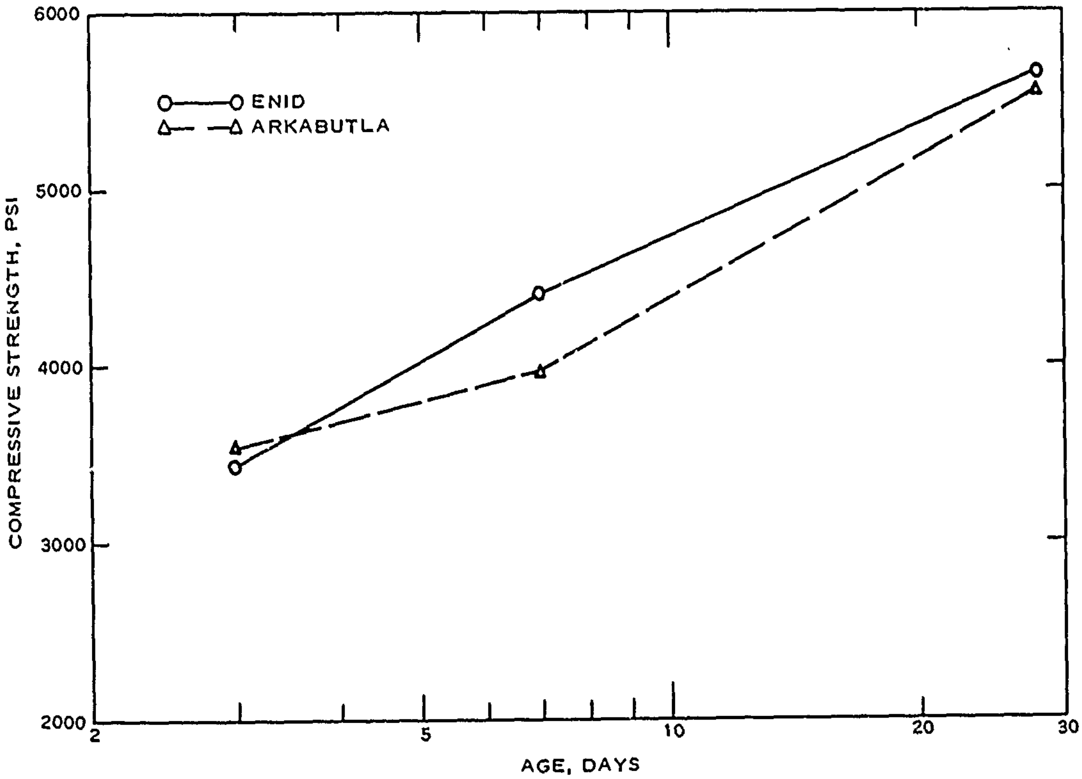


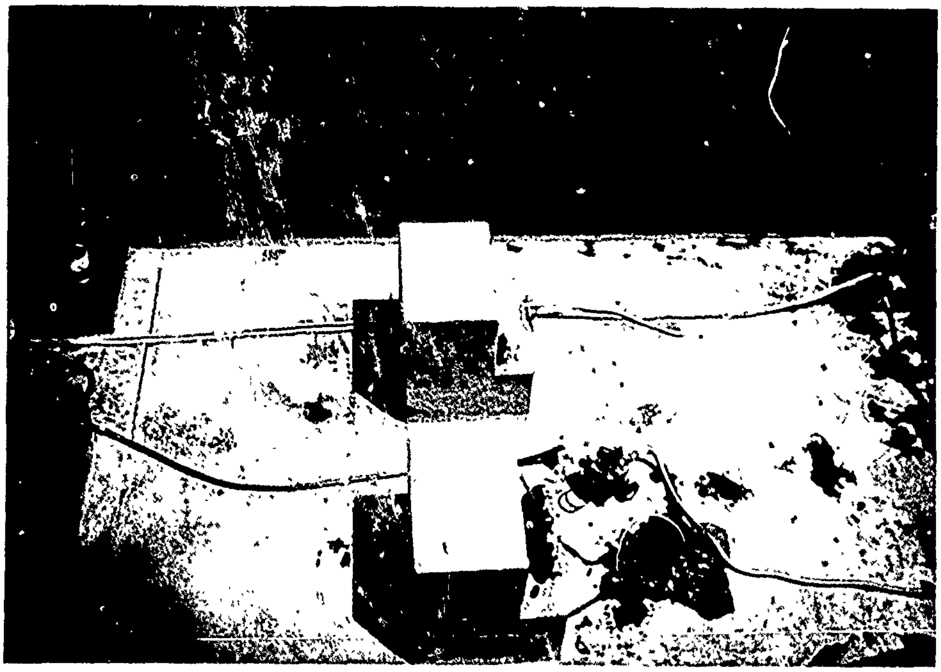
Figure 24. Concrete strengths, Arkabutla and Enid Dams

54. The stilling basin was unwatered for inspection in August 1978, approximately 2 years after completion of repairs. The inspection indicated an excellent bond between the epoxy mortar and fill concrete. With the exception of a relatively small area on each side of the longitudinal splitter wall, the epoxy mortar exhibited good resistance to abrasion erosion. In both cases, erosion occurred in a generally circular pattern (Figure 25) around the upstream baffle nearest the splitter wall. The erosion pattern generally coincided with the areas of maximum erosion prior to repair. The maximum depth of erosion, which occurred in the north passage, was approximately 1/2 in. (Figure 26). The condition of the mortar due to moisture seepage at construction joints was of some concern during the repair. However, the mortar in these areas appeared to have exhibited the same behavior as that elsewhere, with the exception of the joint shown in Figure 26. The original erosion along this joint can be seen in Figure 20. The baffles were in excellent condition with no evidence of erosion.

Pomme de Terre Dam

55. The Pomme de Terre Dam is located on the Pomme de Terre River near Hermitage, Missouri. The outlet works include a reinforced concrete transition section and hydraulic jump stilling basin of natural rock. The design discharge velocity is 74.3 fps. The dam was constructed in 1959, and diversion and storage began in July 1960 and October 1961, respectively.

56. The transition and stilling basin are cut through natural rock. The walls of the 90-ft transition section (Figure 27) are lined with 2-ft-thick reinforced concrete anchored to the surrounding rock. The primary purpose of the transition slab, a 3-ft-thick reinforced concrete slab anchored to the foundation rock, is to prevent undue erosion of the foundation and undercutting of the basin walls and the tunnel. Two concrete mixtures were used. The first layer, containing 3-in. MSA

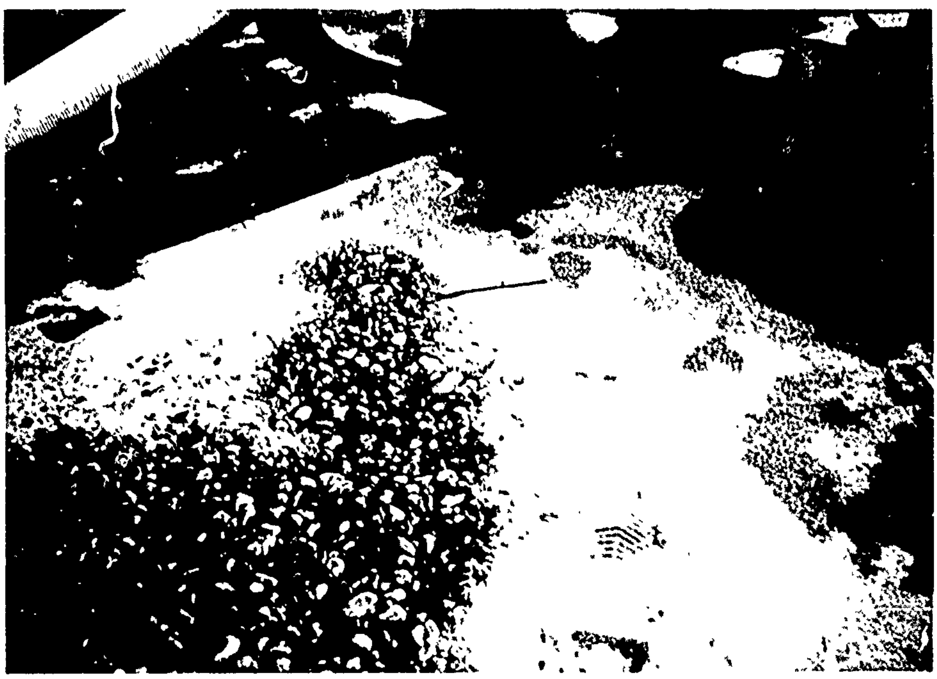


a. Overall erosion pattern

b. Erosion along splitter wall between step and upstream baffle

Figure 25. Erosion of repair in south passage, Enid Dam

a. General view of erosion



b. Close-up view of erosion

Figure 26. Erosion in the north passage, Enid Dam

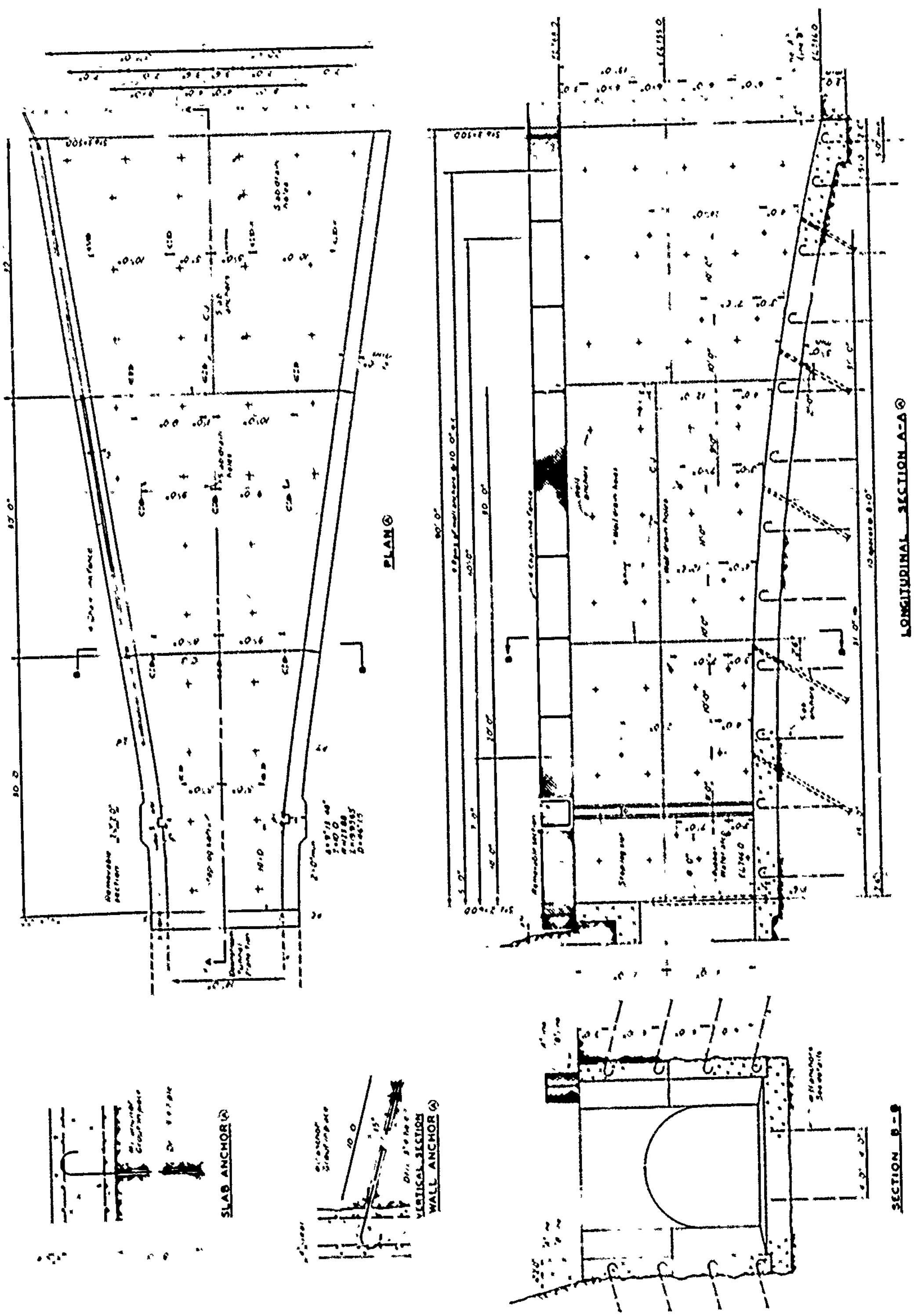


Figure 27. Transition section, Pomme De Terre Dam

and 376 lb/cu yd of cement, was designed for 3000 psi compressive strength. The top layer (12-in. thickness) was proportioned with 0.46 water-cement for 4000 psi as follows:

Material	Weight, lb
Portland cement, Type II	532
Kansas River natural sand	1138
Burlington limestone,	
No. 4 - 3/4-in.	1066
3/4 - 1-1/2-in.	1061
Water	245

Job mixture cylinder strengths ranged from 3325 to 5100 psi with an average of 4191 psi. Air content and slump averaged 3.3 percent and 2-1/2 in., respectively.

57. The stilling basin is 40 ft wide and 200 ft long (Figure 28). The basin has a three-step, 13-ft-high end sill. The basin floor and walls are natural rock.

58. The initial dewatering of the basin was during the first periodic inspection in October 1965. Minor wear at the downstream end of the transition slab had exposed aggregate. Small, fist-size, well-rounded rocks were found just downstream of the transition slab. The natural rock basin floor exhibited erosion of about 1.5 ft.

59. The basin was dewatered again in March 1971 as part of the second periodic inspection. The most significant erosion was on the right downstream end of the transition slab (Figure 29). The wear in this area was 4 to 12 in. Some rebars were exposed and a few rebars had been removed by the erosion forces. However, since the slab was not contiguous with the walls and only protected the foundation rock, the eroded slab did not constitute an immediate threat to the integrity of the structure.

60. The third dewatering of the basin occurred during the third periodic inspection in October 1976. Additional erosion was observed on the transition slab. The major erosion was still on the right downstream portion of the transition slab; however, erosion on the left

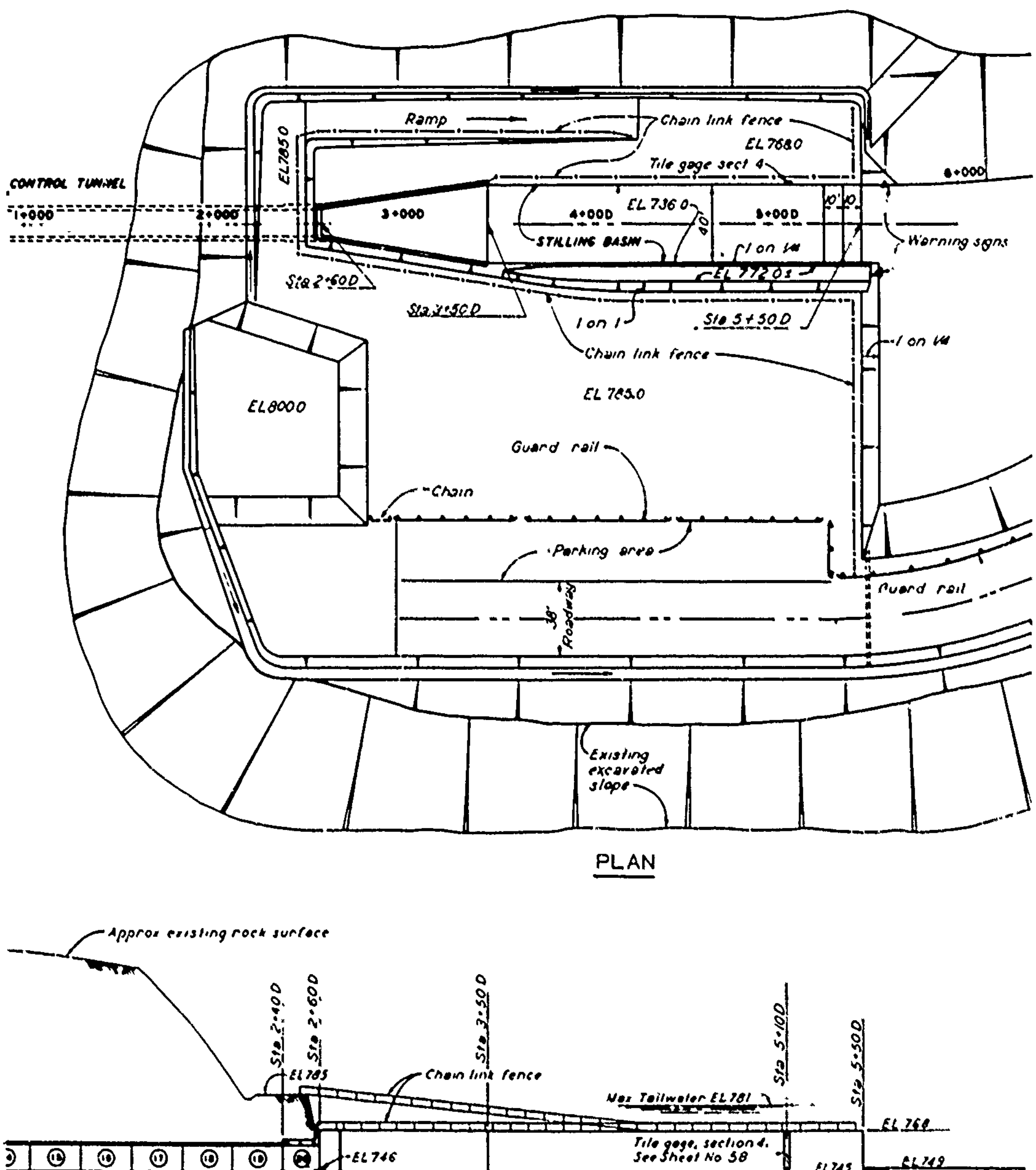


Figure 28. Outlet works, Pomme De Terre Dam

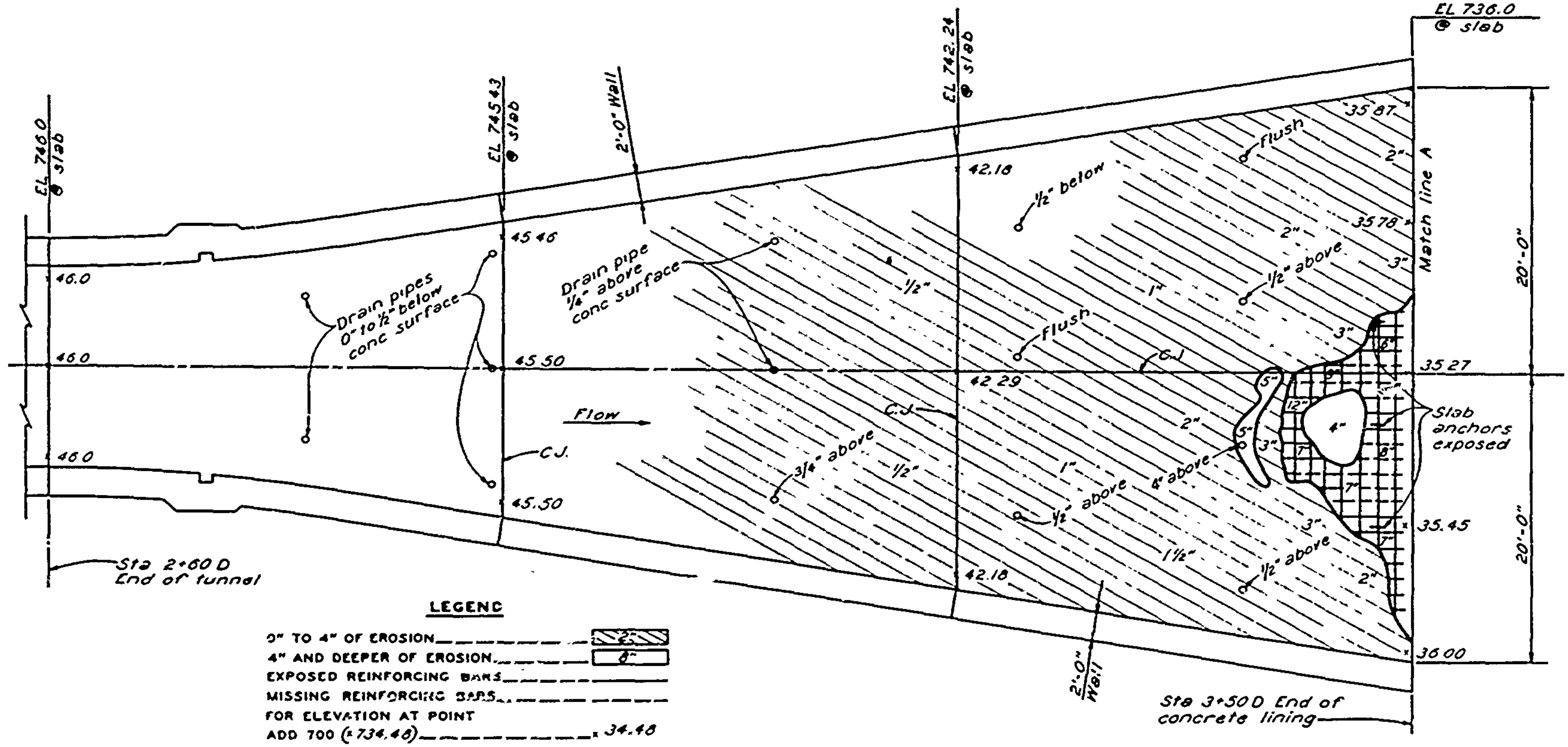


Figure 29. Erosion of transition section, Pomme De Terre Dam

downstream portion of the transition slab had exposed anchors and a few rebars.

61. A temporary repair was made when the basin was dewatered for the 1976 inspection. Sandbags diverted gate leakage to the right edge of the transition slab. Seeping slab drains were plugged or the seepage was piped over the repair area. Concrete in the downstream 12 ft of the slab was removed in those areas where the depth of cover was less than 2 in. The concrete was removed to a depth of 2 in. below reinforcing steel. Reinforcing steel removed by erosion forces was replaced. In areas where the concrete had not been removed by chipping, the surface was cleaned by sandblasting.

62. The repair concrete mixture was proportioned using materials similar to the original concrete for 5000 psi compressive strength as follows.

Material	Weight, lb
Portland cement, Type II	680
Kansas River natural sand	1240
Burlington limestone, No. 4 - 3/4-in.	1640
Water	279

Slump and air content of this mixture averaged 2-1/2 in. and 4.4 percent, respectively. Compressive strengths at 28 days ranged from 4560 to 5640 psi with an average of 5085 psi.

63. The Kansas City District believes the rate of erosion in the Pomme de Terre basin is basically slow, and when good concrete strength is provided, the repair should provide good service. However, the repair, which cost approximately $6000, does not reestablish the original slab conditions. The repaired clear cover over the reinforcing steel averaged about 3 in., or one half the original cover. Minimum cover in the repair area was 2 in. To provide a 6-in. cover would require a much larger overlay and would probably require a change in the transition profile curve. The amount of work and effort to expand the repair is not directly related to the work required to make the temporary repair. An

expanded repair requires the following:

- a. Additional time, which includes time for design of the transition profile, anchors, and methods to handle drainage.
- b. Provisions for minimum flow.
- c. Provisions for handling, for extended time, surface drainage into the basin, and any backwater that may flood the basin.
- d. Provisions to carry gate leakage over the repair area (current repair diverted the gate leakage to a portion of slab not re-repaired).
- e. Provisions to insure the overlay does not fail in bond. (With a larger area of overlay, the chances increase for a localized bond failure to progress and free a large segment of the overlay, which when free may cause additional damage in the basin.)

Probably future discharges and the amount of debris in the basin will be similar to past conditions. The strength of the concrete repair is probably as good as, and may be stronger than, the original concrete strength. The existing slab surface was prepared to provide an adequate bond between the existing concrete and the overlay. Assuming future erosion rates are similar to those in the past and that bond failure does not occur, the Kansas City District estimates that reinforcing steel in the repair area and just upstream of the repair will be exposed in 5 to 7 years.

64. The stilling basin is scheduled for dewatering during the next periodic inspection in 1981.

Pomona Dam

65. Pomona Dam was constructed in 1960 on the Hundred and Ten-Mile Creek near Vassar, Kansas. Diversion flow began in July 1962 and lake storage began in October 1963. The reinforced concrete transition and stilling basin has a design discharge velocity of 57.8 fps. The hydraulic jump stilling basin is 35 ft wide and 80 ft long (Figure 30). The basin is of U-wall design in which the basin walls are structurally continuous with the basin slab. Two staggered rows of baffles, 3 ft wide

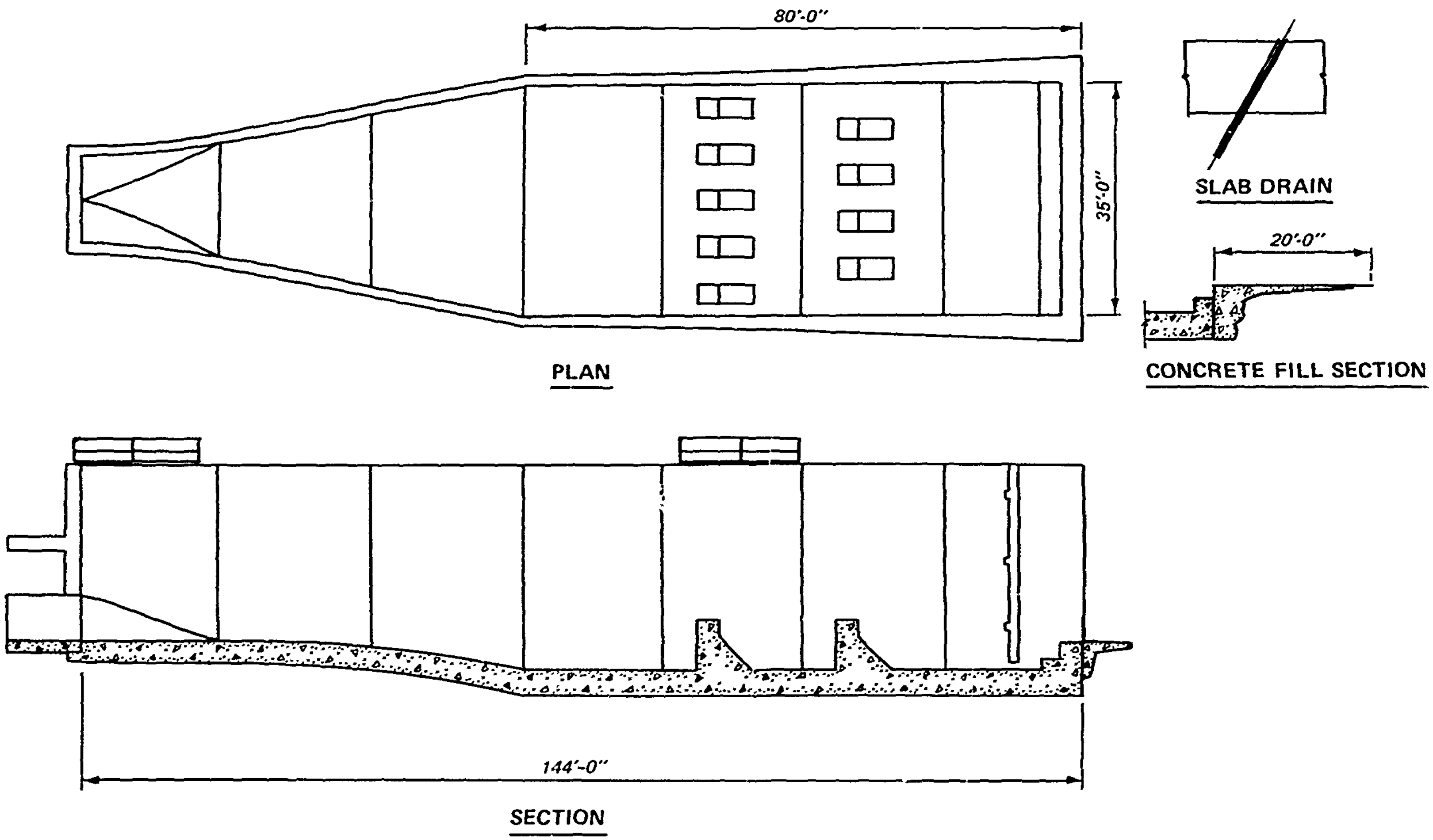


Figure 30. Stilling basin plan and section, Pomona Dam

and 5 ft high are spaced 7 ft center to center. A two-step vertical face end sill is 4 ft high. Fill concrete was placed the width of the stilling basin for a distance of 20 ft downstream from the end sill.

66. Concrete materials and mixture proportions were as follows.

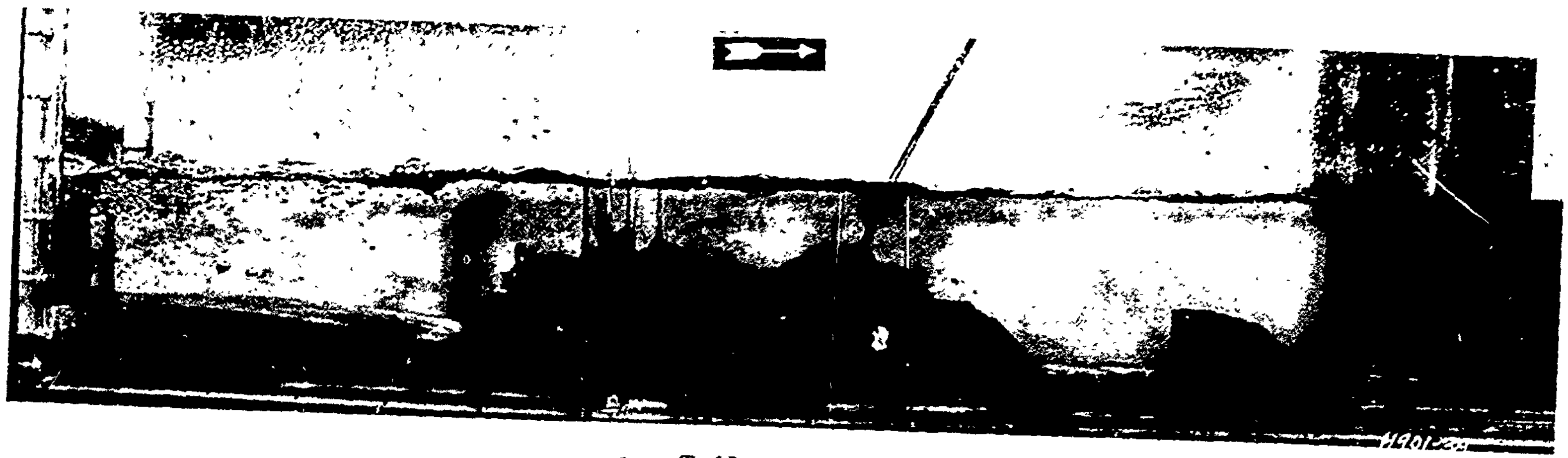
Material	Weight, lb 2-C	2-E	2-F
Portland cement, Type II	527	526	426
Kansas River natural sand	1130	1070	1120
Stoner limestone,			
No. 4 - 3/4-in.	1158	1027	1049
3/4 - 1-1/2-in.	1024	1132	1016
Water	227	216	179

Average test results were as follows:

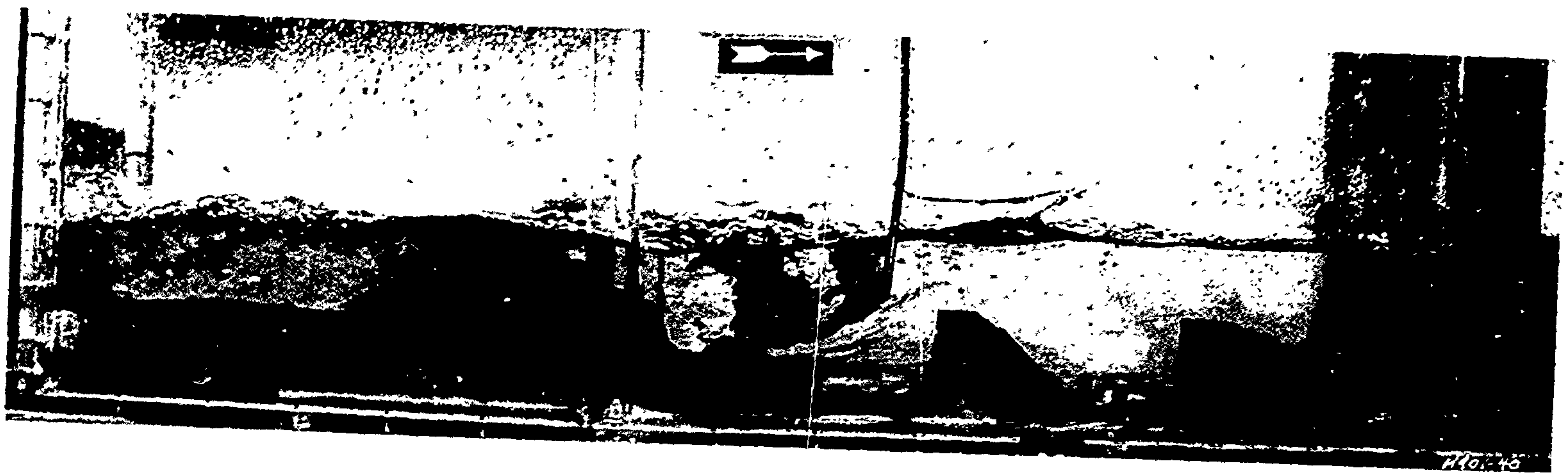
Test	2-C	2-E	2-F
Slump, in.	1.5	1.5	1.25
Air content, percent	4.0	4.9	4.6
28-day compressive strength, psi	5638	5625	5023

67. The initial dewatering of the basin was made in February 1968 as part of first periodic inspection. Erosion caused by the abrasive action of rocks and other debris had occurred at the downstream end of the transition slab and on the upstream one third of the basin slab. Reinforcing steel was exposed in the upstream left third of the basin and just upstream of the baffles. A supplemental inspection of the stilling basin was made in October 1970. The inspection revealed significant additional concrete erosion and extensive reinforcing steel exposure. The major damage was attributed to the flow conditions caused by relatively low discharges, since approximately 97 percent of the releases have been 500 cfs or less.

68. Model tests of the existing stilling basin (Oswalt 1971) verified that severe separation of flow from one sidewall and eddy action, strong enough to circulate stone in the model, within the basin occurred for low and intermediate discharges and tailwaters common to the site. Photographs (Figures 31 and 32) of subsurface upstream flow in the right

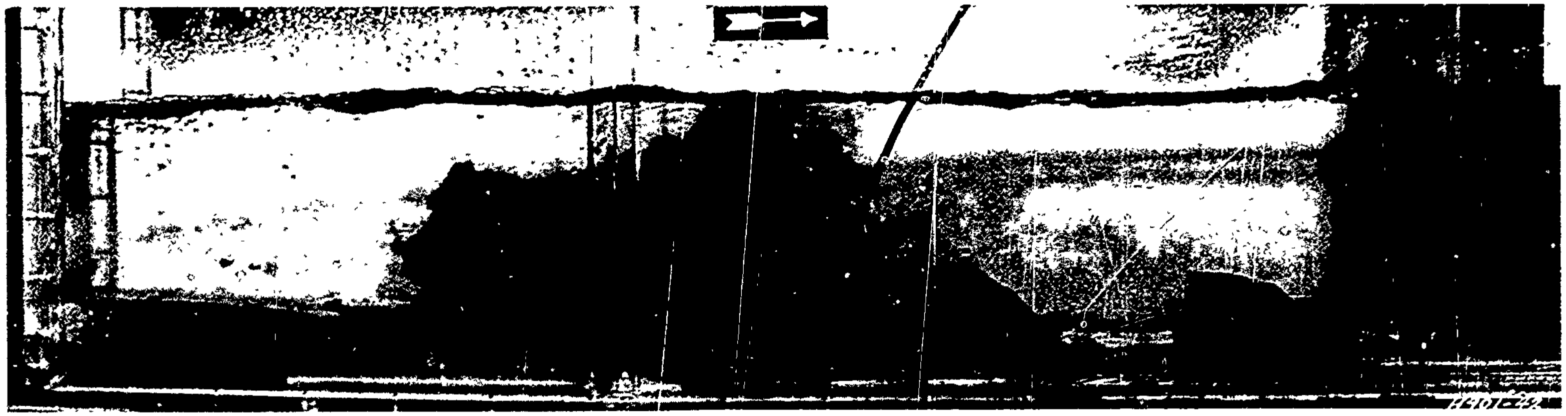


a. Tailwater el 930.0

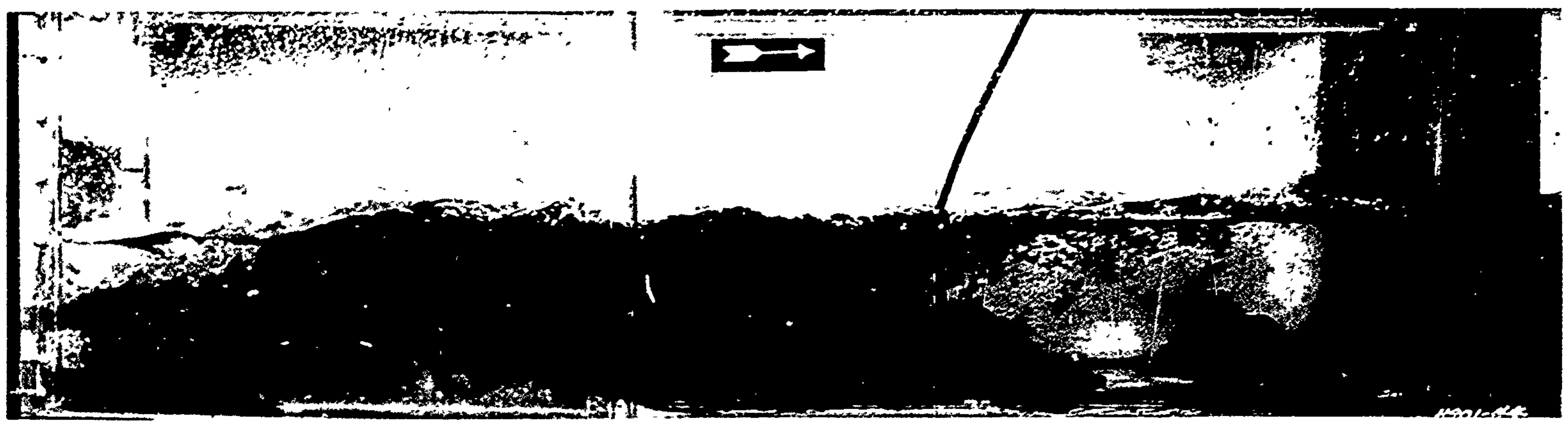


b. Tailwater el 927.0

Figure 31. Stilling basin flow conditions at pool el 974 and discharge 500 cfs, Pomona Dam (from Oswalt 1971)



a. Tailwater el 933.0



b. Tailwater el 928.5

Figure 32. Stilling basin flow conditions at pool el 974 and discharge 1000 cfs, Pomona Dam (from Oswalt 1971)

side of the basin show the results of discharge rates of 500 and 1000 cfs. It is apparent that the abrasion damage observed in the prototype stilling basin was a result of the eddy action, and debris and rock were always found when the basin was dewatered. Although it is possible that some of the visiting public might have thrown rock into the basin, the model indicated that with a flow of about 4200 cfs, the eddy within the basin was sufficiently large and strong to generate considerable reverse flow from the exit channel into and along one side of the basin. It is possible that this return flow could transport riprap from the exit channel into the basin, particularly with the stepped, vertical end sill provided originally.

69. Various modifications including raising the apron, installing chute blocks, constructing interior side walls with lesser flare, and providing a raised hump downstream of the outlet portal were investigated to evaluate their effectiveness in eliminating the undesirable separation of flow and eddy action within the basin. Based on these tests, it was recommended that the most practical solution was to provide a 3-ft-thick overlay of the basin slab upstream of the first row of baffles; a 1-1/2-ft overlay to the basin slab between the two rows of baffles; and a 1 on 1 sloped face to the existing end sill. This solution provided a wearing surface to the area of greatest wear and provided a depression at the downstream end of the basin for trapping rocks. However, flow separation and eddy action were not eliminated by this modification, and damage would be expected to continue to occur if rock and debris were present. Therefore, it was recommended that a fairly large discharge sufficient to create a good hydraulic jump without eddy action be released periodically to flush the rock from the basin. Guidance as to the discharge and tailwater relations required were provided.

70. The basin was dewatered for inspection and repair in October 1972. Additional erosion was observed; however, the rate of erosion appeared to be less. The reduced rate of erosion was attributed to a decrease in the number of days with discharges above 500 cfs. The erosion observed before the repair is shown in Figures 33 and 34.

71. During repairs to the structure, minimum downstream flow

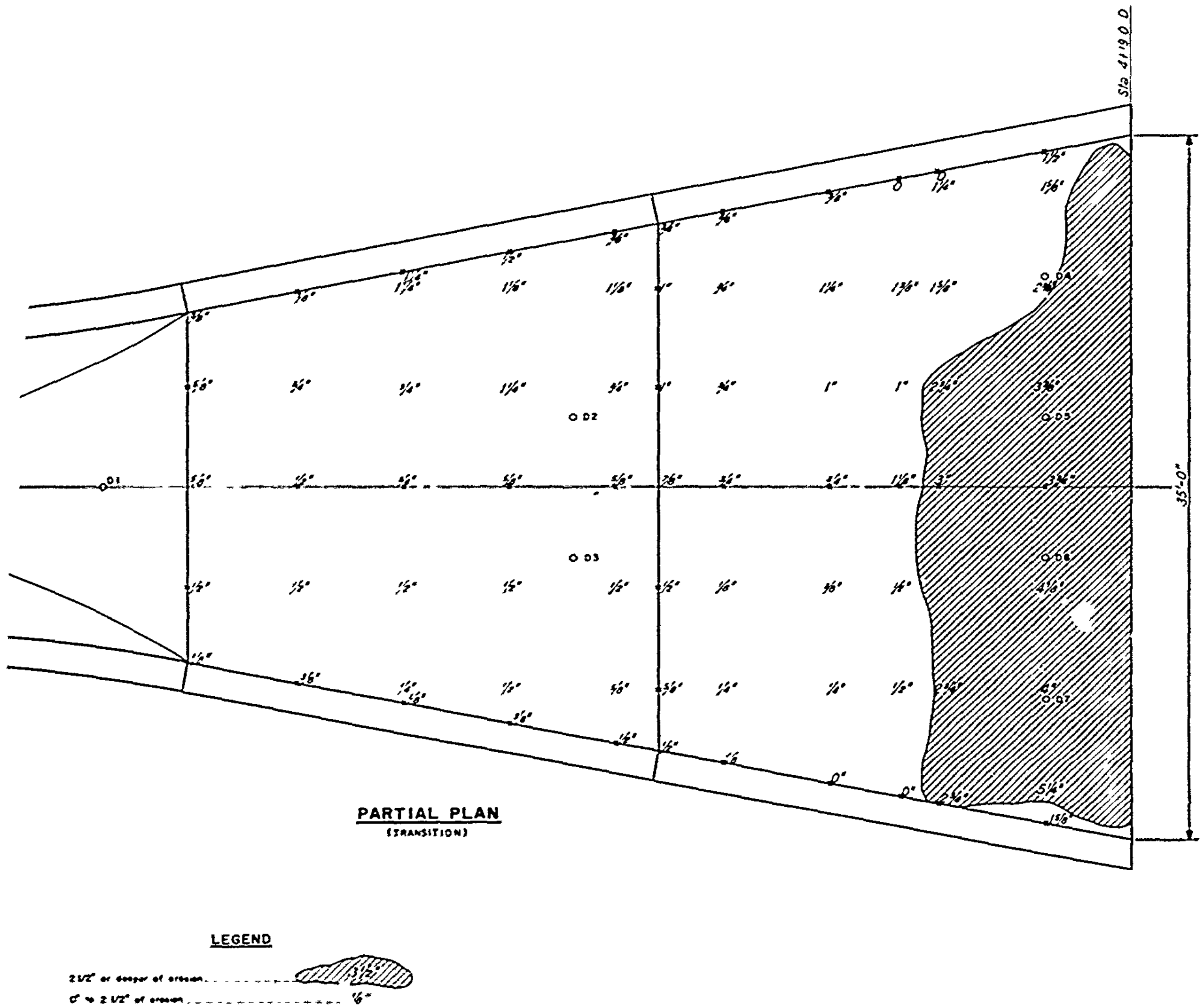
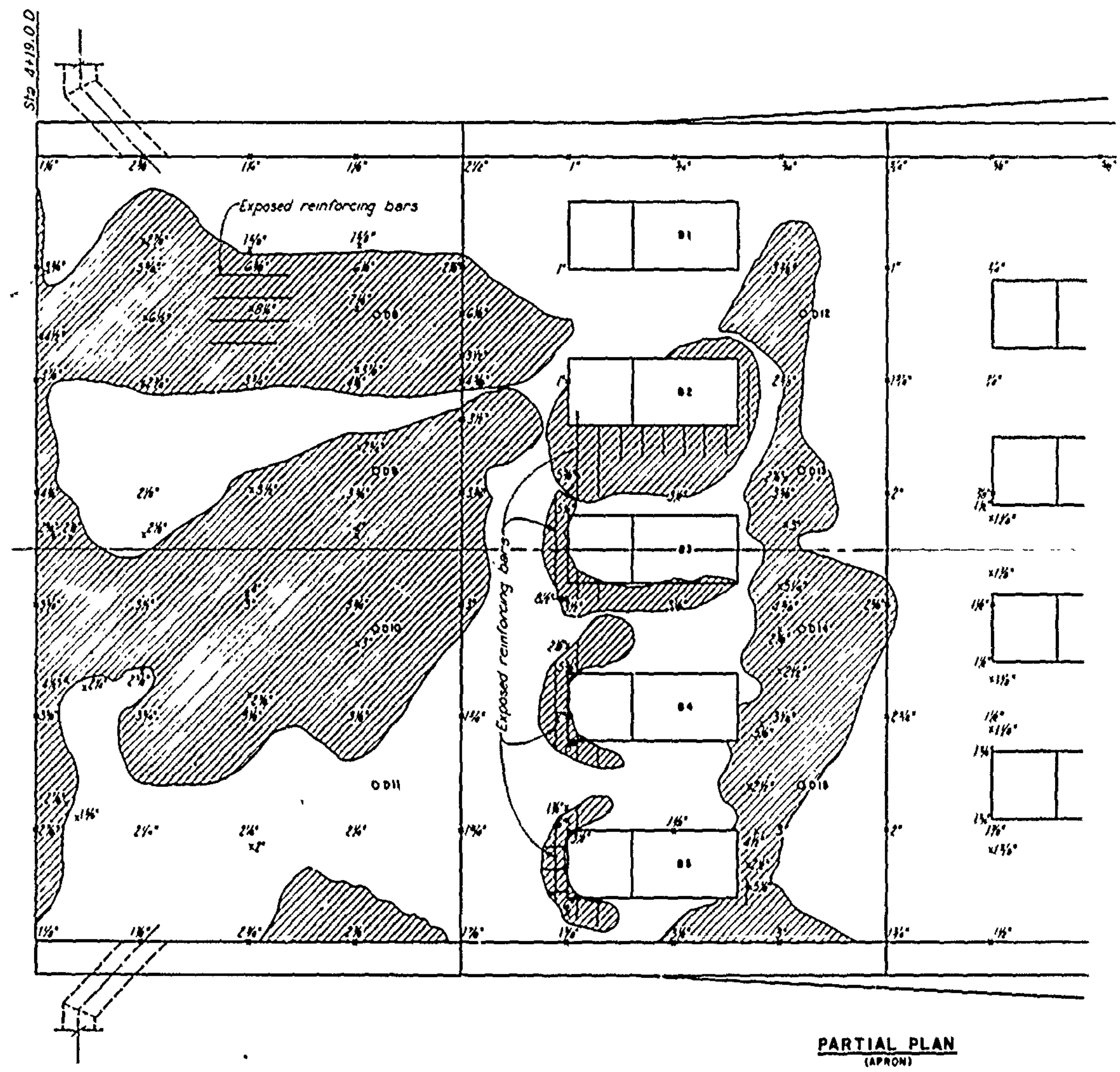


Figure 33. Transition erosion, Pomona Dam



PARTIAL PLAN
(APRON)

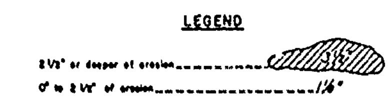


Figure 34. Apron erosion, Pomona Dam

requirements were fulfilled by pumping lake water through the spillway. The pumping was accomplished by Corps of Engineers field personnel. The repair, including dewatering, debris removal, and the control of leakage, seepage, and surface drainage, was accomplished by contract. The repair included (a) a 1/2-in. minimum thickness epoxy mortar applied to approximately one half of the transition slab; (b) an epoxy mortar applied to the upstream face of the right three upstream baffles; (c) a 2-ft-thick concrete overlay slab placed on the upstream 70 percent of the basin slab; and (d) a sloped concrete face added to the end sill.

72. The surfaces to receive the epoxy mortar were cleaned by sandblasting to expose approximately 50 percent aggregate. The cleaned surface was primed with epoxy resin binder just prior to placement of the epoxy mortar, which consisted of approximately five parts silica sand to one part epoxy binder. Specifications required the epoxy coatings to be kept dry and above 60°F for a period of one week.

73. During the early stages of epoxy application, the epoxy mortar set quickly and left part of the surface uneven. A portion of the epoxy mortar surface below the required elevation was recovered with a thin, 1/8-in., epoxy mortar overlay. During a period of rainy weather, backwater flooded the epoxy overlay. At the time of flooding, approximately 80 percent of the epoxy mortar was placed and had two days of curing with dry conditions and temperatures of about 60°F. After the backwater was pumped from the basin and the slab was cleaned, it was observed that two areas of the epoxy mortar were soft, and they were removed. One of these areas was adjacent to a vertical construction joint in the existing slab. Moisture from seepage through the joint apparently prevented the epoxy from setting. The construction joint was saw cut about 2 in. deep and 1/8 in. wide. Fine sand was placed in the saw cut to serve as a drainage medium and 1/8-in.-diameter plastic tubes were positioned at 6-in. to 2-ft intervals to carry seepage from the joint. The area near the joint was air dried and a quick-setting, moisture-compatible epoxy was applied to the joint. However, several small areas along the joint continued to develop leaks. Patching of these areas was required before a satisfactory overlay was completed. Due to the problems of moisture

seepage, it is very desirable to use an epoxy that will harden when moisture is present.

74. The concrete overlay, a reinforced concrete slab, was anchored to the original basin slab. The upstream end of the overlay slab was recessed into the original transition slab (Figure 35). Iron Mountain trap rock, an abrasion-resistant coarse aggregate, was used. Concrete materials and mixture proportions for the repair concrete were as follows.

Material	Weight, lb
Portland cement, Type II	658
Kansas River natural sand	1064
Iron Mountain trap rock	
No. 4 - 3/4-in.	943
3/4 - 1-1/2-in.	1034
Water	270

Slump and air content of this mixture averaged 2.1 in. and 2.8 percent, respectively. Compressive strengths at 28 days ranged from 5785 to 7380 psi, with an average of 6790 psi.

75. The repair was made between November 1972 and January 1973. Bid costs were as follows:

Dewatering	lump sum	$20,800
Extension of Drain Pipes	lump sum	3,168
Extension of 18-in. Pipe and Grates	lump sum	1,225
Abrasion-resistant Concrete	206 cu yd	18,540
Reinforcing Steel	18,920 lb	8,136
Saw Cutting	40 lin ft	120
Concrete Chipping	260 sq ft	520
Epoxy Mortar	725 sq ft	7,250
Total Bid Costs		$59,759

The cost to provide low flow by pumping was estimated at approximately $9000.

76. The basin was dewatered in April 1977 as part of the third periodic inspection. The depression at the downstream end of the overlay slab, which serves as a rock trap, appeared to function well. Most of

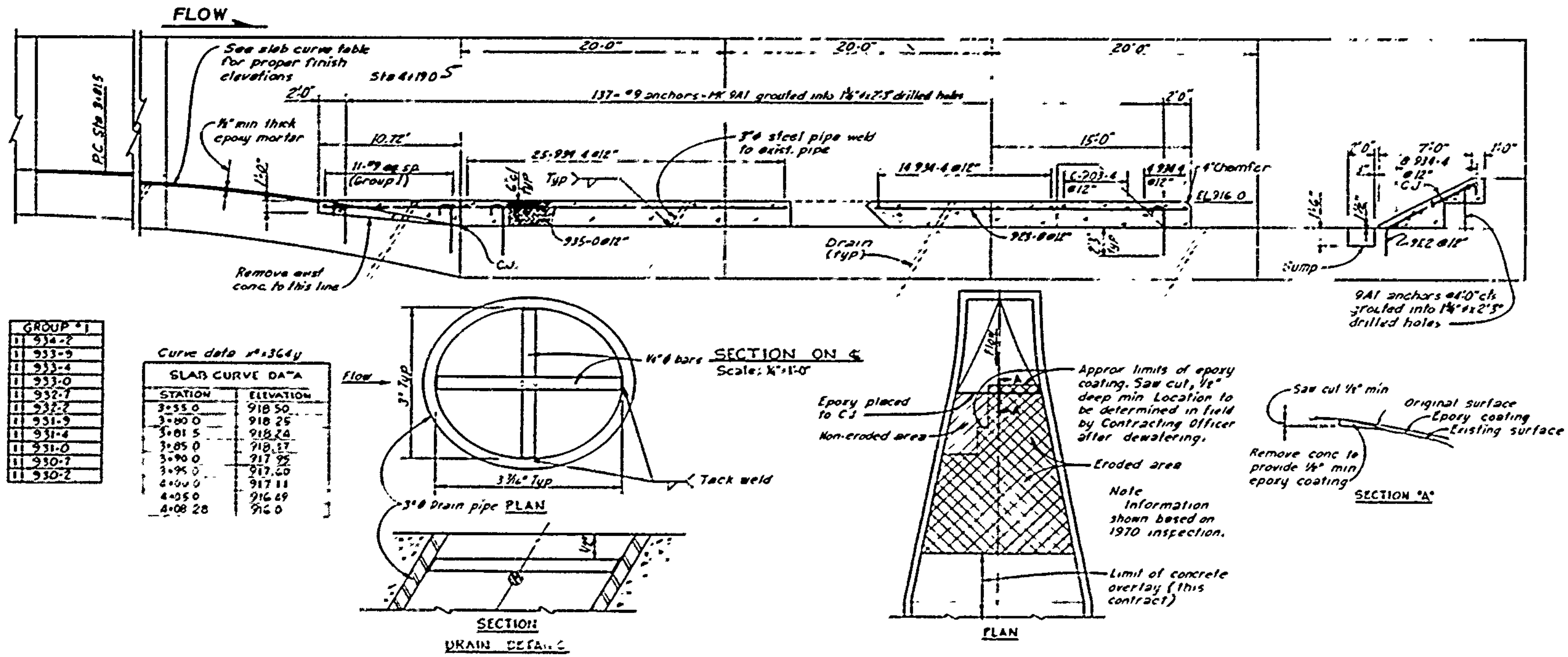


Figure 35. Repair details, Pomona Dam

the rocks, approximately 1 cu yd, were found in the trap and were adjacent to the overlay slab.

77. The concrete overlay had suffered only minor damage. Generally, the erosion depth was about 1/8 in. with maximum depths of 1/2 in. The major wear pattern was on the left third of the overlay slab and was in the upstream half of the basin. The location of wear, the same as the major wear before placement of the overlay slab, indicates rocks are being circulated at some discharge rate. Selected discharge rates with the percent of time and the number of days the basin has operated at these rates for the periods of time between dewaterings are as follows:

Discharge Rate	Feb 68 to Oct 70	Oct 70 to Nov 72	Jan 73 to Mar 77
Less than 500 cfs	829 days, 86.4%	738 days, 96.9%	1271 days, 83.7%
500 to 2000 cfs	110 days, 11.5%	24 days, 3.1%	221 days, 14.5%
More than 2000 cfs	21 days, 2.1%	0 days, 0.0%	27 days, 1.8%

78. The percent of time that the basin operated at the selected rates for the period from January 1973 to March 1977 is about the same as for the period from February 1968 to October 1970. The days operated at the selected rates for flows up to 2000 cfs are significantly greater for the period from January 1973 to March 1977. However, the wear on the basin slab was generally only 1/8 in., 1/2-in. maximum, during January 1973 to March 1977, whereas the wear from February 1968 to October 1970 was generally 1 in. deep with points of maximum wear approximately 2 in. deep. If the above grouping of discharge rates is representative of the discharge rates that cause wear and if the amount of debris present during the discharges for the periods of time given is about constant, then it appears that the repair has definitely reduced the wear on the basin slab. Probably both the rock trap and the abrasion-resistant concrete are factors in the reduced wear.

79. The epoxy mortar overlay had not suffered any visible erosion damage; however, cracks were observed in several areas. In one of these areas the epoxy mortar coating was not bonded to the concrete. When pressure was applied to the cracked epoxy mortar, moisture seeped up

through the cracks. Because of the possibility that large discharges may remove the unattached epoxy, this area of unattached epoxy was removed. An area approximately 5 ft square was removed and saw cut along its perimeter. It was observed that the unattached epoxy area had failed from approximately 1/16 to 3/4 in. within the concrete slab. The failure plane in the remaining epoxy, which was stripped from the 5- by 5-ft area, was both at the epoxy-concrete interface and within the concrete; the majority of the failure occurred within the concrete. Following removal of the epoxy, the slab surface was cleaned and backfilled with a low modulus, low viscosity, moisture-insensitive epoxy mixed with approximately 3-1/4 parts sand to 1 part epoxy. In all other overlay areas, even those with cracks, the epoxy mortar appeared to be attached well. The reason for the cracks in the epoxy is not known. However, several reasons were suggested for the failure of large portions of a similar epoxy mortar on the floor of the tower water passageway at Milford Lake. The Missouri River Division laboratory determined that the epoxy mortar at Milford Lake had a linear thermal expansion coefficient of approximately 17×10^{-6} in./in./°F. This coefficient of linear expansion is approximately three times greater than the coefficient of linear expansion for concrete. Although temperature variations may only range about 30°F, the difference in the thermal coefficients of the two materials may be responsible for the cracks. Two other explanations for the cracks in the epoxy are as follows: (a) properties of the epoxy mortar may change when it is submerged in water and (b) the ability of epoxy mortar to contract without cracking during temperature change decreases with age.

80. The epoxy mortar appeared to offer good resistance to the wear caused by the grinding action of stones moved by currents. However, the epoxy mortar is of little value if it continues to crack and becomes unattached to the existing slab.

81. The next dewatered inspection of the stilling basin is scheduled for 1982.

Tuttle Creek Dam

82. The Tuttle Creek project located on the Big Blue River about 6 miles north of Manhattan, Kansas, consists of a rolled-fill earth dam, a gated spillway, and flood control outlet structures. The outlet structures consist of an intake structure, twin horseshoe-shaped conduits, and a stilling basin. The hydraulic jump stilling basin is 275.42 ft long and consists of a 125.42-ft-long sloped section extending from the conduit exit to a horizontal apron 150 ft long (Figure 36). Stilling basin elements include two rows of 8.5-ft-high baffles, a stepped end sill 18 ft high, a center dividing wall, and curved wing walls. A hydraulic model study (WES 1954) indicated that the maximum discharge velocity in the basin was approximately 50 fps. Bottom velocities, approximately 10 fps, over the end sill and in the exit area were highest for single-conduit operation. In addition, the results of scour tests (Figure 37) indicated that a significant amount of material from downstream of the end sill could be deposited inside the stilling basin during single-conduit operation.

83. The stilling basin was constructed in 1957. Diversion flow and lake storage began in July 1959 and March 1962, respectively. The basin slab is a continuous, reinforced concrete slab anchored to the rock foundation. Portions of the divider wall are reinforced and the divider wall footing forms part of the basin floor. The exterior basin walls are unreinforced gravity walls. The toe of the gravity walls also forms part of the basin slab. Materials and mixture proportions for the original concrete were as follows.

Material	Weight, lb	
	18-5	7-45
Portland cement, Type II	470	424
Big Blue River natural sand	872	833
Farley limestone		
No. 4 - 3/4-in.	638	735
3/4 - 1-1/2-in.	853	885
1-1/2 - 3-in.	1138	1060
Water	207	212

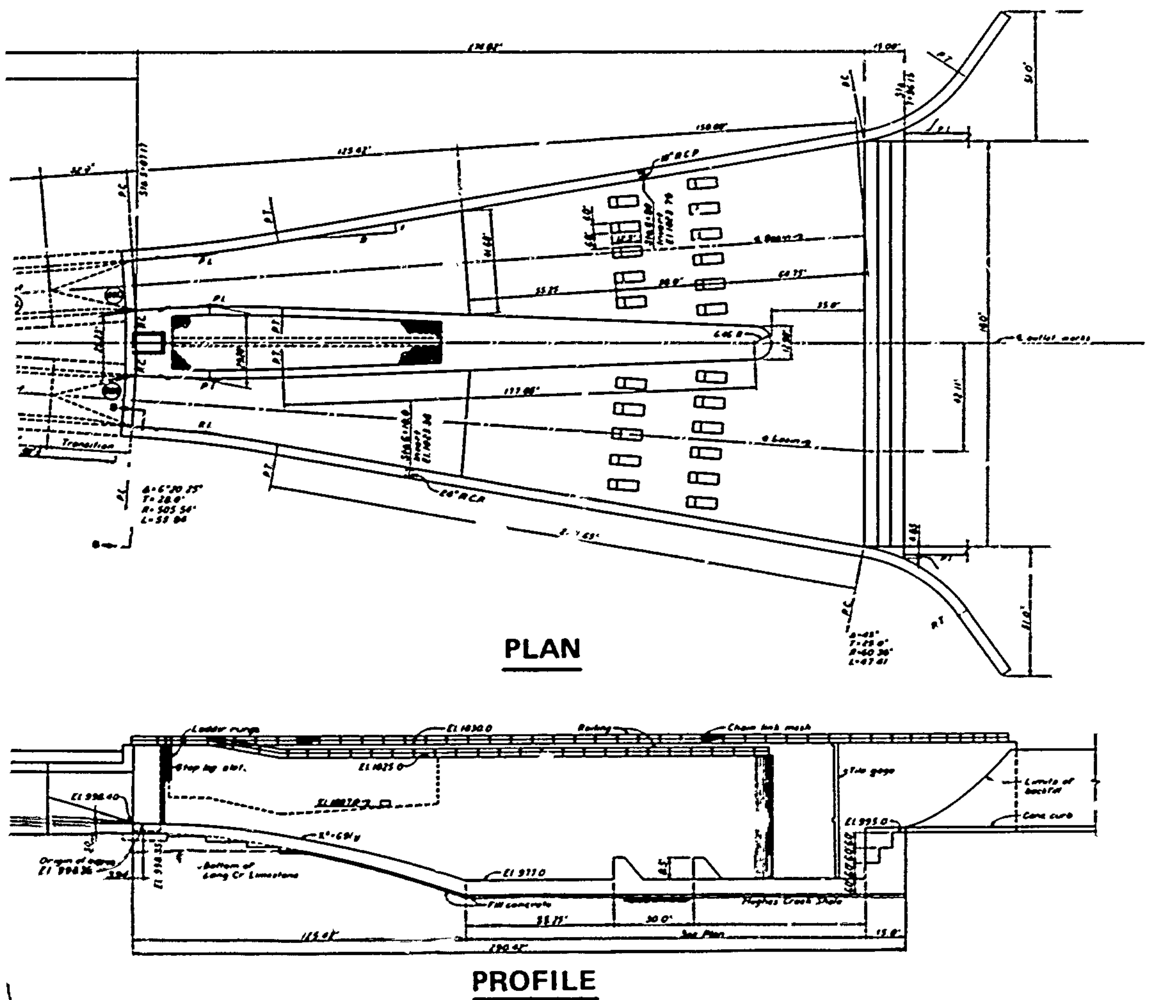


Figure 36. Outlet works plan and profile, Tuttle Creek Dam

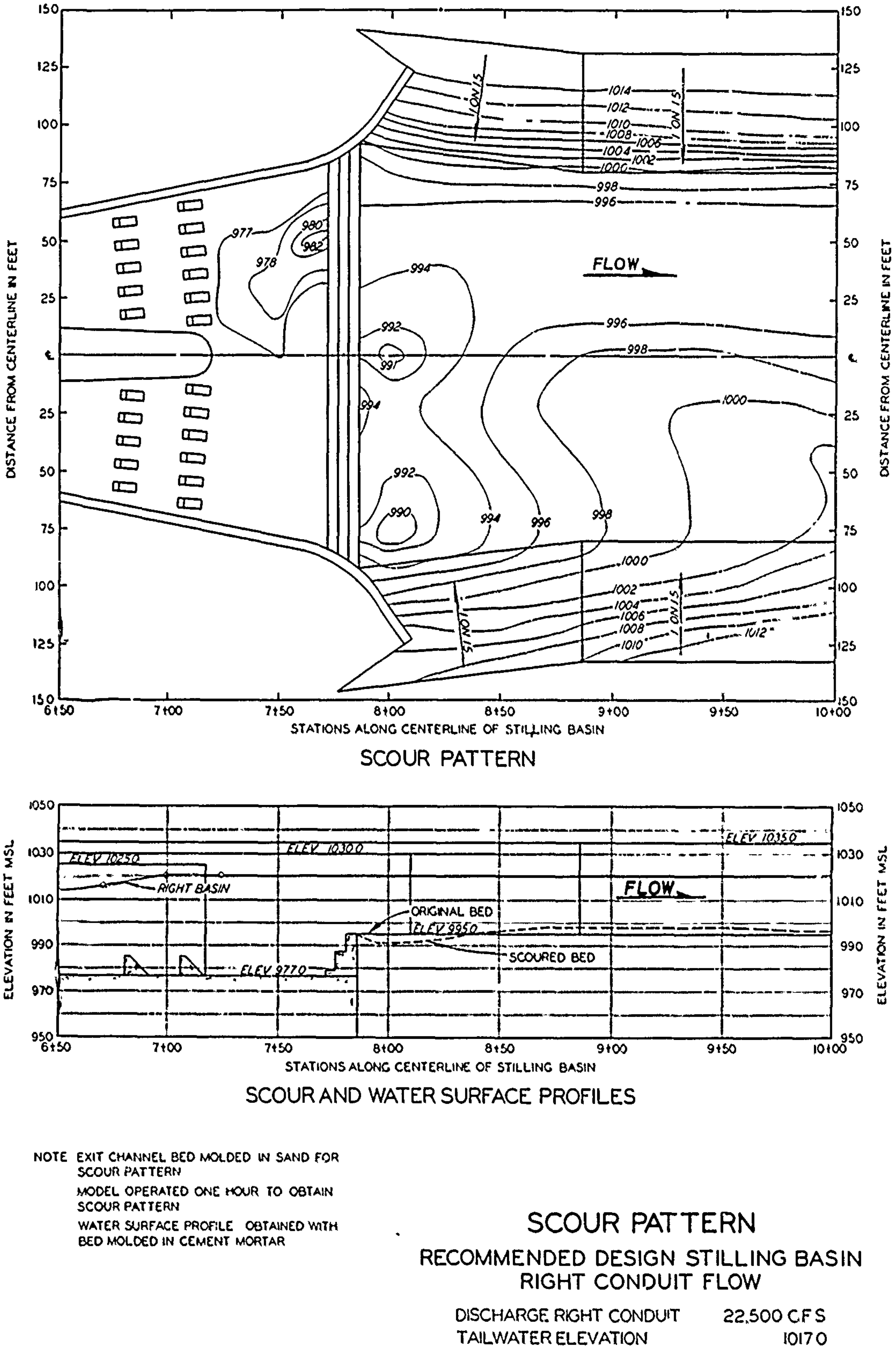


Figure 37. Scour test, hydraulic model study, Tuttle Creek Dam (from Oswalt 1971)

Compressive strengths at 28 days ranged from 4395 to 6120 psi for mixture 18-5, with an average of 4395 psi. In comparison, strengths of mixture 7-45 ranged from 3855 to 4660 psi with an average of 4220 psi. Compressive strengths of four concrete cores taken when a sump was drilled in the downstream portion of the stilling basin ranged from 3530 to 3860 psi. The micrometric air void content of the concrete containing coarse aggregate of 2- to 3-in. maximum size was 13.5 percent. Air content specified was 4 to 7 percent.

84. The first dewatering of the basin was in November 1967 as part of the first periodic inspection. At that time, wear was observed primarily in the downstream portion of the transition slab and the upstream portion of the basin slab. Reinforcing steel was exposed on the basin floor in four areas. The largest area of exposure was approximately 10 by 12 ft. Maximum design velocities had not occurred in the basin. Only a relatively small amount of rock and debris was found in the basin; however, the rock showed signs of wear. It was believed that the wear observed occurred at low discharges and was caused when rocks and other debris were moved by eddy currents.

85. While the basin was dewatered, the four localized areas of exposed reinforcing steel were temporarily repaired. Sandbags were used to isolate the exposed reinforcing steel areas from gate leakage and leakage from other sources. Concrete was progressively removed by jackhammer from the exposed steel where concrete cover was less than 2 in. Below exposed reinforcing steel the concrete was removed to a depth of 2 to 4 in. The area was cleaned and backfilled with a high strength, high quality concrete. The strength of the backfilled concrete was not determined.

86. The basin was dewatered again in March 1973 during the second periodic inspection, and additional areas of erosion were observed (Figure 38). The temporary concrete repair had suffered some erosion; however, reinforcing steel was not exposed in the patched areas. Reinforcing steel was exposed in the original slab at the perimeter of the patches.

87. When the basin was examined in February 1975, generally little

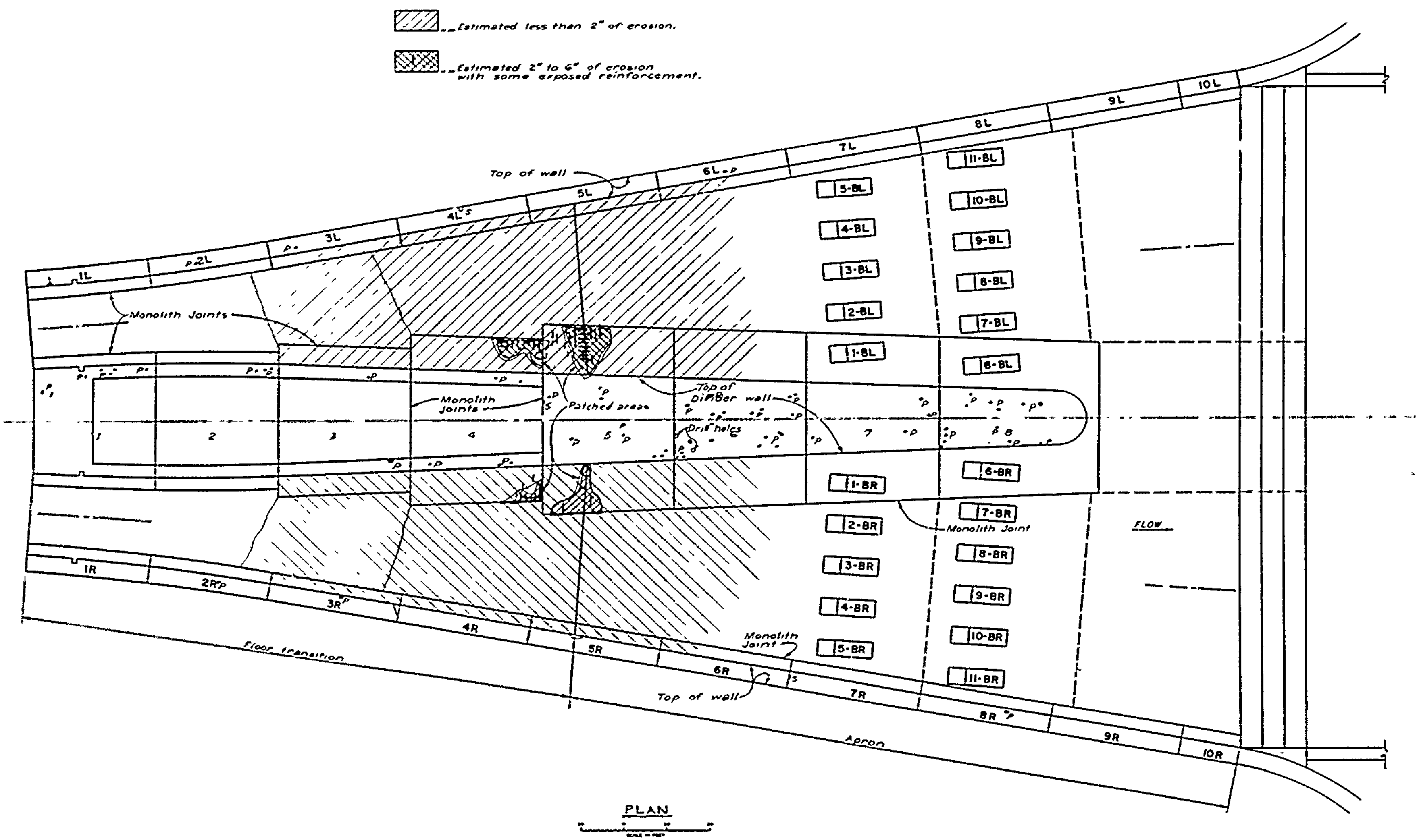


Figure 38. 1973 erosion damage, Tuttle Creek Dam

additional damage was observed. One of the patched areas appeared to have suffered additional wear; however, no reinforcing steel was exposed. Approximately 3 to 4 tons of debris was removed from the basin following the dewatering. Most of the debris was rock, although one engine block and many cap screws were also found in the basin.

88. During January through March 1975 an overlay slab was placed over most of the transition slab and the upstream two-thirds of the basin slab (Figure 39). The overlay slab placement, including initial dewatering and maintaining the stilling basin in a dewatered state, was accomplished by contract. The contract required that the gate leakage, surface drainage, basin drain seepage, and monolith joint seepage be controlled. Gate leakage was intercepted by a sandbag dike in the conduit and piped through the basin.

89. The upstream portion of the reinforced concrete overlay slab (10-in. minimum thickness) was recessed into the existing slab. The concrete overlay contained an abrasion-resistant aggregate, Sioux quartzite, and was proportioned to provide 6000 psi compressive strength as follows.

Material	Weight, lb
Portland cement, Type II	658
Kansas River natural sand	1166
Sioux quartzite, No. 4 - 3/4 in.	1772
Water	243

Slump and air content of this mixture averaged 2-3/4 in. and 5.4 percent, respectively. Compressive strengths at 28 days ranged from 4440 to 5660 psi with an average of 5130 psi. The overlay was designed to be fully bonded to the existing slab. A neat cement grout was brushed onto the existing slab immediately prior to placement of the concrete. Expansion anchors were also provided between the existing slab and the overlay. The design considered that if lack of bond occurred over an area of several square feet, the resistance to the uplift forces would be concentrated at the perimeter of this area and could be large enough to separate the overlay slab from the existing slab. Thus, the anchors

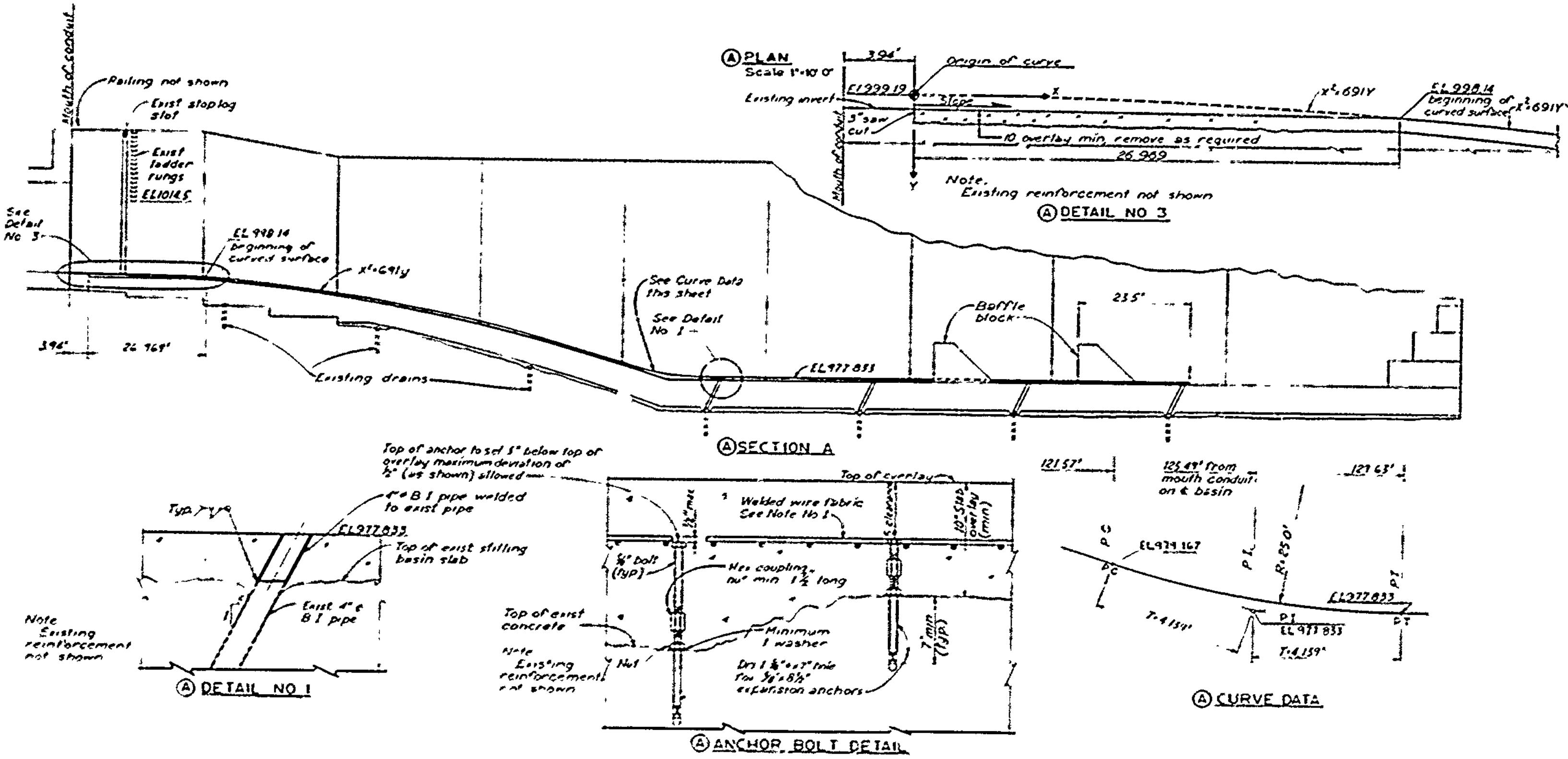


Figure 39. Repair details, Tuttle Creek Dam

were designed to carry the total uplift forces.

90. During placement of the overlay slab, the calking of monolith joints in the basin slab was not satisfactory. Water continued to pond on the existing slab after the grout coat was placed. The bond strength between the overlay and the existing slab was probably much less than intended. The 4-in. spacing on the reinforcing mesh also added to the problem of removing excess water and placing the grout coat. A larger spacing would have allowed easier access to the existing slab surface. Concrete extruded below the forms, and thus monolith joints did not form the plane surface as had been intended.

91. The approximate costs to dewater and place the overlay slab by contract were as follows:

Dewatering and maintaining the basin dewatered	$ 37,900
Concrete removal to recess the overlay slab into the existing concrete	5,000
Concrete overlay slab with cement (605 cu yd)	112,000
Reinforcing steel (16,190 lb)	5,500
Expansion anchors (10,346 anchors)	36,200
Drain pipe extensions and saw cutting	1,400
	$198,000

92. The basin is scheduled for dewatering during the next periodic inspection in March 1980.

Curwensville Lake

93. Curwensville Lake is located on the west branch of the Susquehanna River, in Clearfield, Pennsylvania. The earth-fill dam with reinforced concrete outlet works was completed in 1965. The outlet structure consists of intake tower, 15-ft-diameter conduit, stilling basin, and outlet channel. The concrete stilling basin structure (Figure 40) at the end of the conduit consists of a flared transition section of conduit outlet portal (Figure 41), a stilling basin with baffles, end sill, and wing walls. The floor and cantilever walls of the stilling basin were designed as an integral U-frame. The cantilever walls were designed as retaining wall stems and the floor slabs were designed to

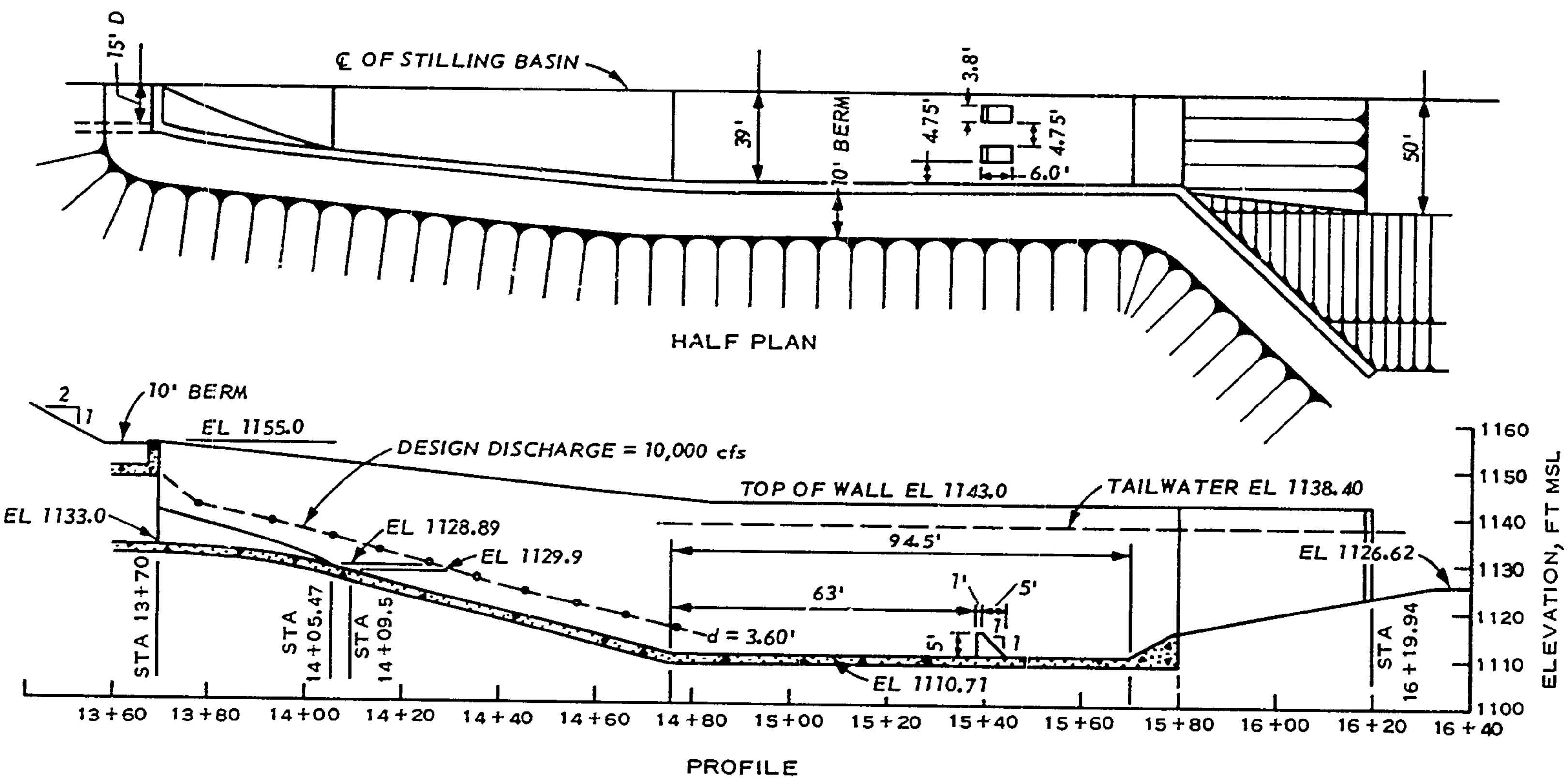


Figure 40. Half plan and profile, Curwensville Lake

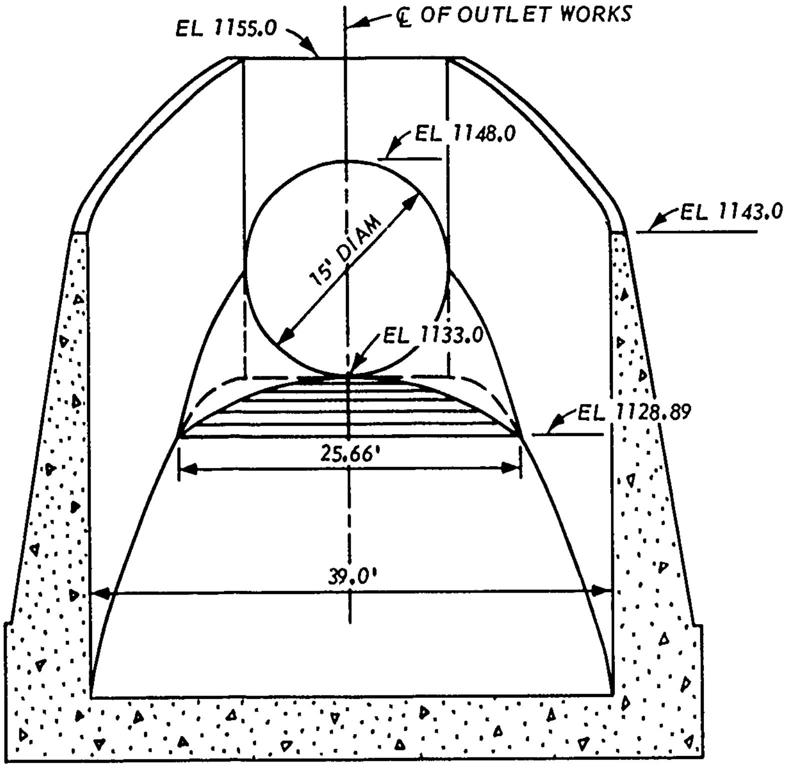


Figure 41. Outlet portal, Curwensville Lake

transmit these loads to the foundation. This rigid frame type structure was also designed to withstand the estimated uplift forces and the uniform foundation pressures without anchorage to the rock foundation. The gravity type wing walls located at the end of the stilling basin were designed as retaining walls.

94. The outlet channel extending downstream from the end sill of the stilling basin was excavated in rock and earth for a distance of about 2000 ft to the existing downstream river channel. The dimensions and alignment of the channel were determined by the hydraulic requirements and by the stable slopes of the materials excavated. The channel bottom slopes upward at one vertical to five horizontal from the top of end sill el 1115.7 to el 1126.6, and then continues on a descending slope of 0.14 percent to the existing river bottom. The bottom width of the outlet channel is 50 ft with side slopes at one vertical to two horizontal.

95. By 1973, scour in the channel rock below the end sill and

erosion downstream in the river channel were such that repairs were deemed necessary. Channel improvements consisted of dewatering the stilling basin down to the existing end sill and putting in a new concrete apron, in addition to excavating the outlet channel and placing riprap along both banks. The reinforced concrete apron has a minimum thickness of 12 in. and design compressive strength of 3000 psi. The apron slab slopes upward from the end sill of the stilling basin one vertical to five horizontal and flares from a width of 39 ft of the end sill to 60 ft at a distance of 59 ft downstream. Side slopes from the apron to the wing walls were also paved. Concrete paving details are shown in Figures 42 through 44. Photographs taken during the construction operations are shown in Figures 45 and 46. Estimated quantities of concrete paving and riprap were 450 and 13,500 cu yd, respectively. The contract bid was $723,365 and work was completed in 1974.

Lac Qui Parle Dam

96. The Lac Qui Parle Dam was built in 1936 on the Minnesota River near Watson, Minnesota. The earth-fill dam has a reinforced concrete control structure on timber pilings with 12 bays, each having 17-ft clearance between piers (Figures 47 and 48). The discharge velocity, original material used, design, and construction details were unavailable.

97. In September 1970, bays 1 through 4 were dewatered for inspection. Results of the inspection indicated remedial work was required. Therefore, a contract was awarded in June 1974 for the repair of the bridge and control structure. Cofferdams both upstream and downstream were included in the contract. These were constructed of granular fill with a 1 on 3 slope. The slope of the cofferdam exposed to wave action was covered with a 2-ft layer of clay and a 6-mil layer of nylon reinforced polyethylene with a series of sandbags for counterweights. Low water levels during the summer of 1974 required the use of 4-in. PVC pipe to maintain flow in the river. Approximately ten 4-in. pipes were used to siphon flow. The top 1.2 ft of the ogee in bay 12 was removed to allow flow of the river during construction.

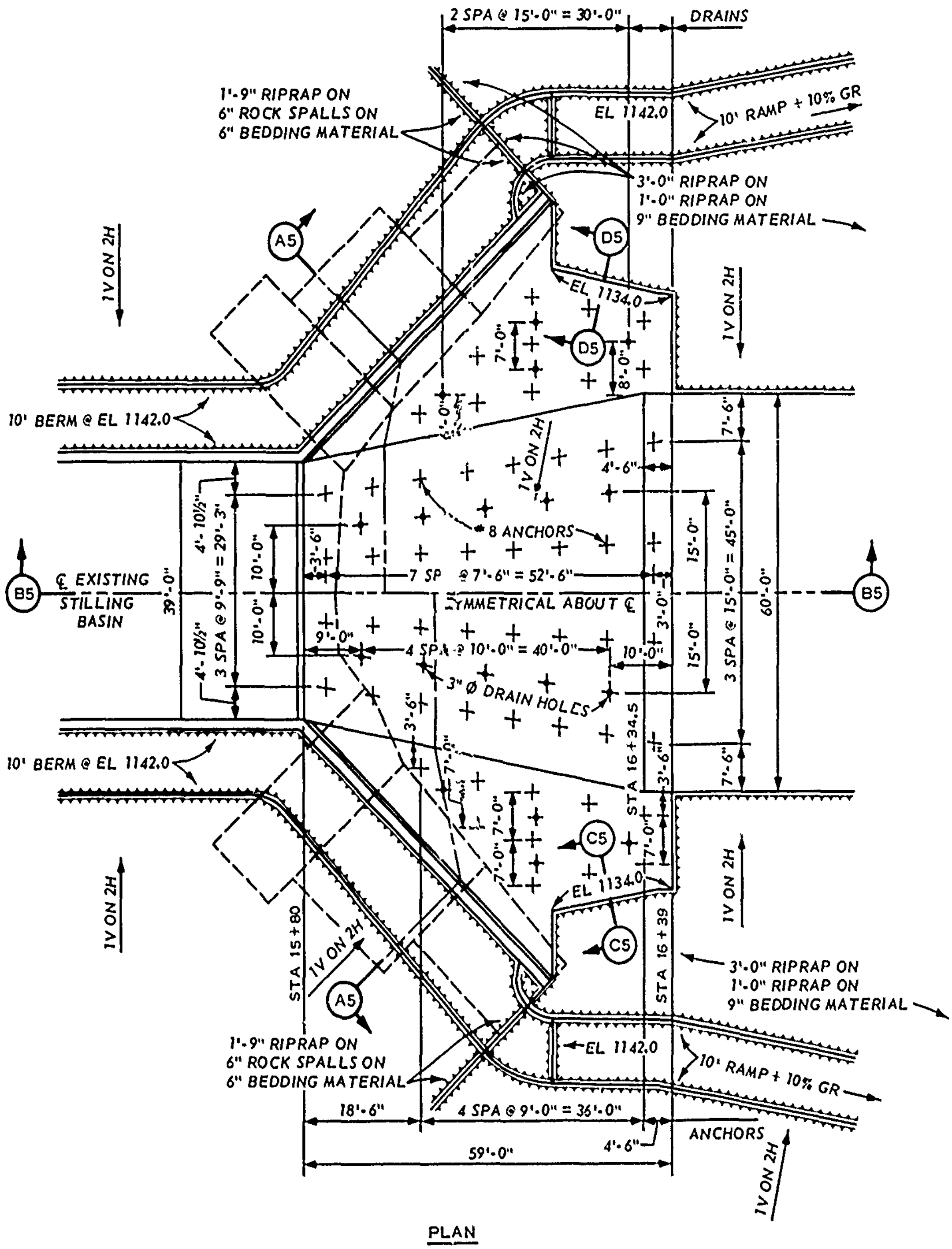


Figure 42. Plan, concrete paving details, Curwensville Lake

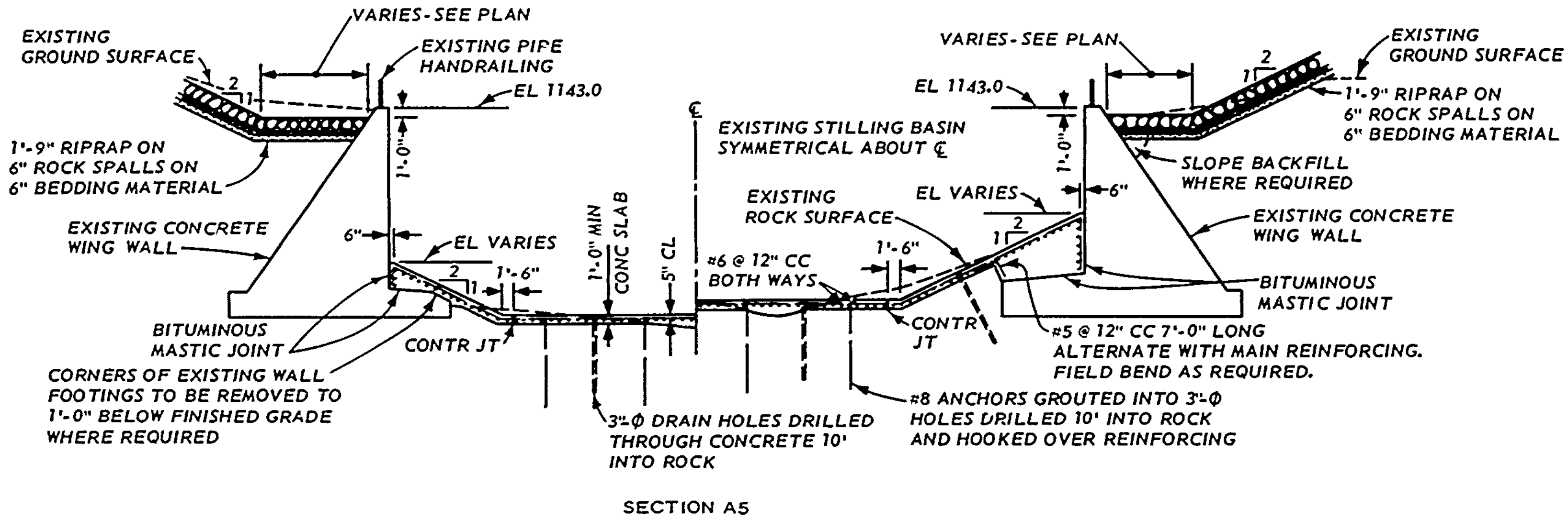


Figure 43. Section A5, concrete paving details, Curwensville Lake

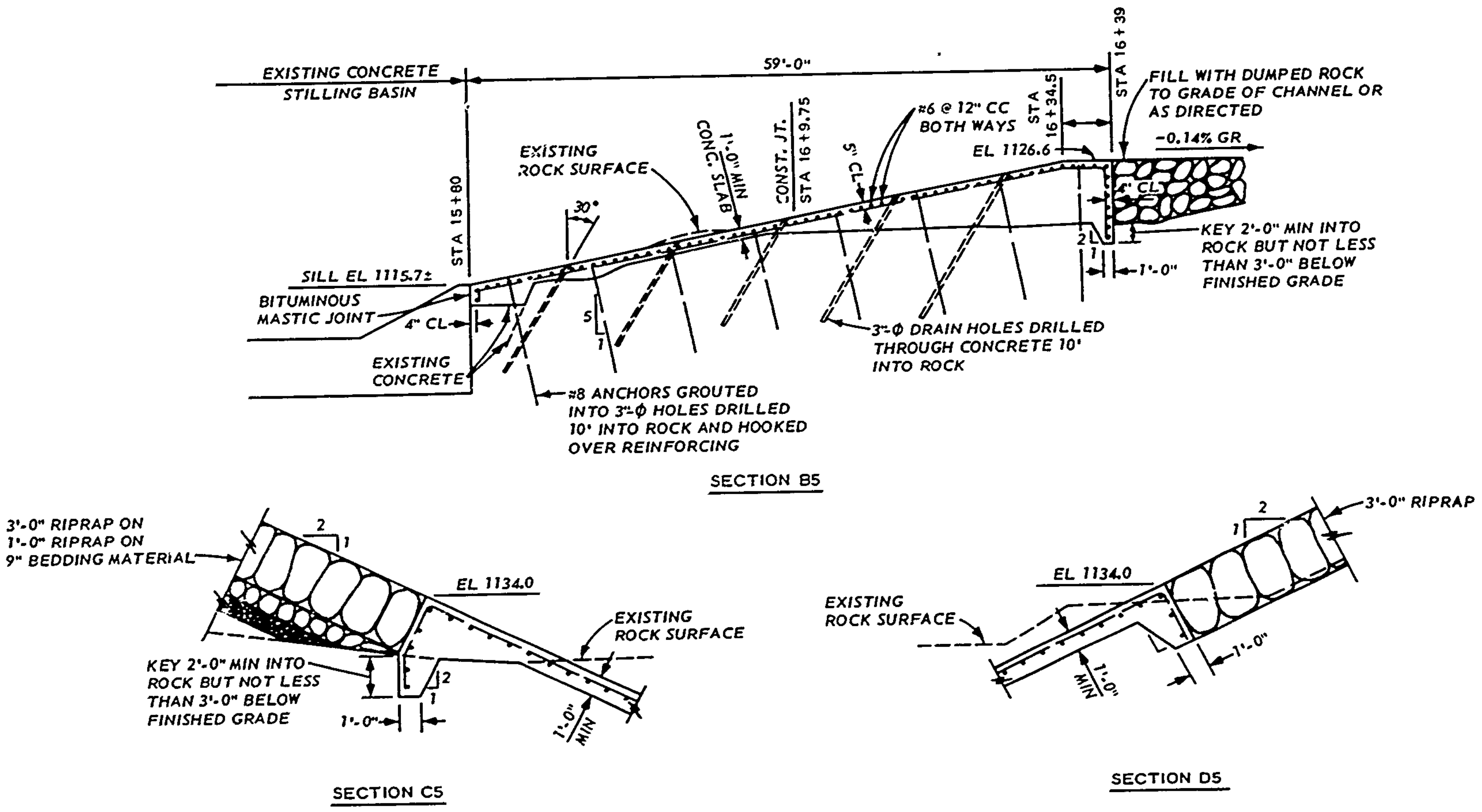
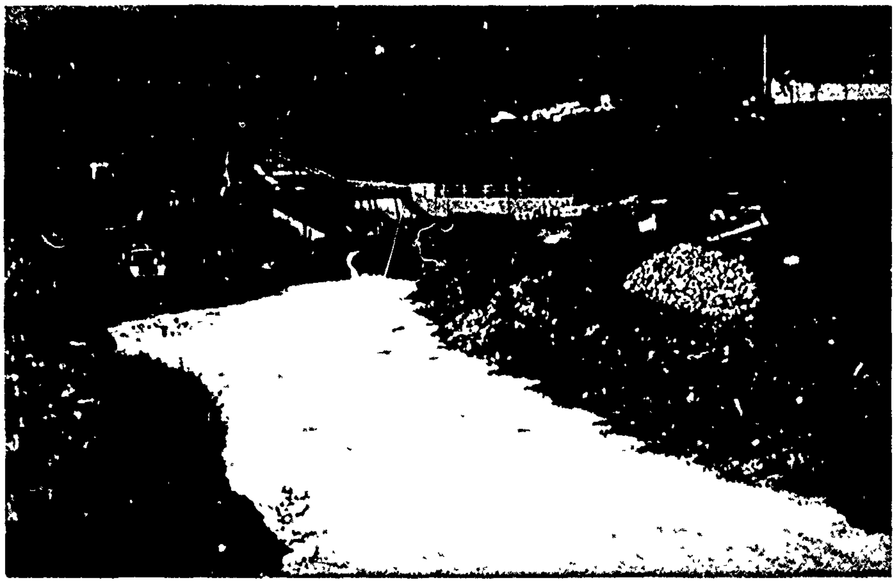


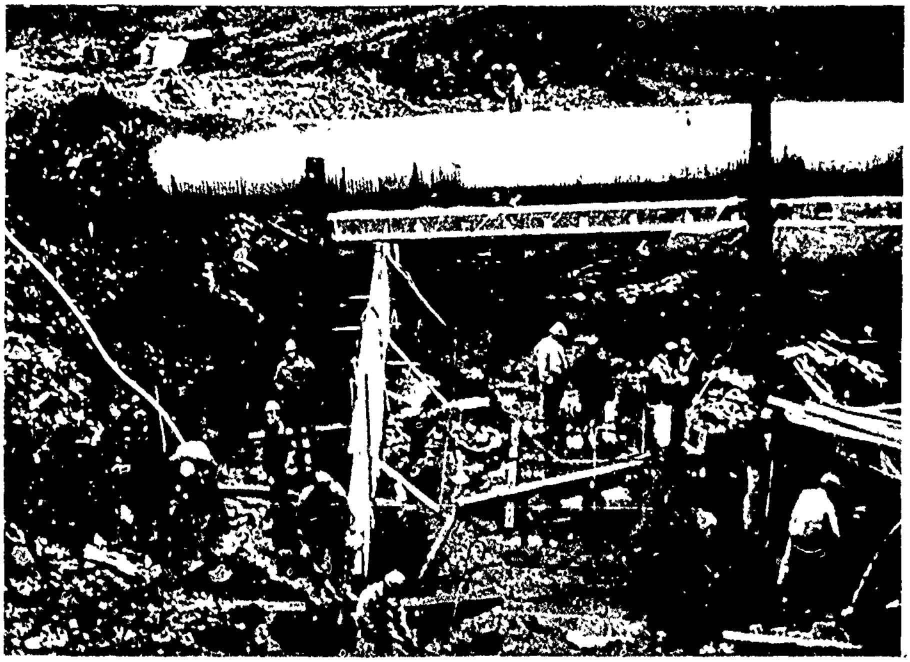
Figure 44. Sections B5, C5, and D5, concrete paving details, Curwensville Lake

a. Diversion through the stilling basin

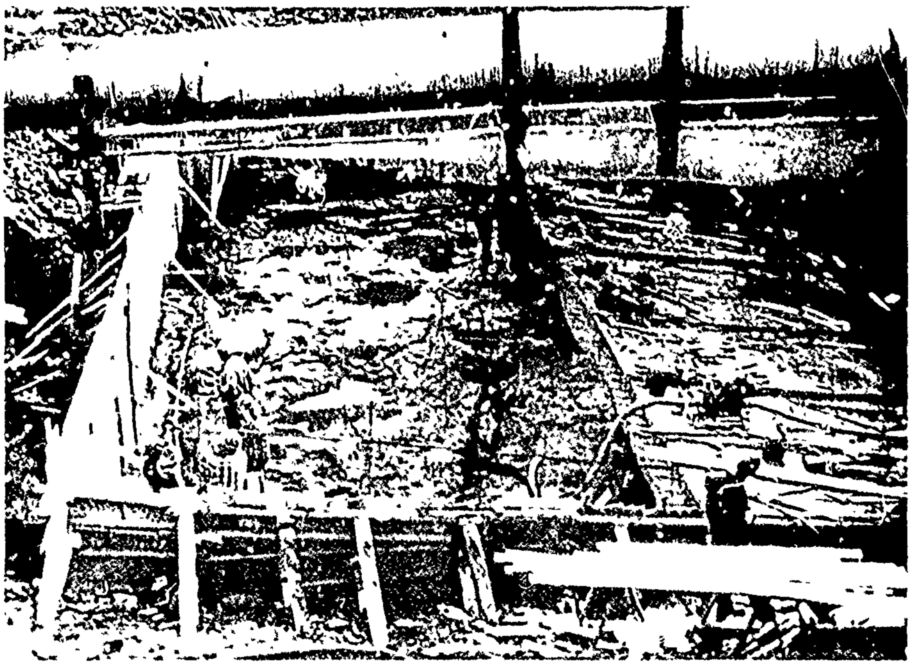


b. Diversion downstream of stilling basin

Figure 45. Diversion facility designed for a release of 50 cfs



a. Placement preparations, 1 Nov 1973



b. Placement preparations, 2 Nov 1973

Figure 46. Preparation for placement of reinforced concrete slab

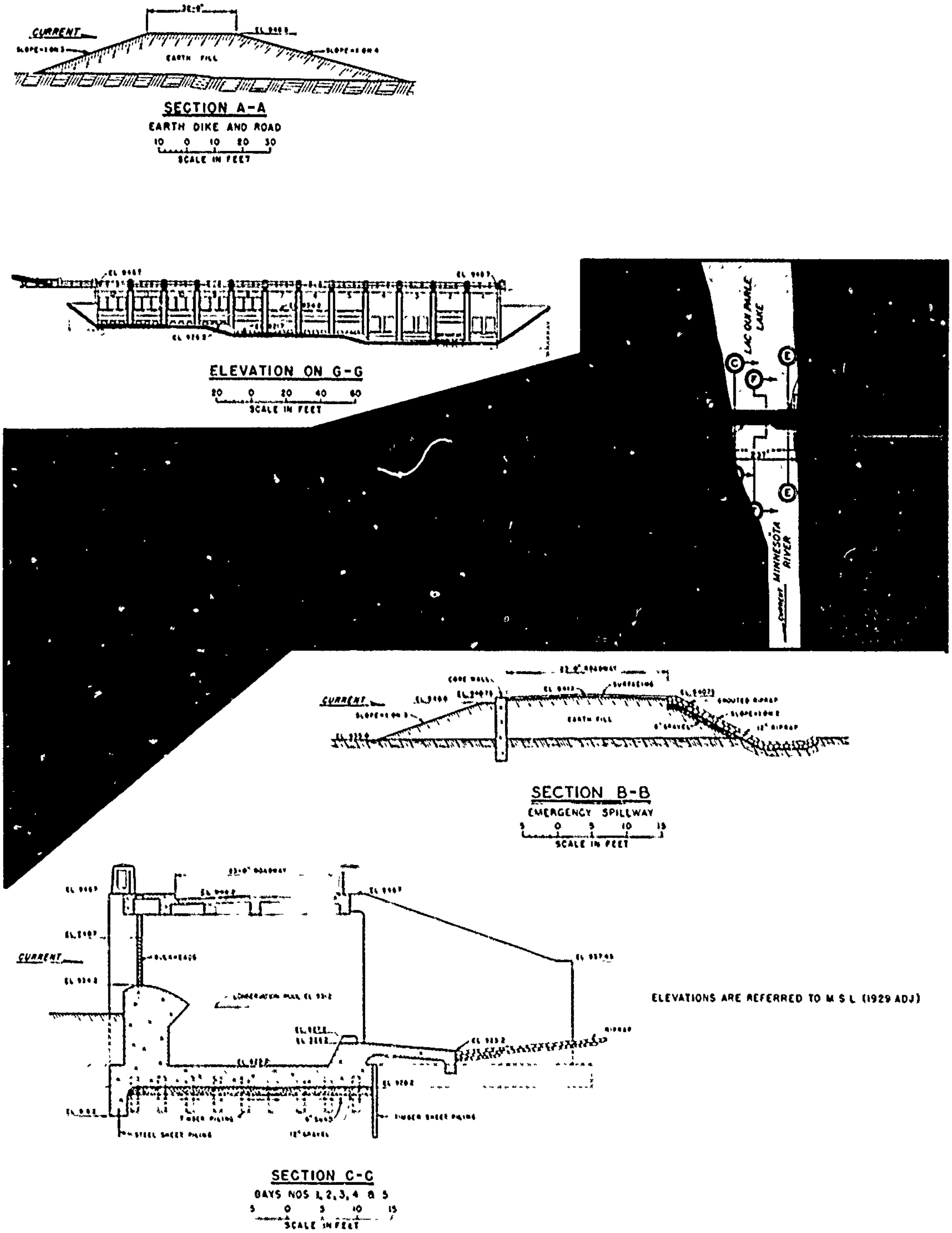


Figure 47. Plan and sections, Lac Qui Parle Dam
(from St. Paul District 1974)

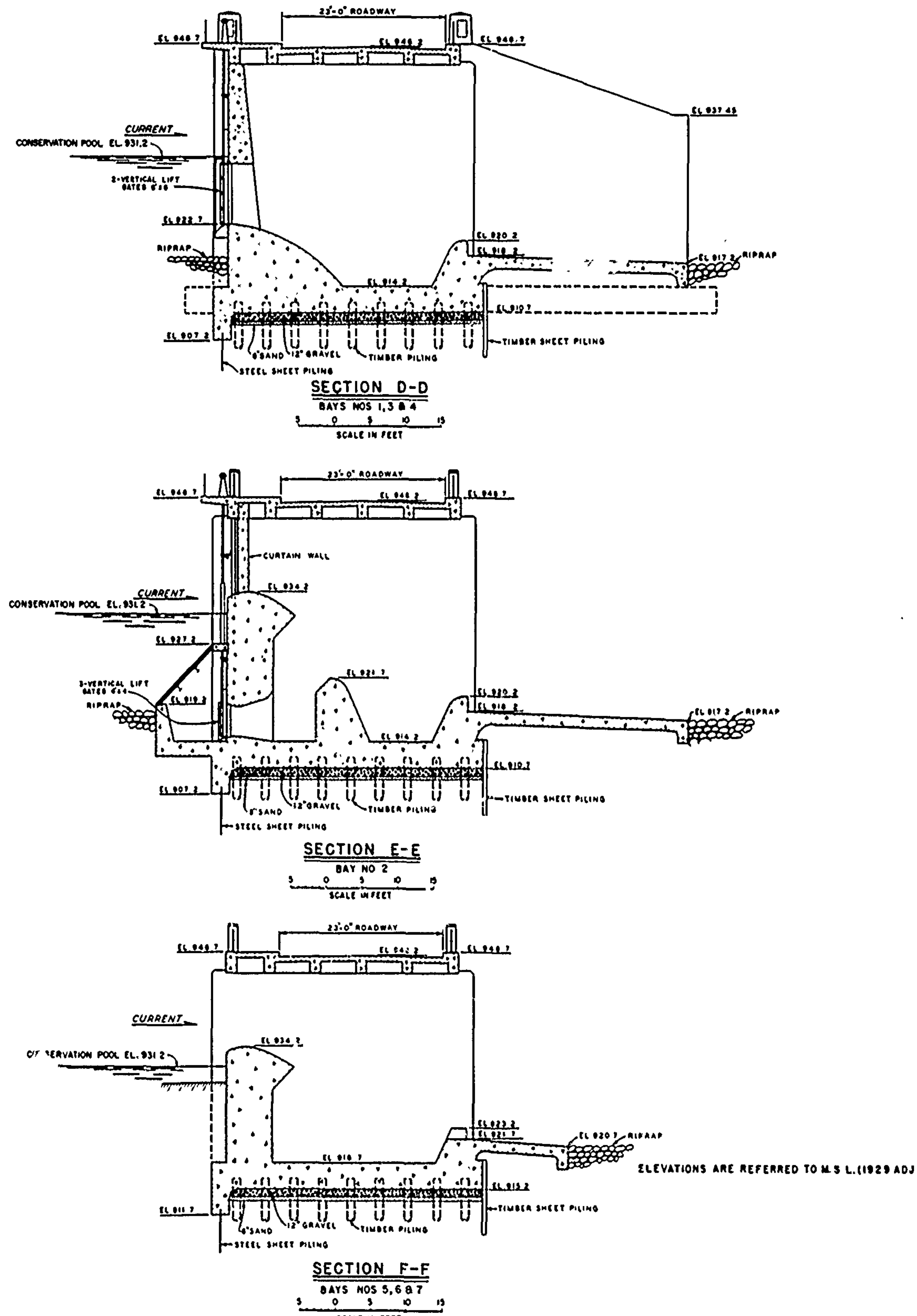


Figure 48. Sections, Lac Qui Parle (from St. Paul District 1974)

98. After the cofferdams were constructed and the structure was dewatered, representatives of the U. S. Army Engineer District, St. Paul, inspected the structure (St. Paul District 1974) to determine the exact extent of repair required. At the time of the inspection, all bays of the control structure, except bay 12, were dewatered, allowing for a thorough inspection of each bay. Several areas of concrete deterioration (Figure 49) were found in the piers, generally along the waterline in each bay. Several base slabs exhibited severe wear and exposed reinforcement. The most severe erosion was in bay 2 (Figure 50) where there was a hole about 1 ft in diamter and 2 ft deep caused by rocks trapped between the gates and weir. Other examples of the concrete deterioration are shown in Figure 51.

99. The damaged concrete was removed using hand and power tools. Saw cuts 1 in. deep were made at the edges of the damaged areas to prevent feather edges (Figure 52). Damaged concrete was removed to a depth of 7 in. below all exposed reinforcements. The damaged areas were sandblasted prior to being patched. The slabs were repaired with epoxy bonding material, conventional concrete, and No. 4 bars on 12-in. centers. The damage on the piers at the waterlines was repaired with a latex mortar (Figure 53). The mixture proportions for the latex mortar were as follows.

Material	Ratio to Weight of Cement
Portland cement	1.00
Sand	3.25
Latex solids	0.15
Water	0.35-0.40

100. In order to allow for the exit of abrasive materials from bay 2, two vertical 30-in. cuts were made in the weir. At the completion of repairs, the cofferdams were removed and the concrete on the ogee in bay 12 was replaced.

101. The cost to repair the surface concrete on the slabs was \$8.75/sq ft. The cost to repair the surface concrete on the ogees was \$15.00/sq ft. The cost of repair of the surface concrete on the

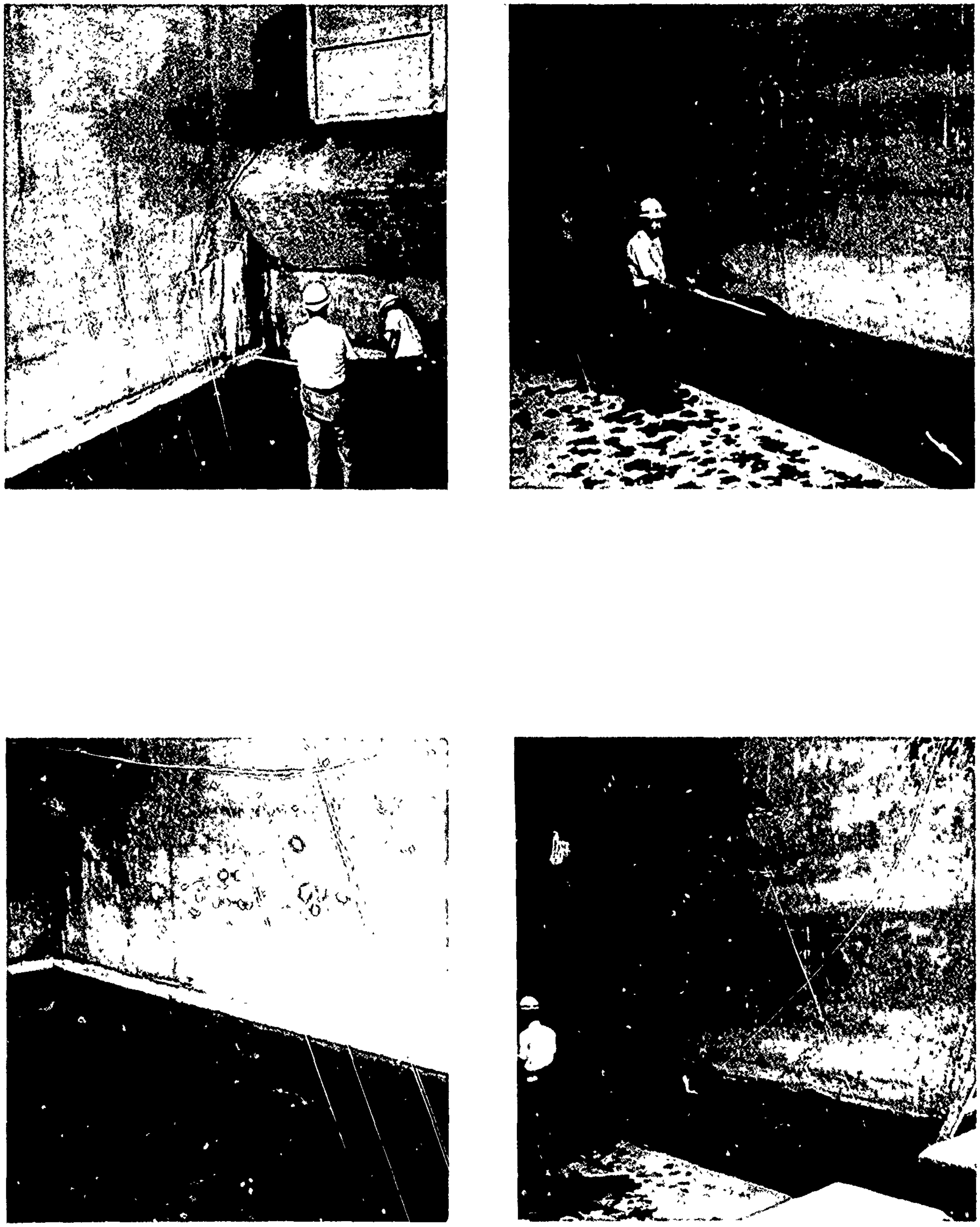


Figure 49. Different views of concrete deterioration along waterline, Lac Qui Parle Dam (from St. Paul District 1974)

a. Close-up view of base slab erosion

b. Base slab erosion

Figure 50. Erosion of base slab in bay 2, Lac Qui Parle Dam (from St. Paul District 1974)

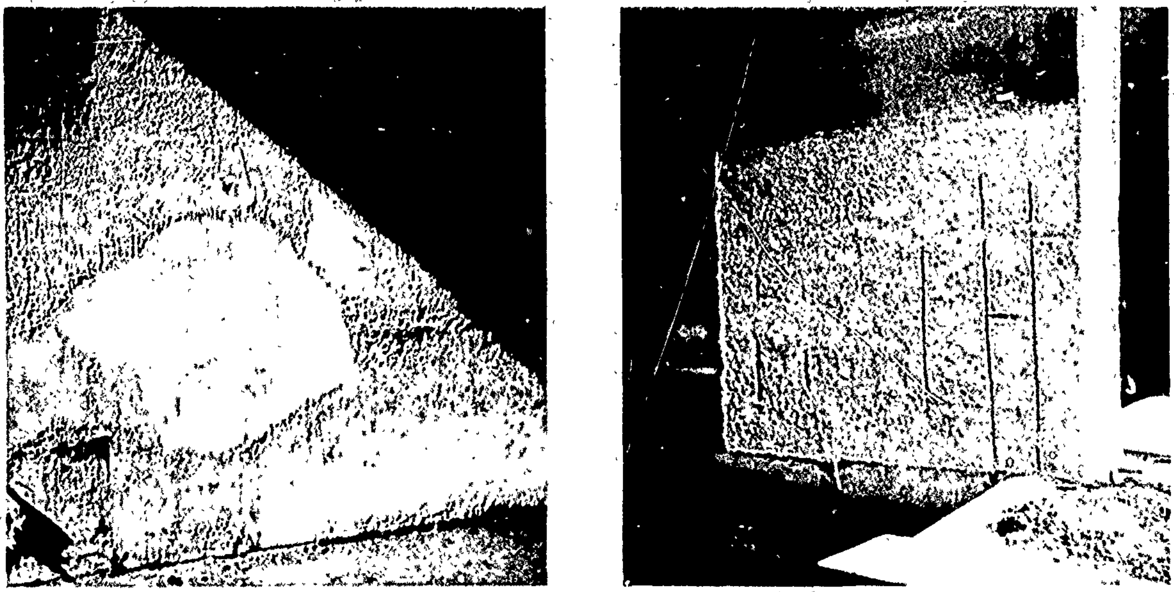
a. Erosion between baffle of wall

b. Miscellaneous debris

c. Baffle deterioration

Figure 51. Examples of concrete deterioration, Lac Qui Parle Dam (from St. Paul District 1974)

Figure 52. Views of spillway repairs, Lac Qui Parle Dam (from St. Paul District 1974)



a. Completed repair

b. Concrete surface prepared for repair

Figure 53. Latex mortar repair of piers, Lac Qui Parle Dam (from St. Paul District 1974)

vertical pier face was $7.50/sq ft. The cost of the miscellaneous latex mortar repair was $.21/cu in.

102. The next periodic inspection is scheduled for September 1979.

Upper St. Anthony Falls Lock

Description

103. The Upper St. Anthony Falls Lock is located on the Mississippi River in Minneapolis, Minnesota. The lock is a reinforced concrete lock of the dry dock type (Figures 54 and 55). It has relatively thin walls and is constructed integrally with a bottom slab. The lock has an overall length of 704.5 ft and a clear chamber length of 400 ft. The width of the lock is 56 ft with the top of the lock wall at el 806.0. The normal lift is 49.2 ft with a clearance of 13 ft over the sill at minimum controlled pool elevation.

104. The lock filling and emptying system consists of four intake ports in each side of both lock walls upstream from the upper gates, wall culverts generally 10 ft square connecting the intake to 10 chamber laterals and three discharge laterals from the land wall culvert, one discharge manifold from the river wall culvert to the river, and reverse tainter valves to control flow during filling and emptying operations. The lock was completed in 1963.

105. The lock was dewatered from December 1975 to March 1976 to repair a damaged miter gate. During this period an examination (St. Paul District 1976) of the filling and emptying laterals and discharge laterals revealed considerable abrasion-erosion of the concrete to maximum depths of 23 in. (Figures 56 and 57). This erosion was caused by rocks up to 18 in. in diameter that had made their way into the laterals. There was no evidence that the rocks entered the laterals from the culvert. The size of the rocks would not have allowed them to pass the intake screen, and there was no sign of erosion in the culvert or any accumulation of rocks in the culvert. The rocks probably either were deposited in the laterals when the 1965 flood of record (91,600 cfs) was discharged through the lock chamber or were transported embedded in ice prior to being deposited. Subsequent filling and emptying of the lock during normal operation agitated these

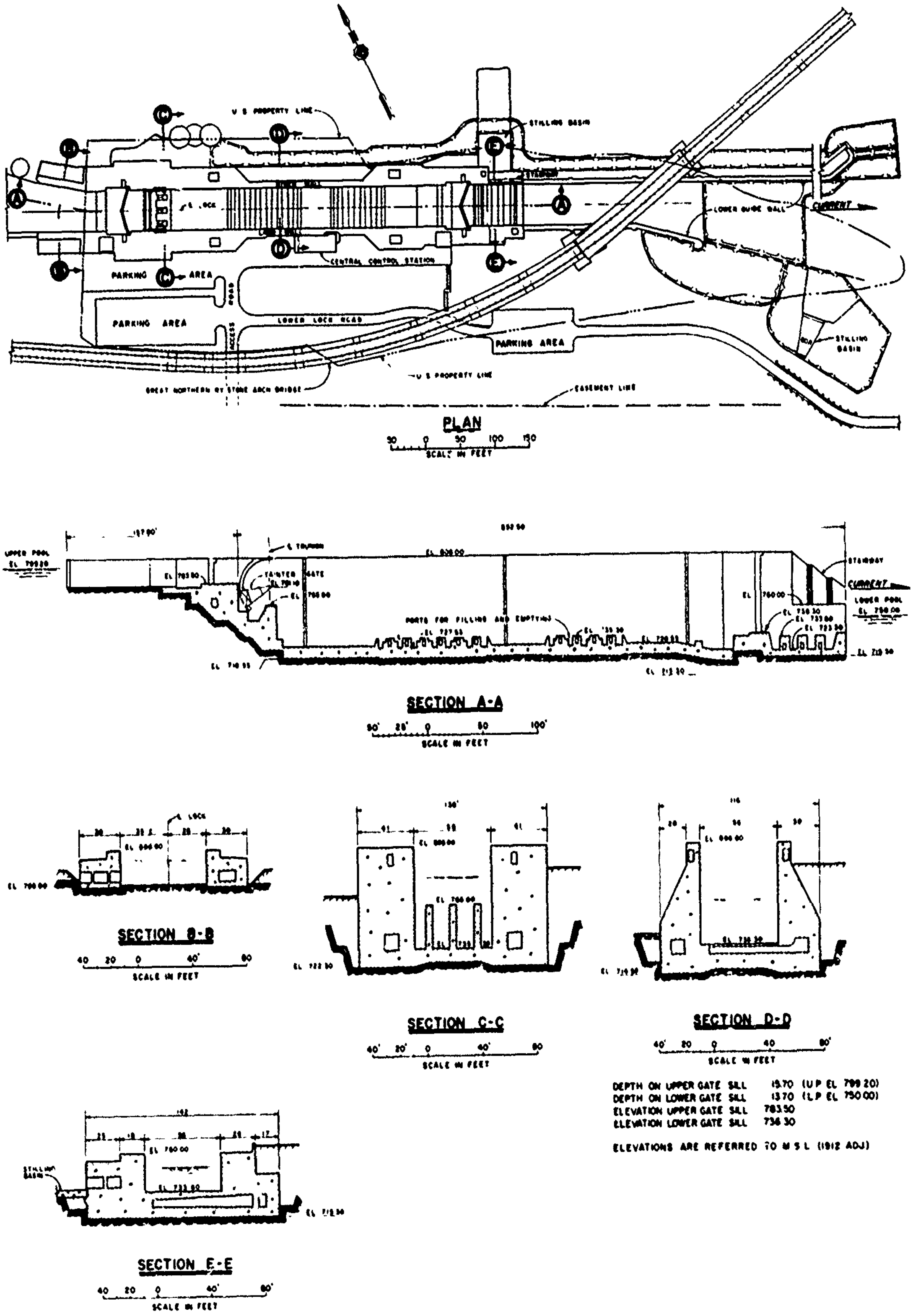


Figure 54. Plan and sections, Upper St. Anthony Falls Lock (from St. Paul District 1976)

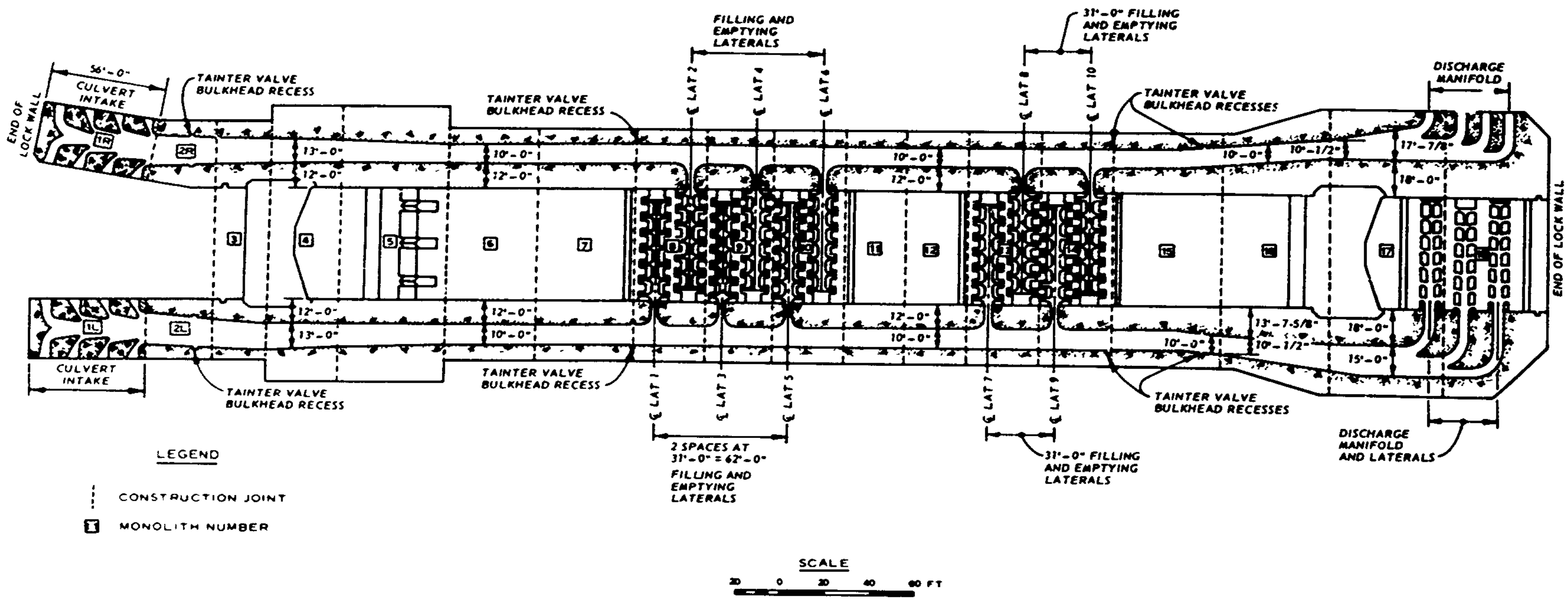
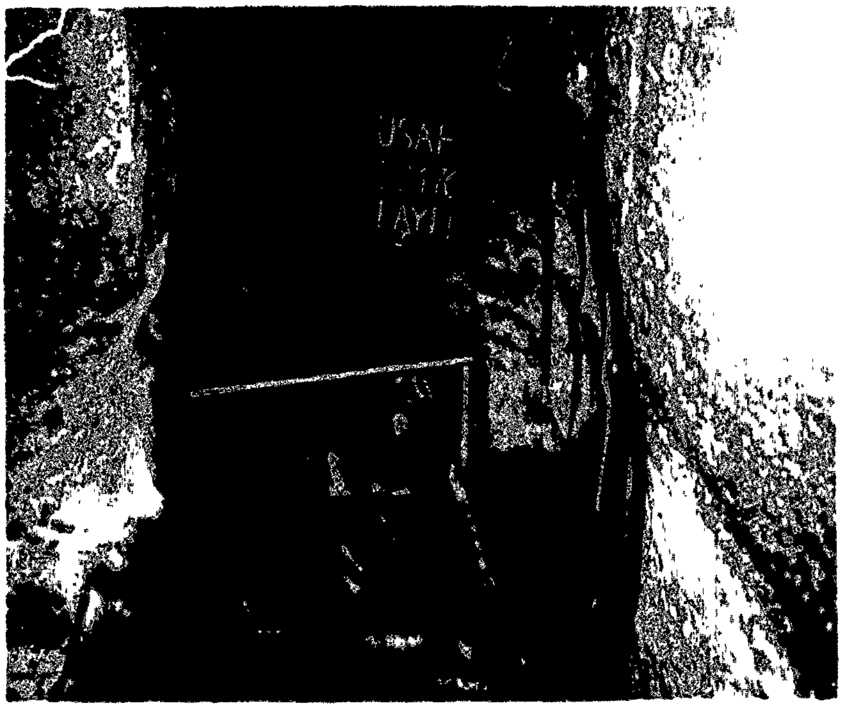


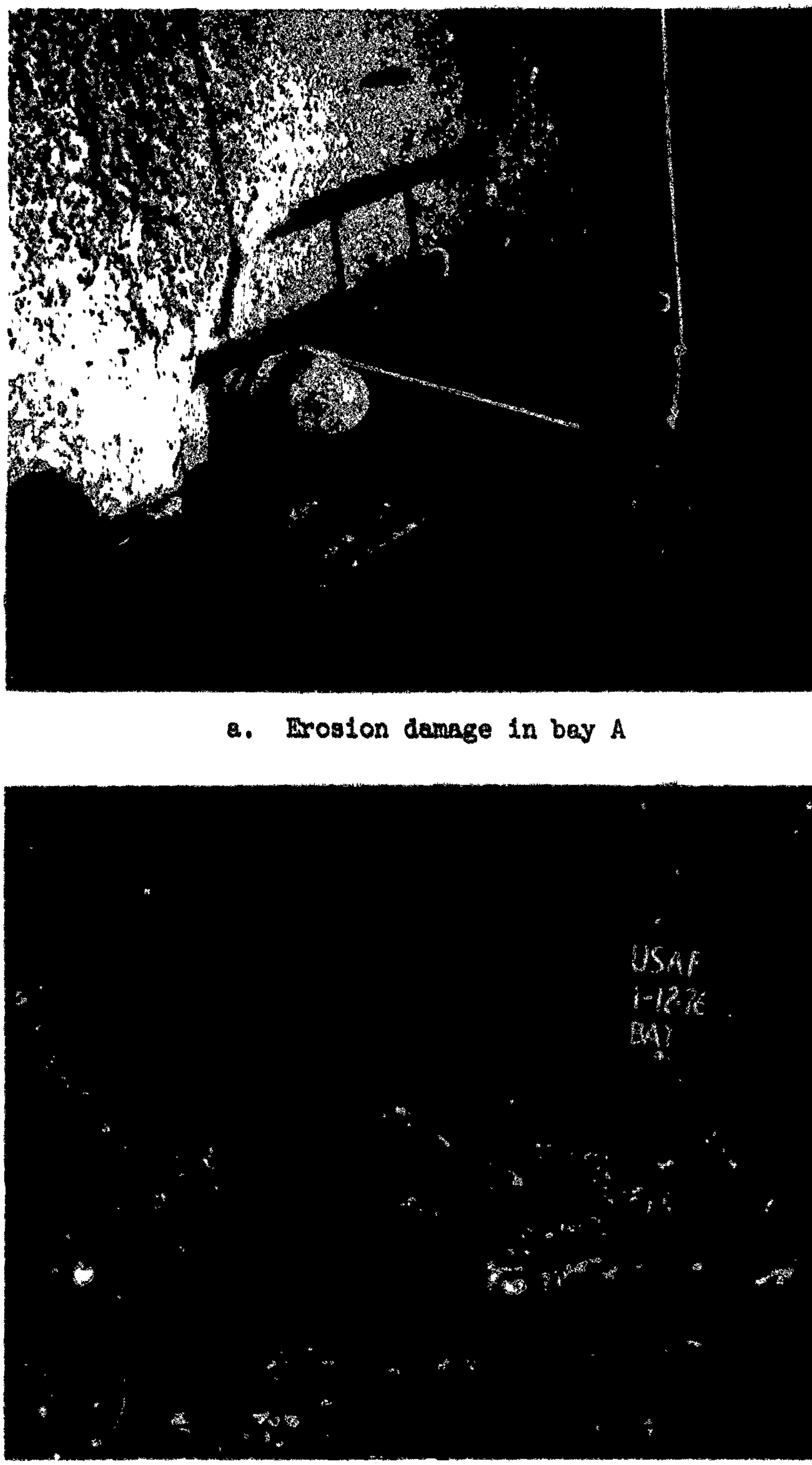
Figure 55. Plan, Upper St. Anthony Falls Lock (from St. Paul District 1976)



a. Filling and emptying lateral, port 2

b. Filling and emptying lateral, port 3

Figure 56. Damage to filling and emptying lateral, Bay H Upper St. Anthony Falls Lock (from St. Paul District 1976)



a. Erosion damage in bay A

b. Erosion damage in bay B

Figure 57. Typical erosion damage in the discharge laterals, Upper St. Anthony Falls Lock (from St. Paul District 1976)

rocks, causing them to erode the concrete.

Investigation

106. A laboratory and field investigation of the concrete was conducted to determine whether the erosion was due to any deficiency of the concrete itself. Ultrasonic pulse velocity measurements in 78 selected areas of the eroded concrete ranged from 15,870 to 18,020 fps with a mean velocity of 17,120 fps. Similar results and a mean velocity of 17,083 fps were obtained on concrete cores. Compressive strength test results of these same nine cores ranged from 7,460 to 12,980 psi with an average of 9,760 psi. A petrographic examination indicated that the air-entrained concrete was of high quality and free from evidence of deleterious chemical reactions or damage by freezing and thawing. All tests indicated that the concrete was sound and of excellent quality and did not contribute to the erosion problems.

Repairs

107. Repairs to the damaged sections were initiated in January 1976. Prior to that time the damaged laterals had been covered and radiant heat had been installed to melt ice that had formed during the dewatering process. The three discharge laterals below the lower land leafs were dried out so that work could begin. Four filling and emptying laterals in the lock chamber itself were also dried out for inspection of the damage. One of these laterals in monolith 13 (bay H) had extensive damage and exposed rebar (see Figure 56). The other three, although damaged, were not as bad. Since repairs to all the laterals required heavy electrical loads and amounts of time, the other laterals were not deiced.

108. Damaged concrete was removed with the use of power hand tools to a minimum depth of 3 in. and all damaged reinforcing steel was repaired or replaced (Figure 58). A slurry of neat concrete was applied to the floors of the laterals and epoxy was applied to the wall before the placement of concrete. Approximately 40 cu yd of steel fiber reinforced concrete (FRC) was used in the repair of the discharge laterals (Figure 59). Mixture proportions for 1 cu yd of fiber concrete were as follows.

Figure 58. Preparation for fiber concrete placement, Upper St. Anthony Falls Lock (from St. Paul District 1976)

Figure 59. Completed repair using fiber concrete, Upper St. Anthony Falls Lock (from St. Paul District 1976)

Material	Weight, lb
Cement	900
Fine aggregate	1500 (SSD)
Coarse aggregate (3/8-in. MSA)	985 (SSD)
Steel fibers (1 in.)	200
Water	360

A 1-1/2-cu-yd trial batch of fiber concrete was mixed on 9 February 1976 by a local ready-mix supplier, under the direction of the steel fiber supplier. A representative of the Concrete Laboratory, WES, observed this demonstration, conducted slump and air content tests, and made cylinders and beams for test. The concrete was described as very workable and cohesive. Test results for this trial batch were as follows:

Test	Result
Air content, percent	4.5
Slump, in.	2-1/2
Flexural strength, psi	670 (7 days)
Flexural strength, psi	840 (28 days)
Compressive strength, psi	5270 (7 days)
Compressive strength, psi	7040 (28 days)

Seven placements of fiber concrete were made in the discharge laterals between 12 February and 24 February 1976. Average test results for these placements were as follows.

Test	Result
Air content, percent	5.8
Slump, in.	3-1/2
7-day strengths, psi	
Flexural	765
Compressive	5020
28-day strengths, psi	
Flexural	965
Compressive	6760

At this point 1/2-in. fibers were substituted for the 1-in. fibers, and four placements were made. Average results of tests on the plastic FRC were 4-1/4 in. and 5.5 percent for slump and air content, respectively. Strength specimens were fabricated from only one placement with results as follows.

	7 Days	28 Days
Flexural strength, psi	620	730
Compressive strength, psi	4450	5770

In addition to the fiber concrete repair, a 30-ft section of II steel was anchored on the floor and walls in lateral B, the center discharge lateral (Figure 60).

109. Test repair materials. It was decided to make the floor of the filling and emptying lateral in monolith 13 (bay H) a test section for different repair materials, as described in the following.

a. Epoxy mortar. A low-modulus epoxy mortar, 1/2- to 1-in. thickness, was to be used to overlay two sections of conventional concrete (Figure 61). Concrete to receive the overlay was left approximately 1 in. below grade. The mortar consisted of one part epoxy to three parts silica sand. Conditions during the repair were described as fair to poor with too much standing water to do a good job.

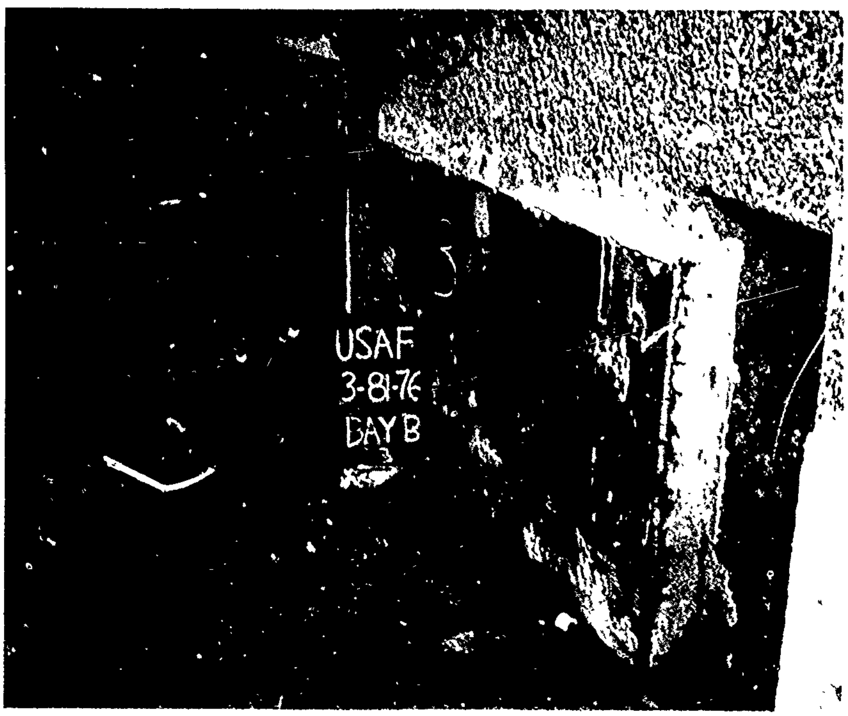


Figure 60. Sections of steel anchored to floor and walls of discharge lateral, Upper St. Anthony Falls Lock (from St. Paul District 1976)

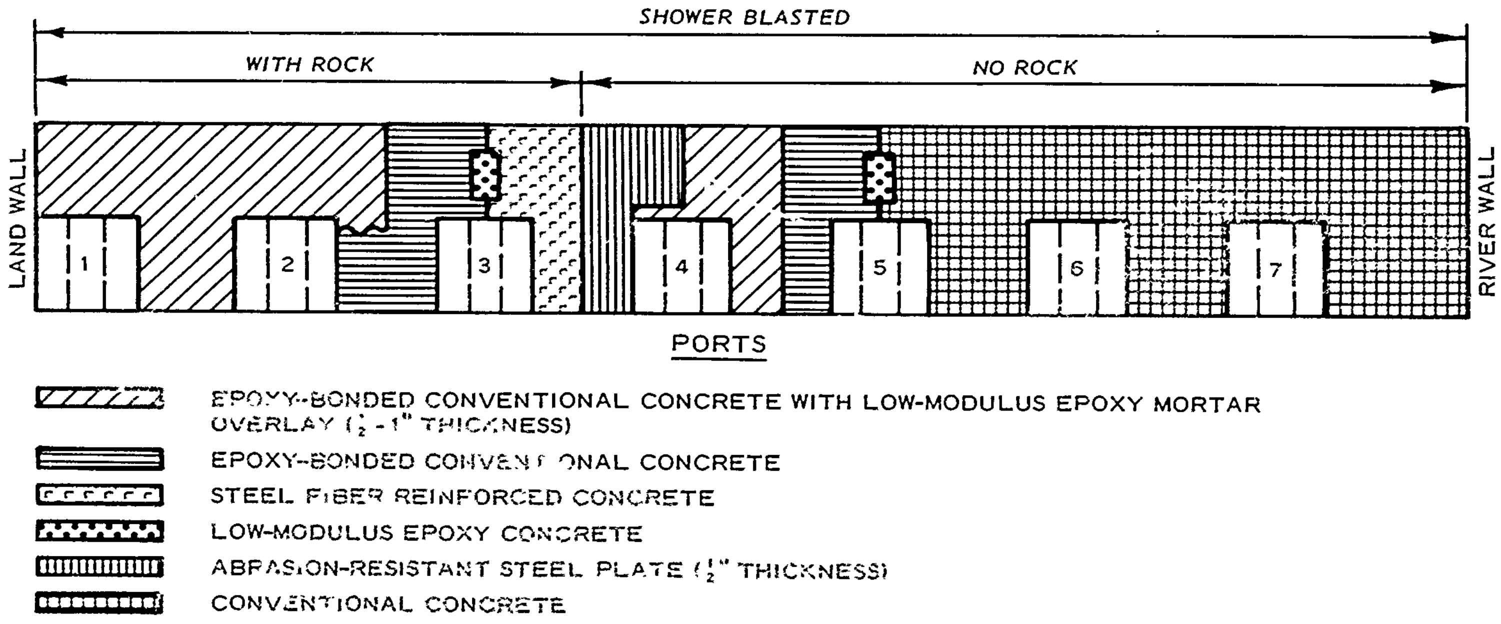


Figure 61. Experimental lateral (bay H), Upper St. Anthony Falls Lock

b. Conventional concrete. Conventional concrete bonded to the old concrete with an epoxy-bonding agent was used in the repair of two sections. No record of mixture proportions or strength characteristics of this concrete could be located. The concrete appeared to have 3/8-in. MSA, and there was speculation that the fiber concrete mixture, minus the fibers, was used.

c. Fiber concrete. In the test section, 1/2-in. steel fibers were substituted for the 1-in. fibers used in the discharge laterals. Apparently, no test specimens were fabricated from this placement; however, test results for a placement of this mixture in the discharge laterals were previously reported (paragraph 108).

d. Epoxy concrete. Two 12- by 24- by 6-in. sections were repaired with low-modulus epoxy concrete. Mixture proportions (yield approximately 1/2 cu ft) were 1 gal of epoxy, 25 lb of grit, and 20 lb of silica sand. It was the opinion of the epoxy representative present during the repair that the chances of getting good results were slim due to free water and sand in the holes prior to placement.

e. Steel plate. One section of an abrasion-resistant steel plate (1/2-in. thickness) was anchored to conventional concrete. After completion of repairs and prior to filling the lock chamber, rocks that caused the erosion damage were returned to their original positions in the lateral (Figure 61) to provide a positive test of the repairs.

110. Test repair results. Approximately 2 years after the repairs, dewatering of the lock allowed an examination of the repairs with results as follows:

a. Epoxy mortar. In spite of the pessimism during placement, the epoxy mortar repair appeared to have performed fairly well (Figures 62 and 63). With the exception of some minor erosion along the edges of the repair and a few small localized areas within the repair where the overlay appeared to be very thin, the epoxy mortar was in good condition. The section between ports 4 and 5 did not receive an overlay (see Figure 61).

b. Conventional concrete. There was general erosion of the entire epoxy-bonded concrete section exposed to abrasion by the rocks. At maximum depths (approximately 6 in.), erosion extended completely through the repair and into the old concrete (Figures 62 and 63). The section between ports 4 and 5 was not subjected to the abrasive effects of waterborne rocks and appeared to be in

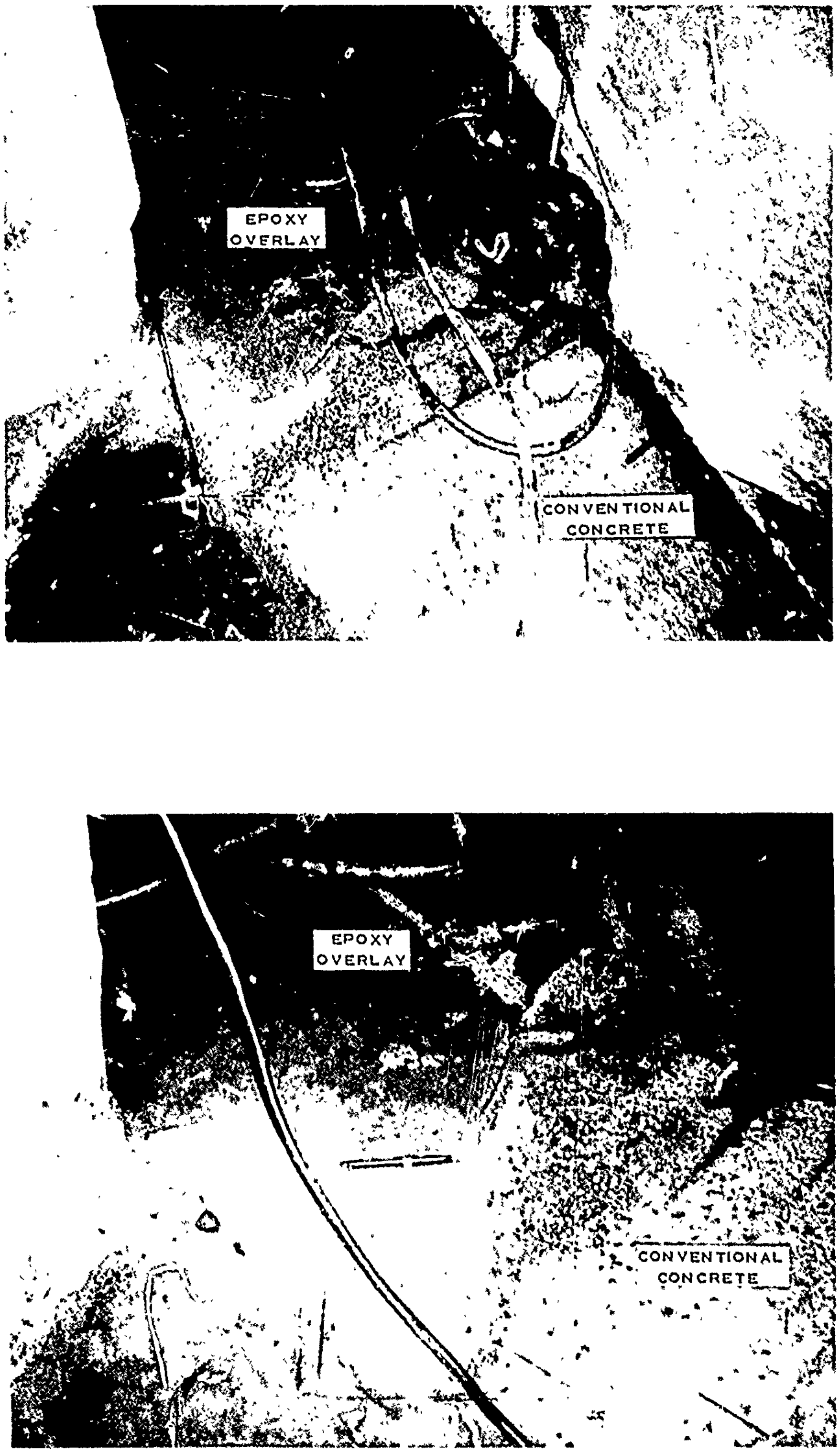


Figure 62. Views of epoxy mortar overlay and conventional concrete, Upper St. Anthony Falls Lock

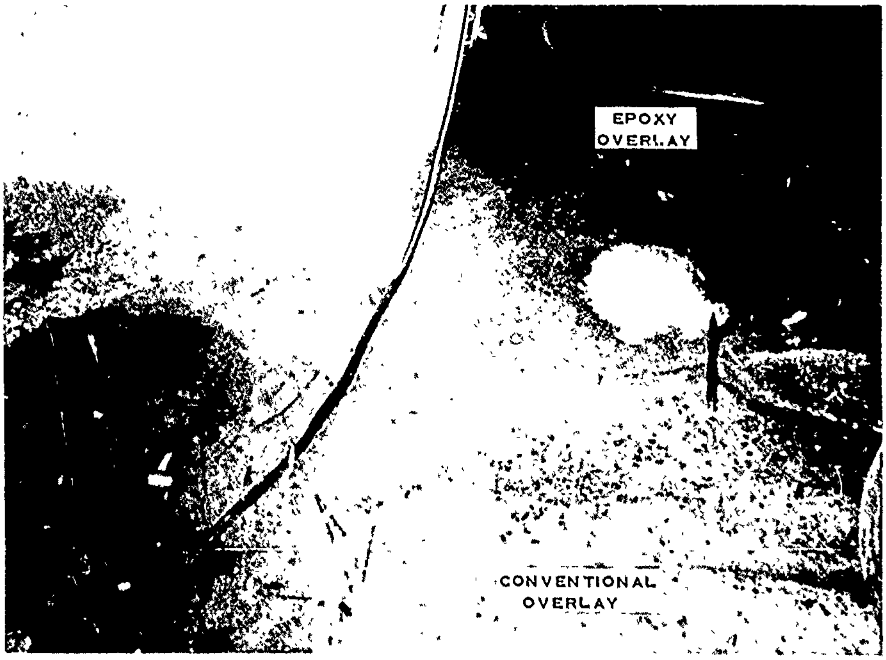


Figure 63. Epoxy mortar overlay and conventional concrete, Upper St. Anthony Falls Lock

essentially original condition. This was also true for the large section of conventional concrete adjoining the river wall.

c. Fiber concrete. The fiber concrete in the discharge laterals was not subjected to the abrasive effects of waterborne rocks in the laterals, and erosion in these areas was negligible. In comparison, fiber concrete in the test section exposed to abrasion by rocks exhibited considerable erosion (Figures 64 and 65). The pattern and extent of erosion were almost identical to the adjacent conventional concrete repair. There was speculation that the relatively slick finish on the epoxy mortar and steel plate on the boundaries of the eroded areas of fiber and conventional concrete may have contributed to a concentration of rocks in the eroded areas. In fact, the major areas of erosion in the conventional and fiber concretes (Figures 66 and 67) were connected immediately in front of port 3, indicating a transition or rock between the two areas.

d. Epoxy concrete. As was the case with the epoxy mortar, pessimism during placement appears to have been unfounded. The section of epoxy concrete repair subjected to abrasion was essentially intact with only slight erosion of the two corners that extended into the

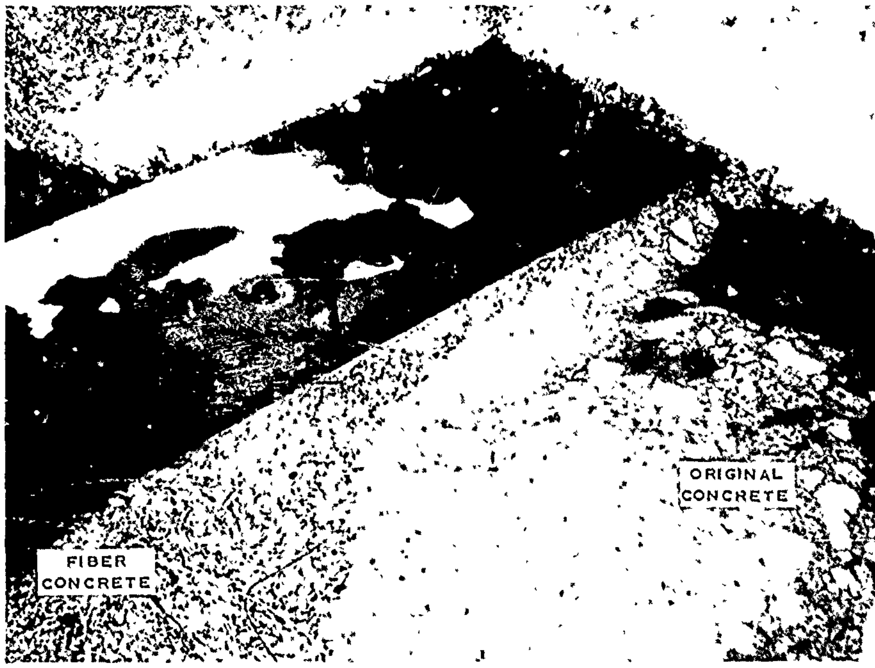


Figure 64. Erosion extending through the fiber concrete (along the steel plate) and into the original concrete, Upper St. Anthony Falls Lock

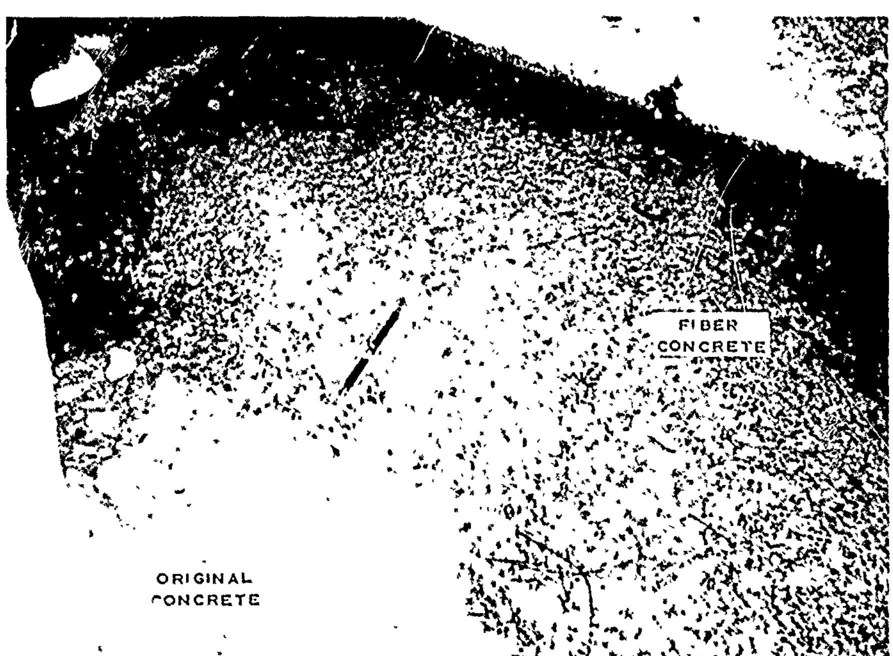


Figure 65. Erosion of fiber concrete extending into the original concrete, Upper St. Anthony Falls Lock

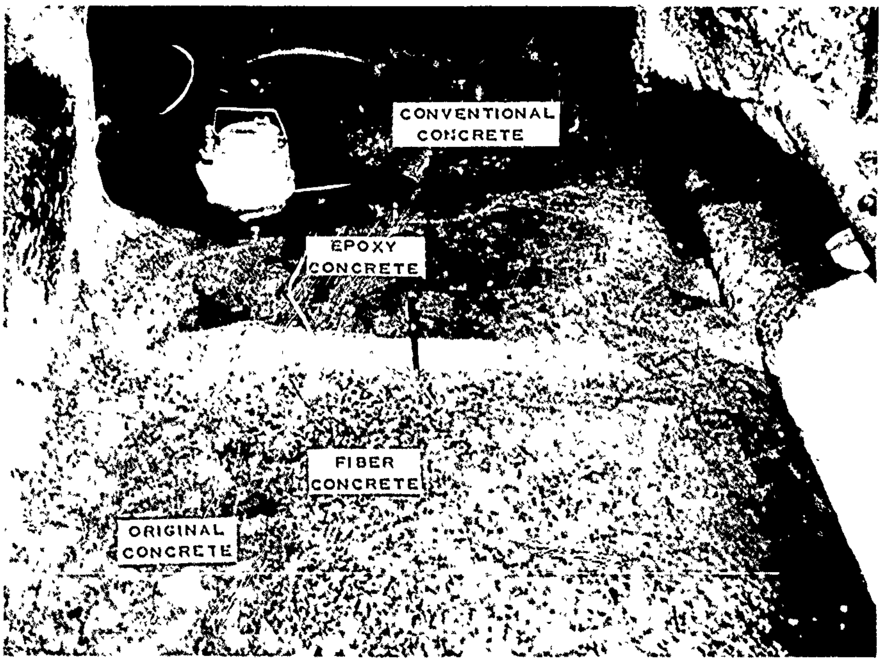


Figure 66. Erosion areas in fiber concrete, epoxy concrete, and conventional concrete, Upper St. Anthony Falls Lock

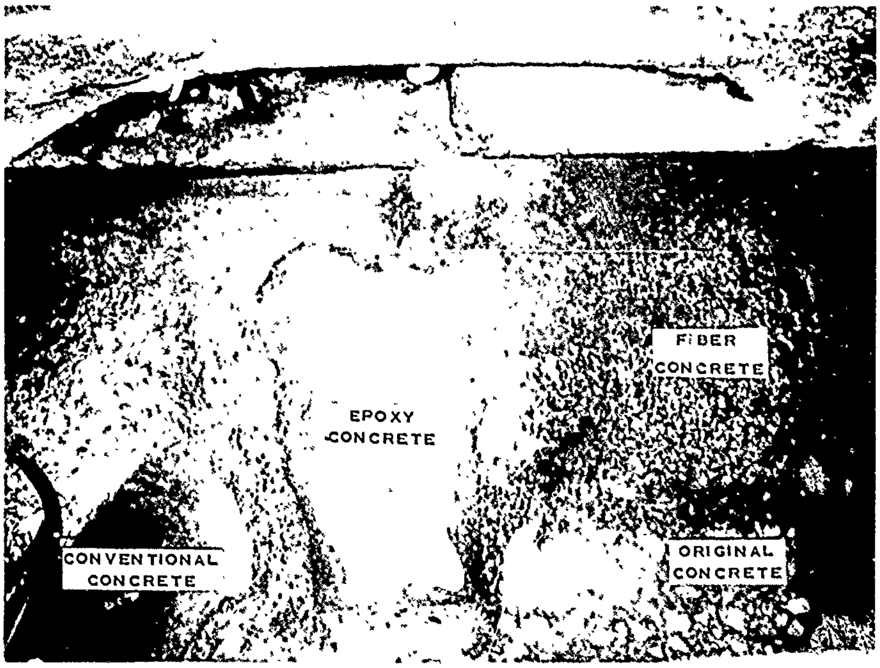


Figure 67. Erosion areas in epoxy concrete, conventional concrete, and fiber concrete, Upper St. Anthony Falls Lock

transition path between the fiber and conventional concrete erosion (Figures 66 and 67). The erosion resistance of the epoxy concrete is particularly significant, considering that erosion extended to the full depth and into the original concrete in portions of the adjacent conventional and fiber concrete. However, abrasion by boulders within the lateral may not have been as severe as areas between the ports because the repair area is small and is directly opposite the filling and emptying port. The second area of epoxy concrete was not subjected to the abrasive effects of rocks and appeared to be in essentially original condition.

e. Steel plate. The plate was on the boundary between the rock and no-rock sections. As might be expected, it exhibited little sign of wear, although the adjacent fiber concrete was eroded (Figure 67) and the nut on one of the anchor bolts was missing.

Bonneville Dam

111. Bonneville Dam is located on the Columbia River (Figure 68) between the states of Washington and Oregon. The Bonneville spillway consists of eighteen 50-ft-wide bays and seventeen 10-ft-wide piers. The spillway crest is at el +24, and the 63-ft-wide baffle deck is at el -16. The baffle deck has two rows of trapezoidal shaped baffles 6 ft high. Downstream of the baffle deck is a 77-ft-wide concrete apron, which has a minimum thickness of 6 ft, resting on bedrock. The spillway was constructed during 1935-37 and first put into use in the spring of 1938. Due to diversion through the south half of the spillway during construction, the baffle deck was used before 1938. The spillway discharge velocity is 50 fps.

112. Although the spillway design flood for the Bonneville Dam Project was set at 1,600,000 cfs, the spillway ogee crest and stilling basin have experienced severe erosion from the beginning of operation at river discharges less than 800,000 cfs. In addition, there has been considerable scour of the riverbed downstream from the lower apron. Divers made frequent surveys in the stilling basin for the first few years to follow the progress of the concrete erosion.

113. In the low-water seasons of 1941-42 and 1947-48, limited

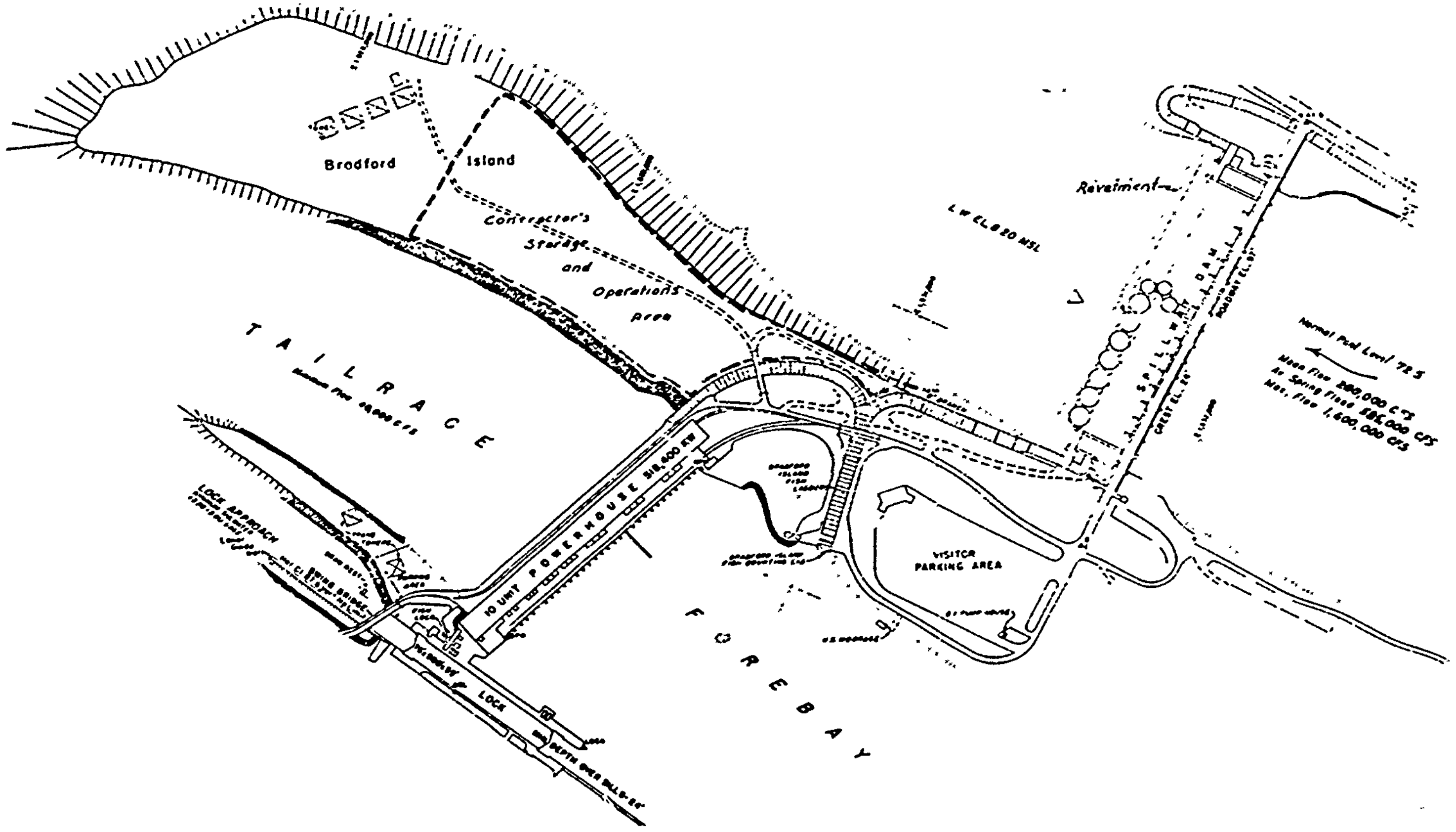


Figure 68. Site map, Bonneville Dam

areas of the baffle deck were repaired using a caisson method that enclosed two baffles. Because of the difficulty of sealing the caisson on the eroded baffle deck and the small area that could be repaired at each setting, the cost of repairs by the caisson method proved to be very high.

114. Prior to major repairs in 1954, various combinations of baffle designs, both with and without end sills, were investigated in a 1:36 scale, three-bay model to develop a stilling basin that would reduce scour of the riverbed and improve pressure conditions on the baffle piers. For economical and structural reasons, no change in the length of the baffle deck or in baffle pier locations was considered.

115. The adopted stilling basin design consisted of one row of baffles and a solid end sill (Portland District 1954). Sloping-faced baffles 7.5 ft wide by 6.0 ft high were adopted in place of the original upstream row of 6-ft-wide baffles. Pressures on the baffles were appreciably increased from -12 ft by replacing the 9-in. radii on the upstream corners with elliptical side curves. The solid end sill was located over and shaped to the cross section of the existing second row of baffles. It was necessary, for structural reasons, to raise the baffle deck from el -16 to el -15 upstream from the end sill, leaving the baffle deck downstream from the end sill at el -16.

116. The annual spring high discharge of the Columbia River limits the construction season to the 8-month period of 1 August to 1 April of the following year. Use of the full length of the spillway must be available for the remainder of the year; therefore, all cofferdams must be removed at the end of each working season. Therefore, it was proposed to award a contract for cofferdamming and repair of the south half of the spillway and baffle deck to be done during one low-water season.

117. In 1954, the south half of the stilling basin, bays 10 through 17, was dewatered and major repairs were made to the baffles, baffle deck, pier noses, and the lower slopes of the ogee crest. Typical erosion of the spillway and stilling basin prior to these repairs is shown in Figure 69. Concrete condition ranged from areas having

a. Baffle and deck erosion

b. Pier nose and apron erosion

Figure 69. Erosion in south half of spillway prior to 1954 repairs, Bonneville Dam (from Portland District 1967)

little or no erosion to areas having eroded holes several feet in depth from which concrete as well as the reinforcing steel had been removed.

118. Details of the apron and baffle repairs are shown in Figures 70 and 71. The upstream row of baffles was reconstructed whereas the downstream row was covered by the new end sill. The baffle deck was repaired by anchoring a reinforced concrete slab (12-in. minimum thickness) over the existing deck.

119. In an effort to obtain concrete with the greatest possible resistance to erosion and cavitation, the water-cement ratio was not allowed to exceed 0.40 by weight. Also, forms were lined with 1/2-in.-thick absorptive form liners or vacuum mats in order to eliminate voids and case-harden the concrete surfaces. Shallow unformed areas of erosion were gunited or repaired by stoning a mixture into the surface. After application of the mortar by either of these methods, the surfaces were cured and then ground to produce a smooth slick surface.

120. Diver surveys in 1958 and 1965 showed increasing but not critical erosion of the repaired baffle deck downstream of the upstream baffles and only minor erosion on the pier noses and the solid downstream baffle (Portland District 1967). The diver inspector reported that the erosion in the basin floor appeared much like a horseshoe downstream and behind the baffles. This new erosion pattern indicated a different scouring flow, probably due to the solid baffle (Portland Ditrict 1973).

121. During the next inspection in 1968, two baffles in each bay, the pier noses, the solid baffle in the south half, and portions of the end of the apron were checked. No increased erosion was found. The erosion in the north half had not changed, and remedial treatment similar to one used in the south half was not considered warranted at that time. Downstream of the apron below the stilling basin floor, the divers found gravel piled back against the south solid baffle wall, and found considerable loss (estimated 40 percent) of boulder fill placed to retard erosion below the apron in the north half. In some areas large holes had eroded below and under the apron.

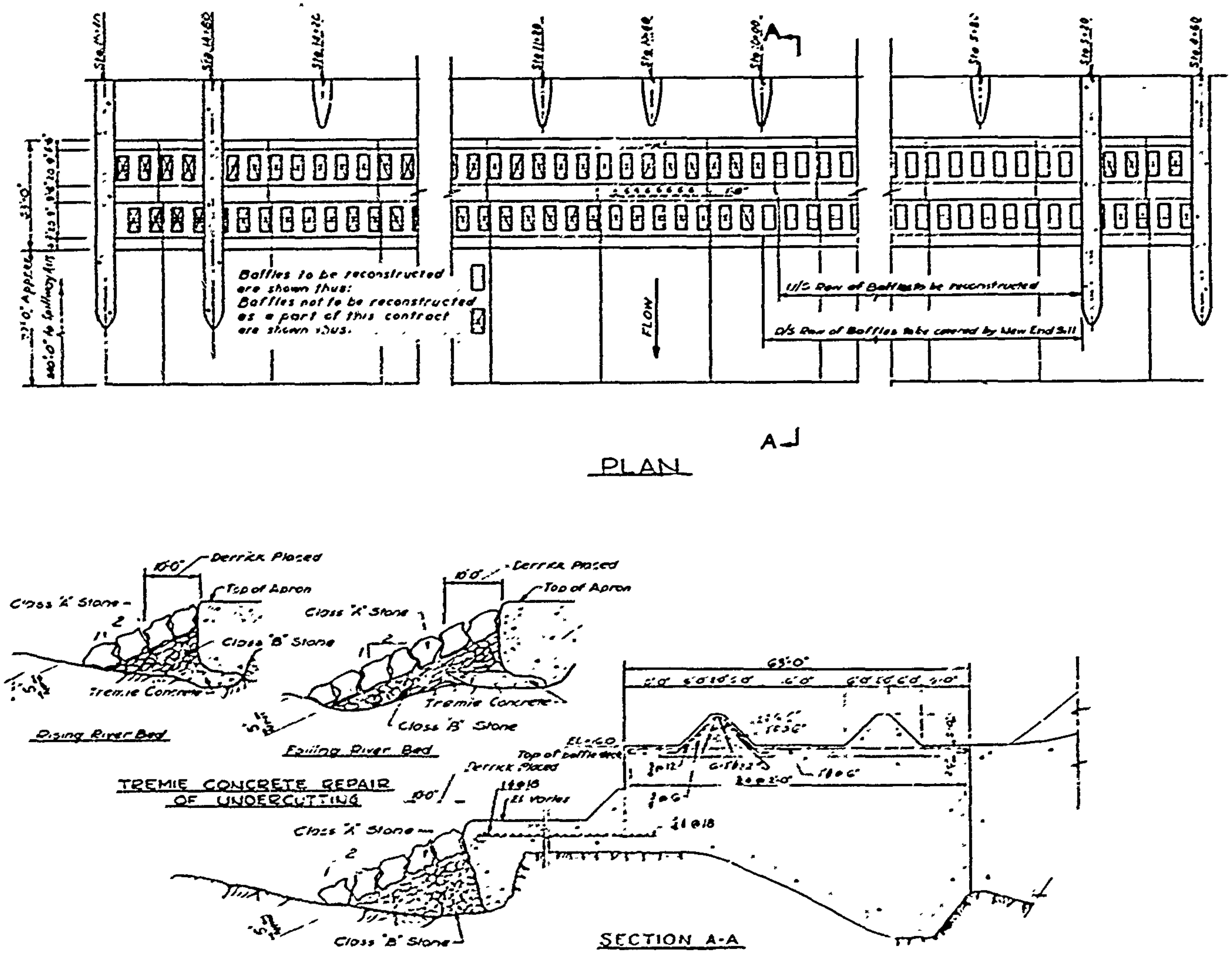


Figure 70. Apron and baffle repair, Bonneville Dam

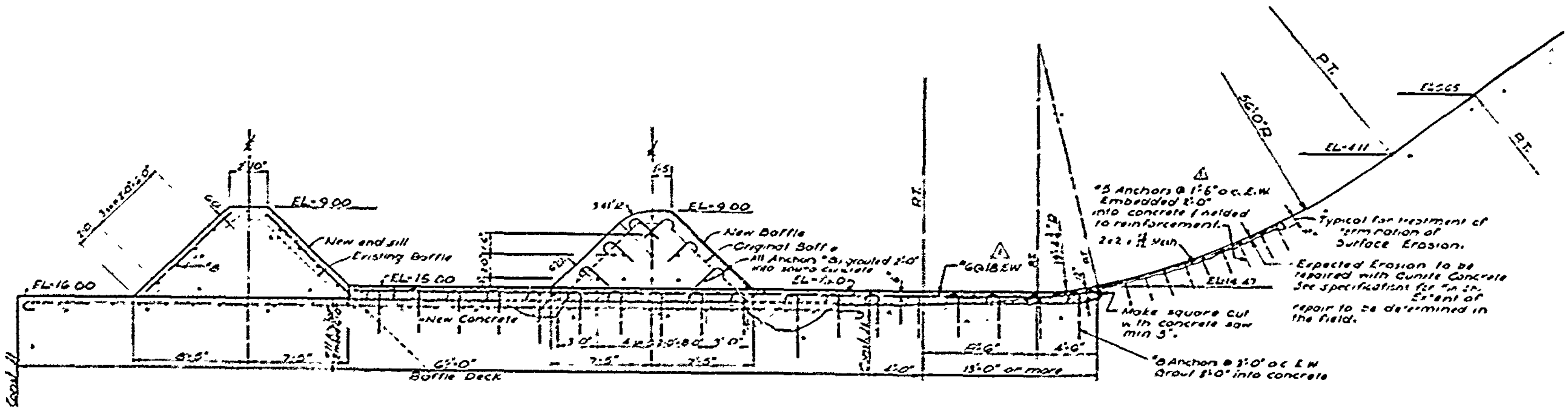


Figure 71. Baffle deck overlay details, Bonneville Dam

122. No significant changes from the previous inspection were noted in a diver inspection of the stilling basin in the fall of 1972.

Chief Joseph Dam

123. Chief Joseph Dam is located on the Columbia River near Bridgeport, Washington. Major features of the project are the dam, overflow spillway, and powerhouse (Figure 72). The stilling basin is designed to dissipate the energy of 1,250,000 cfs of water over the spillway. The annual peak flow of 350,000 to 500,000 cfs can be expected over a 5- to 7-month period. The stilling basin is approximately 920 ft wide and 220 ft long and is divided into four rows of concrete slabs approximately 65 ft wide, 50 ft long and 5 ft thick (Figure 73). The slabs are anchored to the foundation rock with grouted anchor bars. The slabs

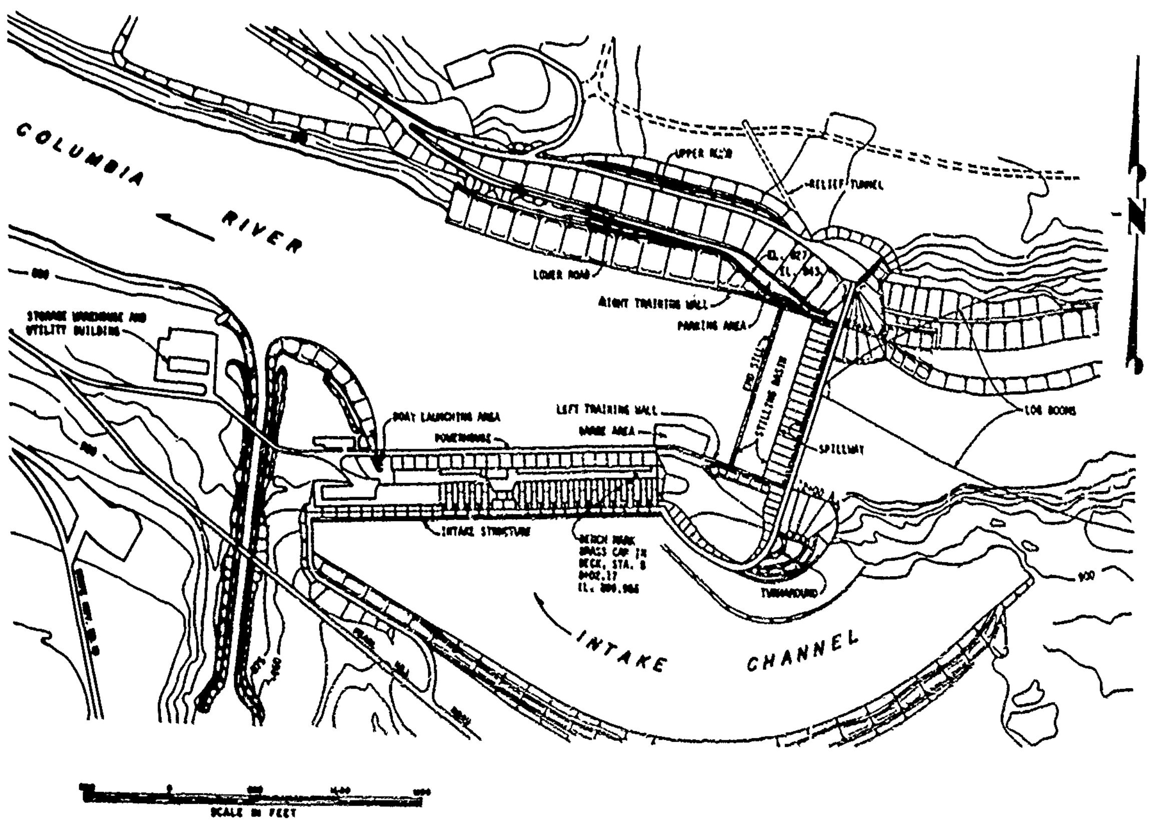


Figure 72. Location plan, Chief Joseph Dam

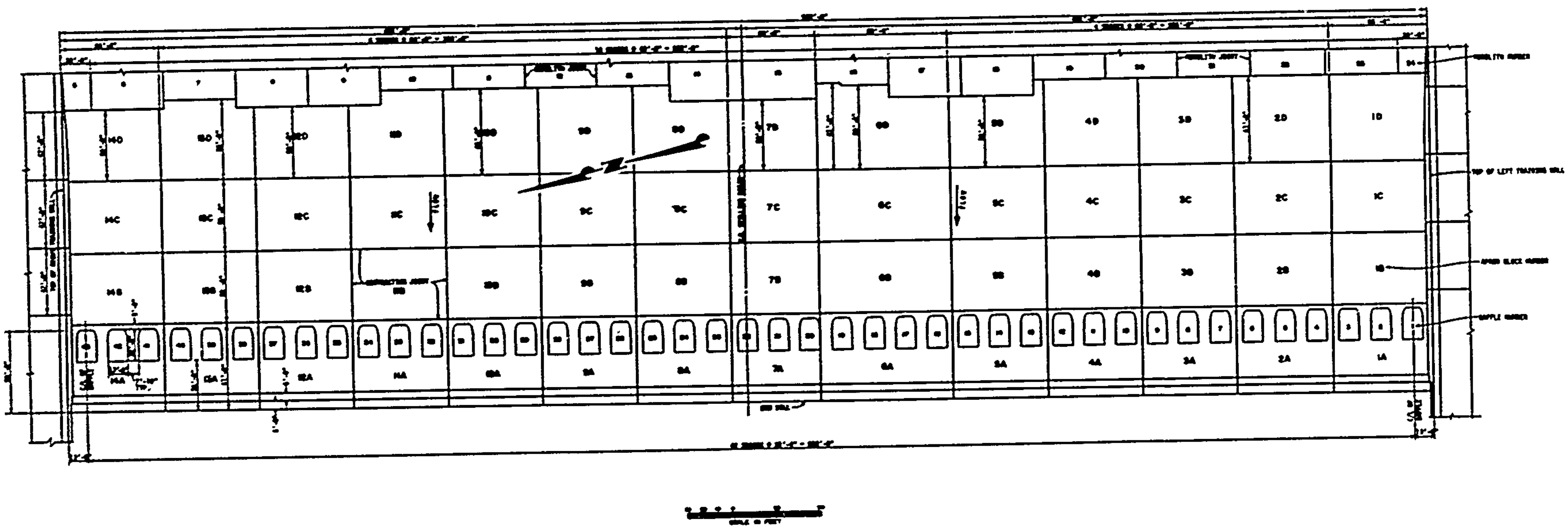


Figure 73. Stilling basin plan, Chief Joseph Dam

are unreinforced except the downstream row, which carries a single row of baffles and the end sill.

124. Extensive areas of eroded concrete were discovered during an underwater inspection in March 1957, two years after the project became operative. An underwater survey during November 1957 substantiated and located the areas of widespread damage. Areas were found in which there was erosion of concrete in excess of 12 in. in the slab and in the baffles, which exposed the reinforcing steel. Severe damage was concentrated in localized areas immediately upstream of some baffles, between baffles and between baffles and the end sill. Two small scour holes approximately 5 ft in depth were discovered between the baffles and end sill. Additional damage to the stilling basin, observed during the March 1960 survey, was found to be minor as compared with the pattern of major damage found in the 1957 survey.

125. The major damage was apparently caused by abrasion of large-size rock particles together with concentration of river flows through constricted sections of the stilling basin during the dam construction. The stilling basin has been subjected to passage of flood flows that cause velocities in excess of 100 fps at the bucket of the spillway. Under such circumstances, progressive deterioration from flow impingement and cavitation of the damaged areas and areas adjacent to the damaged areas could be expected. Therefore, laboratory studies were initiated to determine the most practical methods for repairing the stilling basin.

126. The high costs of cofferdam construction to make repairs in the dry basin required that consideration be given to underwater methods of repair. Results of laboratory experiments to determine methods for forming, placing, and finishing concrete under water indicated that underwater repair techniques might be successful and encouraged proceeding with trial prototype repairs. An experimental pilot repair was made during the period August 1960 to January 1961 by the Seattle District (1961) to develop construction techniques and cost data and to evaluate the effectiveness, practicability, and costs of an underwater method of repair.

127. An eroded section of concrete 30 by 49 ft in the stilling basin floor (Figure 74) was selected for the pilot repair. The damage in this area was typical of eroded areas with exposed aggregate and an erosion depth ranging from 3 to 16 in. This area in blocks 28 and 38 presented problems typical of many damaged areas, repair of which required a flat concrete overlay slab and a sloped upstream transition slab.

128. The concrete overlay slab was placed, utilizing underwater intrusion grouting, in six 10- by 20-ft sections for the flat overlay slab portion and three 10- by 9-ft sections for the sloped upstream transition slab portion. The sloped upstream transition slab was provided to minimize cavitation. Fillets having a slope of 1 on 2 were placed against the downstream left and right sides of the slabs to cover the rough vertical sides of slabs that were formed against sandbags. The flat overlay slab was placed with the finish grade 4 in. above the original finish grade of the stilling basin floor. This ensured a minimum slab thickness of 4 in. and held chipping of existing concrete under the sloped upstream transition slab to a minimum.

129. Repairs were accomplished under water at a depth of about 26 ft and in a current of approximately 0.5 fps. Repair work was done by two divers in hard helmet deep-sea diving gear. The repair site was located by measurements from the stilling basin contraction joints. The sequence for placing the various slabs is shown in Figure 74.

130. The repair work was observed during construction by engineers through the use of a 35-ft-long underwater scope equipped with a 6-in.-diameter bottom glass and a high power telescope. Eroded concrete was found to be thoroughly clean after cleanup with wire brushes, water jet, and air-actuated dredge. Surfaces of newly placed concrete were smooth except for small isolated areas which had scaled off to a depth of 1/16 to 1/18 in. Some wavy, shallow depressions were found that had been formed by wrinkles in the polyethylene sheet placed under the top form panels.

131. Beam (6 by 6 by 24 in.) and cylinder (6 by 12 in.) test specimens filled with aggregate were intruded with grout drawn from the mixer

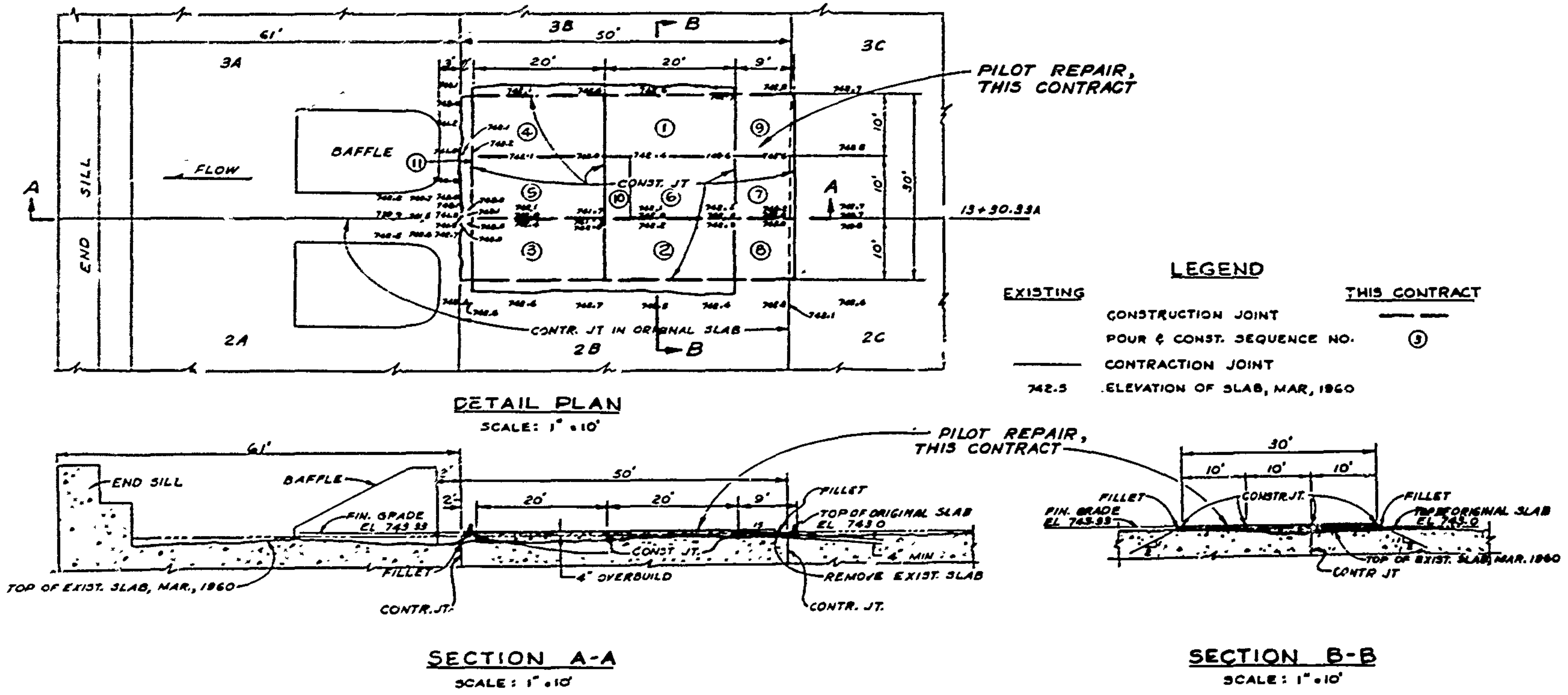


Figure 74. Pilot repair, stilling basin, Chief Joseph (from Seattle District 1961)

at the time of intrusion of seven pours. Specimens were cured under water in laboratory tanks held at river temperatures until after final set of concrete. After the forms were removed, the beams and cylinders were placed in the river until time of testing except for one day required to transport them to the testing laboratory. Flexure and compressive strengths were as follows.

Beams	Age Days	Flexure Strength, psi			Range of River Temperatures During Curing, °F
		Maximum	Minimum	Average	
	35	330	330	330	44 to 39
	49	410	380	395	45 to 39
	64	500	490	495	49 to 39
	72	395	350	375	51 to 39
	83	420	415	418	53 to 39
	91	490	450	470	56 to 39
	164	440	--	440	58 to 39

Cylinders	Age Days	Compressive Strength, psi			Range of River Temperatures During Curing, °F
		Maximum	Minimum	Average	
	35	2260	2120	2190	44 to 39
	49	2810	2370	2590	45 to 39
	64	3350	3020	3185	49 to 39
	72	2600	2460	2530	51 to 39
	83	3310	2680	2995	53 to 39
	91	3310	3000	3155	56 to 39
	164	4365	4050	4210	58 to 39

132. Construction cost of the experimental pilot repair was $60,000. The unit cost to repair 1690 sq ft of eroded concrete, including the sloped upstream transition portion and fillets, was $35.50/sq ft or $1200/cu yd. Construction and mobilization of a floating plant, forms and miscellaneous equipment, and experimental work of the diving crews accounted for a large percentage of the total cost. Cost of future construction would be considerably less as the floating plant forms and accessory equipment are now available at the project. Experience gained from the operations of the pilot repair could be utilized to increase efficiency of labor and services, also reducing cost of future repair work.

133. Based on the pilot repair, it was concluded that methods and

procedures were developed to the extent that underwater repair of eroded concrete in the stilling basin was considered to be physically feasible and practical. The desired strength, bond, and finished surface of concrete in the underwater pilot repair appeared to be satisfactory. Construction costs, although high in certain instances, would be reduced for large-scale repair. The cost per square foot of slab for the pilot repair excluding plant, mobilization and demobilization, forms, and job supervision was $19.50, or $660/cu yd of concrete. Evaluation of the quality and effectiveness of the repair to withstand high velocity flows would require observation over a period of several years. Therefore, an inspection of the repair work was planned annually during the fall months, starting in 1961, to determine whether an adequate bond between concrete surfaces was obtained and to determine which of the three concrete finishes (by finisher, by sloping top form panels, or by horizontal top form panels) could best withstand the high velocity flows.

134. Divers made the initial inspection (Seattle District 1962) of the repair slab between 13 and 15 November 1961. Elevations of top of concrete were obtained and compared with those obtained at the time repair work was completed. Underwater photos were taken and compared with earlier photos taken at the same locations. The condition of the pours was described by means of telephone communication with divers and recorded by the engineer supervising the inspection.

135. Prior to the inspection, the repair slab had been exposed almost continuously to spillway discharge from January to November 1961. Maximum discharge was 446,000 cfs. The average discharge was about 112,000 cfs. The exposure in 1961 was about the average to be expected during any year.

136. Although parts of the slab were found to be damaged (Figure 75), the major portion of the slab was in excellent condition. A large part of sections 1 and 2, which are each 10 by 20 ft, was gone. The original concrete surface was exposed, indicating a bond failure of repair concrete to original concrete. The bond failure is attributable to several factors. Considerable difficulty was encountered when the side forms were removed from sections 1 and 2. An excessive amount of

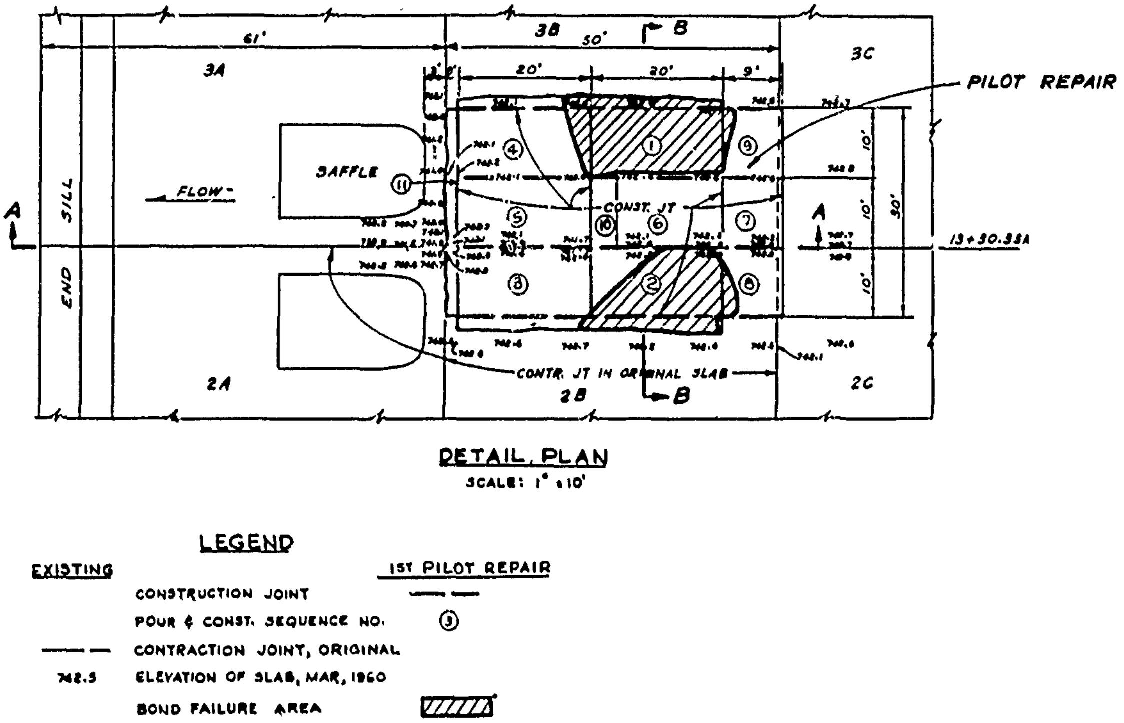


Figure 75. November 1961 inspection of pilot repair, Chief Joseph Dam (from Seattle District 1962)

grout was intruded into the two sections and flowed over the top and along the outside of the forms. This overflow caused the side forms to be concreted in and necessitated the use of 15-ton capacity railroad jacks to remove them. The force used to lift the side form pieces out of the concrete could have been sufficient to break the bond between the freshly placed repair concrete and the original concrete. In later sections, the divers used extreme care to avoid overtopping the side forms. This precaution permitted forms to be removed with relative ease.

137. A secondary cause of failure may have been the lesser degree of fluidity of grout in the first two sections than in succeeding pours. Flow cone measurements of grout in sections 1 and 2 varied from 21 to 24 seconds while the flow cone measurements of most of the remaining sections varied from 19 to 20 seconds. The greater degree of fluidity in the later sections may have improved the intrusion flow around and

between the preplaced coarse aggregate lying in contact with the original concrete, resulting in a better bond of new to old concrete.

138. Small parts of sections 4, 8, 9, and 11 lying adjacent to the first two sections were torn out due to the failure in sections 1 and 2. The upstream edges of the transition sections 7, 8, and 9 were in good condition. The upstream feather edges of transition sections 7 and 9 resisted stilling basin currents as did the edge of section 8, which was placed in a 4- to 6-in. slot cut into the eroded concrete. A small part of the surface of section 8 had coarse aggregate in relief at the time of completion of repair work due to lack of grout. This area has eroded several inches since placement. The surfaces of all other sections remaining in the repair slab exhibited no signs of erosion.

139. Based on results from 1 year of operation, the following tentative findings were indicated.

a. The repair concrete has a sufficient bond to eroded concrete to withstand high velocity stilling basin currents.

b. The three types of surface treatment (finisher, sloping top form panels, and horizontal top form panels) were found to be equally effective.

c. Upstream edges of transition sections do not appear to require that placements be made in slots dug in eroded concrete.

d. Repair operation must be conducted with special care to prevent breaking of bond between old and new concrete. Templated side forms designed to minimize grout leakage as well as to permit easy form removal should be employed.

140. The second inspection of the experimental repair slab in November 1962 (Seattle District 1963) revealed that additional damage had occurred since the first inspection of November 1961. This damage occurred in sections 4 and 9 and amounted to approximately 50 and 35 sq ft, respectively (Figure 76). The eroded concrete surface of the original stilling basin floor was exposed in these areas, indicating an apparent bond failure of the repair concrete. This damage was similar to that found after the first year of exposure to spillway discharge. The maximum and average discharges during 1962 were 300,000 and

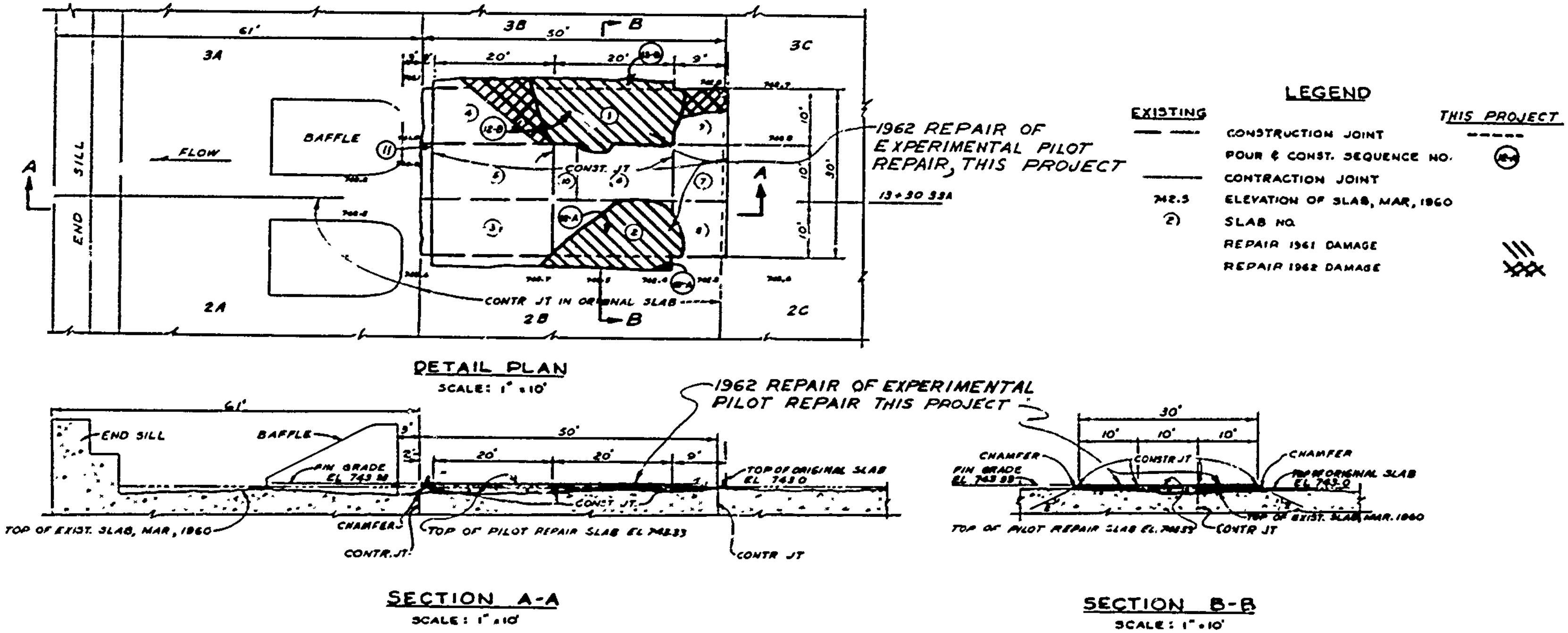


Figure 76. November 1962 inspection and repair of experimental pilot repair, Chief Joseph Dam (from Seattle District 1963)

114,000 cfs, respectively. The apparent bond failures observed after the two inspections could not be readily traced to any single cause. However, in an effort to obtain a better bond, two modifications to the original methods of repair were suggested: (a) clean the eroded concrete surface by underwater sandblasting, and (b) omit large aggregate from part of the repair area.

141. The additional underwater experimental repair was undertaken to refine construction techniques, seek improved methods of underwater concrete surface cleaning, and replace damaged portions of the pilot repair. The 1962 repair was confined to the damaged areas within the original pilot repair slab. The area repaired was divided into two separate sections, as shown in Figure 76. The larger of the two sections, 12-B, is approximately twice the size of the smaller section, 12-A. Concrete chamfers, 13-A and 13-B, were placed against sections 12-A and 12-B, respectively.

142. The repair concrete was placed using underwater intrusion grouting in two flat overlay sections (12-A and 12-B) with surface elevations set to match the existing pilot repair. The upstream end of the placement was sloped to conform with the existing transition section. In addition to the flat overlay sections, chamfers having a slope of 1 on 2 were placed against the north and south sides of the new placements to replace damaged chamfers. Sequence for placing the various sections is shown in Figure 76. Section 12-A was cast using aggregate and intruded grout. Section 12-B and chamfer placements 13-A and 13-B were cast with intruded grout only. The repair required approximately 4 cu yd of minus 1-1/2 in. aggregate and 13-1/2 cu yd of grout. Aggregate gradings, mixture proportions, and construction techniques are described in detail in the Seattle District (1963) report.

143. Construction cost of the 1962 repair was $24,000. The unit cost to repair 525 sq ft of eroded concrete, including the sloped upstream transition portion and chamfers, was $45.50/sq ft, or $1350/cu yd, of concrete. The unit costs of future large-scale repairs of the stilling basin were expected to be much lower because of the experimental nature of the 1960 and 1962 repairs.

144. A 1963 inspection of the repairs revealed that the grout mix area failed completely except for occasional thin patches of grout bonded to the original stilling basin slab. The remainder of the pilot repair slab was in excellent condition.

145. By 1966, erosion in the stilling basin had progressed to maximum depths of approximately 6 ft. The most severe erosion was located in areas between the row of baffles and the end sill. Also, there was significant erosion between some of the baffles. Therefore, it was decided to repair portions of blocks 2A, 3A, and 7A-12A (Figures 77-83) using pumped concrete and preplaced aggregate concrete.

146. Prior to the concreting operations, debris was removed from the repair areas, exposed reinforcement was cleaned, and an estimated 6200 sq ft of concrete surface area was cleaned. After holes for anchors were drilled, they were filled with grout, the anchors were embedded, and the horizontal reinforcement was positioned.

147. In order to maintain a 4-in. minimum slab thickness of preplaced aggregate concrete at the upstream terminus, an estimated 120 sq ft of concrete had to be removed from block 2A. This was accomplished with a barge-mounted air drill. A diver guided the placing of the point using a metal guide and loaded the broken concrete into a ship. Concrete buckets containing the coarse aggregate were guided into position and dumped by a diver. Screeds placed on the preset edge forms were then used by the divers for leveling the aggregate to the proper grade before placing the top form panels. Grout pipes were driven the full depth of the coarse aggregate.

148. A firm experienced in producing preplaced aggregate concrete was retained to provide equipment and to supervise the grouting operation. The mixer and pumps were set up on the south training wall, and grout was pumped first through grout pipes in the deepest area of the placement until a return showed through the vent pipes surrounding that area. The pipes were plugged with corks when good sanded grout appeared. When grout showed in the next row of vent pipes, the grout hose was moved to an adjacent pipe and the grout hole was plugged. This procedure was continued until the entire form had been pumped. Approximately

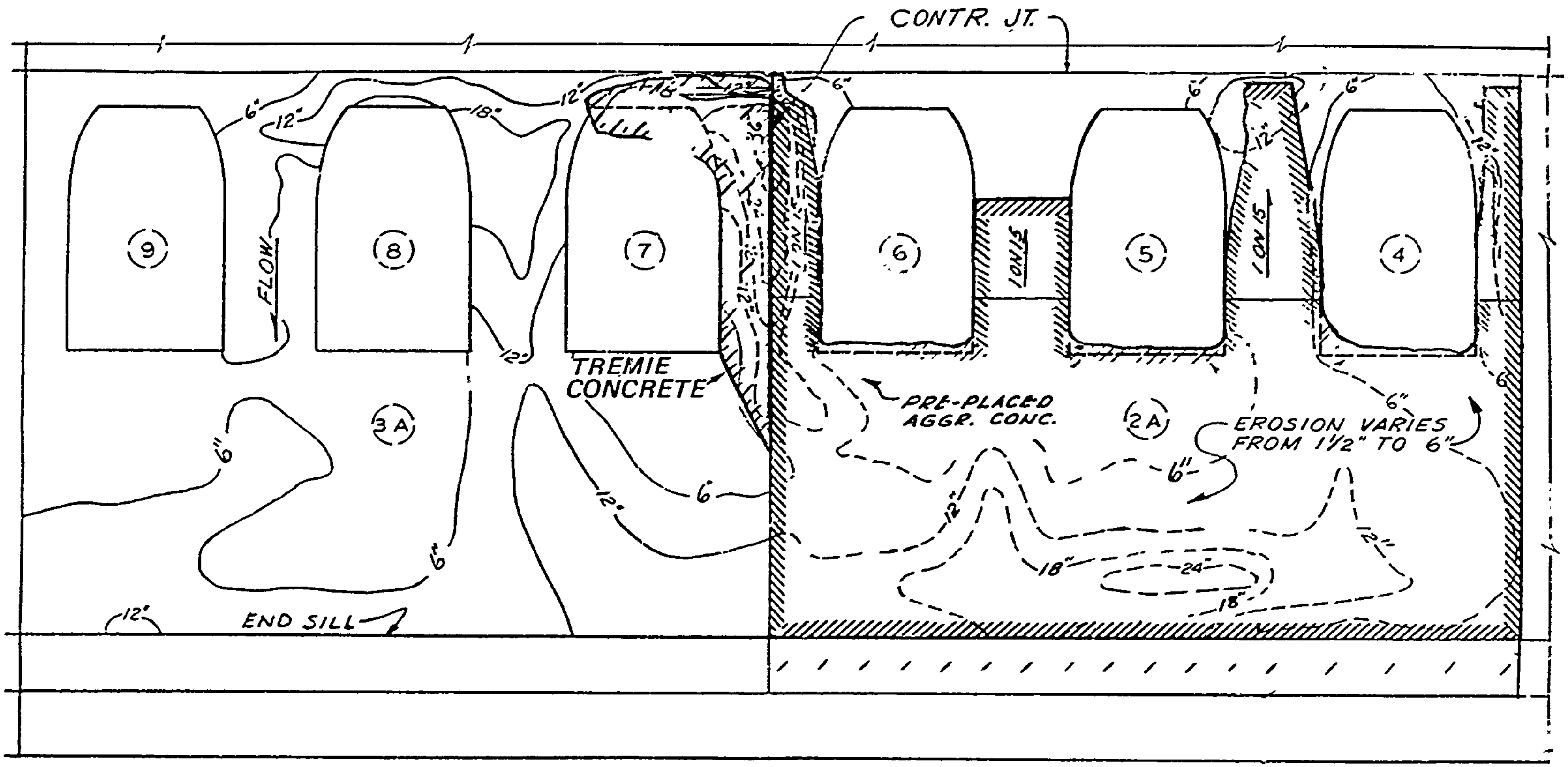


Figure 77. Preplaced aggregate concrete repair of stilling basin erosion, blocks 2A and 3A, Chief Joseph Dam

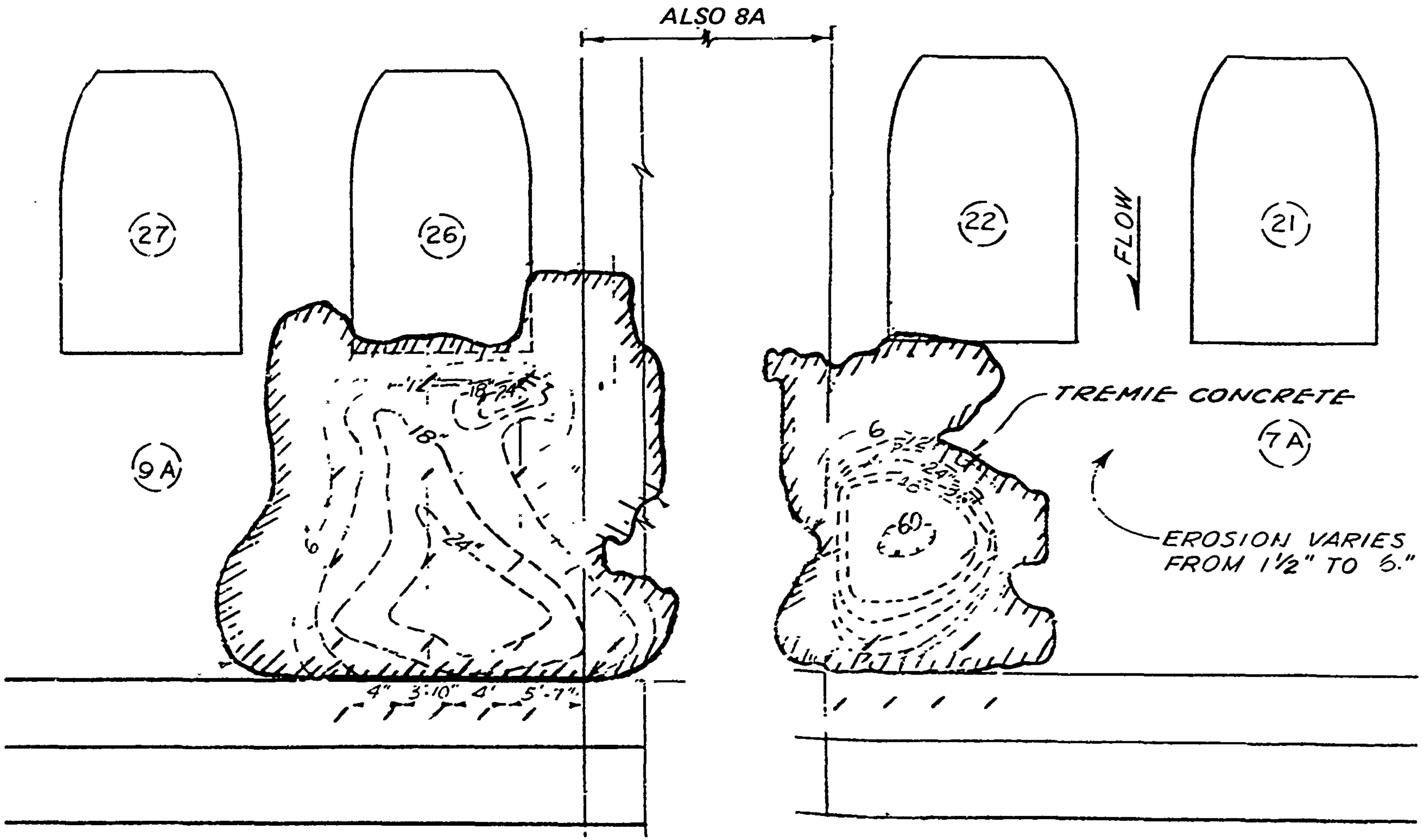


Figure 78. Tremie concrete repair of stilling basin erosion, blocks 7A-9A, Chief Joseph Dam

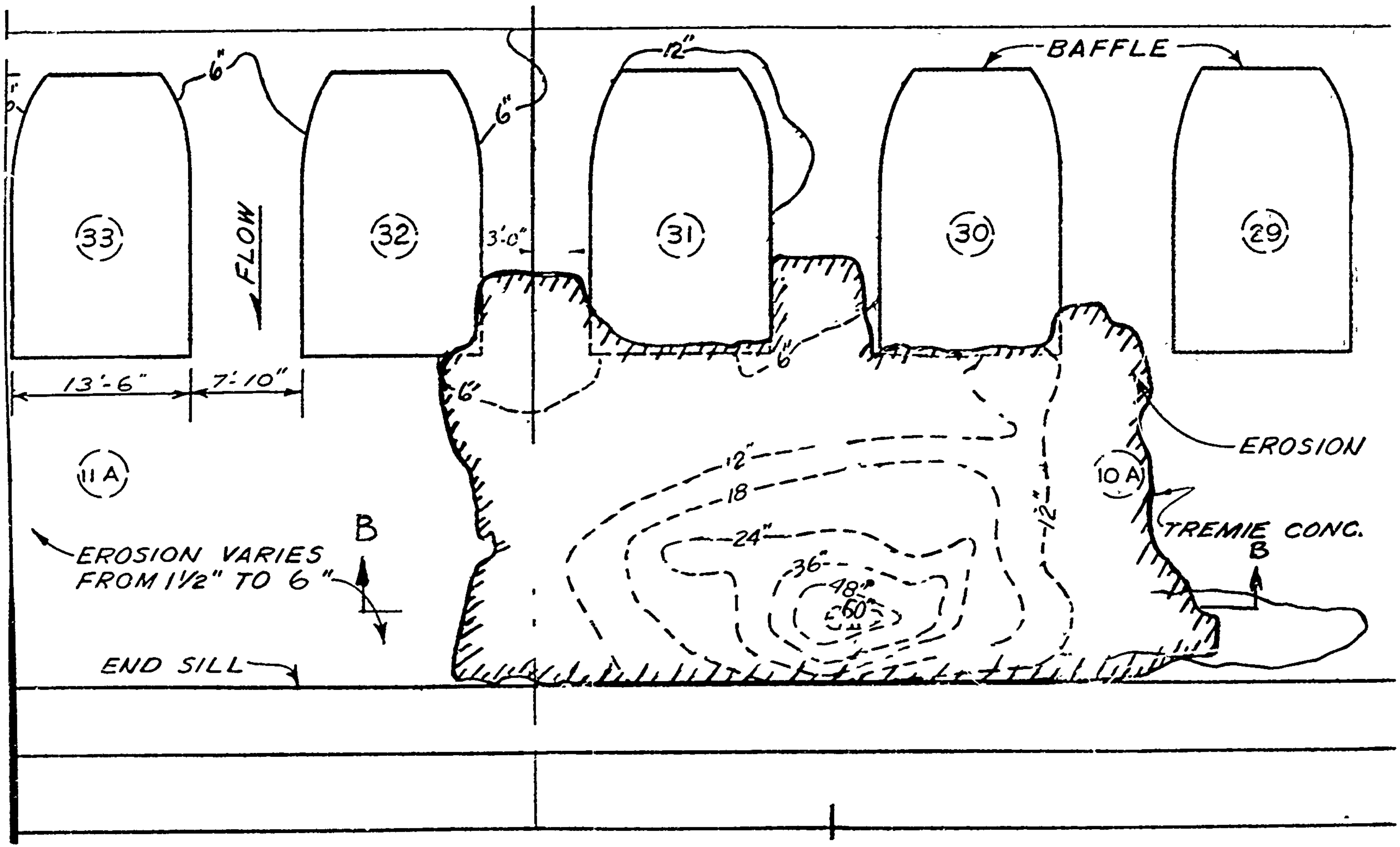


Figure 79. Tremie concrete repair of stilling basin erosion, blocks 10A and 11A, Chief Joseph Dam

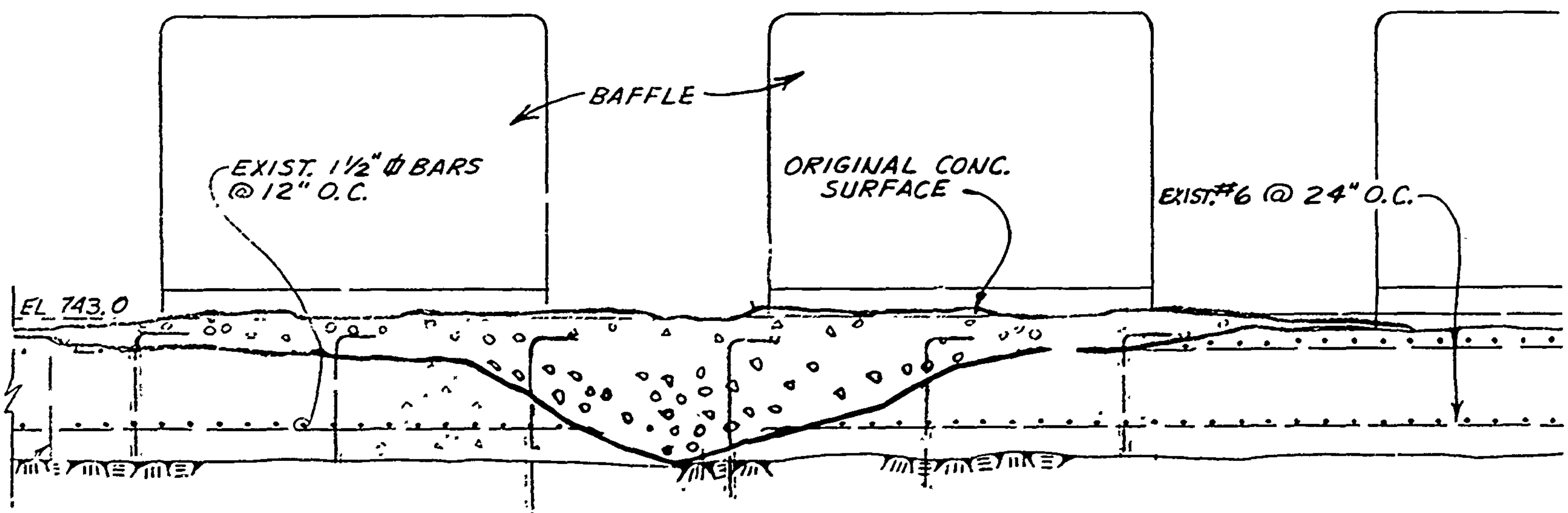


Figure 80. Typical transverse section, blocks 10A and 11A, Chief Joseph Dam

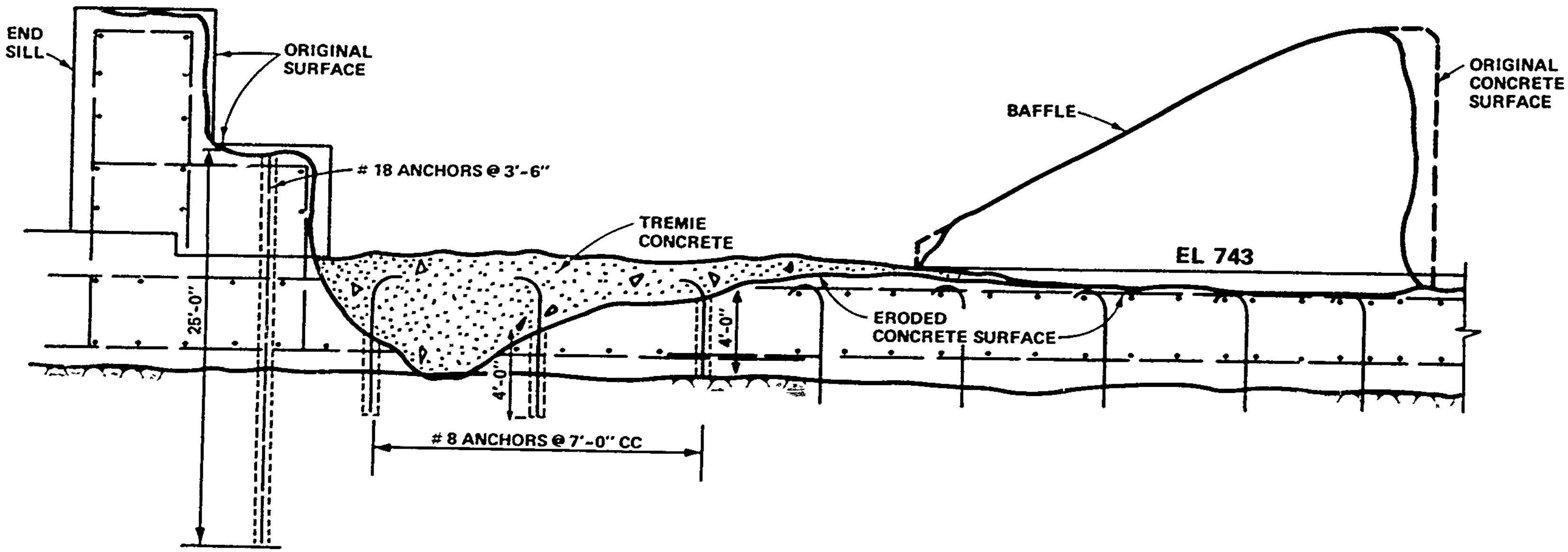


Figure 81. Typical longitudinal section, block 10A, Chief Joseph Dam

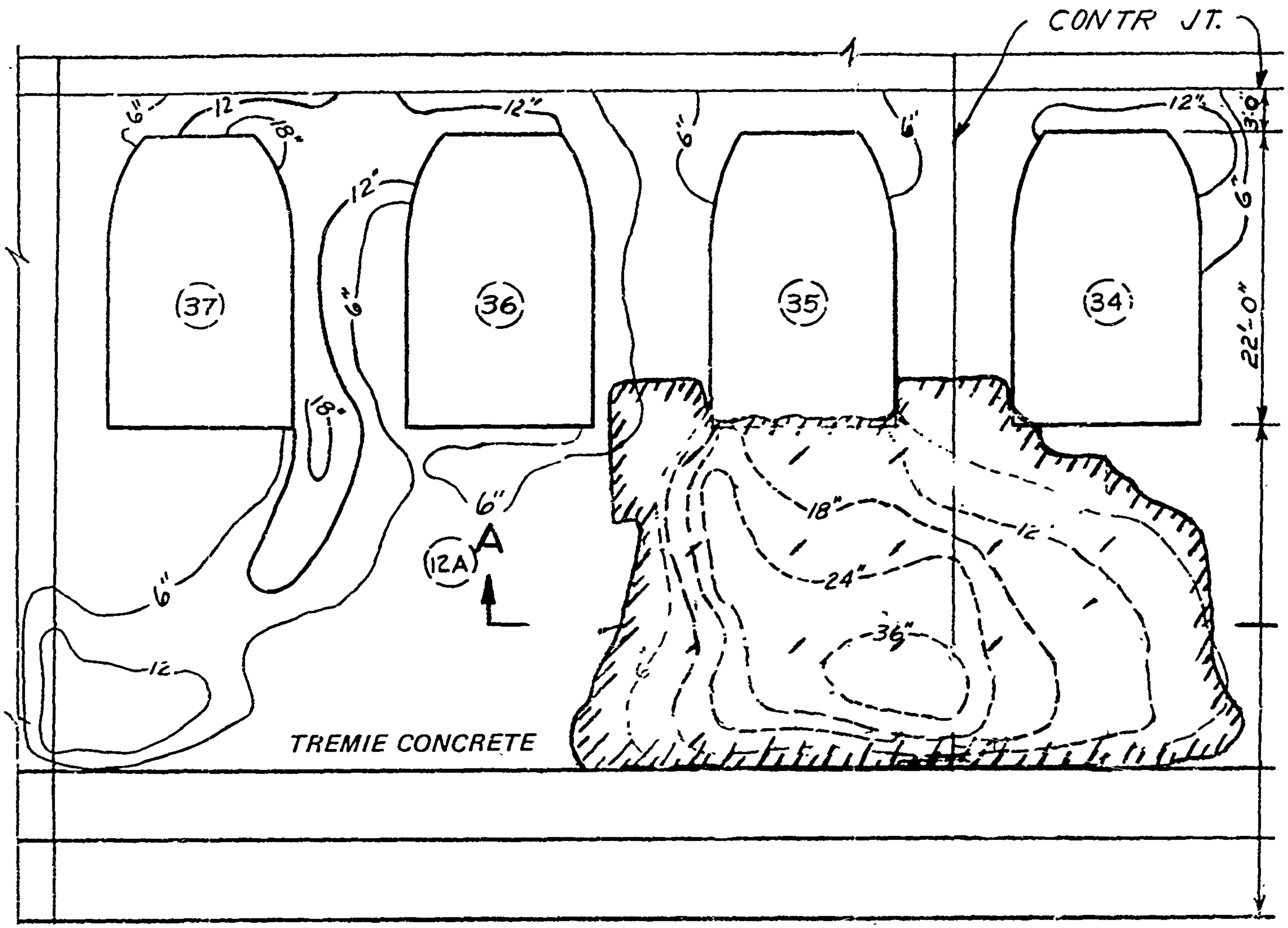


Figure 82. Tremie concrete repair of stilling basin erosion, block 12A, Chief Joseph Dam

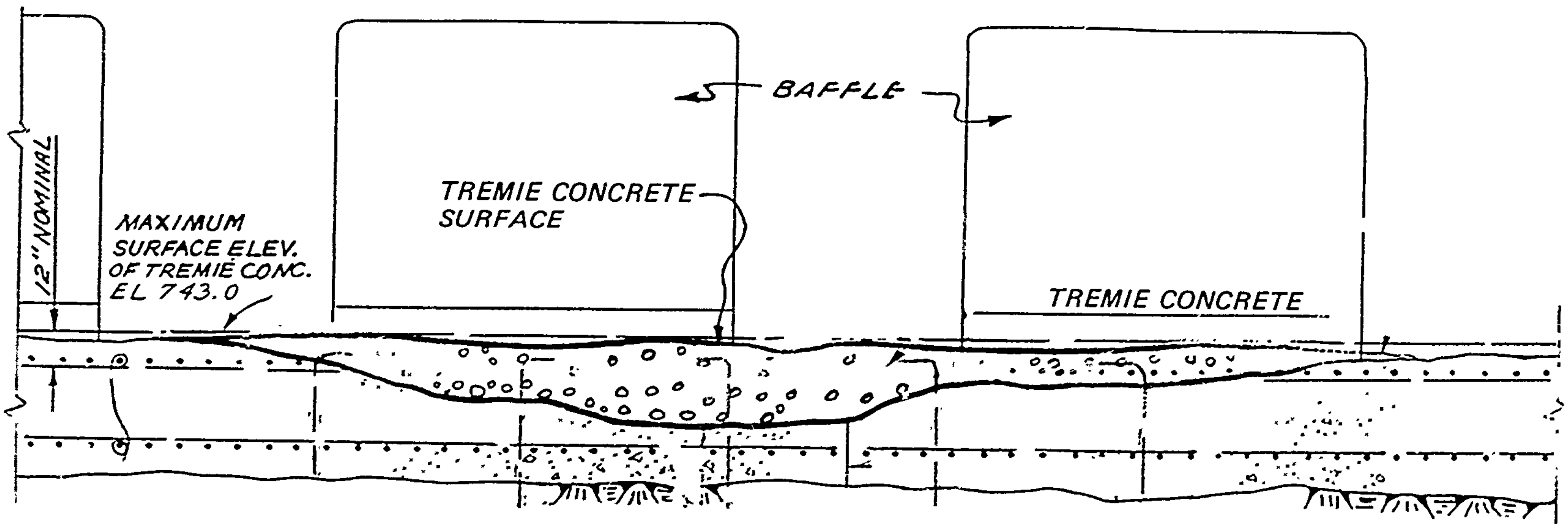


Figure 83. Typical transverse section, block 12A, Chief Joseph Dam

80 cu yd of preplaced aggregate concrete was required in the repair of block 2A.

149. Pumped concrete was used in the repair of blocks 3A, 7A-12A. The contractor used 4- by 24-ft platforms placed on the stilling basin floor and spanning upstream-downstream over the tremie placement areas. These platforms provided surface control and a walkway for divers controlling placement. These platforms were framed with 6-in. tubular steel and were supported on jackscrews. The platform surfaces were plywood sheets bolted to the bottom of the tubular frames.

150. The concrete was batched by a local supplier and delivered to the site in truck mixers. At the site, the material was placed in a collection hopper on the south training wall and delivered from this point through a 10-in. pipe into concrete buckets on a power barge for ferrying to the sites of pump work. The concrete was placed with a concrete pump delivering through a 3-in. hose. The last 2 ft of this hose were fitted with a metal tube to allow ready insertion of the conduit into the concrete when the surface leveled off. A clamp was located just above the metal tube to provide the required valve action and to allow the placement to be controlled by the diver.

151. Pumped concrete placement began in block 7A. The mix design first used was not altogether satisfactory. A sufficiently flat surface slope on the concrete (approximately 8:1) was achieved, but too much movement of the placement hose was required. Also, the concrete had a tendency to stop flowing at the edge, thus leaving an unsatisfactory transition to the existing concrete surface. After the pumping of concrete in block 7A was completed, the mix design was changed to produce a more workable concrete. One hundred pounds of sand per cubic yard was replaced by 100 lb of coarse aggregate. All other material proportions remained the same. This revised mix produced a workable concrete that was easily placed and produced a smooth, even surface that flowed out well. Surface slopes estimated at 12:1 were achieved. This revised concrete mix was used for the remaining blocks. Surface slope on pumped concrete in block 7A is therefore 8:1, and in the other blocks it is 12:1.

152. The measured and estimated surface areas of pumped concrete were as follows:

Block	Estimated Contract Quantity sq yd	Measured Quantity sq yd
3A	16.2	21.8
7A	17.3	44.3
9A	61.0	92.4
10A	194.2	159.3
11A, 12A	101.3	121.2
Totals	390.0	439.0

153. Apparently, the increase is due in part to the achievement of a flatter surface on the pumped concrete than was assumed possible in design. The remainder of the increase in surface area is due to an average 3-in. overbuild, which at the flat surface slope achieved, resulted in an additional 66-sq-yd area.

154. The repairs were accomplished in September through December 1966. During the high-water season in 1967, the repairs were subjected to peak discharges of 432,000 cfs, a flow with a frequency of recurrence of about once in 6 years.

155. A December 1967 inspection of the repairs showed the preplaced aggregate concrete surfaces to be in excellent condition. Inspection of several surface marks previously noted during acceptance inspections showed no discernible change. Vertically formed sides were undamaged. Laitance, noted during acceptance inspections, on transition slab surfaces and at junctures with existing concrete had been generally removed by water action. Two surface irregularities were produced during construction when grout pressure slightly raised top forms and allowed grout to run over adjacent surfaces. Both irregularities were chipped back during construction to meet surface tolerances. The condition of the raised faces of these irregularities had changed little since completion of the work. The remaining preplaced aggregate surfaces were unchanged from the placement condition.

156. The December 1967 inspection also showed the pumped concrete

surfaces to be in good condition with only minor surface damage noted. The worst damage occurred to the first placement of concrete where the design mix proved to be too stiff and had a tendency to form a roll at the outside edges. Small depressions in pumped concrete surfaces were noted during the acceptance inspection in December 1966. No change in these depressions was found in December 1967.* The remaining repaired surfaces showed little change during the 1-year exposure to the spillway discharge.

157. A detailed inspection of the stilling basin (Seattle District 1975) conducted in 1974 indicated that there had been no extensive erosion of the stilling basin since the repairs were made. The average measurements of erosion of the stilling basin slab ranged from +0.05 ft (growth) to -0.58 ft.

158. In general, the A-blocks had a roughened surface with exposed aggregate in relief (0.2- to 0.3-ft erosion) with the exception of portions of those blocks repaired in 1966. At the repaired location the surfaces were generally very smooth and no erosion had occurred in these areas from 1966 to 1974. The contraction joint erosion, generally V-shaped and more extensive than on the block surface, was 6 in. wide at the top and 3-1/2 in. deep. There were some areas in the basin where the aggregate was missing and some rebar exposed; however, this erosion was present prior to 1964 and the rate of erosion has been minimal from 1964 to 1974.

159. In general, the B- and C-blocks had roughened surfaces with exposed aggregate in relief (0.2- to 0.3-ft erosion) similar to the A-blocks. However, there were not as many areas that had missing aggregate. The contraction joints were V-shaped, similar to those of A-blocks, but not as deep. Negligible erosion has taken place in these areas since 1964. The D-blocks in general had a roughened surface with aggregate in relief, the erosion growing from 0.1 to 0.4 ft. The contraction joints between the D-blocks and the monoliths had some aggregate missing (0.6- to 0.8-ft erosion).

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* Inspection data for 1966 and 1967 received from the Seattle District in response to Engineer Circular 1110-2-181 (OCE 1977).

Libby Dam

160. Libby Dam is a straight concrete gravity structure (Figure 84) located on the Kootenai River in northwestern Montana, approximately 17 miles upstream of Libby, Montana. The dam, 420 ft above bedrock and 2900 ft long at the crest, consists of nonoverflow monoliths, powerhouse intake monoliths, and spillway monoliths. Flow over the two-bay ogee spillway, crest el 2405, is controlled by two 48-ft-wide by 59-ft-high tainter gates. The spillway design discharge is 145,000 cfs with normal full pool (el 2459). Three rectangular sluices, each 10 ft wide by 22 ft high, controlled by tainter gates 10 ft wide by 17 ft high, regulate discharges at pool elevations below the spillway crest. Each sluice is designed for a discharge of 20,000 cfs.

161. A hydraulic jump stilling basin (Figure 85), designed for a discharge velocity of 120 fps, provides energy dissipation for both

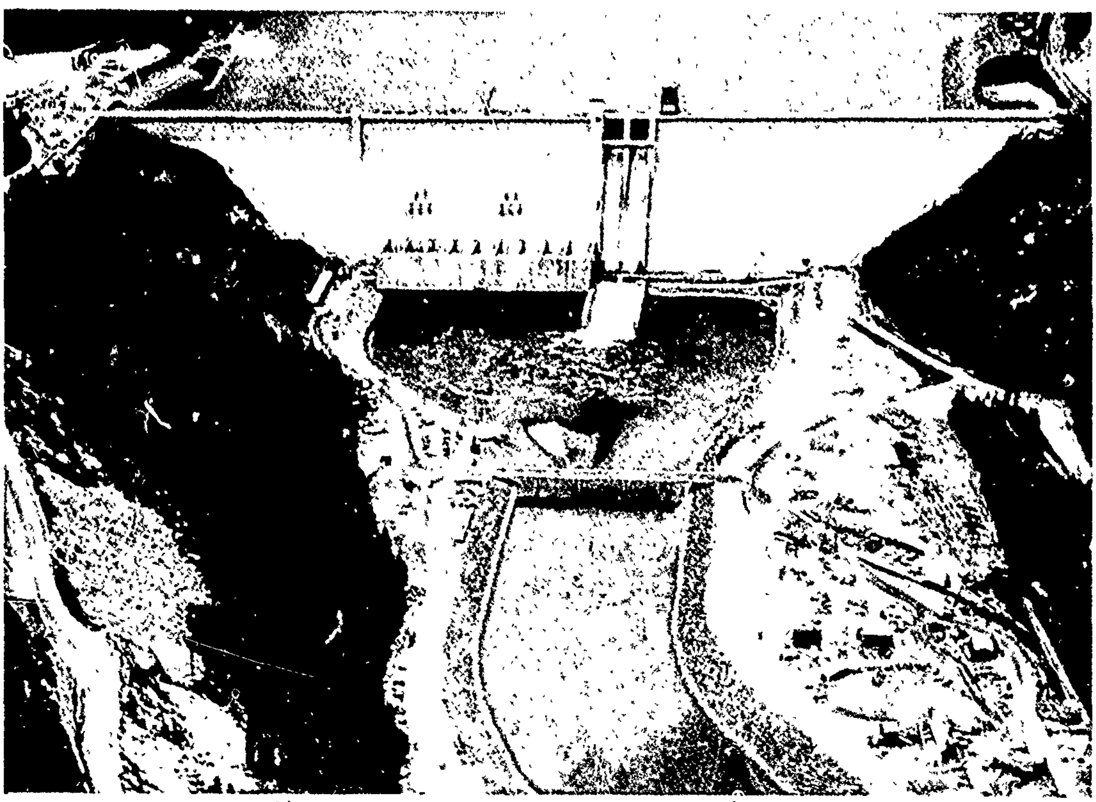


Figure 84. Aerial view of Libby Dam (from Smith and Nissila 1975)

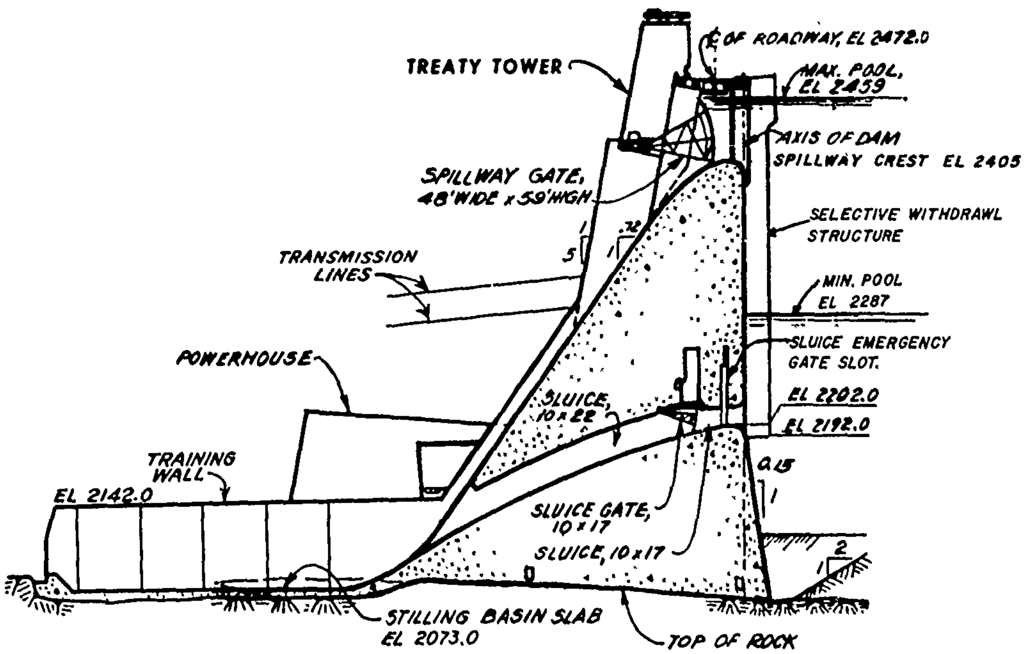


Figure 85. Section through stilling basin, Libby Dam (from Seattle District 1973)

sluice and spillway flow. The horizontal basin floor, 252 ft long and 116 ft wide, is 332 ft below the spillway crest. The 12-ft-high end sill is sloped at a 45-degree angle. The stilling basin was constructed of structural concrete (1-1/2-in. MSA, 5000 psi), drained and anchored. Construction of the stilling basin was completed in 1968, the same year concrete placement for the dam began.

162. Natural fine and coarse aggregate from the Yarnell Terrace source were used in the Libby concrete. The coarse aggregate was hard, very moderately weathered, well-rounded to subrounded gravel, predominantly quartzitic in composition, with small amounts of argillite, limestone, granite, and miscellaneous stones. The well-graded sand had an average fineness modulus of 1.90 and approximately 20 percent of the material passed the No. 100 mesh sieve. During processing by conventional classification methods, excess fines were wasted and a product conforming to the desired coarser grading limits resulted.

163. The tailrace channel downstream from the basin is composed essentially of fluvial gravels, with a few very large bolders infrequently spaced throughout the gravels. The invert of this channel is at

el 2110, 37 ft above the basin slab. The runout slope from the stilling basin end sill to el 2110 was constructed on a 1 vertical to 6 horizontal slope and the first 50 ft of this slope downstream from the end sill was armored with 3000- to 5000-lb derrick stone.

164. Following the 1968 construction season, the river was diverted over low blocks on the left side of the stilling basin (Figures 86 and 87). Prior to the low block diversion, the stilling basin was thoroughly cleaned. Later in the construction sequence diversion over the low blocks was stopped, and the river was diverted through three temporary sluices (Figure 88) also located on the left side of the stilling basin. During these two diversions, strong eddy currents transported approximately 8000 cu yd of sand and silt with a small amount of gravel and boulders into the basin. In the spring of 1972, most of this material was removed by suction dredge. However, about 50 cu yd of large rock and boulders remained near the end sill. Before

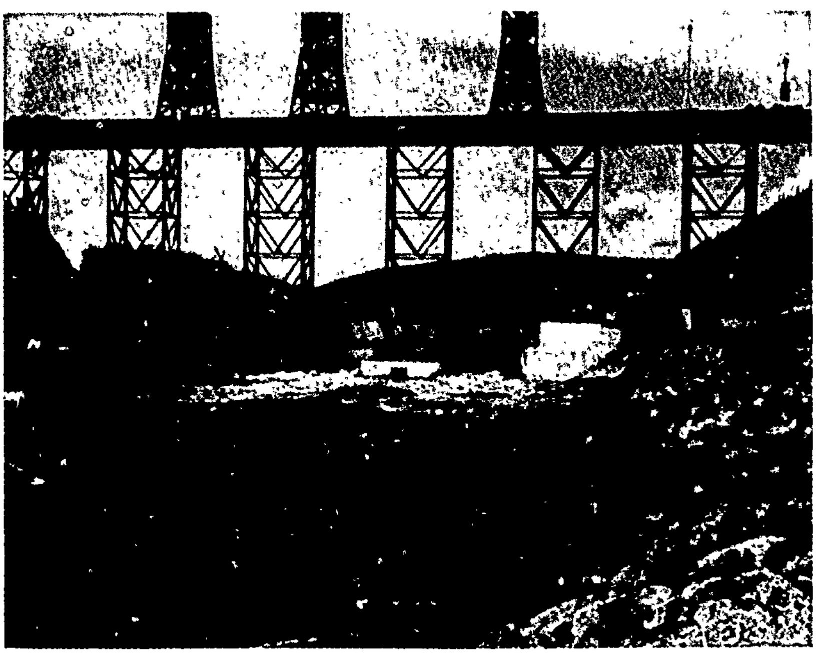


Figure 86. Upstream view of diversion flow over low blocks on the left side of the Libby Dam stilling basin, 1969 (from Seattle District 1973)

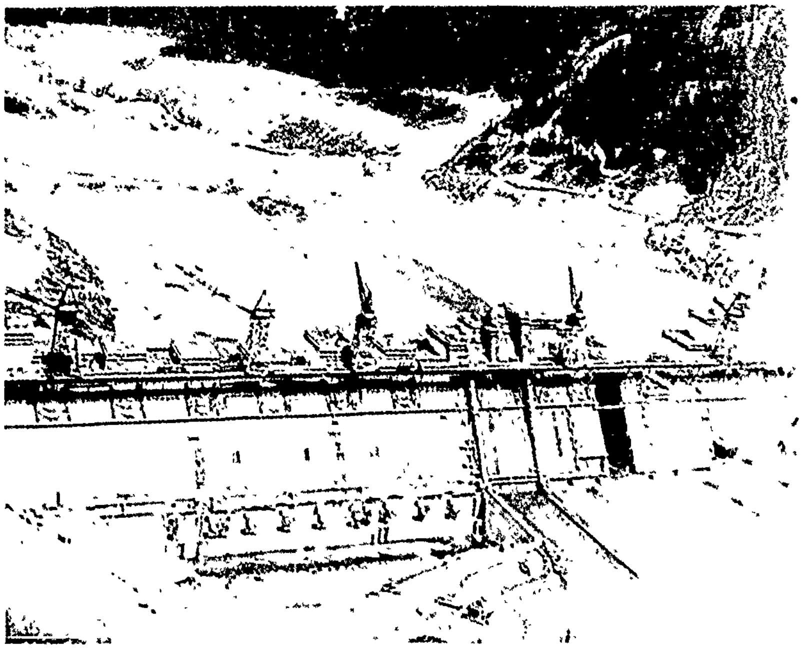


Figure 87. Riverflow through low monolith 33 and temporary sluices, July 1971 (from Seattle District 1973)

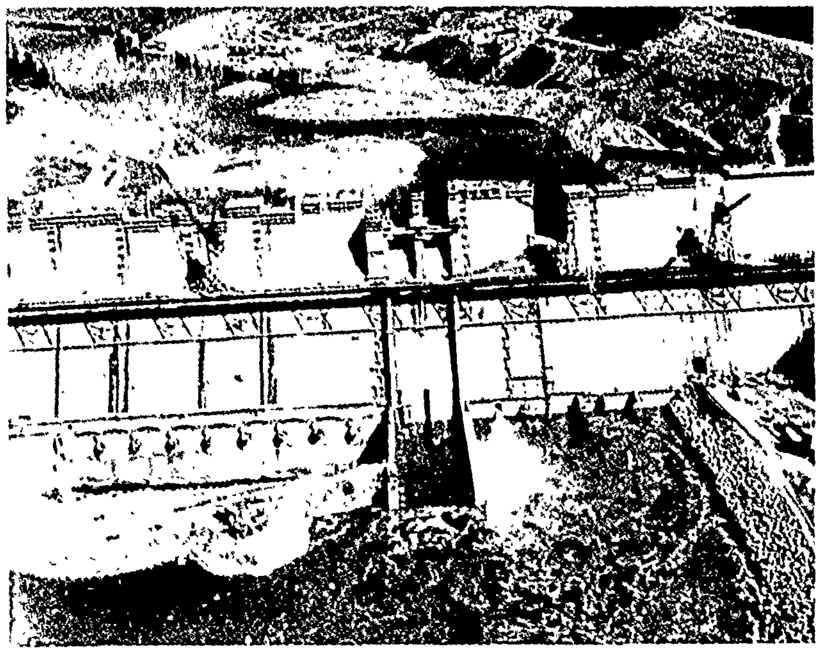


Figure 88. River diversion through temporary sluice in monolith 31, March 1972 (from Seattle District 1973)

this remaining material could be removed, the temporary sluices were plugged, and the stilling basin was placed in operation with flow from the permanent sluices on 27 March 1972.

165. Four months later a diving inspection of the stilling basin showed that approximately 3000 cu yd of additional rock and debris had accumulated in the stilling basin since the dredging operation earlier in the spring. Localized areas of erosion were noted, the largest of which was at the junction of the floor slab and the end sill. This area was about 30 ft in length, 2 to 2-1/2 ft wide, and up to 4 in. deep.

166. The stilling basin was next inspected in October 1974. During this 2-year interval the stilling basin was in continuous operation with the majority of the flow discharging from the sluices. This inspection was accomplished by the use of a two-man submarine and covered the entire stilling basin floor and the bottom 5 ft of both training walls. The inspection indicated severe erosion in the floor of the basin from the downstream end of the bucket to the end sill and across the entire basin width. Throughout the majority of this area, the tops of the J-bar anchors were exposed. Between the J-bars, concrete had eroded considerably deeper in elliptical dish-shaped pockets. These holes were estimated to be up to 12 in. deeper than the top of the J-bar anchors, thus indicating erosion in excess of 2 ft at maximum depth. The major erosion damage was located in the middle half of the stilling basin, although exposed aggregate was observed throughout the basin floor. Along the right wall and floor intersection, starting at the end sill and proceeding upstream for approximately 100 ft, extensive erosion had occurred that resulted in a large amount of reinforcing steel being exposed; some was bent away from the concrete, but the majority was missing. Inspection of the end sill showed that considerable erosion had taken place on the entire upstream 45-degree face of the sill. This erosion had resulted in the exposure of the end sill reinforcing steel. A major portion of the rebar located in the floor was either missing or bent upward away from the floor surface. This inspection did not show much loose rock on the basin floor.

167. A diver inspection was made in December 1975 after another full year of stilling basin operation, and then a detailed diving inspection took place in December 1976. During the interval between the 1975 and 1976 dives, basin use was minimal due to the fact that the powerhouse was completed and the majority of the flow was through the turbines. The detailed inspection included depth measurements on 15-ft centers or closer and visual inspection using recorded television pictures. This inspection showed severe erosion over the entire basin floor, with the area of maximum erosion in the center third of the basin. In this area the erosion occasionally extended into bedrock, and many valleys and ridges existed due to deep erosion in the concrete. The deepest penetration appeared to be about 6 ft below the original basin floor surface. Reinforcing steel was exposed and broken off along the base of the right training wall and the end sill. The walls of the basin showed erosion extending about 6 to 8 ft above the floor. The erosion was most predominant near the floor. Reinforcing steel was exposed on the right wall with some bars bent away from the concrete and some missing. The top of the end sill had been eroded; but due to large eddy currents in this area resulting from powerhouse discharge, measurements were not possible. The sloping face of the sill had eroded to the reinforcing steel in places. A large amount of reinforcing steel was located at the base of the end sill. Numerous pockets of well rounded rocks were observed. Based on the 1976 inspection, erosion contours and debris locations within the basin were plotted (Figure 89).

168. When constructed, the stilling basin exit channel was lined with derrick stone from el 2080, 5 ft below the top surface of the end sill, to el 2088.3. Underwater observation of the channel adjacent to the end sill showed a 4-ft accumulation of rock, riprap, and derrick stone to within 1 ft of the end sill top surface. Near the right training wall, loose rock projected above the end sill. No displacement of derrick stone was observed downstream from the end sill except for one 4-ft-deep hole. Gravel deposits had accumulated in the downstream channel between the stilling basin exit channel and the project construction bridge to el 2122.

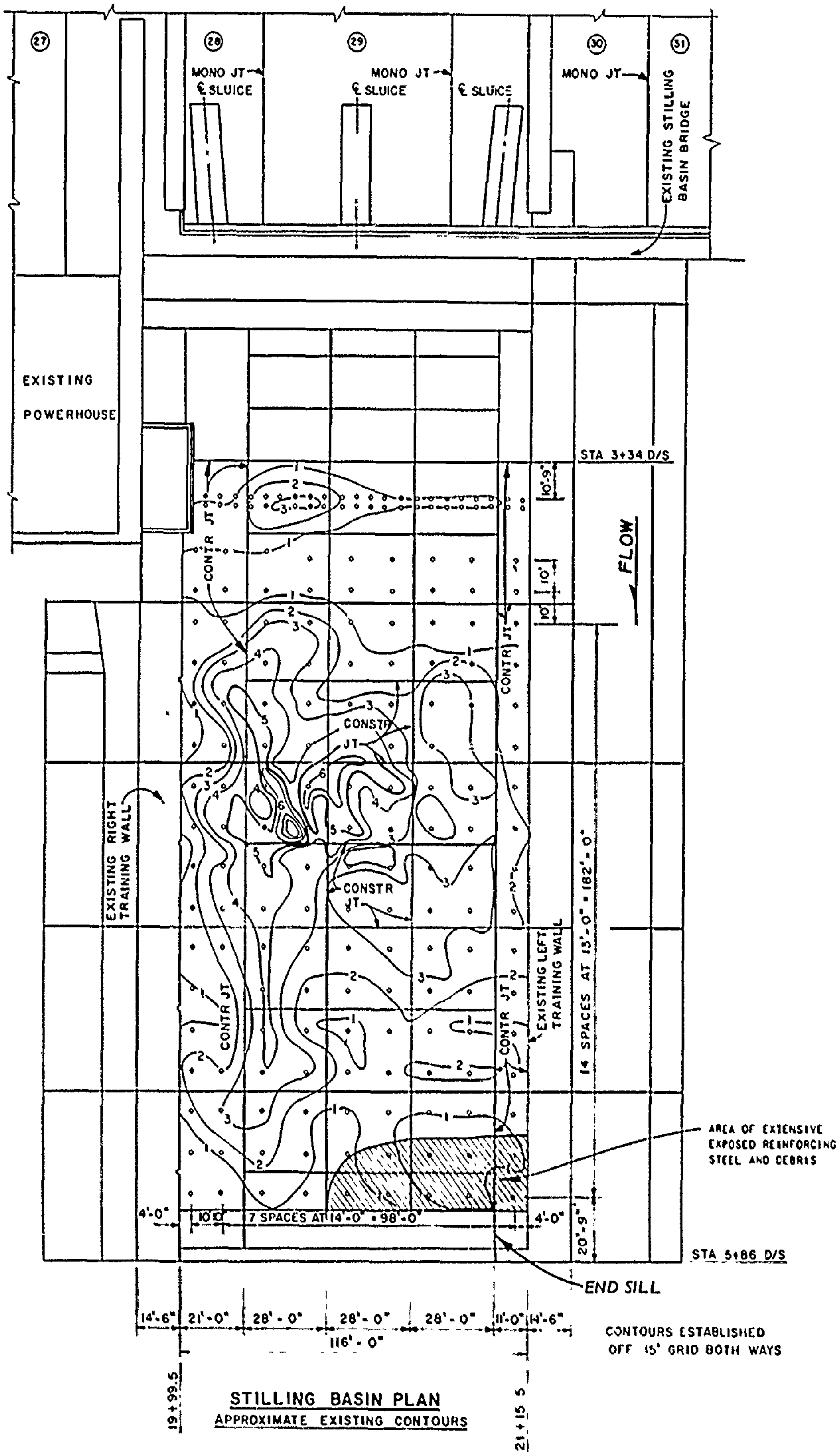


Figure 89. Erosion contours based on 1976 inspection, Libby Dam

169. An analysis of the stilling basin damage indicates that the low, unbalanced flows prevalent throughout the sluice operation transported boulders and gravel from the tailrace into the stilling basin. Large, balanced flows from the spillway (discharge greater than 25,000 cfs) required to flush this debris out of the basin occur infrequently. The majority of the material responsible for the stilling basin damage is probably from the displacement of the channel bed that occurred immediately downstream from the low block and temporary sluice diversion facilities. Many thousands of cubic yards of material were eroded from this area and deposited in the tailrace area downstream from the stilling basin. Model studies (U. S. Army Engineer Division, North Pacific 1966) showed a tendency for some of this eroded material to be deposited in the stilling basin. Some of the material deposited in the basin may have come from other sources such as cofferdam removals and contractor cleanup.

170. Eliminating stilling basin damage similar to that at Libby Dam would require having at least one or two generating units available for use upon initial pool raise; bulkheading the stilling basin off during all stages of diversion; prohibiting unbalanced flow of any magnitude through either the sluices or the spillway; and providing considerably more armoring of the channel invert downstream from the stilling basin (Regan 1977).

171. A contract in the amount of $2,200,000 was awarded in August 1977 for dewatering and repair of the stilling basin. Dewatering was accomplished by floating a prestressed concrete bulkhead into position above the end sill between the stilling basin training walls and by pumping the water from the basin (Figure 90). Total cost of dewatering was approximately $715,000.

172. Erosion of the stilling basin slab was about as expected, based on the previous diver surveys. The area of severest erosion (approximately 6 ft deep) was located near the center of the basin (Figure 91) with a few small areas of exposed bedrock. Almost all of the top layer of reinforcing for the right training wall footing had been exposed and had failed in fatigue (Figure 92). Failed concrete around the

Figure 90. Prestressed concrete bulkhead in position, Libby Dam

circumference of these bars indicated significant vibrations in this area.

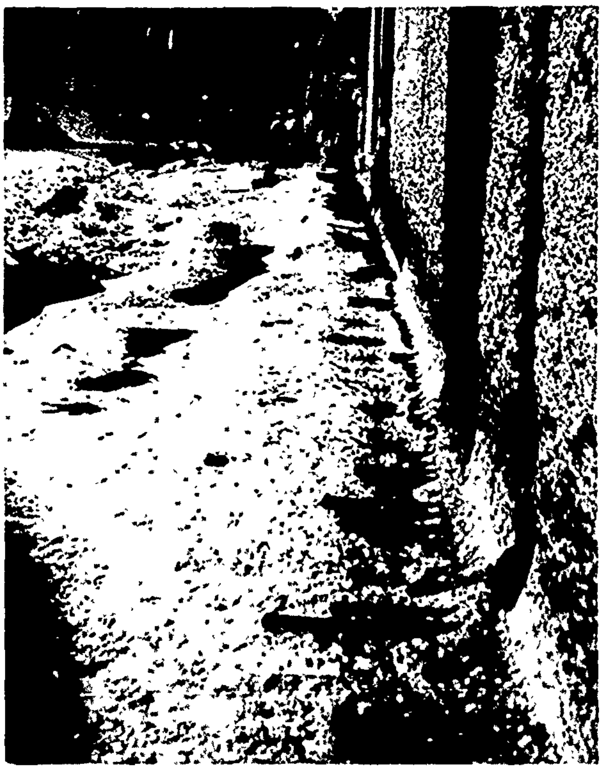
173. Compared with the major erosion of the basin slab, the walls of the stilling basin exhibited little significant erosion. The maximum depth of erosion was less than 4 in., and this was generally confined to one monolith near the middle of the basin (Figure 93). Erosion of the transition slab below the sluices, particularly the center sluice (Figure 94), was worse than anticipated. In general, this erosion ranged from 1- to 4-in. depths. The repair contract was modified to include repair of this area. Significant erosion of the end sill was generally confined to a relatively small area adjacent to each training wall (Figures 92 and 95). Debris in the stilling basin consisted primarily of reinforcing steel, cobbles, and small boulders (Figure 96).

174. Where necessary, the existing concrete in the floor slab was chipped out and removed to provide a minimum thickness of 18 in. for the repair concrete. A minimum 2-in.-deep saw cut was provided normal to the concrete surface prior to chipping. Damaged portions of reinforcing

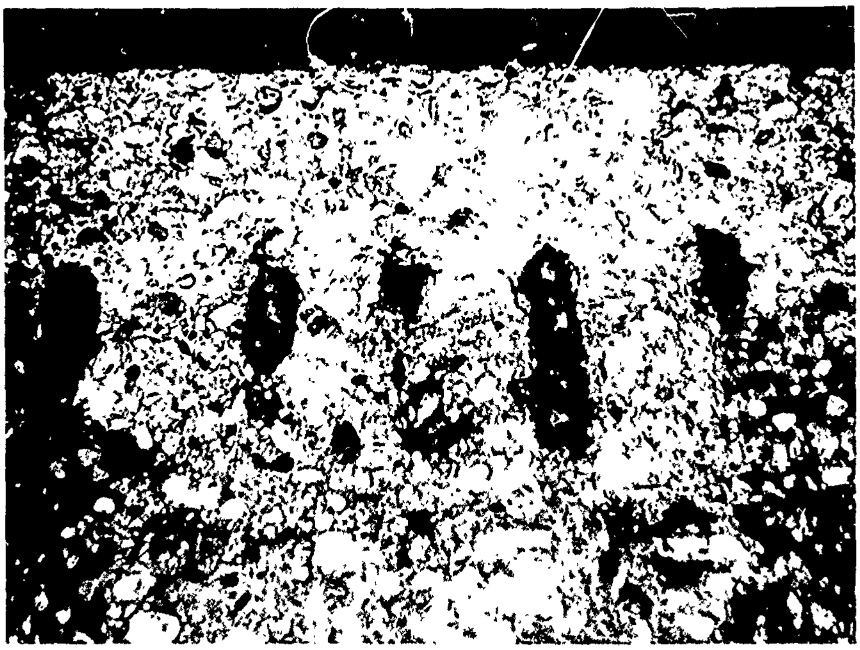
a. Major erosion near center of slab

b. Closer view of erosion

Figure 91. Erosion of stilling basin floor slab, Libby Dam



a. General view along right training wall



b. Fatigue failure of reinforcing steel

Figure 92. Exposed reinforcement, Libby Dam

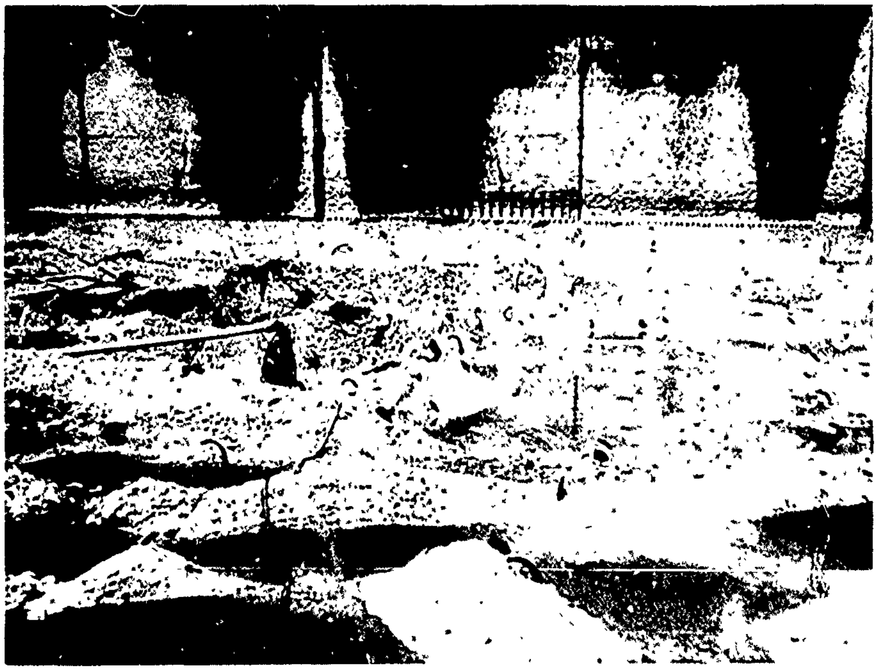


Figure 93. Erosion of stilling basin floor slab and right training wall, Libby Dam

Figure 94. Erosion of transition slab below center sluice, Libby Dam

Figure 95. Erosion of end sill adjacent to left training wall, Libby Dam

steel and anchor bars were removed and replaced. Existing uplift pressure relief drains were inspected, and those not weeping water were plugged with concrete. Steel pipes were provided to extend the remaining drains above the final concrete surface. No. 8 anchor bars were grouted with nonshrink grout in holes drilled 4 ft on centers each way.

175. Following a thorough cleanup, structural grade unreinforced fill concrete was used as necessary (Figure 97) to bring the stilling basin floor up to el 2071.5 (18 in. below finished grade). This concrete, 1-1/2-in. MSA, was placed by pumping into formed areas coinciding with the original construction joints. A fiber-reinforced concrete topping, 18-in. minimum thickness, was used to bring the basin floor slab up to final grade. A local ready-mix producer furnished the fiber concrete from a 35-cu-yd/hr capacity batch plant (Figure 98) set up on the site. The steel fibers (0.01 by 0.02 by 1.0 in.) were batched through a shaker onto the aggregate conveyor belt simultaneously with the aggregate. Four 8-cu-yd-capacity truck mixers, each with 6 cu yd of concrete, were used for mixing and for transportation to the stilling basin.

a. Reinforcing steel and other miscellaneous debris

b. Cobbles

Figure 96. Stilling basin debris, Libby Dam

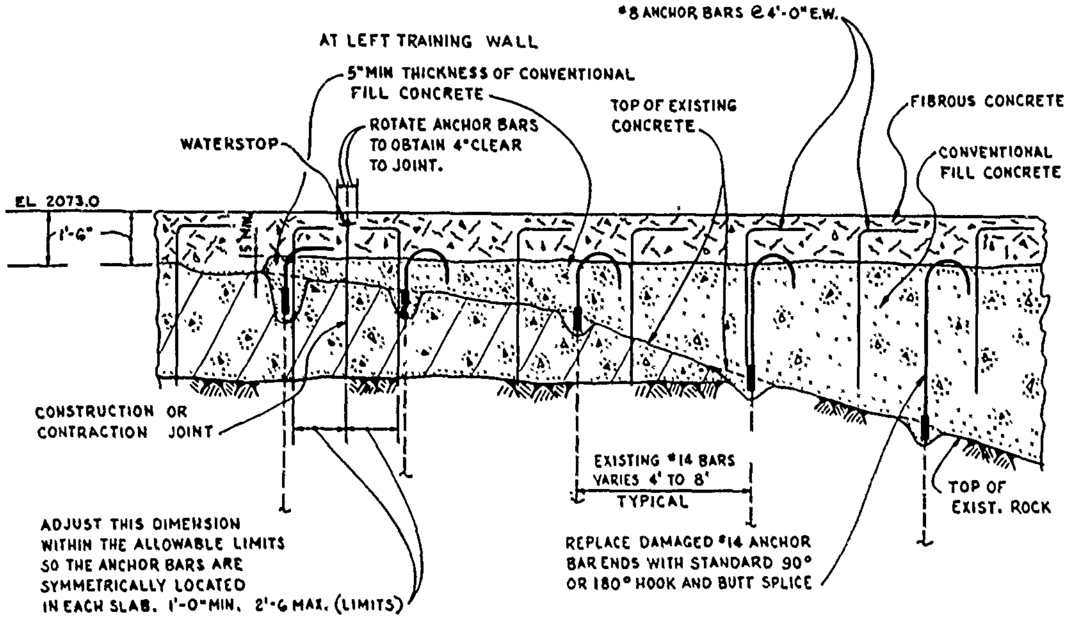


Figure 97. Repair detail, stilling basin slab, Libby Dam

Figure 98. Batching fiber concrete, Libby Dam

The fiber concrete was placed by pump with a 6-in. line. A form-type vibrator was attached to the truck chute during discharge to assist the concrete in sliding down the chute. In addition, a spud vibrator was attached to a 2- by 3-in. auxiliary screen over the pump hopper (Figure 99) to help break up fiber balls before they could get into the pump. Internal vibrators were used to consolidate the concrete and a vibrating screed was used to finish the floor slab. Waterstops were placed along construction joints as shown in Figure 97. Curing was by white pigmented curing compound.

176. The fiber concrete mixture proportions for a 1-cu-yd batch were as follows:

Material	Weight, lb
Portland cement, Type II	447
Pozzolan	189
Fine aggregate	1591
Coarse aggregate (No. 4 - 3/4 in.)	1353
Steel fibers	141
Water	275

Figure 99. Fiber concrete pumping operation, Libby Dam

A typical batch of concrete contained 3 percent air and 2-3/8 in. slump and had a flow of 8.5 seconds. The unit weight was 150.1 lb/cu ft. Compressive strengths were 3890 and 6420 psi at the 3- and 7-day ages, respectively.

177. Erosion of the stilling basin training walls was such that only one monolith (see Figure 93) required repair. A fiber concrete pourback was used to repair this area (Figure 100). Erosion of the transition slab below the sluices was repaired by removal of concrete to a minimum depth of 12 in. and by placement of fiber concrete. Prior to placement of the concrete, No. 8 anchor bars were placed in holes drilled on 4-ft centers and grouted with a nonshrink grout. Also, a horizontal mat of No. 6 reinforcing bars on 12-in. centers was hooked to the anchor bars (Figure 101). The end sill was repaired using an 18-in. topping of fiber concrete anchored with No. 8 anchor bars to the existing concrete (Figures 102 and 103). On this sloping surface the fiber concrete was placed in 2-ft layers and vibrated internally prior to placement of another layer.

178. After the fiber concrete had been placed and cured, 4-ft sections along the construction joints were polymer impregnated. The polymer impregnation process consisted of drying the concrete, saturating

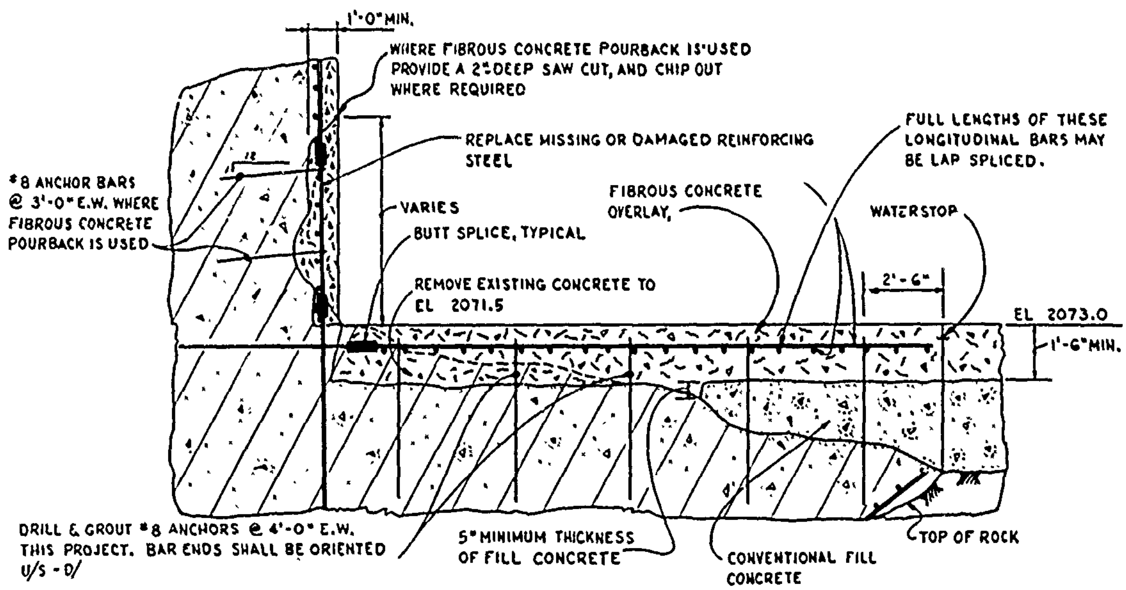


Figure 100. Repair detail at right training wall, Libby Dam

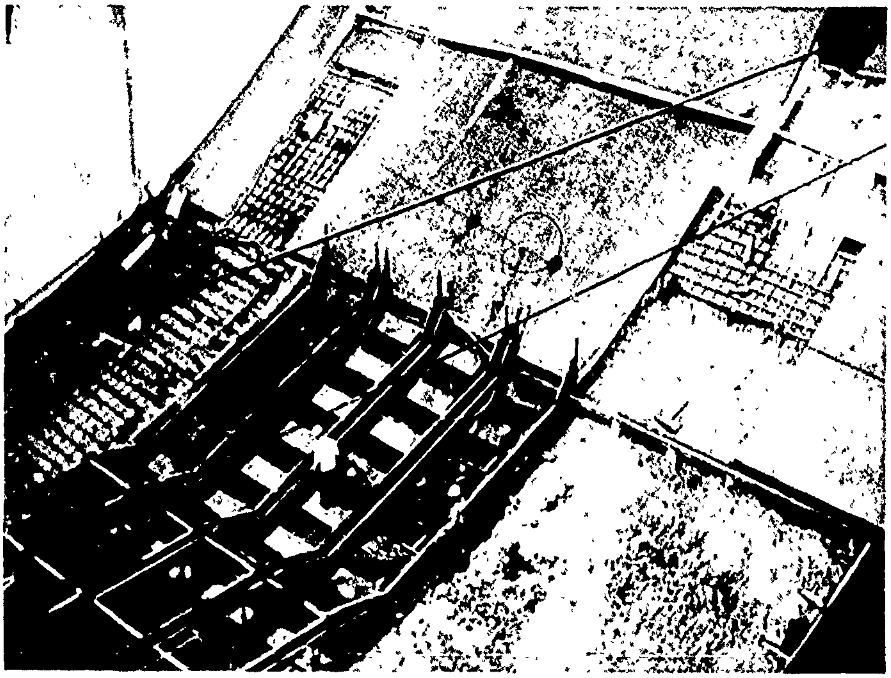


Figure 101. Repair of transition slab below sluices, Libby Dam

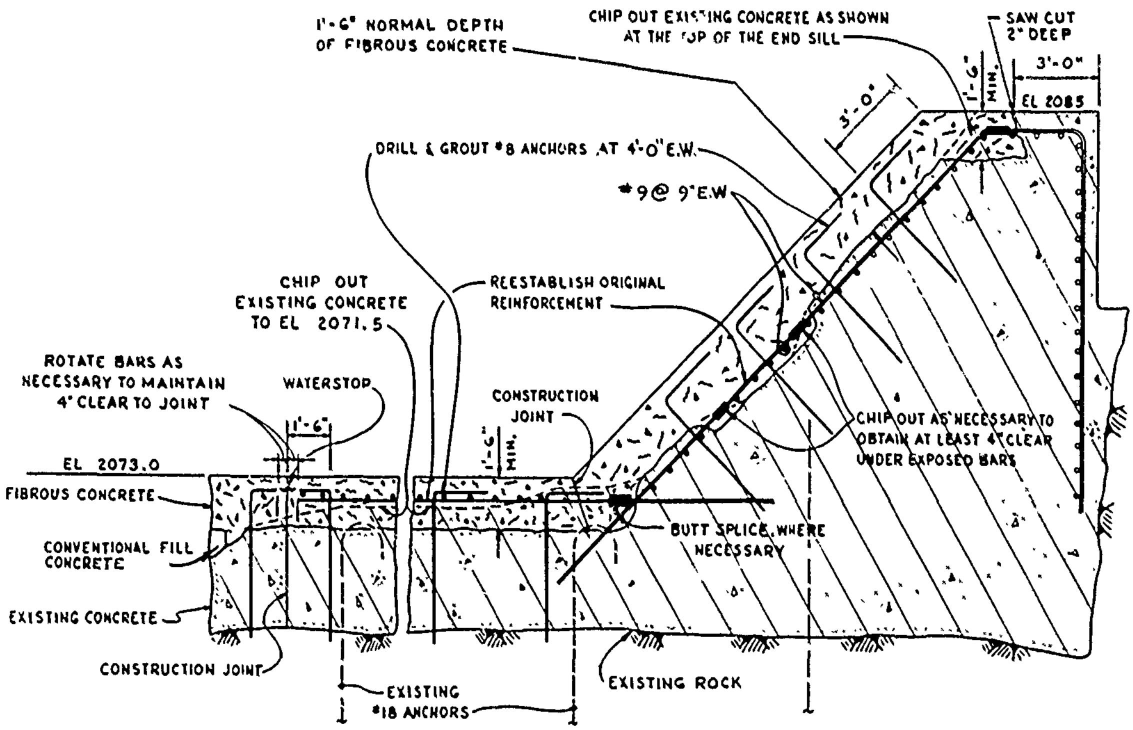


Figure 102. Repair detail at end sill, Libby Dam

Figure 103. End sill prior to forming and concrete placement, Libby Dam

the concrete with a monomer solution, and then applying heat to polymerize the monomer solution. A. V. Munch and R. M. Oedewaldt* described polymerization procedures in detail and these are summarized in the following paragraphs.

179. Enclosures consisting of plywood and steel forms in lengths of 80 ft or more were positioned along construction joints (Figure 104) to concentrate the heat on concrete surfaces 2 ft either side of the joint. Radiant heat was used to dry the concrete at temperatures between 250 and 345°F for 8 to 10 hr. Thermocouples attached to the

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* A. V. Munch and R. M. Oedewaldt. "Polymerization of Fibrous Concrete for Stilling Basin Repair at Libby Dam (unpublished)," Libby Dam Resident Office, CE, Libby, Mont.

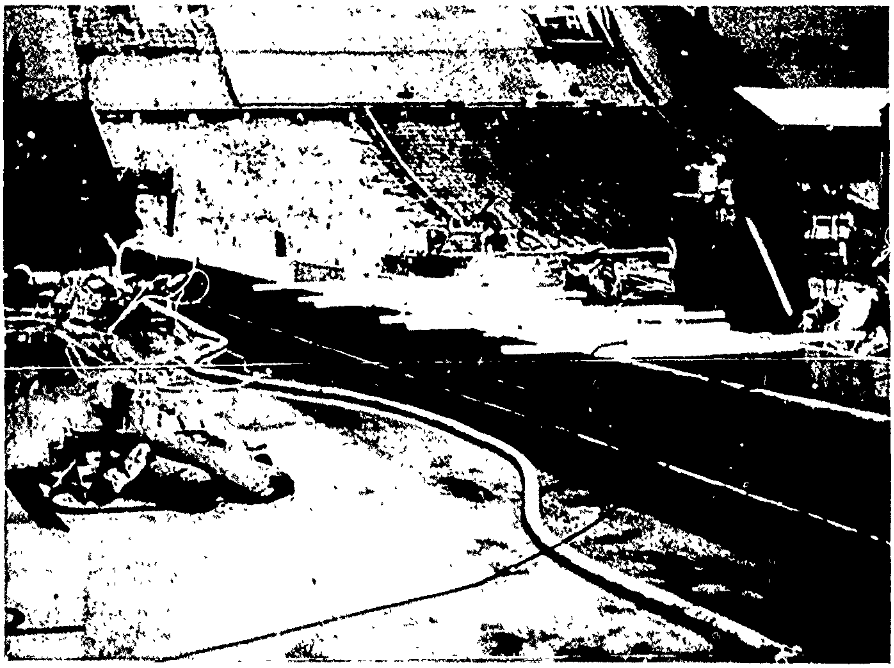

Figure 104. Enclosure used in polymerization of concrete along construction joints, Libby Dam

concrete surface were used to monitor temperatures. After the concrete was dried, it was allowed to gradually cool to 75-105°F. Exposure of the concrete to air was not permitted until the concrete temperature was within 25°F of ambient to avoid thermal shock.

180. After it cooled, the concrete within the enclosure was saturated with monomer solution. A 3/8-in. layer of dry sand was used to help distribute the monomer and reduce evaporation losses of the monomer, which was ponded within the enclosure. Monomer was applied in the range of 0.07 to 0.15 gal/sq ft. Soak times ranged between 5 and 8 hr. During the soak period a sheet of polyethylene was draped over the enclosure to reduce evaporation and control fumes. The monomer solution was 95 percent methylmethacrylate (MMA), 5 percent trimethylolpropane trimethacrylate (TMPTMA, cross agent S-9A0), and 0.5 percent azobisisobutyronitrile (AIBN) catalyst.

181. Polymerization of the monomer-saturated concrete was accomplished using steam to heat the concrete surface to 150-260°F for periods ranging from 30 min to 3 hr. After polymerization, the

enclosure was moved to another construction joint and cleanup of the polymerized area was begun.

182. The contractor's first trial polymerization effort was not successful and the following adjustments were made:

a. The steam pipes were lowered so that they were 6 in. above the concrete surface, instead of the original 2 ft.

b. The concrete sand aggregate used for the soak cycle was replaced by a silica sand having the following gradation:

Screen Size	Percent Passing
No. 8	100
No. 16	95-100
No. 30	40-70
No. 50	5-10
No. 200	0-1

This change was made to reduce the amount of fines in order to minimize sand adherence to the surface. A similar gradation was used successfully at other projects.

c. A larger steam generator with a steam pressure of 100 psi was procured.

d. Aluminum foil was applied to the sides and ends of the enclosures to improve heat reflection.

e. The amount of monomer applied to the sand was reduced through trial and error. The determination on when to begin the steam curing was made by adopting the absorption slump test for sand, which establishes when sand has reached the saturated surface dry state. Although this method has some basic deficiencies, due to the fact that the viscosity of the monomer changes with time, it did provide a reasonably reliable index to establish when the monomer had evaporated sufficiently so that curing could commence without sand adherence to the concrete surface.

183. Water was always present on the floor of the stilling basin during the repair because of either the relief drains or precipitation. The control of this water was a continuing problem. The contractor sealed the edges of the enclosures with bentonite to prevent surface water from entering the enclosure; however, it was soon discovered that water was entering the enclosure through the control joints. The

problem was eventually solved by drying one half of the stilling basin floor by means of dikes and pumps. Polyurethene sheets prevented precipitation from entering the enclosure.

184. Considerable experimentation was done to solve the problem of the sand blanket adhering to the surface of the concrete. The use of the sand cone was reasonably successful, except for the following cases:

- a. The curing temperature was not reached promptly.
- b. The monomer was not uniformly distributed.
- c. Excessive time was required to achieve satisfactory monomer evaporation from the sand.

It is suspected that under one or a combination of these conditions a reverse evaporation takes place and the monomer begins to evaporate from the concrete, resulting in uneven penetration.* Examination of some cores tend to confirm this, since it was noted in some cases that although the depth of monomer penetration was somewhat uniform, the concrete a short distance below the surface showed no penetration. In areas where the water problem is severe, water could be reabsorbed by the concrete, thus displacing the monomer.

185. An attempt was made to solve the sand adherence problem by flooding the units with water prior to curing them. This was only partially successful because the sand would polymerize once the curing temperature was reached. However, if the sand was removed immediately after the polymerization period, when it was still plastic, the procedure was successful. The final solution, on this project, was to remove the sand completely immediately after the soak cycle, then flood the area and begin the curing cycle.

186. Depth of polymerization was successfully determined by drilling 3-in.-diam cores to a depth of 4 in. Core locations were generally selected at random and 12 percent of the locations were at joints. The cores were immersed in hydrochloric acid for 2 min or less, flushed with water, and examined visually. In general, polymerization occurred at

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* Munch and Oedewaldt, op. cit.

depths ranging from 3/4 to 1-1/4 in.; greater depths were achieved in areas of cracks and joints. Initially, there was some concern over the widespread development of thermal stress cracks on the surface of the concrete, but an examination of the cores confirmed that the cracks were sealed. In general, no bond developed between concrete surfaces at construction joints.

187. Based on results of the polymerization operation, a number of recommendations* for future work of this type were offered.

- a. A limitation should be placed on the amount of time acceptable to achieve the desired drying temperature. A long drying period creates a greater depth of heat penetration, which has the adverse effect of resulting in premature polymerization of the monomer due to the retention of heat below the surface of the concrete. It is suggested that the equipment used should be capable of attaining the minimum drying temperature desired within 1 hr.
- b. The sand blanket should be removed at the completion of the soak period and the area to be polymerized should be covered with water prior to beginning the curing cycle. If this procedure is not desirable, the development of a reliable testing technique to determine when the monomer has evaporated from the sand in sufficient quantities to minimize sand adherence to the surface is suggested.
- c. A maximum time of 5 min should be established for attainment of the minimum temperature for curing.
- d. A mechanical, readily removed device that would hold a thermocouple in contact with the concrete surface should be developed.
- e. The data recording and analysis, monitoring of results, and adjustments to procedures within the broad guidelines established by the specifications should be the sole responsibility of CE personnel or an independent testing laboratory under the direct supervision of the CE, not quality control personnel.
- f. A flat finish of the concrete surface should be specified. The trowel finish attained on this project was too tight and impervious, and it undoubtedly decreased the monomer penetration rate.
- g. If at all possible, an expert in the field of polymerization should be present during the initial operations.

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* Munch and Oedewaldt, op. cit.

188. Upon the completion of concreting operations, replacement uplift pressure relief drain holes were drilled vertically. In addition, one row of new drains was drilled in the footing of each training wall. These new drains were diamond cored to depths of at least 6 in. into bedrock.

189. Repairs were completed and the stilling basin was flooded in June 1.78. Total cost of the repairs, including contract modifications, was approximately $3,150,000. Future inspections and evaluation of the repairs will depend on the freqency and magnitude of spills.

Dworshak Dam

190. Dworshak Dam is a straight concrete gravity structure located on the North Fork of the Clearwater River near Orofino, Idaho. The structure has a two-bay overflow spillway with tainter gates. The outlet works consist of three conduits, 12 by 17 ft in cross section, cast with the dam. The outlet inverts follow a parabolic curve through the dam, dropping 137 ft to the point where they meet the spillway above the stilling basin (Figures 105 and 106). The stilling basin is cast-in-place concrete, 114 ft wide and 266 ft long, with a horizontal floor and vertical face end sill 20 ft high. Project design discharges for the outlets and spillway are 40,000 and 150,000 cfs, respectively, at reservoir elevation 1600. However, discharges into the stilling basin have generally been on the order of 1,000 to 25,000 cfs. Velocities range from 122 fps in the outlet works immediately downstream of the valve to 160 fps in the stilling basin.

191. The initial periodic inspection was conducted while the dam was still under construction (Walla Walla District 1972). At the time of the May 1972 inspection, concrete in the high monoliths was at el 1455 with the lowest monolith at el 1435. The stilling basin had been in operation less than 1 year and no distress was visible; however, a complete inspection of the stilling basin with divers was not scheduled.

192. A diver inspection in May 1973 indicated that rubble and

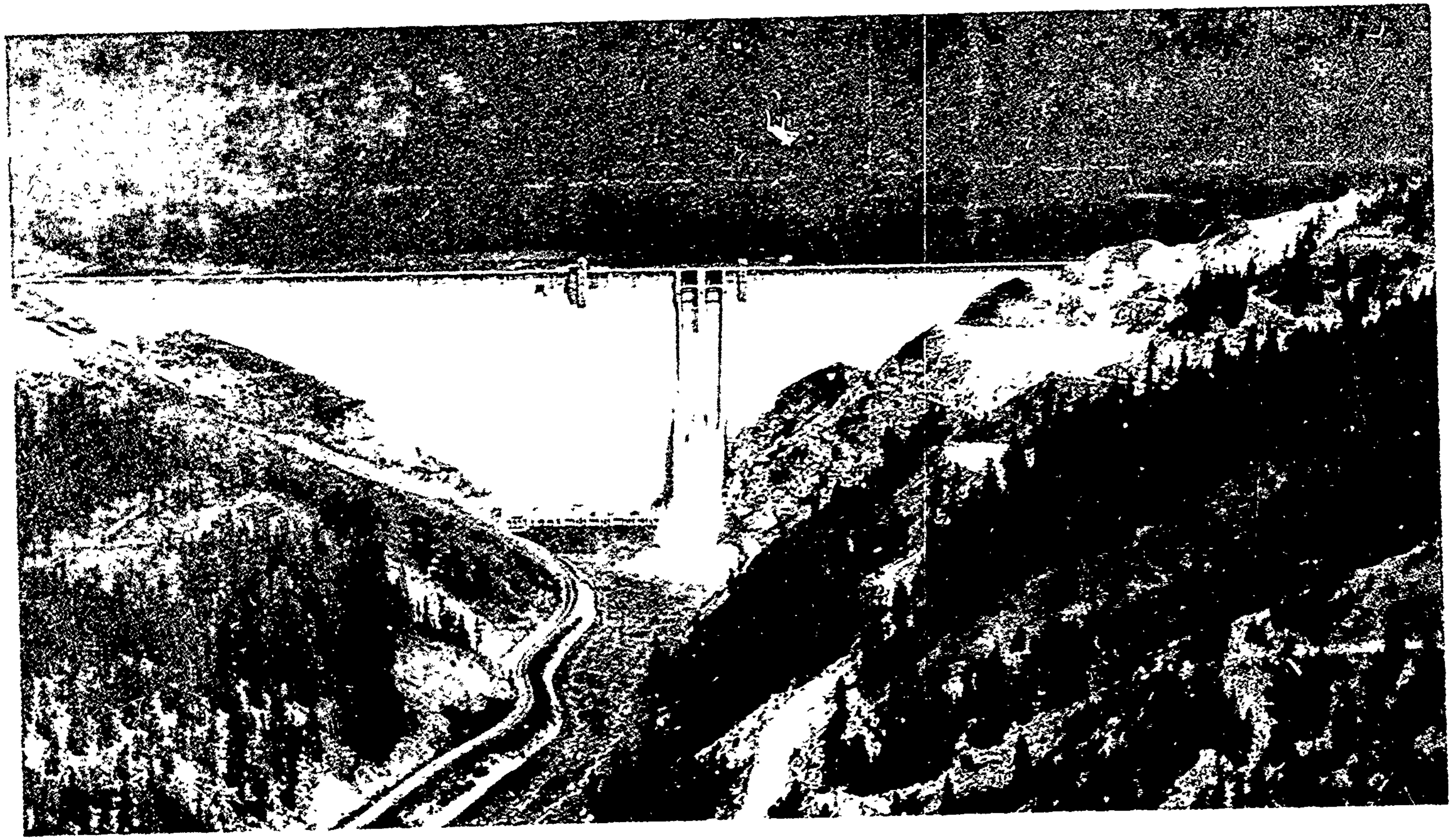


Figure 105. Low-level outlets discharging into stilling basin, Dworshak Dam
(from Walla Walla District 1972)

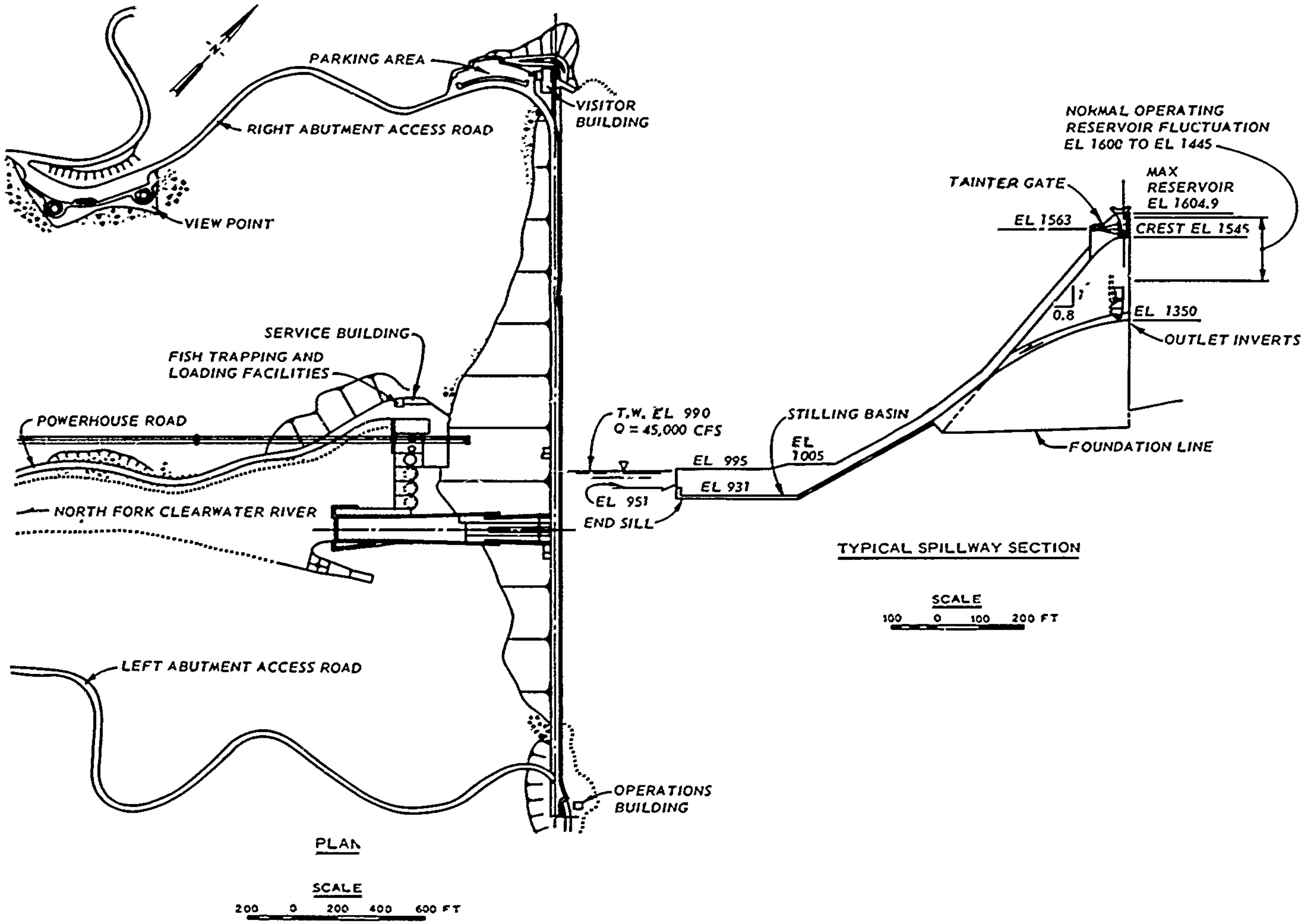


Figure 106. Plan and spillway section, Dworshak Dam

materials from construction and operation of the dam had entered the stilling basin and had caused severe damage to the concrete.

193. The fourth periodic inspection was conducted in June 1975 (Walla Walla District 1975a). At this point, the estimate of damage included erosion of 2000 cu yd of concrete with bedrock exposed in some areas. A topographic survey of the damaged floor was conducted (Figure 107), and soundings were made to determine the condition of the outlet channel downstream of the stilling basin (Figure 108).

194. The stilling basin was dewatered for repair in August 1975 as reported by the Walla Walla District (1976), Schrader and Kaden (1976), and Murray and Schultheis (1977). A 70-ton, 114-ft-long steel box girder was floated into position at normal tailwater elevation between the right and left stilling basin training walls and above the end sill. Tailwater elevation was lowered by controlling the powerhouse discharge so that the beam positioned itself into supporting brackets at the training walls. When the box girder was positioned (Figure 109), steel sheet piling was placed against the downstream face of the girder and the end sill steel plate. A polyethylene sheet was placed against the upstream face of the sheet piling, and leaking joints of the sheet piling were sealed using a mixture of manure and sand. A masonry block wall two courses high was placed along the end sill to trap water that still leaked through the bulkhead. Water was pumped from this sump back to tailwater and kept the stilling basin fairly dry under normal circumstances.

195. Approximately 30 cu yd of debris (e.g. reinforcing steel, boulders, and cables) was found on the stilling basin floor (Figure 110). The surface of the stilling basin concrete at the time of dewatering was irregular but smooth, indicating wear or erosion. However, just downstream of the end sill, large blocks of concrete, estimated to range in weight from 50 to 1500 lb and containing reinforcing steel that had been snapped off, could be seen. These blocks were jagged and were apparently not the result of erosion. It is possible that cavitation forces occurred at some time and removed the large blocks of jagged concrete and that general erosion at a later date caused the smoothly

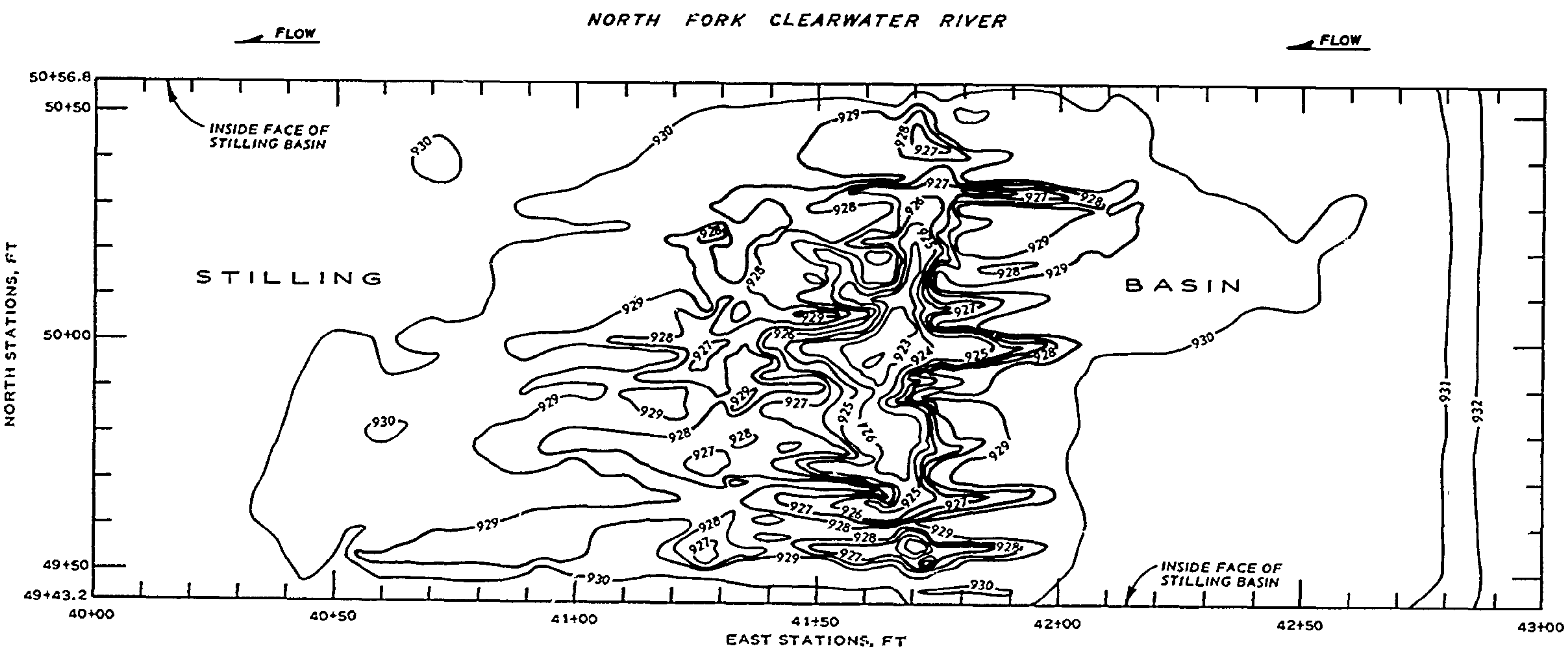


Figure 107. Survey of stilling basin damage, Dworshak Dam (from Walla Walla District 1975a)

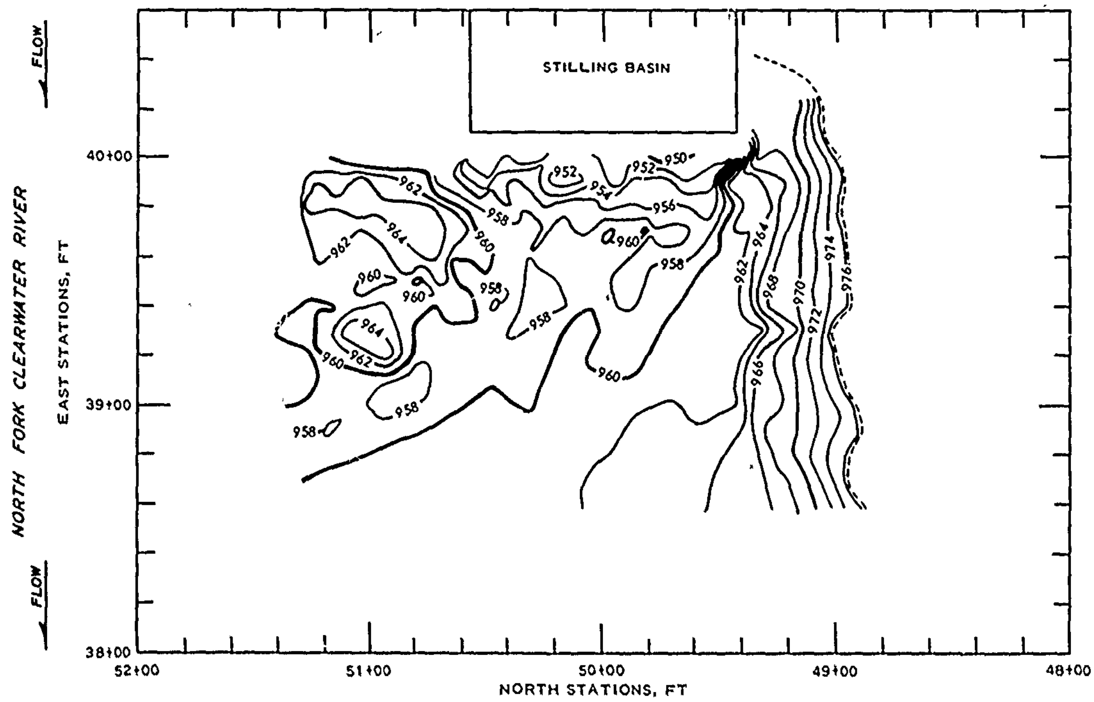


Figure 108. Results of soundings downstream of the stilling basin, Dworshak Dam (from Walla Walla District 1975a)

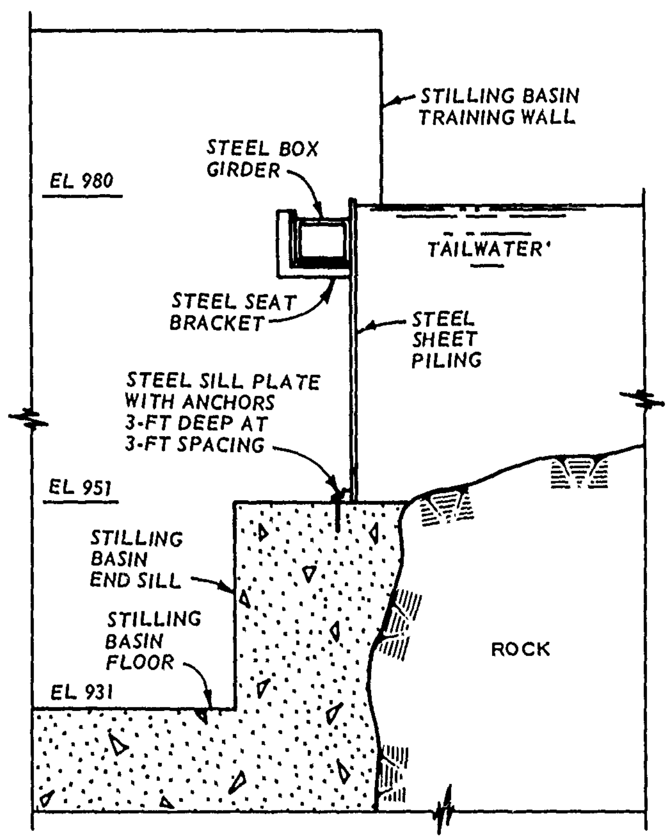


Figure 109. Bulkhead for dewatering, Dworshak Dam (from Walla Walla District 1976)

Figure 110. Stilling basin debris, Dworshak Dam

worn surface. The majority of the stilling basin floor received some degree of severe damage (Figures 111 and 112). At the deepest point (approximately 10 ft), erosion had progressed completely through the concrete and into the granite bedrock. Apparently, erosion of the stilling basin floor started at the joints in the slab. All 1049 uplift pressure drainholes placed during original construction were plugged with gravel, nails, reinforcing bar ends, and miscellaneous debris.

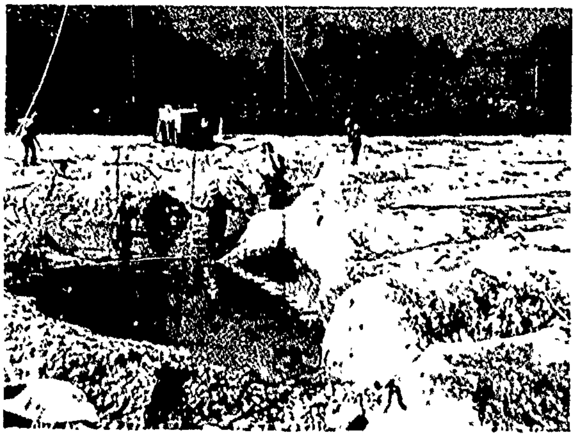
196. Compared to the major erosion of the floor, the walls of the stilling basin were in reasonably good condition. The maximum depth of erosion was approximately 6 in. and was generally confined to the bottom 4 ft of the wall.

197. The extensive damage in the stilling basin was attributed to two factors: (a) large amounts of construction debris deposited prior to and during initial operation, and (b) unbalanced flow into the basin because of inoperable or faulty gates in the spillway and outlet works. In addition to churning the debris around inside the basin, unbalanced flow can draw material from downstream into the basin. Based in part on the experience at Dworshak, OCE (1976) circulated guidance outlining practical measures to reduce the possibility of abrasion damage to stilling basins.

198. Following a thorough cleanup, eroded areas of the stilling basin floor were filled to within 15 in. of final floor elevation with 730 cu yd of structural grade unreinforced fill concrete. This concrete, with 1.5-in. maximum size aggregate, was pumped 65 ft straight down from the top of the training wall to a horizontal pipe that ran out an additional 50-250 ft to the placement area.

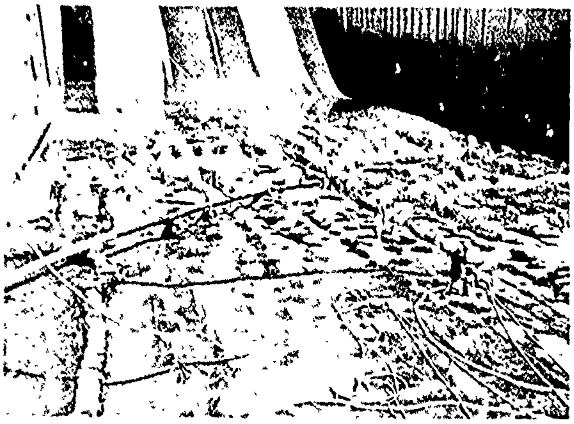
199. It was virtually impossible to follow existing construction joints, or any other preferred construction jointing, during the

Figure 111. Erosion of stilling basin floor slab, Dworshak Dam (from Walla Walla District 1976)



a. Maximum erosion,
Station 41 + 65

b. Erosion at
Station 40 + 50



c. General stilling basin
erosion

d. Typical wall erosion

Figure 112. Erosion damage, Dworshak Dam
(from Walla Walla District 1975a)

placement of fill concrete. As the placement would proceed from where the erosion was shallow towards another location where erosion was much deeper, the volume of material needed to maintain a "live" face on the fresh concrete became greater than that the pump could supply. Cold joints at unplanned locations became unavoidable. These joints were considered acceptable since the concrete filling these large holes merely acted as large blocks of solid fill material held in place with the fibrous concrete topping and prestress bars.

200. Prior to the placement of fiber-reinforced concrete topping, No. 8 anchor bars were placed in rotary percussion holes on 5-ft centers and grouted with a nonshrink cement grout. A horizontal mat of No. 6 reinforcing bars on 15-in. centers was hooked to the anchor bars (Figure 113). The fiber concrete mixture proportions were as follows:

Material	Weight, lb
Portland cement	715
Fine aggregate	1450
Coarse aggregate (3/4 in. MSA)	1400
Steel fibers (10 by 22 mils, 1 in. long)	120
Water	293
Water reducing and retarding admixture	51 oz
Air-entraining admixture	*

* As necessary for air content of 3 to 5 percent.

Flexural strengths of this mixture were in the range of 775 to 860 psi at 7 days and approached 1000 psi at 28 days. Compressive strengths at 7 days were in excess of 6000 psi and were approximately 8000 psi at 28 days. Although these strengths are impressive, the primary reason for using fiber concrete was its extended fatigue life and ability to absorb energy, thereby resisting damage caused by impact.

201. Existing construction joints in the stilling basin were continued through the 15-in. topping of fiber concrete. Placements of concrete, normally about 110 cu yd, were accomplished using a crane and two concrete buckets. The rate of placement varied considerably with weather conditions and batch plant operation. The best placement was completed in approximately 4 hr; but some placements, which were

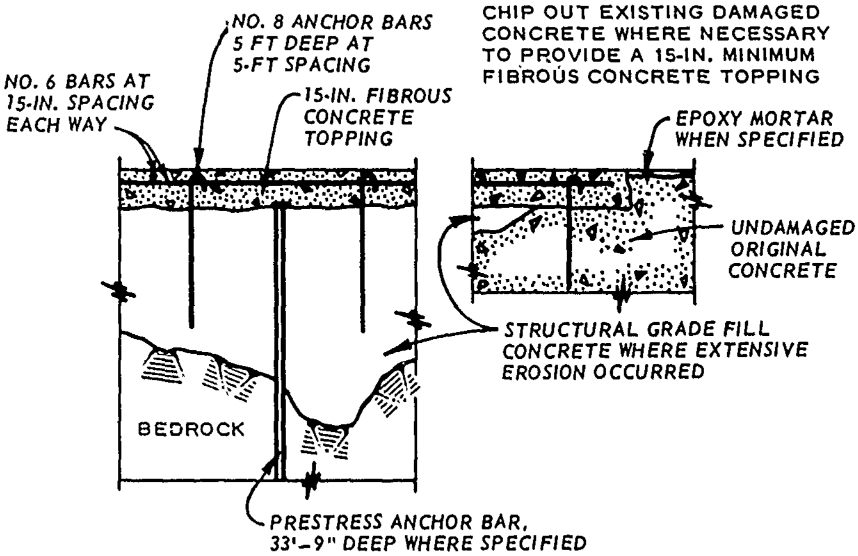


Figure 113. Repair details, Dworshak Dam (from Walla Walla District 1976)

prolonged by cold weather, rainstorms, and batch plant breakdowns, took up to 12 hr. Six-inch-diameter air vibrators were used for internal concrete consolidation, and a vibrating screed was used to strike off the surface.

202. After it had been placed, the concrete was impregnated with methyl methacrylate (MMA) monomer. It was the original intent to polymer impregnate the entire stilling basin floor. However, due to the limited time available and the fact that impregnating the entire basin floor would not allow a comparative analysis of the effects of impregnation, only the right half of the stilling basin was impregnated.

203. Each area (720 sq ft) to be polymerized was enclosed (Figures 114 and 115) and dewatered with a wet vacuum to reduce the time required to dry the concrete. Sand was then applied by hand within the enclosed area to a depth of approximately 3/8 in. The sand acted as a wick to assure uniform distribution of the monomer during the saturation phase because the concrete could not be finished absolutely level. Drying of the concrete within the enclosure was accomplished using infrared heat lamps. The temperature of the concrete surface was elevated to approximately 300°F in 4 hr and was maintained at that level

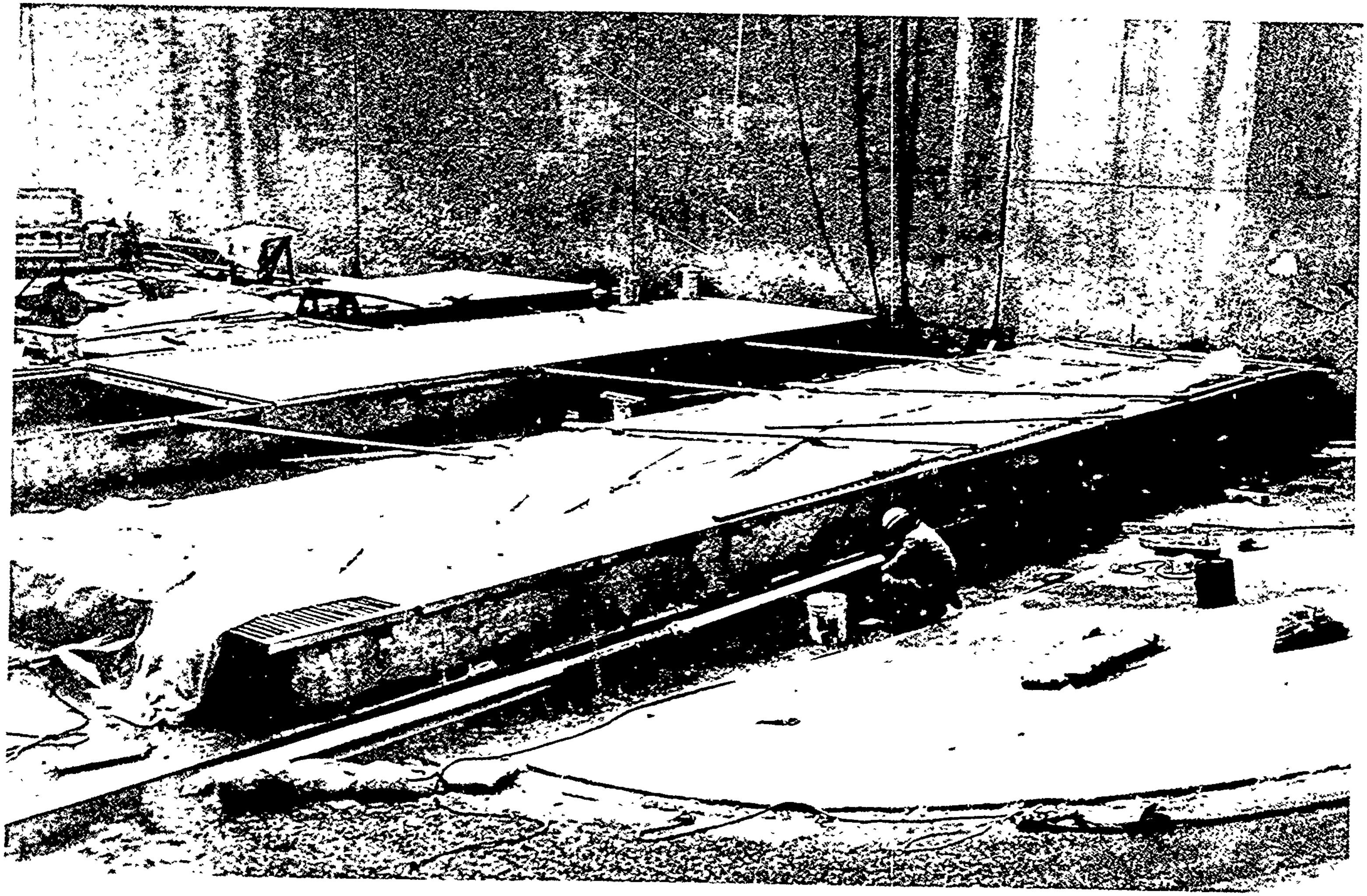


Figure 114. Enclosures for polymerization operations, Dworshak Dam (from Walla Walla District 1976)

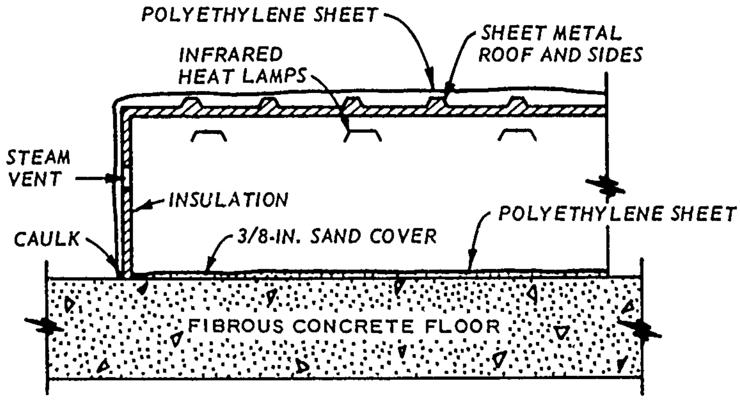


Figure 115. Typical section through polymerization enclosure, Dworshak Dam (from Walla Walla District 1976)

for a period of 7 hr. The heat lamps were then turned off and the concrete was allowed to cool to approximately 120°F. During the cooling period of approximately 8 hr, the enclosed area had to be maintained in a dry condition.

204. Roof sections of the enclosure were then removed and monomer was applied over the sandbed by gravity through a spray bar extending the width of the enclosure. Application of the monomer at the rate of 1 gal/sq yd required only a few minutes. At the conclusion of the monomer application the insulated steel cover was replaced and the entire enclosure was covered with polyethylene sheeting to prevent evaporation during the soak period. Loss of monomer to the atmosphere was minimal and no hazardous levels were recorded on a vapor monitor. Using a 6-hr soak period, monomer absorption by the fiber concrete was slightly less than 1 lb/sq ft of area. At the conclusion of the soak period the monomer was polymerized by heating. Heating of the saturated concrete was done with steam heat distributed for a period of 1 hr at approximately 200°F. A typical polymerization cycle is shown in Figure 116.

205. The original monomer system consisted of 95 percent methyl methacrylate, 5 percent cross-linking agent (TMPTMA), and about 0.5 percent catalyst (VAZO 64). This proportion was soon changed to approximately 97 percent MMA, 2.5 percent cross-linking agent, and 0.5 percent catalyst. The cross-linking agent was reduced so that polymerization

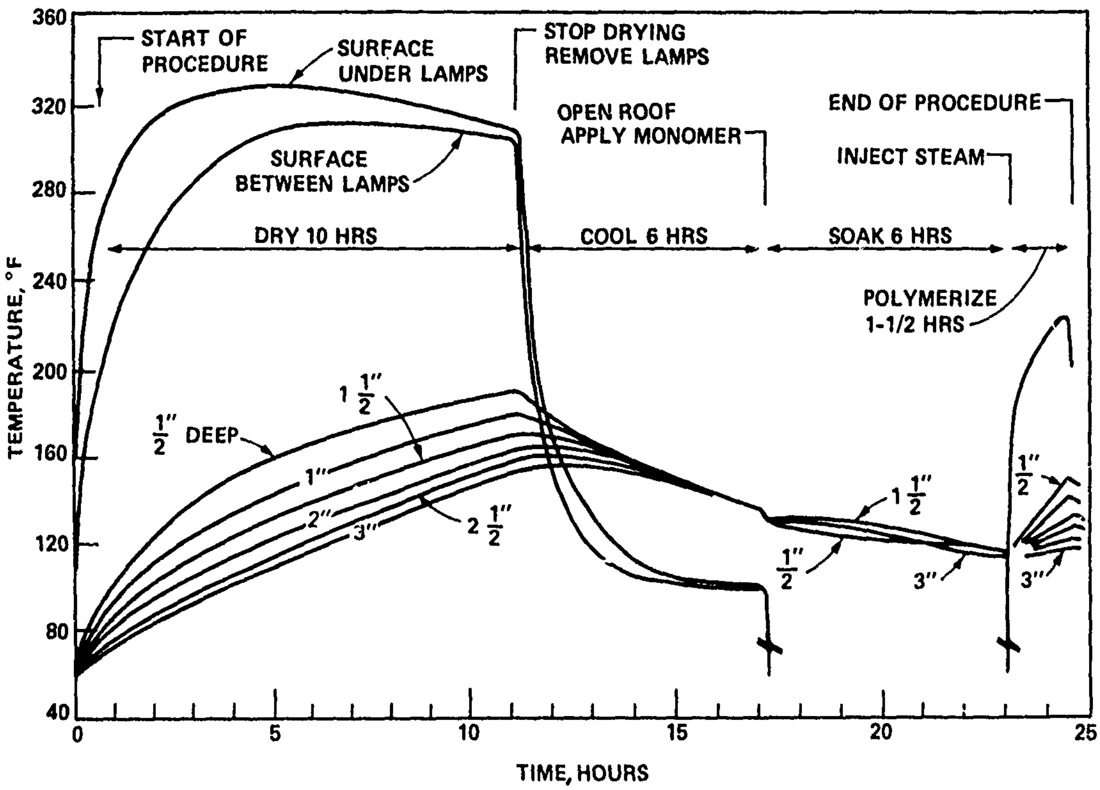


Figure 116. Typical polymer-impregnation cycle, Dworshak Dam

would occur at a slightly slower rate and the monomer could be applied before it had cooled to a temperature below that at which polymerization would be initiated.

206. After the concrete was polymerized, the enclosure was removed to a new area and cleanup of the impregnated area began. The contaminated sand was removed and buried in a predesignated area. From the initial placement of the enclosure to the removal of the polymerized sand, the procedure required approximately 24 hr. With two enclosures, full polymer impregnation of 720 sq ft was successfully accomplished within each 24-hr period. At this rate, approximately half of the stilling basin floor, or 15,000 sq ft, was completed in 11 days.

207. In areas where erosion of the original concrete had not progressed to the 15-in. minimum depth specified for the fibrous concrete topping, the design called for excavation of the remaining concrete to this depth (see Figure 113). However, one section of the stilling basin

floor (approximately 2160 sq ft) and the lower 20 ft of the spillway (2950 sq ft) exhibited only minimal erosion to a maximum depth of about 4 in., and the original reinforcing mat was not exposed. Therefore, it was decided to repair both sections with an epoxy mortar topping.

208. The epoxy work was completed using several types of epoxy mortar. The primary one was a stress-relieving material that was slow curing and had a low exotherm. This allowed mixing and placing in thick sections up to several inches deep without requiring layered systems. Several problems occurred with the epoxy during application that were primarily the result of workmanship, weather conditions, and failure to enclose the work area. Under the cool conditions that existed, the epoxy mortar probably did not fully cure before the stilling basin was put back into service. The original thought for using the epoxy was that it would be economical and fast, but this was not the case, and the quality of work was less than desired.

209. The repair contract called for cleaning of the uplift pressure drains and extending them through the replacement concrete; however, the existing drains were virtually impossible to clean. Several alternatives to the drains were considered before a decision was made to install prestress units to restrain uplift forces. Two hundred fifty-three prestress units were installed in rotary percussion holes drilled to a depth of 33.75 ft (see Figure 113). The bottom of the hole was filled with fast-setting, prepackaged polyester resin capsules and the upper portion of the hole was filled with prepackaged slow-setting polyester resin capsules. A high-strength prestress bar was then pushed through the polyester resin packages and rotated until it reached the bottom of the hole. Within 15 min. the bottom 5 ft of the bar was anchored with the fast-setting materials. The bar was then stressed to 98 kips and anchored at the jacking end with a nut. The slow-setting polyester resin later hardened and totally encased the upper portion of the bar with a structurally bonded material.

210. It was originally planned to repair the walls of the stilling basin using procedures similar to that for the floor. However, the walls were in reasonably good condition; consequently, they were rapidly and

economically repaired with a high-strength shotcrete mortar.

211. The cost of mobilization and dewatering of the stilling basin was approximately $700,000, and the cost of the polymer-impregnated floor surface was about $6.50/sq ft. The cost of the epoxy mortar repair was about $15.50/sq ft, and the cost of the fibrous concrete was approximately $5.60/sq ft. The contract was completed on 15 December 1975, and the stilling basin was filled.

212. A diver inspection of the stilling basin repairs was made in July 1976 (Walla Walla District 1977). During the time between completion of repairs and inspection, the basin was subjected to a total of 53 days usage (9 days from the spillway gates and 44 days from the regulating outlets). The spillway and outlet gates were operated symmetrically (or very close to it) for all spills. Total flows ranged between 2,100 and 20,000 cfs, with the majority being on the order of 3,000 to 10,000 cfs.

213. The inspection required approximately 1-1/2 hr of underwater time during which the diver walked approximately 2000 ft in the basin. Visibility was 2-3 ft; therefore, approximately 15 percent of the stilling basin and lower spillway area was physically inspected. Results of this inspection are described briefly in the following paragraphs.

214. There was no major erosion or damage. There were isolated accumulations of gravel, rebar, and debris at a number of locations throughout the basin. Some of this was located near the end sill, some near the training walls, and some near the lower portion of the spillway curve. The size of these accumulations ranged from 5 gal or less to an area about 12 by 20 by 1-2 ft deep. The debris consisted of gravels to 6-in. size and rebar about 1 in. in diameter. The diver also found two piles of large steel pipe and angles about 14 ft long with 8-10 pieces in each pile. Since the stilling basin was totally cleaned prior to being filled after the major repairs of 1975 and since there was no current erosion sufficient to expose reinforcement or produce large size gravel, the debris must have come from downstream of the end sill. The stilling basin walls exhibited a small amount of surface erosion

(less than 1 in.). There were several areas at the junction between the floor and wall with erosion up to 3 in. deep.

215. The epoxy mortar that had been placed on the lower spillway and a portion of the stilling basin slab were failing. This area was filled with failures where the epoxy had broken loose at the concrete interface. An estimated 25 percent of the surface was gone. The epoxy failure areas ranged from a fraction of an inch deep up to 4 in. deep. Tensile forces at the surface apparently pulled up the epoxy and caused it to fail. These forces could have been caused by indirect tension from impact or by the high velocity water flow. The latter seems more probable because the diver found one piece of epoxy mortar about 1 to 2 in. thick and 1 ft square near the opposite end of the stilling basin. Indirect tensile forces from impact would not have broken loose a piece of this size. Concrete below the epoxy mortar has apparently failed. The joint between epoxy mortar and fibrous concrete typically has areas where the epoxy failed and the fibrous concrete edge is about an inch higher than the nonfibrous side.

216. The fiber concrete (both polymer impregnated and not impregnated) was generally in good condition. There were several areas in the center of the basin (where severe damage to approximately 10 ft deep had previously occurred) that were several feet in diameter and dished out up to an inch deep. Except for joints and open cracks, the diver described fibrous concrete areas as looking like a "new garage floor." In general, the impregnated side was probably a little better than the nonimpregnated side, but both sides were very good. He found much of the polymer-impregnated concrete to still have the 1/8-in.-thick (plus) sand layer stuck to it where this had adhered during the impregnation process. Joints and open cracks in the entire basin (including fiber concrete) have been the most susceptible to damage. Typical joints and open cracks in the fiber concrete had eroded up to about 1 in. deep at the joint and had tapered out to the original floor surface within a few inches to a foot of the joint. Because of moisture in the joints and cracks during the repair, concrete at joints and cracks was not impregnated.

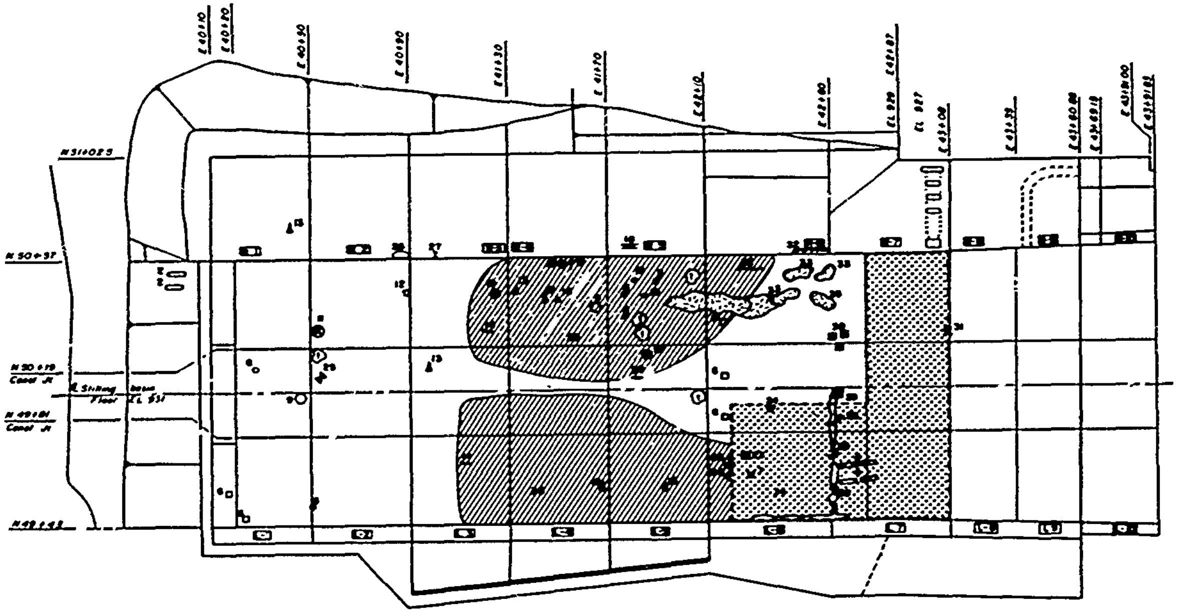
217. The end sill was in good condition and the sill plate for the temporary bulkhead still in place. The area just downstream of the end sill (for perhaps 20 ft) was inspected. It has an erratic bedrock surface with high, waterworn peaks. Rock at the left end of the sill drops down about 4 ft for about 10 ft downstream and then has a general slope upward. Rock at the rest of the sill slopes up immediately from the top of the sill. Just past the right side of the sill (off the right training wall) the rock goes immediately upward to within an estimated 5 ft of tailwater at tailwater el 977. Crevices in the rock contained a small amount of gravel and quite a bit of reinforcing steel, angle iron, etc. Some of this steel was wedged tightly into rock joints.

218. After some additional usage of the stilling basin, a diver was employed in November 1976 to clean the debris from the basin and provide more detailed information on the condition of the floor. Results of the diver inspection are shown in Figure 117. Photographs of the debris are shown in Figure 118. Significant comments resulting from the stilling basin inspection are as follows:

- a. Even with an initially clean basin and with symmetrically operated outlet discharges, debris is brought back into the basin.
- b. There is some general wear at the joints. The joints were not successfully impregnated with monomer, due to moisture.
- c. The large area of epoxy mortar repairs has failed.
- d. There is some "grooving" in the concrete surface near the center of the basin. The grooves run only in the upstream/downstream direction.
- e. Inspections to clean the basin and to report on its condition should be considered necessary and routine procedures after each major use.

Ice Harbor Dam

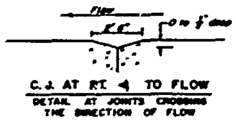
219. Ice Harbor Dam is a concrete gravity structure (Figure 119) with powerhouse, navigation lock, and embankment segments located on the Snake River near Pasco, Washington. The structure has a regulated



NOTES

THE AREA NUMBERED 35 HAS GROOVES IN THE AREAS AS SHOWN. THE GROOVES ARE PARALLEL WITH THE FLOW AND SPACED APPROXIMATELY 2"-5" APART.

DISHING AT JOINT LINES IS APPROXIMATELY 0 TO 1/2" DEEP. SEE DETAIL. THE FLAT EPOXY AREA ON THE FLOOR (STA. 42+70 88 TO STA. 42+20) IS DAMAGED THROUGHOUT. ONLY THE WORST DAMAGES ARE NOTED FOR CLARITY. FROM THE DIVERS REPORT IT IS ESTIMATED THAT ABOUT 80% OF THE EPOXY IS DAMAGED OR MISSING IN THIS AREA.



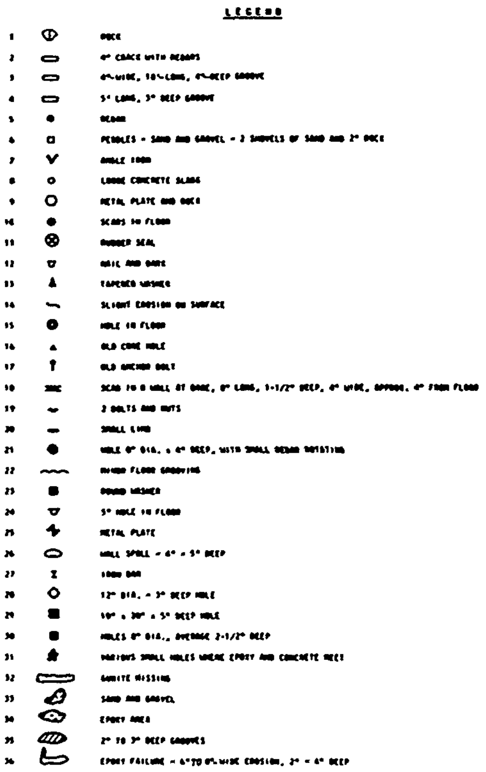
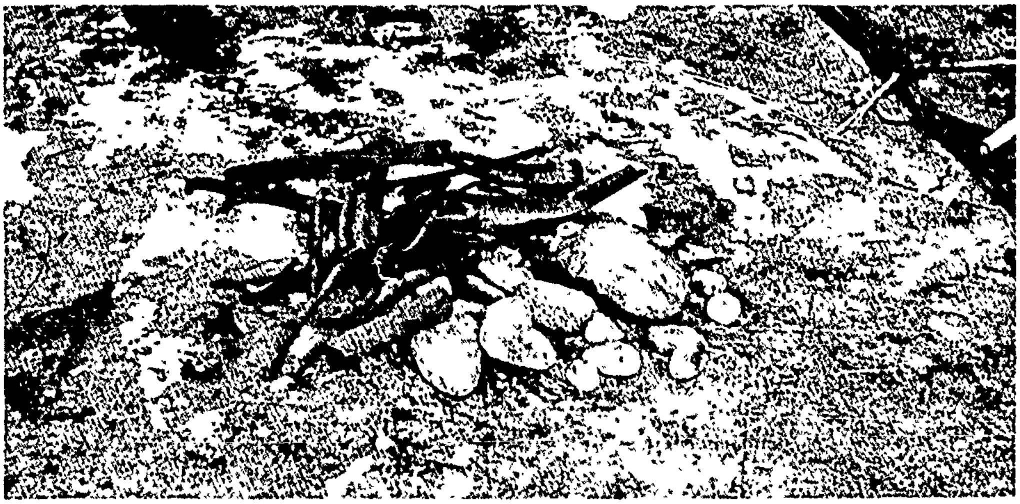


Figure 117. Stilling basin condition, Dworshak Dam (from Walla Walla District 1977)

a. General gravel and non-metallic debris



b. Metallic debris and cobbles

Figure 118. Typical debris removed from stilling basin during Nov 1976 cleaning, Dworshak Dam (from Walla Walla District 1977)

overflow spillway, 10 bays with ogee sections, and tainter gates (Figures 120 and 121). Spillway deflectors have recently been built into

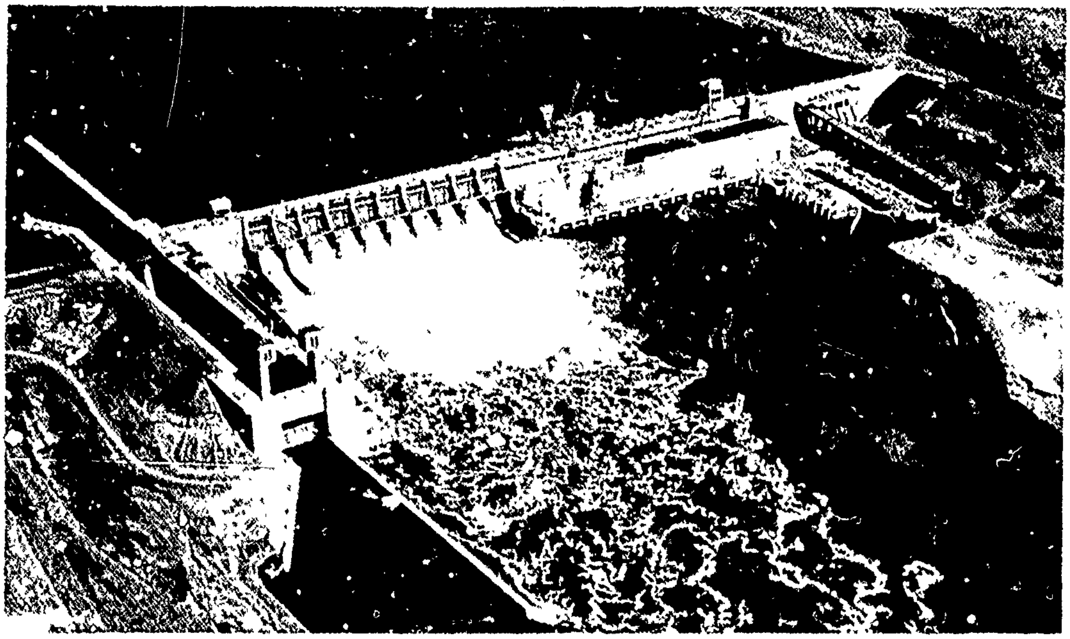


Figure 119. Ice Harbor Lock and Dam (from Walla Walla District 1975b)

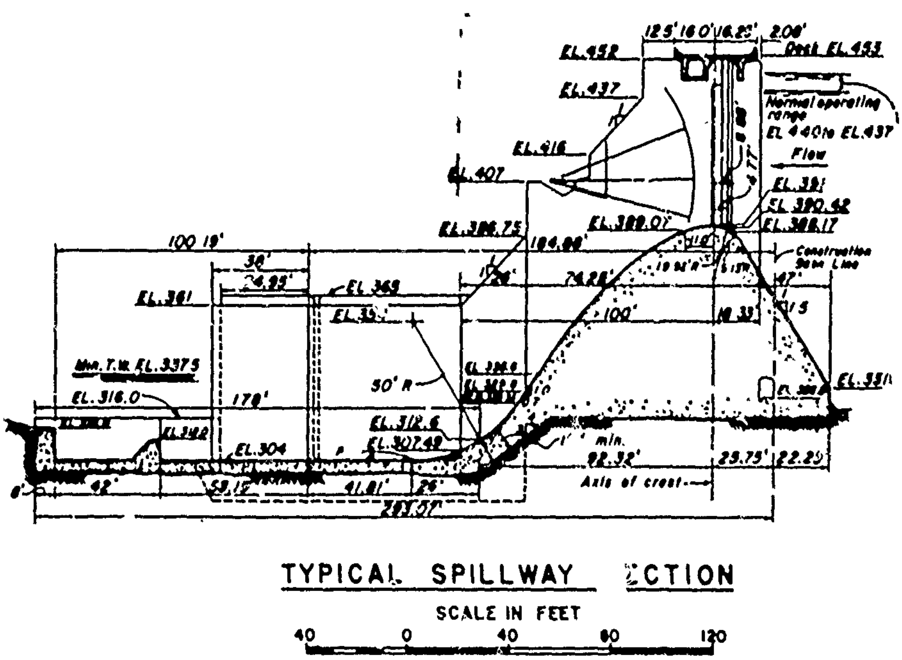


Figure 120. Spillway and stilling basin, Ice Harbor Dam (from Walla Walla District 1975b)

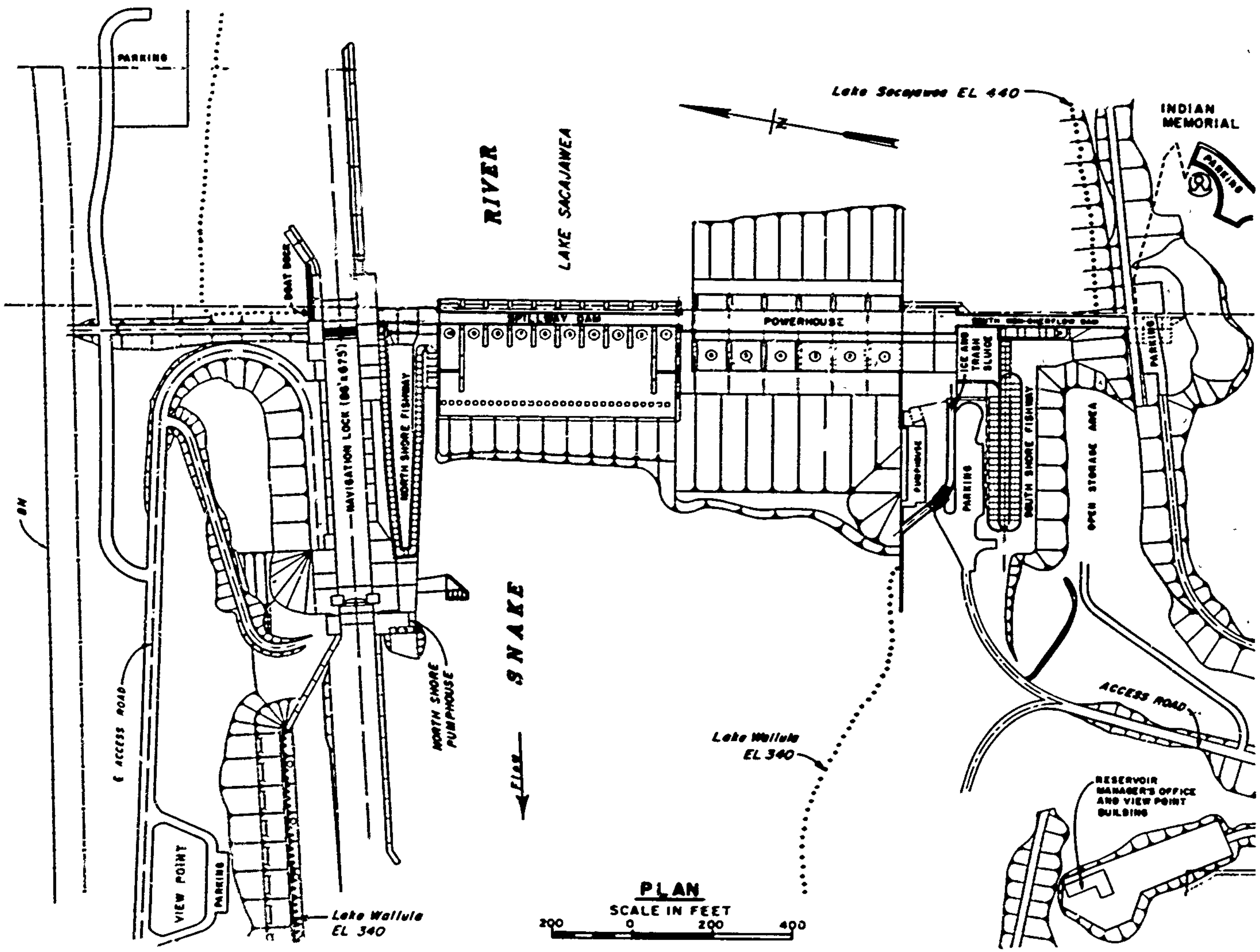


Figure 121. Ice Harbor Lock and Dam (from Walla Walla District 1975b)

the ogee sections. The spillway design discharge is 850,000 cfs. The stilling basin is cast in-place concrete 590 ft wide and 168 ft long. The horizontal floor slab has one row of concrete baffles 8 ft high and 10 ft wide and a vertical face end sill 12 ft high.

220. After the stilling basin had been completed, it was watered up and used for diversion of the river cha [illegible] during construction of the lock and right abutment embankment. [illegible] verflow went between the spillway piers and over the base concrete [illegible] which the spillway ogee sections would later be placed. During this diversion, debris from construction and the river bottom apparently accumulated in the stilling basin. In August 1963, divers inspected the stilling basin (Walla Walla District 1966) and found it full of rocks, reinforcing steel, steel plates, and other debris. The basin had eroded to bedrock in an area about 20 by 20 ft. The overall erosion zone was about 50 by 50 ft. Concrete in front of bays 1 and 2 was also badly damaged. Contracts were let immediately and the basin was cleaned of debris and repaired with tremie concrete prior to flood time in the spring of 1964. In September 1965, a diver inspection indicated that the repaired and original concrete appeared to be in good condition with no evidence of further erosion. No accumulations of debris were reported. In late 1974, soundings of the basin were made (Walla Walla District 1975b) and no serious erosion was noted.

221. Additional powerhouse units were installed at the project in about 1976. This has greatly reduced the frequency of use and quantity of flow that the spillways and stilling basin are subjected to.

Barren River Dam

222. Barren River Dam was completed in 1964. Located on the Barren River in Kentucky, the dam is 13 miles southwest of Glascow and 95 miles south of Louisville. The outlet works of the earth and rockfill dam include a U-shaped stilling basin (Figure 122) 40 ft wide and 150 ft long, a vertical face end sill approximately 7 ft high, and a single row of baffles. The design discharge of the basin is 10,000 cfs

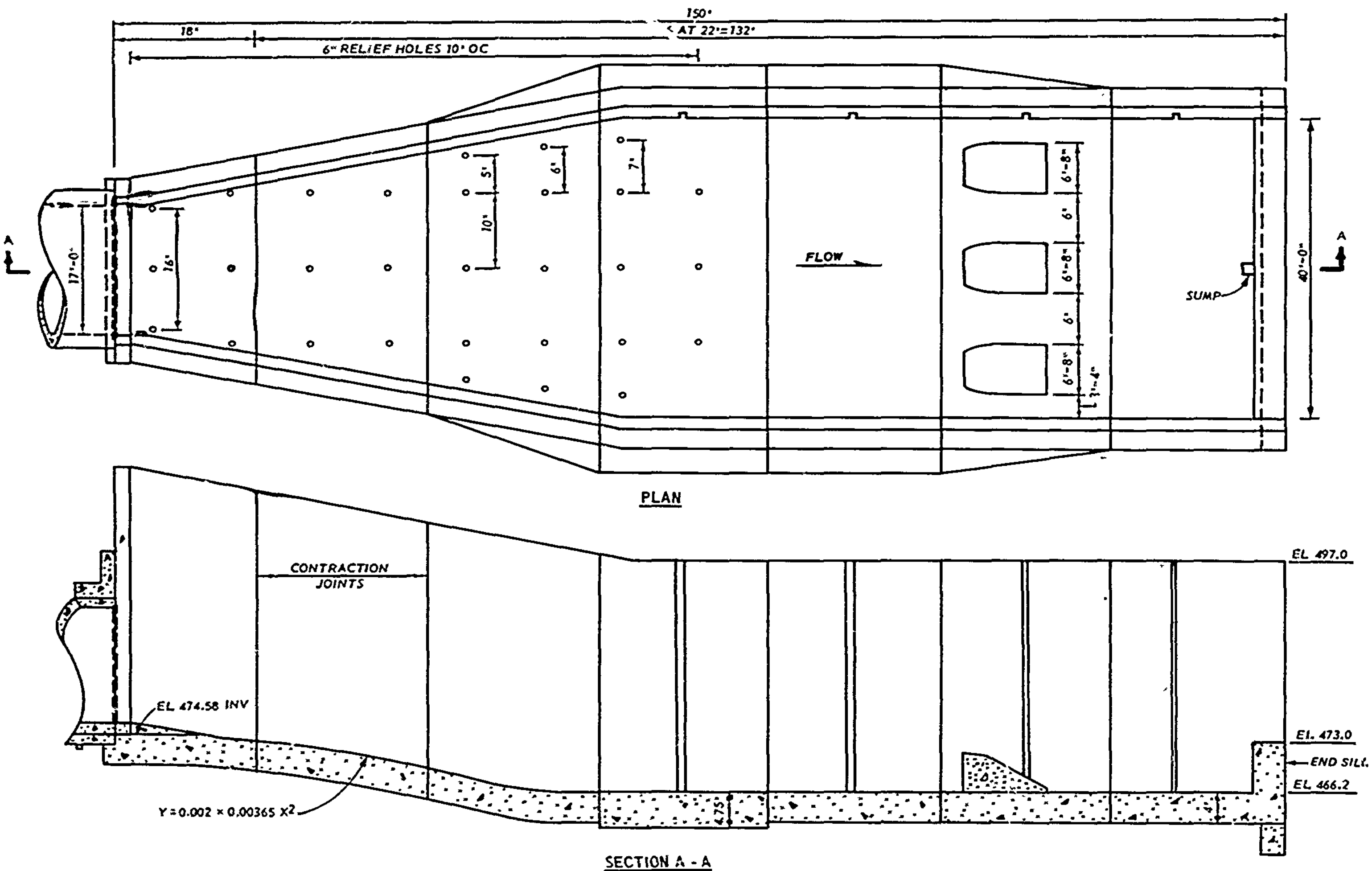


Figure 122. Stilling basin plan ar section, Barren River Dam

with an average velocity of 56.4 fps entering the basin. The original material in the stilling basin was 3000-psi reinforced concrete.

223. Movement of rocks within the basin eroded approximately 80 percent of the basin floor to an average depth of 6-8 in. with a maximum depth of 10 in. The stilling basin was dewatered for repair by constructing a rock-fill cofferdam downstream in the outlet channel and by using a suction pump to remove the water. A bypass conduit was used to handle gate leakage. Water pressure under the basin floor was relieved with temporary pipes.

224. Edges of the eroded area were sawcut and the concrete was chipped to a minimum of 9 in. below finished grade. Existing reinforcement exposed was removed and the surface was cleaned by waterblasting. Dowels and reinforcing each way were added and an epoxy bonding agent was applied prior to the placement of conventional concrete (5000 psi). In an attempt to prevent future damage, the bottom of the retreat channel was paved and the riprap slopes were slush-grouted for 126 ft downstream of the stilling basin. Cost of the repair, completed in the fall of 1976, was $104,000. The first inspection of the repair is scheduled for the summer of 1979.

Nolin Lake Dam

225. A rock- and earth-fill dam, Nolin Lake is located on Nolin River in Edmonson County, Kentucky. The dam's outlet works include a U-shaped stilling basin with baffles (Figure 123). Major dimensions of the stilling basin are 40-ft width, 174-ft length, 7-ft-high end sill, and 35-ft-high sidewalls. The stilling basin design discharge is 12,000 cfs with an average velocity of 61.2 fps entering the basin. Built of reinforced concrete (3000 psi), the project was completed in 1963.

226. During the September 1974 periodic inspection, a diver partially inspected the stilling basin and found rock, debris, and exposed reinforcement. The conduit and stilling basin were then dewatered and inspected in October 1974 to verify and determine the extent of damage. Erosion had occurred in the lower portion of the parabolic

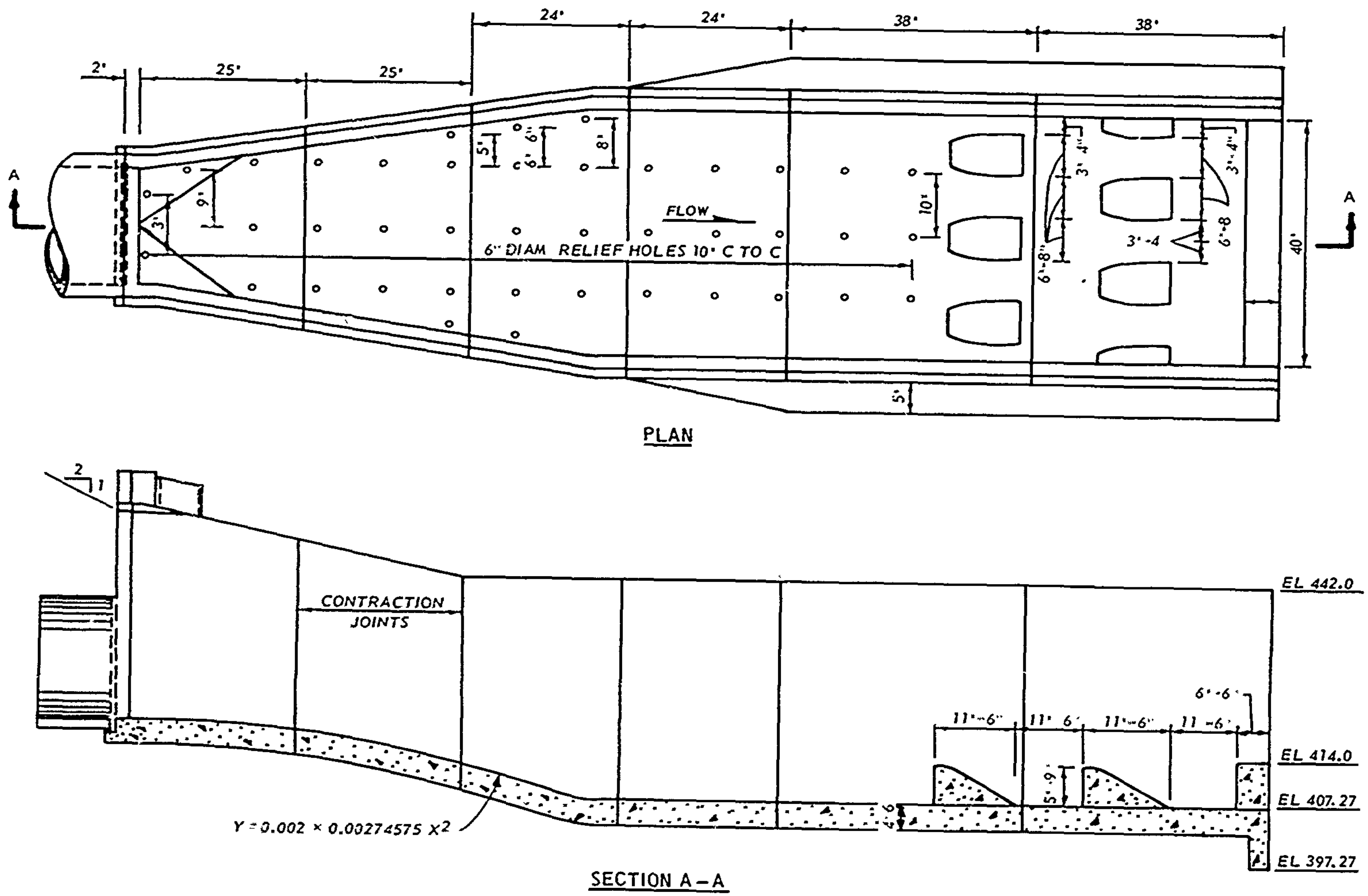


Figure 123. Stilling basin plan and section, Nolin Dam

section, in the stilling basin floor, in the lower part of the baffles, and along the top of the end sill. The most severe erosion (Figure 124) had occurred in the area between the wall baffles and the end sill where holes 2 to 3 ft deep had been eroded into the stilling basin floor along the sidewalls. The stilling basin floor erosion was most severe in the vicinity of the 6-in. relief drain holes.

227. The top layer of reinforcing steel was exposed on the lower portion of the parabolic section (Figure 125) and on the lower downstream portion of the baffles (Figure 124). The top reinforcement steel was missing in the middle portion of the stilling basin floor between the parabolic section and the baffles and also at the holes in the downstream corners of the stilling basin. Elsewhere, the stilling basin floor top reinforcement steel was in place, but exposed. At the end sill, the top reinforcement steel was either exposed or missing. Several feet of wall reinforcement steel were also exposed adjacent to the holes in the downstream corners of the stilling basin. After the inspection, Operations personnel removed the rocks and loose reinforcement (Figure 126) and returned the stilling basin to service until repair work could be scheduled.

228. Dewatering was initiated in May 1975 and included the use of a rock-fill cofferdam downstream in the outlet channel, suction pumps, and bypass conduits. The structural repair work included raising the stilling basin floor elevation 9 in. and raising the end sill elevation 1 ft. New work included adding end walls at the end of the stilling basin, a concrete pad adjacent to the right stilling basin wall, and a 50-ft-long concrete paved channel section. The additional 9-in. floor thickness adequately covered sidewall erosion near the old floor elevation except at the downstream corners. The existing concrete was saw cut to a depth of 2 or 3 in., where new concrete joined old concrete (Figure 127). The eroded concrete was removed down to sound (competent) concrete. In some cases it was necessary to remove some sound concrete to expose the reinforcement. At the parabolic section, however, existing sound concrete was removed to 9-in. minimum depth below new plan grade. The new parabolic section was revised slightly from the

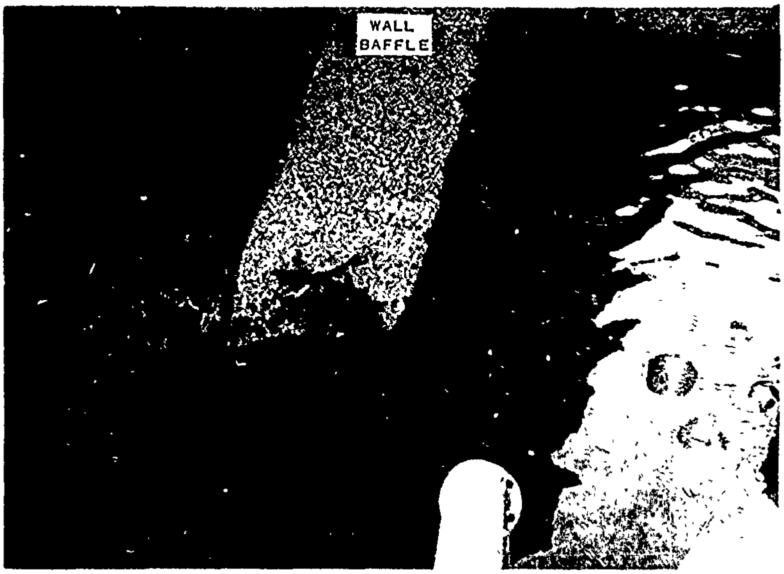


Figure 124. Stilling basin erosion, Nolin Dam

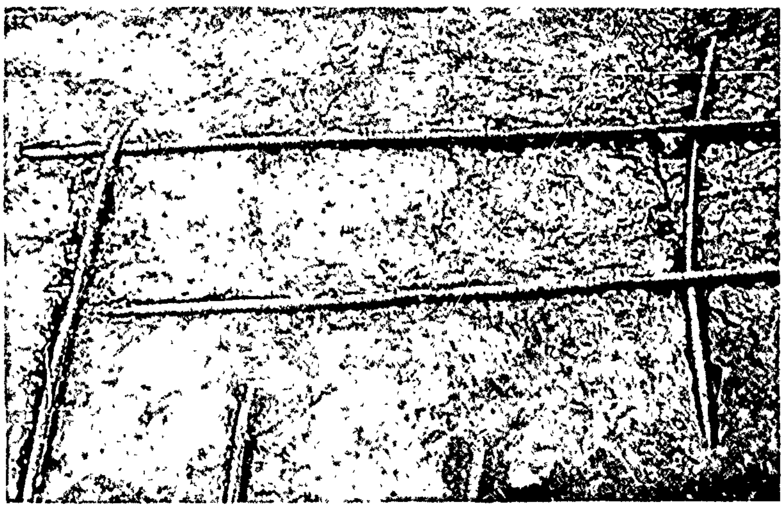


Figure 125. Typical erosion of parabolic section, Nolin Dam

Figure 126. Debris in stilling basin, Nolin Dam

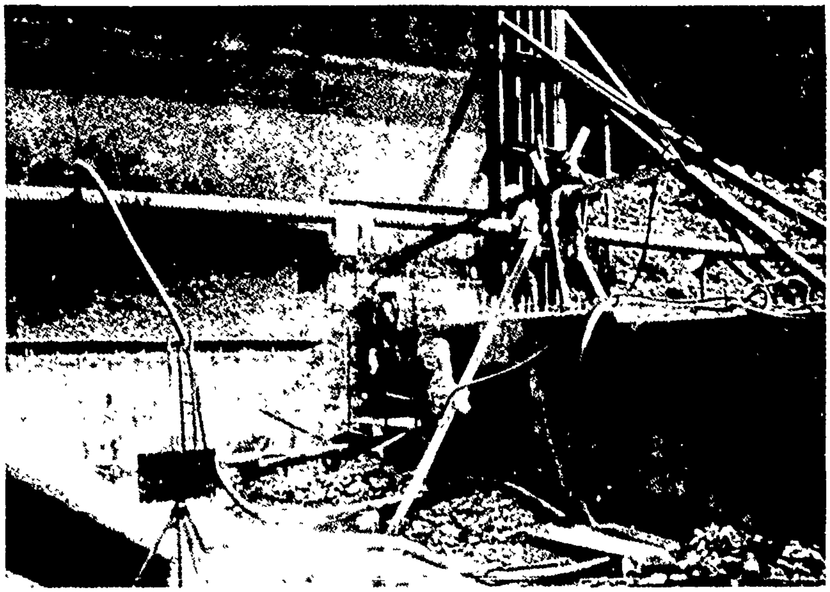


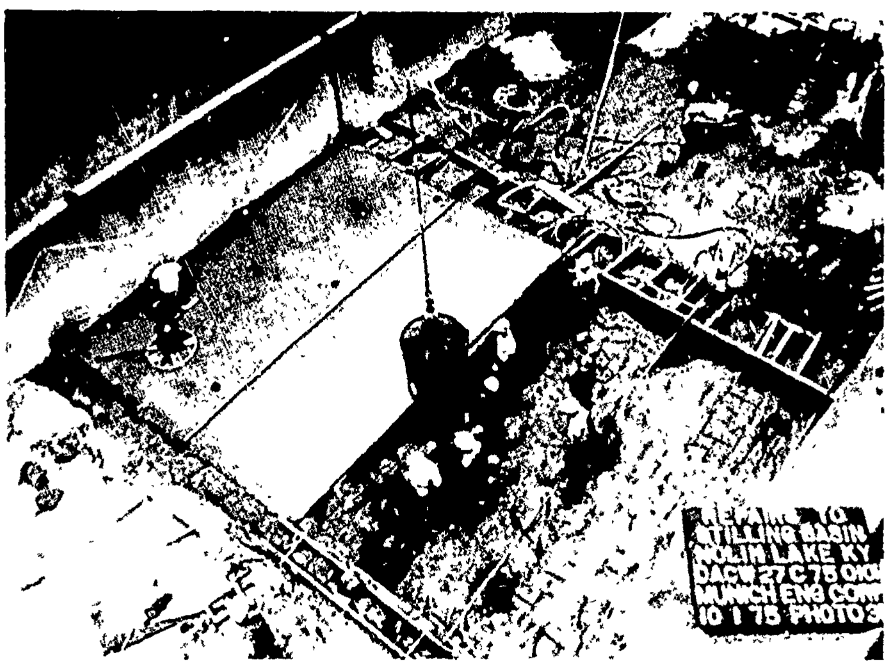
Figure 127. Concrete removal, stilling basin wall and end sill, Nolin Dam

original to meet the new floor elevation.

229. Dowels were used to anchor the new concrete (5000 psi unreinforced) to the old in the parabolic section and the floor (Figure 128). An epoxy bonding agent was used on the wet-sandblasted old concrete surface to bond the old and new concrete. The missing or previously removed stilling basin floor top steel was not replaced as calculations indicated that top steel was not required to carry loading stresses. Since the floor concrete is always submerged, temperature steel replacement was not considered necessary. The existing 6-in. relief drains were extended through the new concrete floor.

230. The new stilling basin end walls (Figure 129) are gravity walls, the top of which will support a removable closure beam at the stilling basin walls. Dowels were used to tie a portion of the concrete paved channel section (Figure 130) to the underlying rock. Where it was not practical to tie the concrete-paved channel section to the underlying rock, a thickened concrete section was placed. A fishing platform was provided on the slope of the paved channel section. The concrete pad adjacent to the right stilling basin wall will permit a mobile crane to place a closure at the end of the stilling basin wall

a. Prior to concrete placement



b. Concrete placement

Figure 128. Stilling basin repairs, Nolin Dam

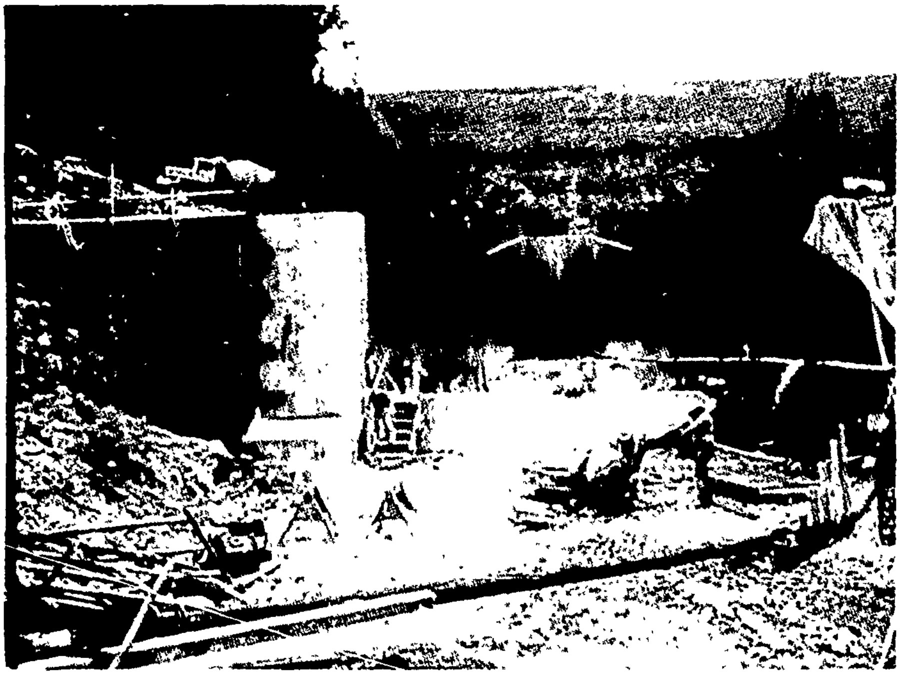


Figure 129. End wall construction, Nolin Dam

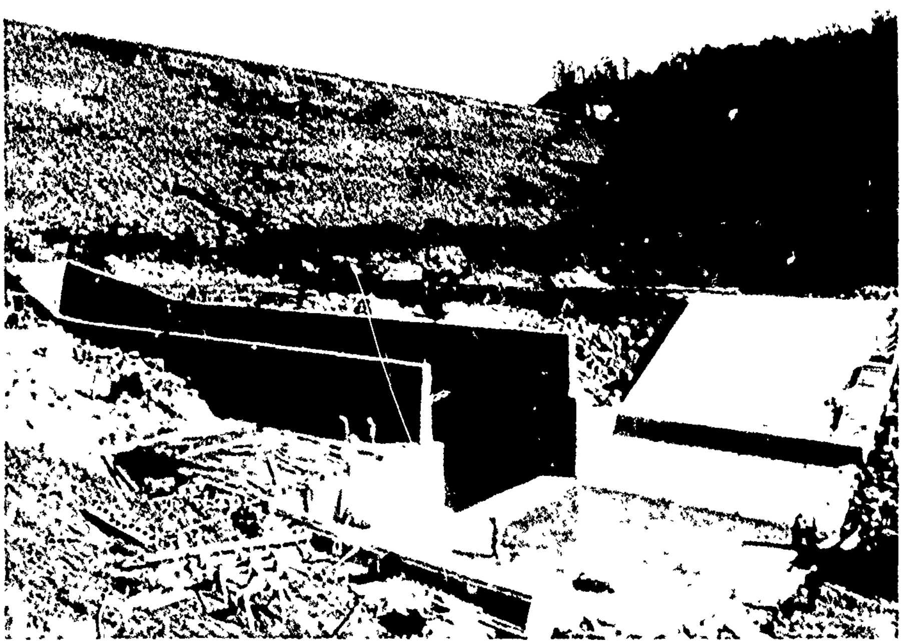


Figure 130. Stilling basin end walls, channel lining and equipment pad, Nolin Dam

for more expeditious dewatering of the basin.

231. The repair work was completed in December 1975 at an estimated cost of $119,000. Total contract cost including the new construction was approximately $356,000.

232. A diver inspection of the stilling basin was made in September 1976 by the Resource Manager, Nolin Dam, and the following damage was noted.

a. Erosion had created a trench 3 in. wide, 2 in. deep, and 30 ft long on the sloped portion of the basin floor. The appearance was similar to what would have occurred if water had run over the concrete before the mortar hardened.

b. Erosion had occurred on both sides of the stilling basin on the downstream side of the wall baffles. The left side of the stilling basin facing upstream showed the greatest erosion (up to 8 in. deep). Rock covered the right side. The downstream face of these two baffles had eroded to a concave shape 2 to 3 in. deep from original surface.

c. Approximately 4 tons of rock was in the stilling basin. The majority of this rock was from 3/4 to 5 in. diameter with some scattered rock up to 12 in. diameter. The rock, piled up to 15 ft deep, apparently entered the basin from downstream.

d. Piled rock up to 18 in. deep was found on the slab downstream from the stilling basin. It was similar in composition and size to that found in the stilling basin.

233. A similar inspection in August 1977 indicated that approximately 1 to 1-1/2 tons of large, limestone rock, all with angular edges, was scattered around in the stilling basin. No small or rounded rock was found. Since the basin had been cleaned during the previous inspection, this rock was thought to have been thrown into the basin by visitors. When the stilling basin was dewatered for inspection in October 1977, no rock or debris was found inside the basin. Apparently the large rock discovered in the August inspection had been flushed from the basin during the lake drawdown when the discharge reached a maximum of 7340 cfs. No additional damage had occurred in the stilling basin since the 1976 inspection.

Center Hill Dam

234. A combination concrete gravity and rolled earth-fill dam (Figure 131), Center Hill is on the Caney Fork River in DeKalb County, Tennessee, approximately 17 miles southeast of Carthage, Tennessee. The ogee spillway has a crest length of 470 ft surmounted by eight tainter gates with top at el 685. The design discharge is 454,000 cfs with a surcharge of 44.2 ft. The outlet works consists of six sluices, 4 ft wide by 6 ft high, with hydraulically operated slide gates. Their discharge capacity with pool at spillway crest el 648 is 9600 cfs. The flood of record occurred in February 1950 at which time the reservoir reached el 680.6.

235. The stilling basin (Figure 132) is of the roller bucket type with the bucket inverts stepped transversely across the spillway ranging from el 458 to 465 to 472. The stilling basin is 470 ft wide and the bucket radius is 50 ft with the lip of the bucket at an angle of 45 degrees with the horizontal. The maximum head available from maximum pool to lowest bucket invert is 243 ft. Releases were made through the

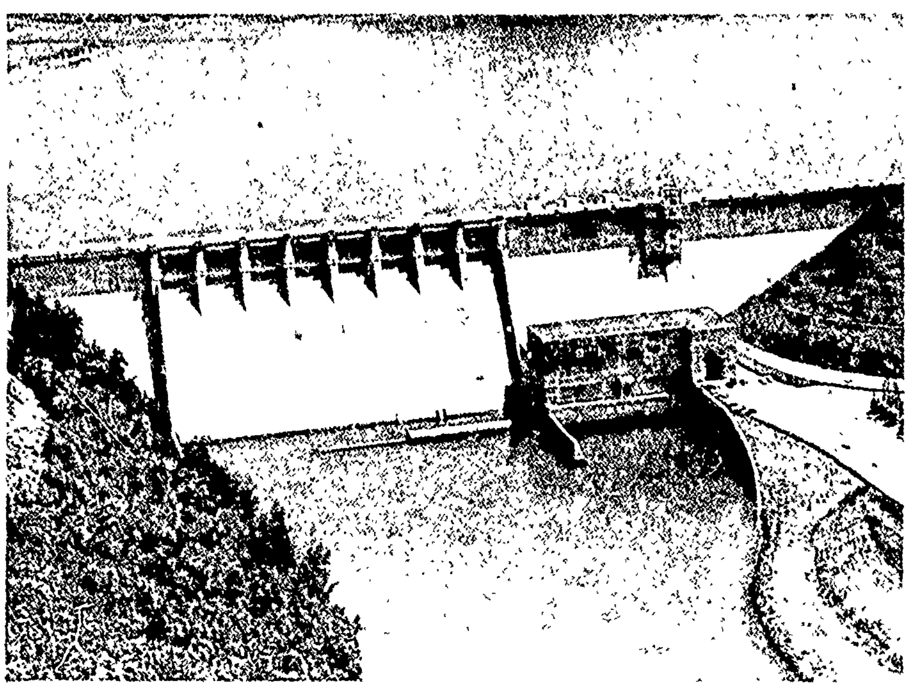


Figure 131. Aerial view of Center Hill Dam

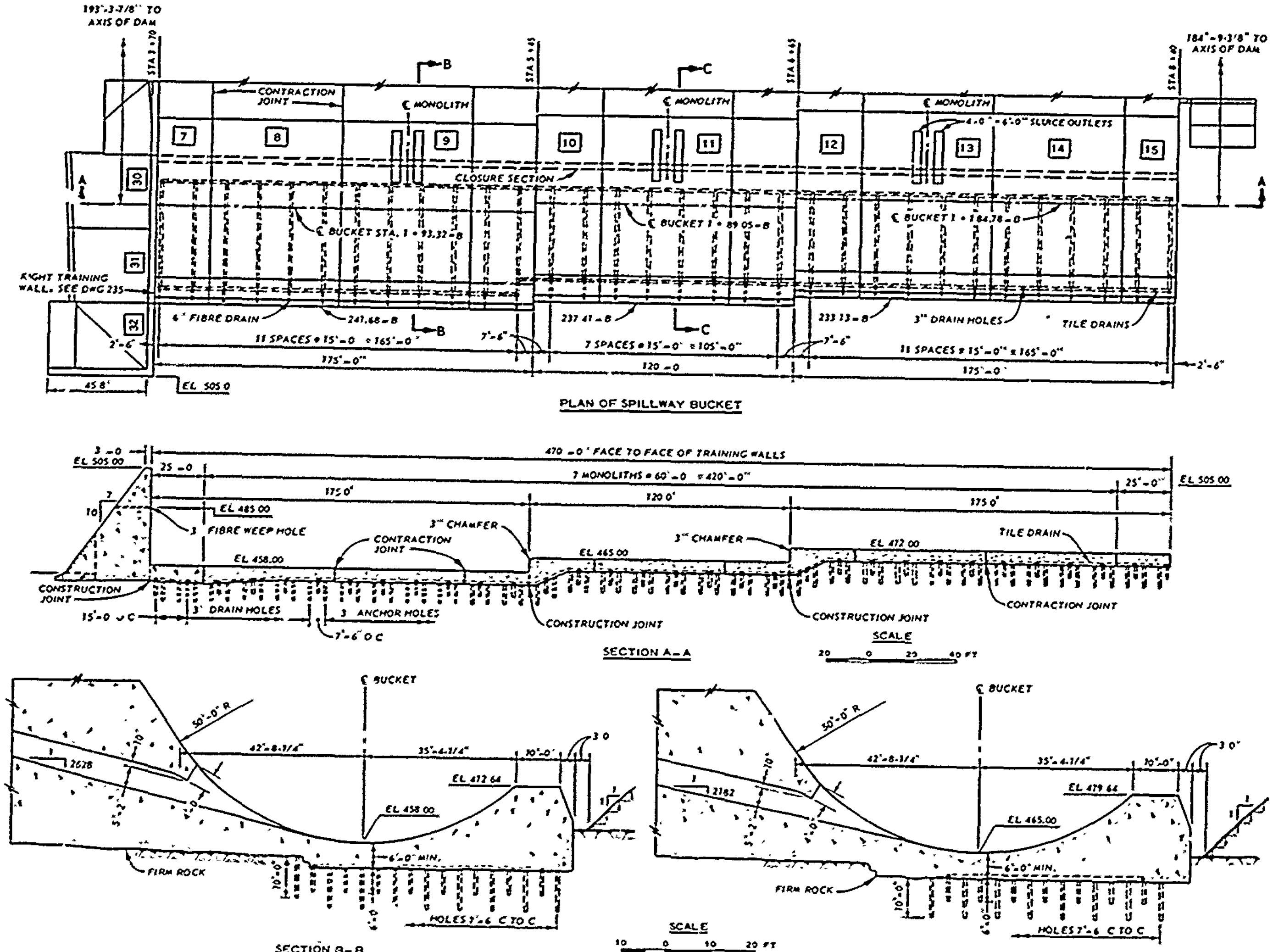


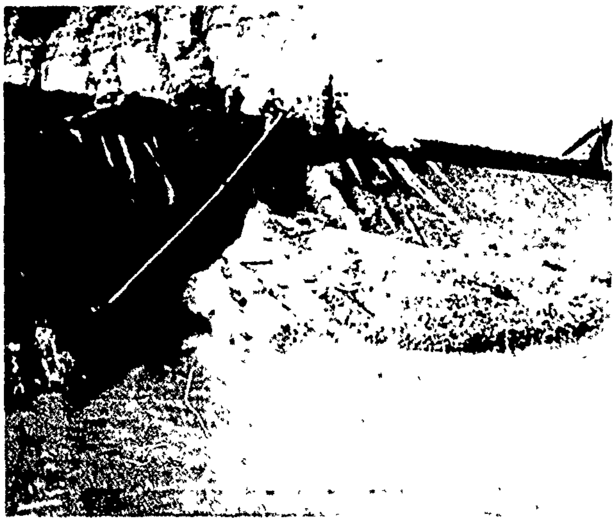
Figure 132. Spillway stilling basin, Center Hill Dam

sluices to maintain river flow during impoundment of the reservoir. The pool reached el 648.86 on 27 January 1950. A section through the basin was analyzed to determine that the resultant force fell within the middle third for spillway discharge. This analysis considered depth of jet, height of tailwater, centrifugal force of jet, and the full hydrostatic uplift. As an additional precaution against the intermediate forces of turbulence and the impact of debris, the basin is anchored to the foundation and tied together with reinforcing both parallel and normal to the direction of flow.

236. When the sluice gates were closed in the fall 1950 to permit the contractor to complete certain excavation in the tailrace and river bed, several dark areas were observed underneath the pooled water in the spillway stilling basin. An investigation disclosed that these areas were cavities in the concrete, and arrangements were made to drain the entire basin area for further investigation and necessary repair. The basin consists of three sections, each of different elevation, with the lip of the right section below tailwater elevation. In order to unwater the stilling basin, it was necessary to sandbag the entire lip of the low section to a height of approximately 4 ft (Figure 133) and use pumps to remove the enclosed water. The lips of the middle and left bucket sections were above tailwater elevation and drained to the right section where the pumps carried off the additional water.

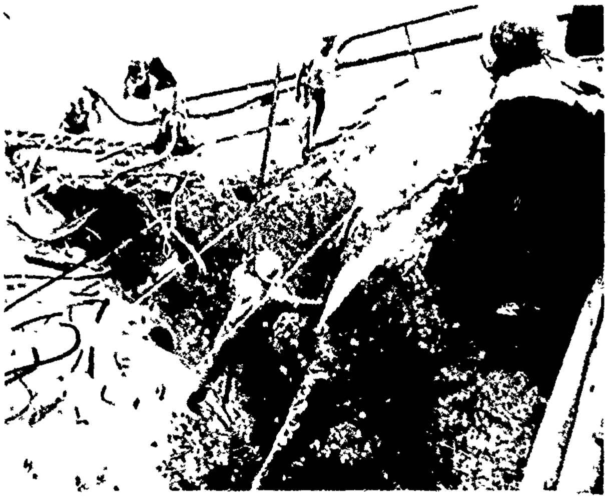
237. A thorough study of the existing concrete in the spillway basin after unwatering revealed the following facts:

- a. The surface of the entire bucket section, including the downstream face of the lip, showed signs of severe attrition. The surface of the concrete appeared as if it had been roughened by the use of bush hammers.
- b. The outlet edges of all the sluice openings were broken and cavitied to a considerable degree. At sluices 1 and 2 the reinforcing steel was slightly exposed.
- c. In the highest of the three bucket sections there were two small cavities approximately 2/3 cu yd each in volume. These were on the downstream side of the low point of the bucket and symmetrical about the two sluice outlets.
- d. The greatest damage was found in the right bucket section (the lowest in elevation of the three sections). It



a. General view of erosion damage looking downstream

b. Close-up view of erosion damage



c. General view of erosion damage from the bucket lip

Figure 133. Damage to right bucket, Center Hill Dam

consisted of a cavity of approximately 107 cu yd in volume, beginning near the bottom corner of sluice 1 and increasing in depth and width to a point nearly one third through the bucket lip (Figures 133 and 134). A cross section generally typical of the wear throughout the remainder of the right bucket section is also shown in Figure 134. The wear on the other two bucket sections was slightly less than this particular cross section. The approximate plan location of a pile of aggregate consisting of sand, river gravel, and stone in an estimated quantity of 64 cu yd is shown in Figure 134. The aggregate deposit was analyzed and it was determined that the stone and gravel were washed from the arm of the cofferdam tying into the high bucket section.

238. String lines were established on the theoretical grades (calculated from the dam drawings) for the 175-ft length of the right spillway bucket section. These strings were used to facilitate taking cross sections of this bucket section. It was concluded that this portion of the bucket was the only one so severely eroded as to necessitate taking of sections. From these field measurements, it was calculated that 125.3 cu yd of concrete had been eroded away.

239. Laboratory concrete records were examined for all pertinent facts relative to the concrete placement in this section of the dam. These records revealed that concrete in monoliths 7, 8, 9, and 10 was placed in September, October, and November 1942. The initial course in each monolith, which tied into the dam and extended part way up on the lip of the buckets, was 6-in. MSA concrete with a cement content of 423 lb/cu yd. The next course in each monolith, which formed the remainder of the lip, had the same cement content with 3-in. MSA. Water-cement ratios ranged from 0.55 to 0.76. Slump test results ranged from 2-1/2 in. to 6-1/2 in. Compressive strength test results ranged from 1711 to 3770 psi at 7 days and from 2586 to 4889 psi at 28 days.

240. Based on the available information, the following conclusions were made:

a. The erosion at the sluice outlets was the result of the combined action of cavitation and impact of large boulders being moved about by the swirling action of the water in the bucket. It was agreed that the design of the sluice outlets could have been improved by using rounded or splayed corners instead of square ones.

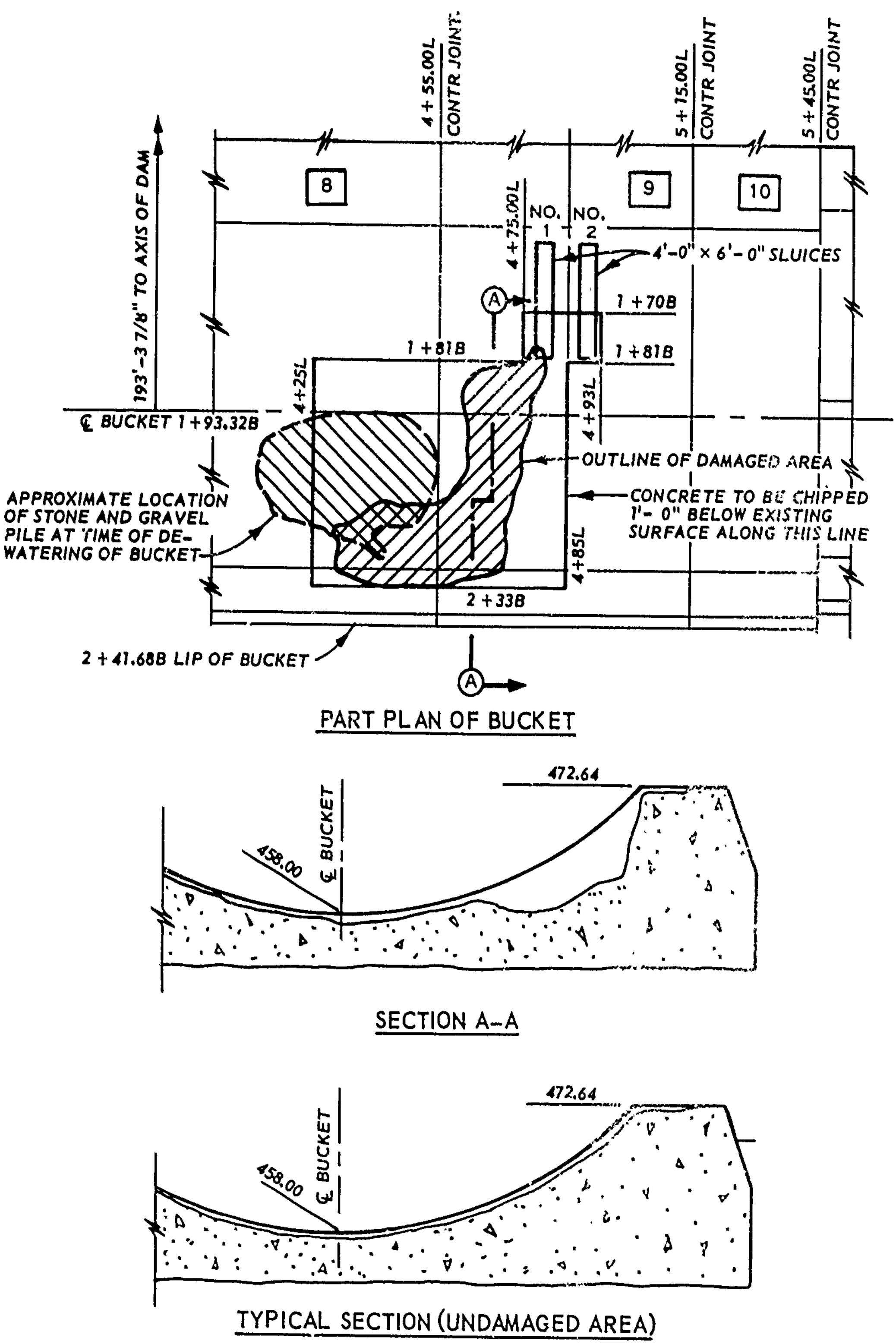


Figure 134. Extent and location of erosion damage, Center Hill Dam

b. The cavities in the stilling basin proper were the result of an abrasive action of stone and water. The proximity and quantity of river stone in the cofferdam created conditions that resulted in the extensive abrasive action.

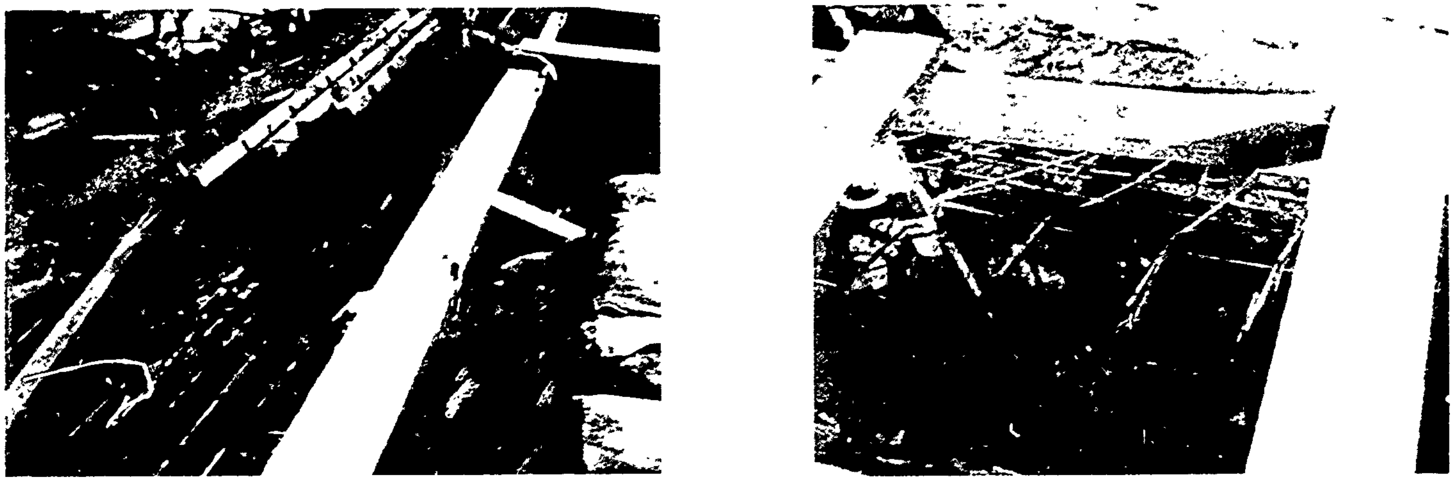
c. The wear and resulting roughness prevalent throughout the entire concrete surface in the basin were also the result of this abrasive action.

241. The sides of the two small cavities in the highest bucket section were dovetailed with jackhammers and filled with 1-1/2-in. MSA-class "A" concrete (564 lb of cement per cu yd). High spots created by uneven erosion were chipped down to a surface level with adjoining concrete. The small pockets, appreciably lower than adjoining concrete, were chipped down to a minimum 6-in. depth, dovetailed, and filled with a relatively dry grout.

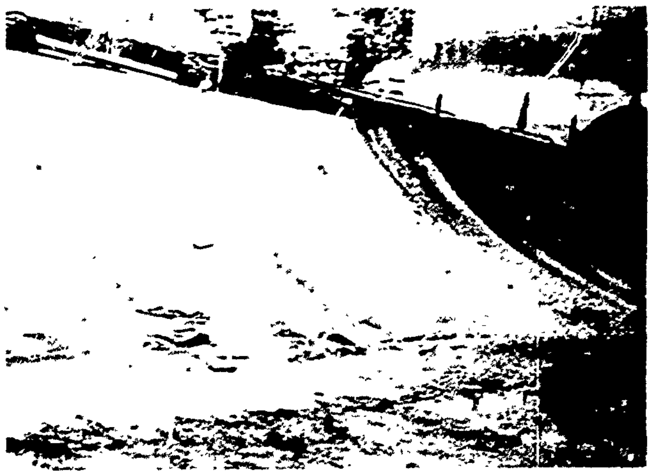
242. The concrete surrounding the large cavity in the right bucket section was chipped down 1 ft below the existing elevation in the pattern outlined in Figure 134. Vertical 1-in. square steel dowels on 4-ft centers were grouted 40 in. into the existing concrete. The ends of the dowels were split and driven onto wedges to provide anchorages before being grouted. A horizontal steel mat consisting of 3/4-in.-diameter bars on 18-in. centers both ways was fastened to the vertical dowels (Figure 135). This steel received a minimum 3-in. cover of new concrete. The concrete consisted of 1-1/2-in. MSA with 517 lb of cement per cu yd. A total of approximately 178 cu yd of concrete was required to fill the cavities and other damaged areas after chipping operations in preparing them for concrete.

243. The outlet edges of the sluice openings were chipped back from 18 in. at the bottom of the openings to 12 in. at the top sides of the openings. These edges were reformed on an 8-in. radius at the bottom of the opening and tapered to a 3/4-in. chamfer at the top sides of the openings. Reinforcing steel consisted of 3/4-in.-diameter bars on 18-in. centers used as a vertical mat. Concrete consisted of 1-1/2-in. MSA-class "A" concrete. Repairs were completed February 1951.

244. Following the flood of 1975, a routine diver inspection of the foundations and stilling basin in 1976 indicated some erosion in the



a. Views of cavity in right bucket prior to concrete placement



b. Completed repair

Figure 135. Right bucket repair, Center Hill Dam

lower bucket section. Another inspection was made in March 1977 to follow up on the problem and to plot the eroded areas more accurately. An eroded area approximately 6 ft in diameter, tapering from nothing to approximately 6-in. deep at the center with one reinforcing bar barely visible in the center, was located in monolith 8, 15 ft from the contraction joint between monoliths 7 and 8. The area was in the bottom of the basin. A slight amount of erosion was located approximately 20 ft from the construction joint above in monolith 8 and 10 ft down from the upstream edge of the apron. The area was approximately 10 ft long and 3 ft wide with a maximum depth of 3 in. Two reinforcing bars were visible in this area. Another area on top of the apron in monolith 8, 10 ft from the construction joint between monoliths 8 and 9 and 5 ft from the stream edge of the apron, was located. The area is 8 ft long and 1 ft wide at maximum point, and 8 to 10 in. deep. The location of this area could indicate a bad mix or placement of concrete. Additional follow-up inspections will be made to determine if conditions have changed.

Kinzua Dam

245. Kinzua Dam, which forms the Allegheny Reservoir, is located on the Allegheny River in Warren County, Pennsylvania, approximately 198 miles above the mouth of the river at Pittsburgh, Pennsylvania.

246. The stilling basin is a level hydraulic jump type (Figures 136 and 137) 178 ft 4 in. long and 204 ft 0 in. wide. It contains nine baffles, which are 8 ft high, 10 ft wide, and 18 ft 8 in. long and are located 55 ft 7 in. upstream of the end sill. The end sill is 11 ft high and 6 ft wide and extends across the entire stilling basin. Both the end sill and the baffles are keyed into the underlying rock by keys extending 6 ft below the stilling basin floor slab. The articulated floor slab has contraction joints at approximately 30-ft centers longitudinally and 21 ft 6 in. in the transverse direction. The minimum thickness of the slab is 5 ft. Anchorage is provided by No. 11 anchors spaced at 7-ft 2-in. centers in the transverse

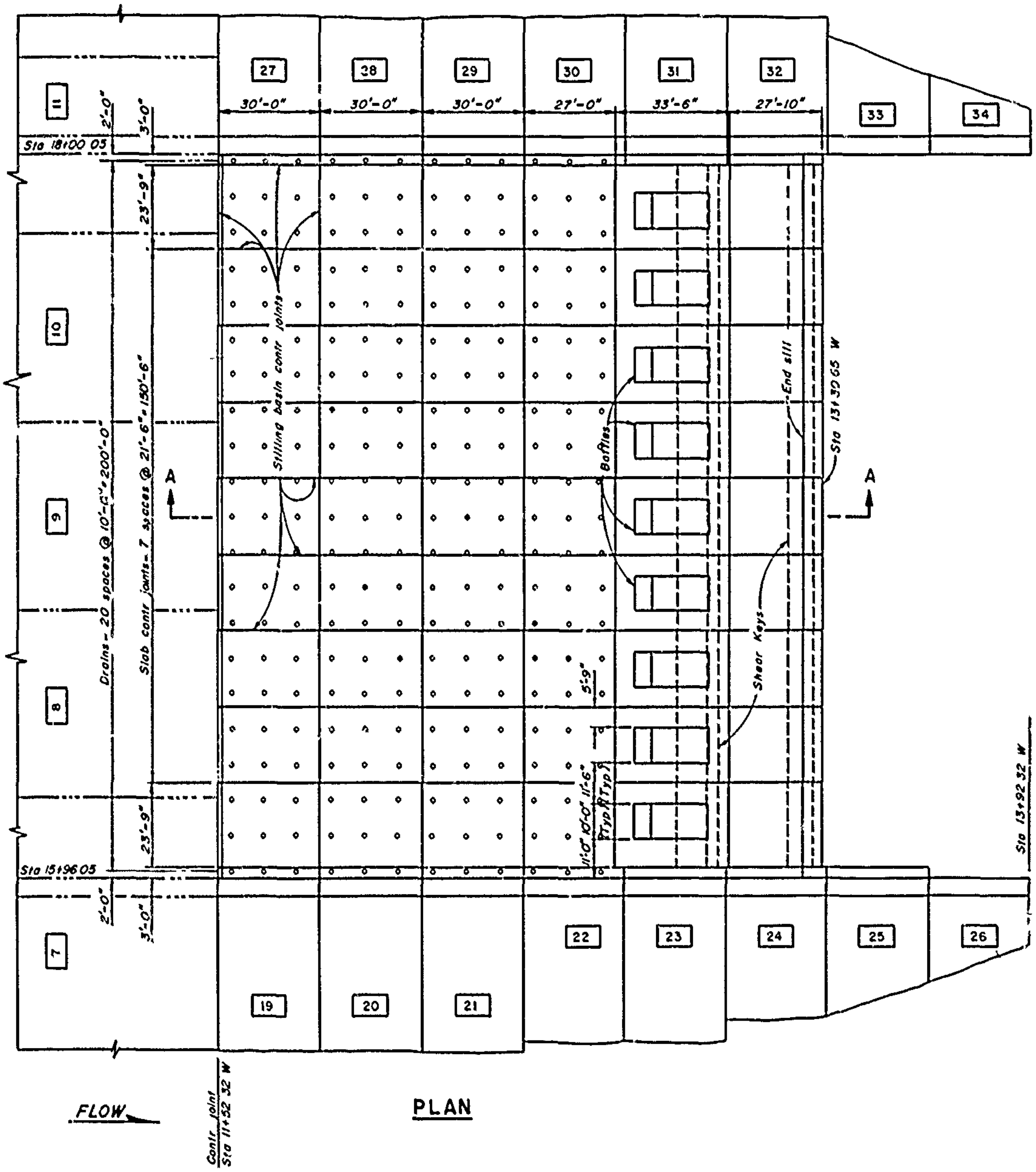
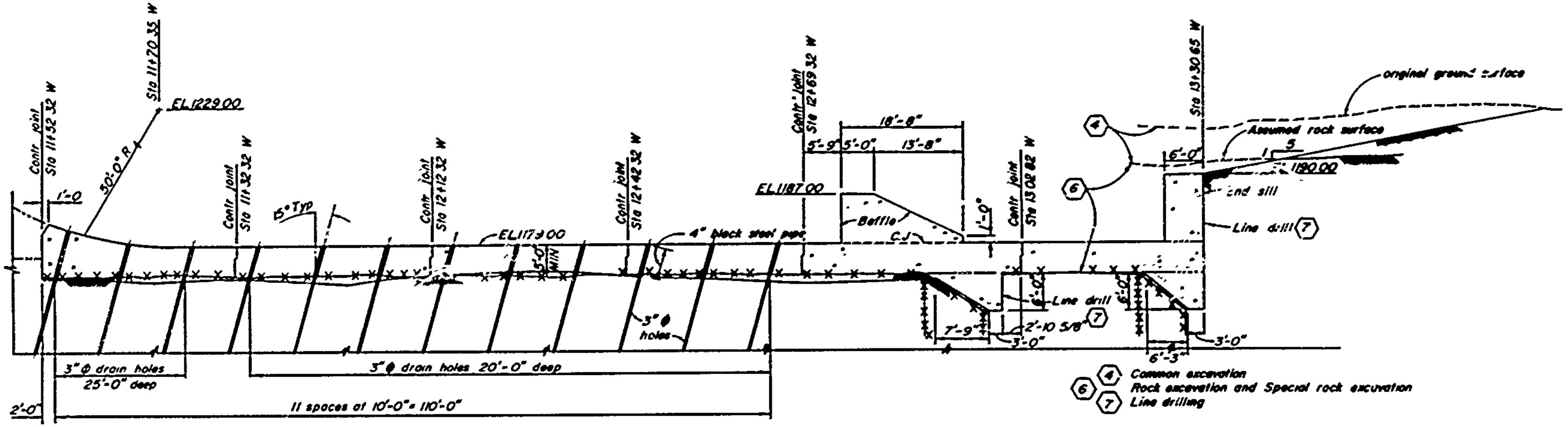


Figure 136. Stilling basin plan, Kinzua Dam



SECTION A-A

Figure 137. Stilling basin profile, Kinzua Dam

direction and 10-ft centers longitudinally. Except for the anchors, no reinforcement was provided in the upstream part of the stilling basin. Reinforcement, however, was provided in the baffles, end sill, and adjacent areas. Three-inch drainholes spaced on 10-ft centers in each direction are provided upstream of the baffle piers. These drains are 20 ft deep over the entire stilling basin except at the upstream end where three rows of drainholes are 25 ft deep.

247. The stilling basin was constructed of concrete with a 28-day compressive strength of 3000 psi. The top of the slab is at el 1179.0 and the top of the end sill at el 1190.0. The slab was designed for 50 percent relief of maximum uplift pressure due to the hydraulic jump under design flood spillway discharge. It is anchored to engage foundation rock to a depth sufficient to resist the unrelieved uplift force. The baffles were designed and anchored to be stable against the full impact force of the design flood spillway discharge jet. The end sill was designed for the force of the jet passing between the baffles. The stilling basin was first used in March 1963 when the stream was diverted through lowered monoliths 8 and 10 to begin second stage construction. Flow through the sluices began October 1964 when final closures were made, and normal operation of the dam began on 15 December 1965 with the final acceptance of the project by the CE.

248. The spillway was designed for a maximum discharge of 140,000 cfs and a maximum velocity of 108 fps. Spillway flow has occurred but was small, due to control by the tainter gates. The maximum discharge of record came during June 1972 when 24,800 cfs was discharged through the sluices. The maximum velocity at the sluice exit was 88 fps with the upper pool at el 1362.12.

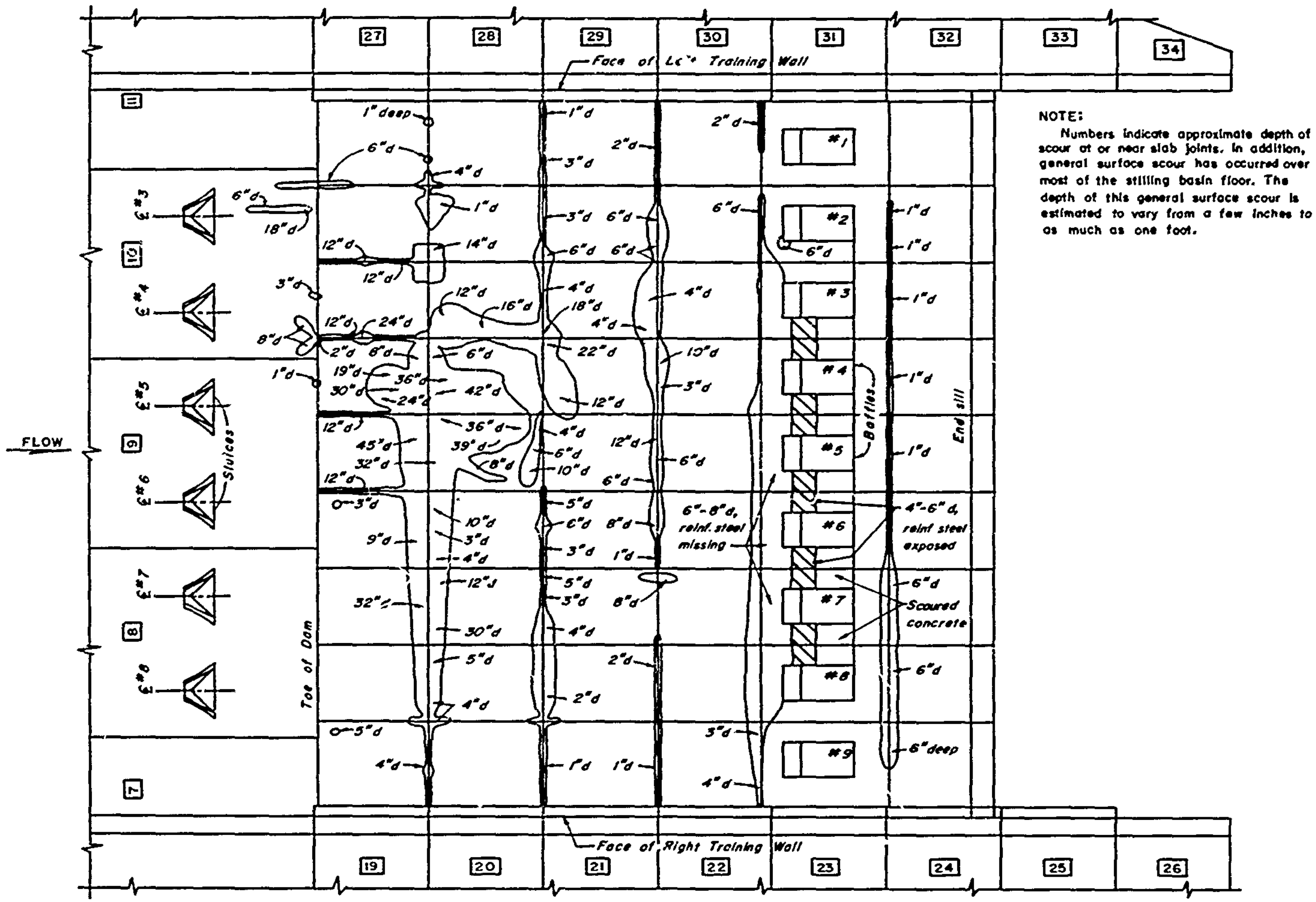
249. Because of the proximity of a pumped storage power plant on the left abutment and problems from spray, especially during the winter months, the right side sluices were used most of the time. Use of these sluices caused a circulatory current that carried debris into the stilling basin. The end sill being below streambed level contributed to the deposition of debris in the basin.

250. As early as September 1969, less than 4 years after the stilling basin had been placed into normal operation, damage to the stilling basin floor had been reported by scuba divers. Later inspection by a hardhat diver and an electronic depth indicator verified the presence of holes and erosion as well as piles of rock, gravel, and other debris. The rock and gravel fragments were found to range from sand size to 8 in. in diameter. Most of the damage was at the contraction joints and the corners of the baffles. Scour holes were up to 42 in. deep.

251. In November 1972 an attempt to remove the debris by the use of a trash pump was not very successful because of the large size of some of the stone. A long-boom truck crane with a clamshell was brought in and was used to remove about 50 cu yd of gravel, rock, and metal. Placing the truck crane on the right bank training wall required the construction of a 200-ft-long road from the toe of the dam embankment.

252. A contract was awarded, and repairs were started on 16 July 1973. The plans called for accomplishing the work in two stages using cellular cofferdams that would enclose about 60 percent of the stilling basin for each stage, permitting stream flow in the unobstructed part of the stilling basin. The cofferdams were pumped out at a rate not exceeding 9 in./hr, and fish entrapped within the cofferdam were removed.

253. Damage to the stilling basin floor was most noticeable in the area downstream of sluices 4 and 5 (Figure 138). The two rows of slabs adjacent to the left training wall had generally minor surface erosion with a maximum depth of approximately 6 in. A large, donut-shaped hole (Figure 139) extended over four slabs downstream of sluices 4 and 5 and had a maximum depth of about 42 in. Erosion was generally deeper at the joints. Damage to the baffles was generally confined to the baffles in the middle of the basin (3-7) with baffles 1, 2, 7, and 8 relatively undamaged. Reinforcing was exposed over approximately two thirds of the upstream face and partially around the corners of the baffles in the worst cases (Figure 140). All faces of the baffles had eroded to varying degrees. The toe of the spillway section was eroded to a maximum depth of 4 in. from the floor of the stilling basin up to approximately el 1192.5.



PLAN

Figure 138. Erosion of stilling basin concrete, Kinzua Dam

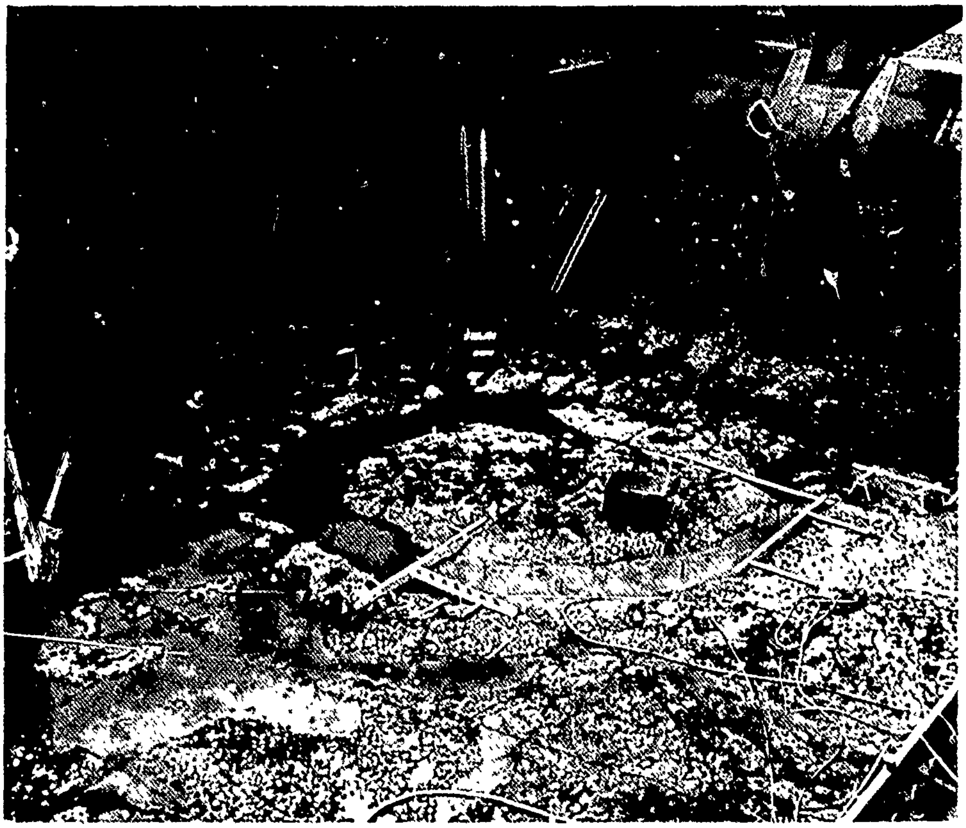


Figure 139. Erosion of stilling basin slab, Kinzua Dam

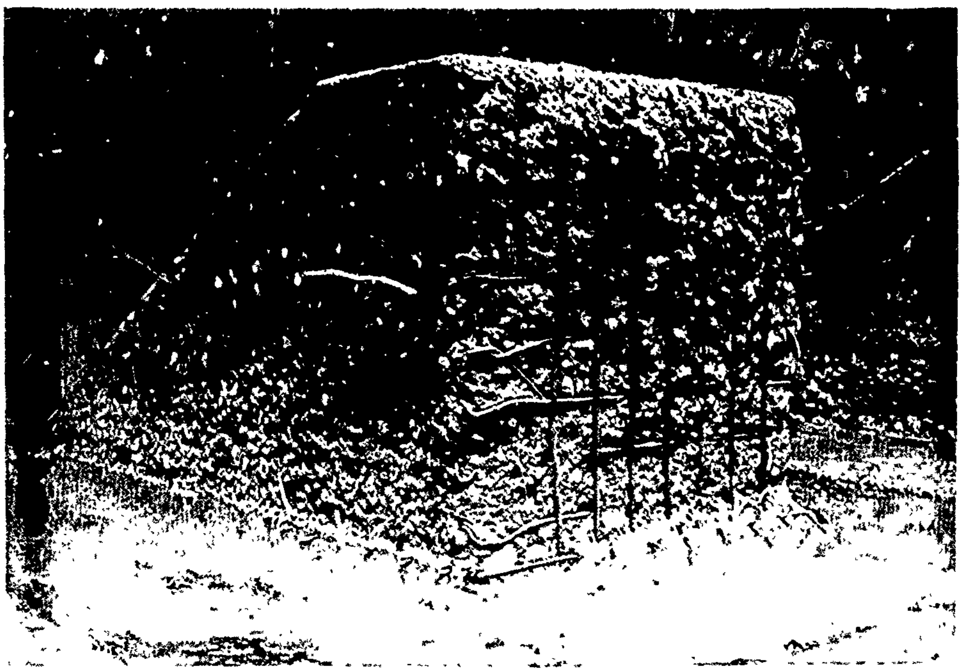


Figure 140. Damaged baffle pier, Kinzua Dam

254. In preparation for the repair, the floor of the stilling basin was cleaned by wet sandblasting. Loose, weak, or deteriorated concrete was removed by chipping (Figure 141). All loose materials and impurities were then removed by washing or wet sandblasting, and No. 8 dowels were installed on approximately 3-ft centers (Figures 142 through 144). The deeper holes were partly filled with dense concrete having a 28-day compressive strength of 3000 psi.

255. Prior to placement of the fibrous concrete, a trial mixing was made at the batch plant to approve the suitability of the operation. Two 1-yd batches of mixtures proportioned by the Ohio River Division's laboratory to have fine to coarse aggregate ratios of 3 to 1 and 3 to 2, respectively. Each batch contained 200 lb of thin, flat steel fibers 1 in. in length and was proportioned for 28-day compressive and flexural strengths of 6000 and 1100 psi, respectively. Based on this trial, the 3 to 2 ratio mix was recommended for use in the repair work. Also, it was recommended that 7-yd batches be mixed in a 10-yd capacity ready-mix truck. The batching sequence was sand, crushed limestone coarse aggregate, steel fibers, cement, and water.

256. The above described procedure was used at the start of placing

Figure 141. Concrete removal and anchor installation, Kinzua Dam

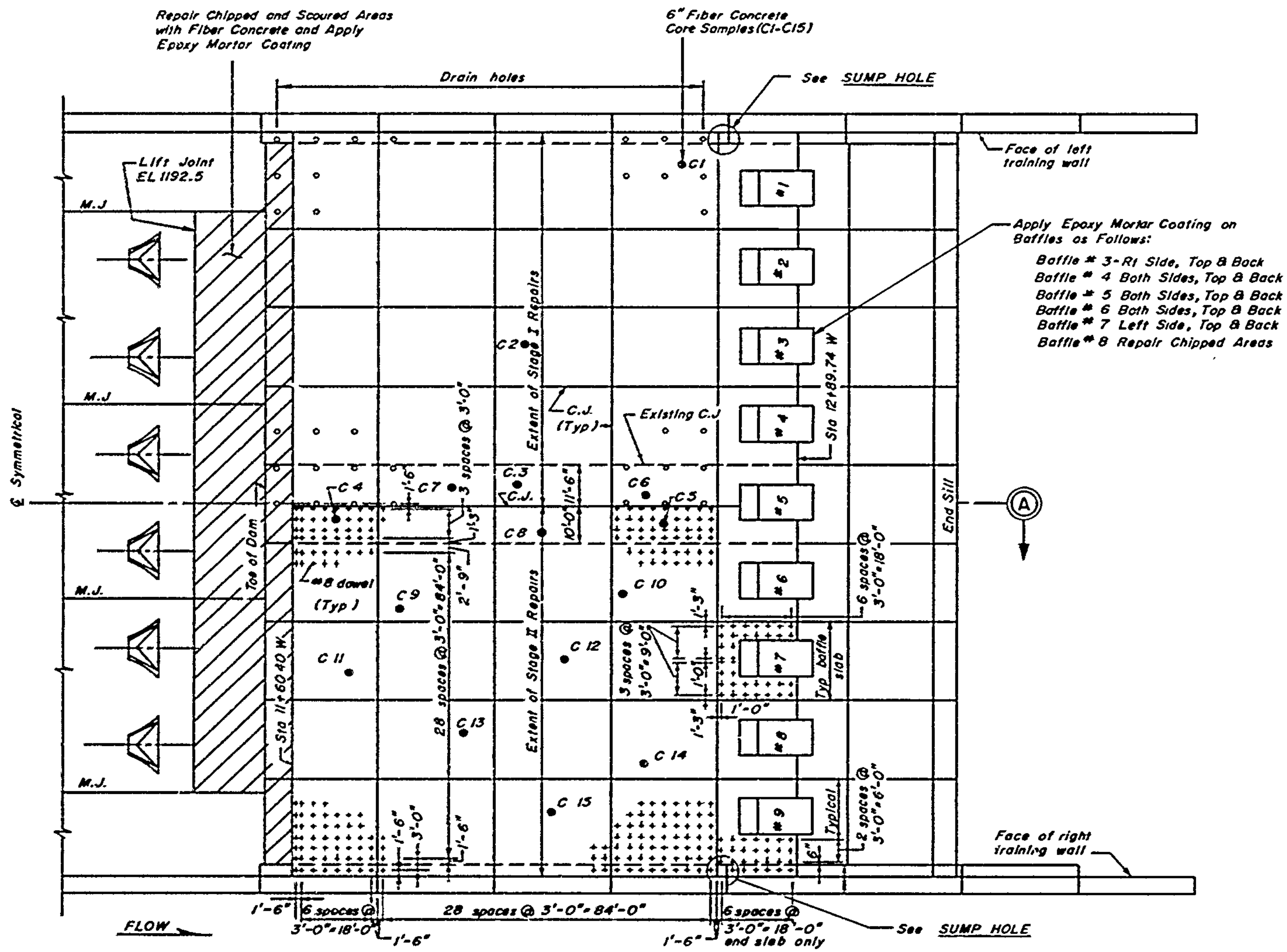
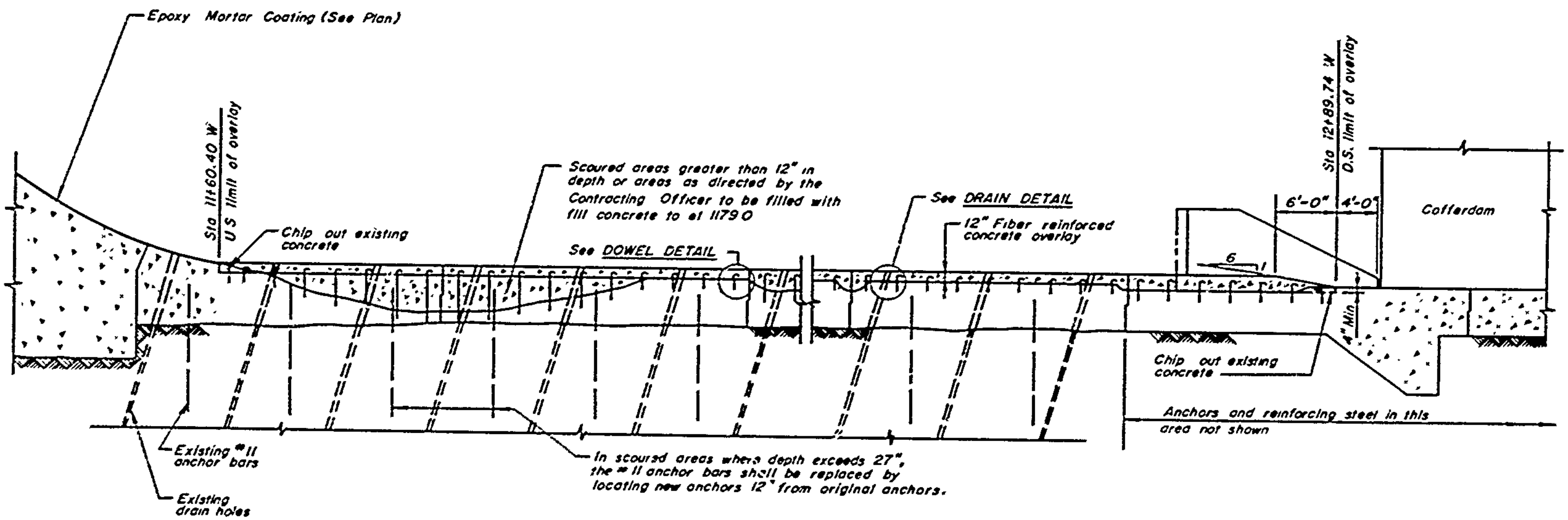


Figure 142. Plan, stilling basin repairs, Kinzua Dam



SECTION A–A

Figure 143. Section, stilling basin repairs, Kinzua Dam

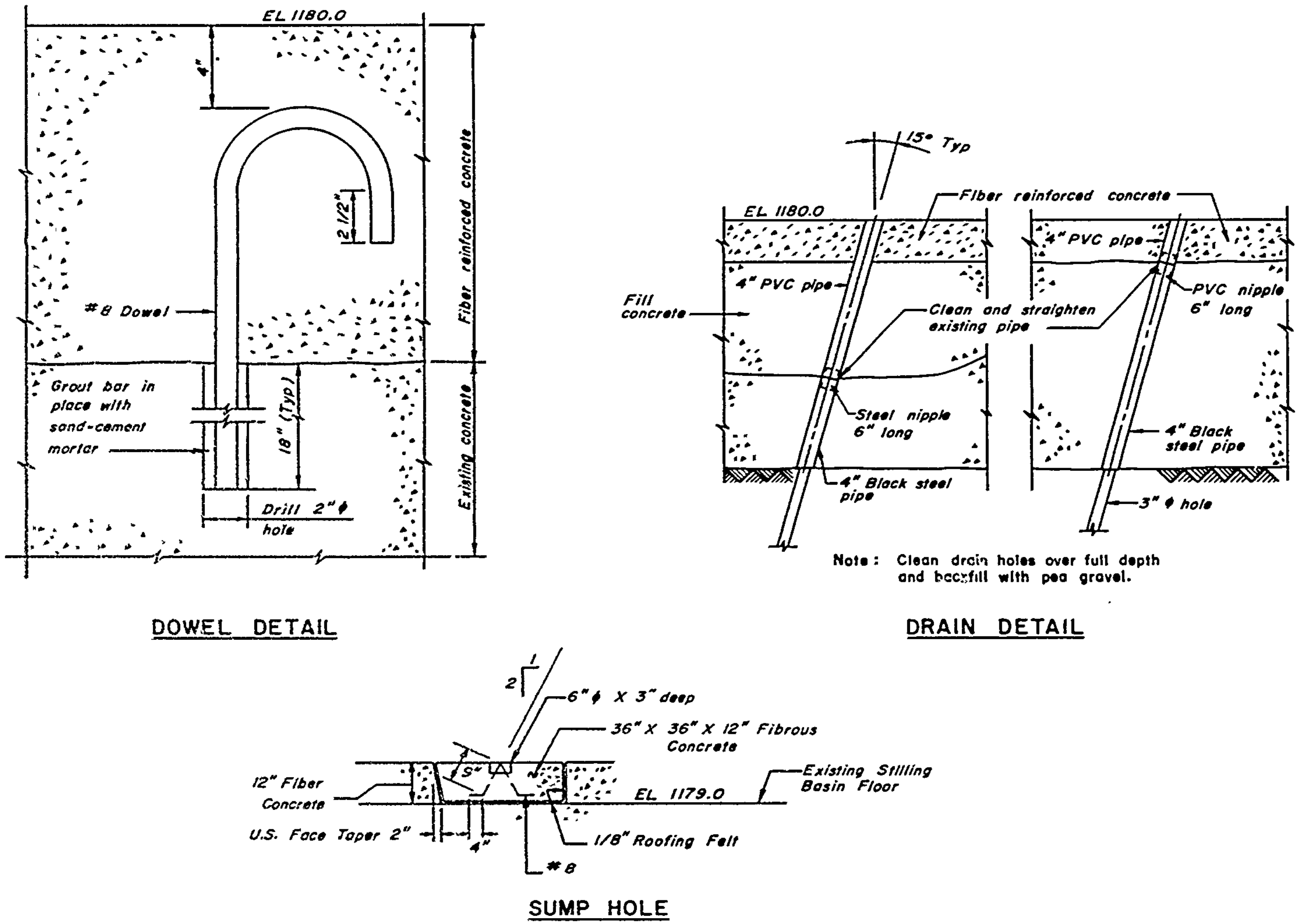


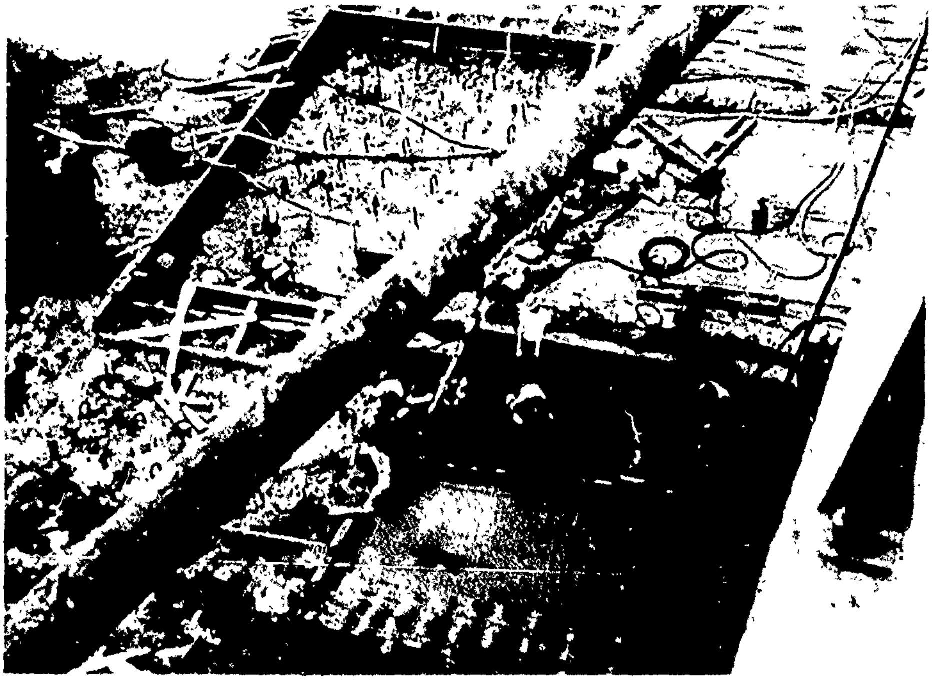
Figure 144. Stilling basin repair details, Kinzua Dam

fibrous concrete with water added at the job site. However, these batches contained excessive "balls" and had to be rejected. The procedure was then adjusted to add all the ingredients at the batch plant. The fine and coarse aggregates were placed in the mixer, and approximately 70 percent of the mixing water was added. The steel fibers were added next, using a high-speed conveyor at a nearly continuous rate. All the cement was then charged into the mixer, and the remaining water was added to the batch prior to transporting it to the job site. Balling of the batches was thus eliminated or reduced to a point where the balls could be removed. Six cubic yards were batched in a 10-cu-yd capacity transit mixer.

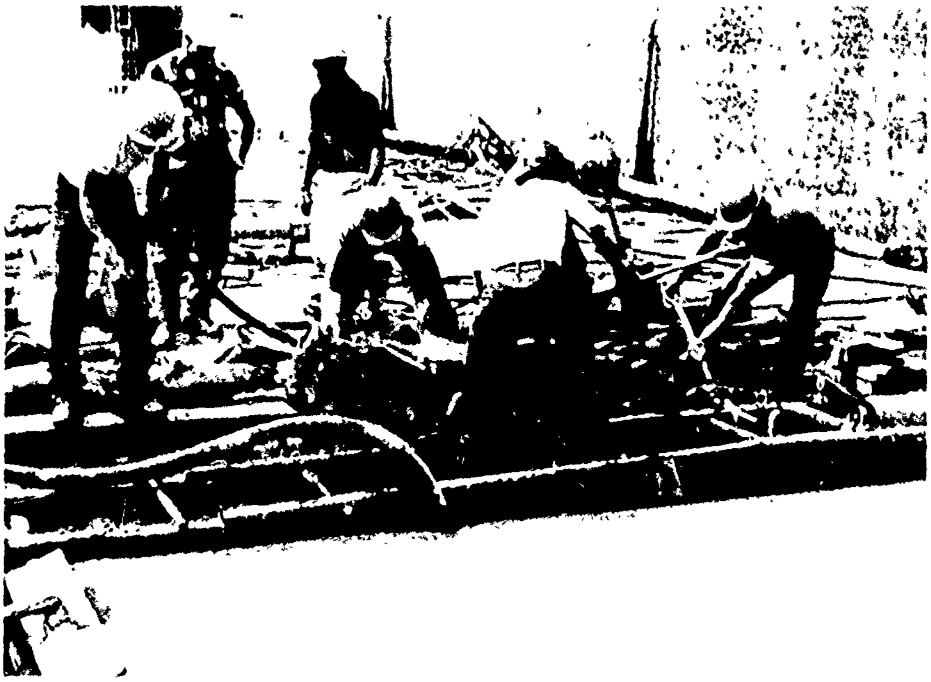
257. A high modulus epoxy bonding compound was placed on the stilling basin floor immediately prior to placement of the fibrous concrete overlay. The overlay was placed in slab sections (Figure 145) conforming to the original slabs. Slab placement was in alternate sections starting at the upstream end of the row adjacent to the left training wall (Figure 146) and proceeding in adjoining rows to the center of the stilling basin. Relief drains were extended through the overlay (Figure 144). Approximately 1400 cu yd of fiber concrete was required for the overlay, which was placed to el 1180.0, 1 ft higher than the original stilling basin floor from the toe of the dam to a point just short of the downstream end of the baffle piers.

258. The baffles were prepared for repair (Figures 147 and 148) in the same manner as the stilling basin slab. Dowels were installed and an epoxy bonding compound applied. The front of the baffles was resurfaced with fibrous concrete and reinforced with No. 8 steel bars. In addition, corner pieces of corrosion-resistant steel plate, 5/8 in. thick, were installed at the upstream corners of the baffles. Both sides, the top, and the back of damaged baffles were coated with an epoxy mortar. This same mortar, a 1 to 1 ratio of epoxy and silica sand, was used to coat the damaged spillway surface to a 1/2-in. average thickness.

259. The only major difficulty encountered during the repair work was that of installing a cofferdam of sheet pile cells while a stream flow was maintained. A minor problem resulted during repairs when the



a. Placement of fiber concrete



b. Close-up view of fiber concrete placement

Figure 145. Fiber concrete placement, Kinzua Dam

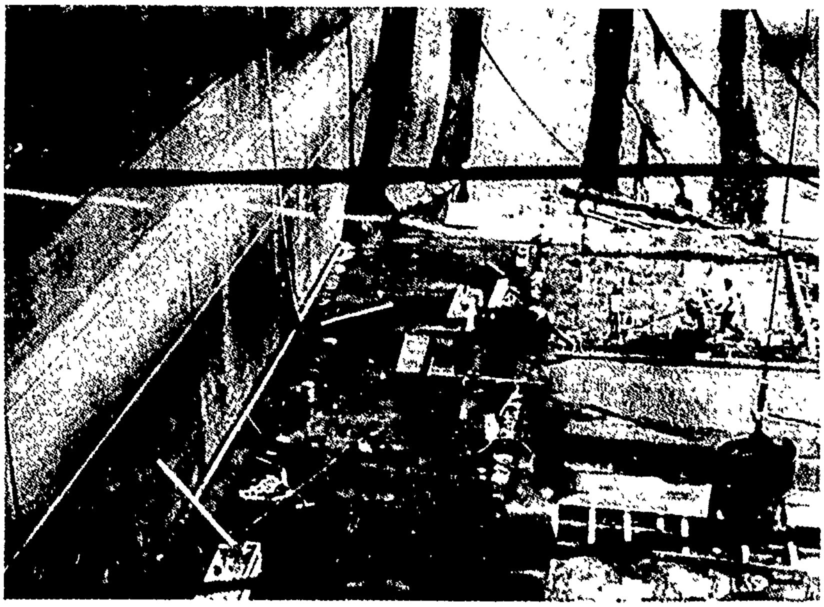


Figure 146. General view of repair operations,
Kinzua Dam

1-in.-long steel fibers used in the fibrous concrete had a tendency to ball. When some of the water was added to the mix before the introduction of the steel fibers, however, this balling tendency was greatly reduced.

260. The preflooding inspection of Stage I was made on 25 October 1973 and for Stage II on 29 August 1974. All work was completed by the contractor on 25 October 1974 at a total cost of $1,714,987. Approximately $734,000 of this total was for construction of cofferdams required to dewater the structure (Table 2).

261. The initial diver inspection of the repair in November 1974 indicated minor concrete deterioration on some of the baffles and in the surrounding floor area. Two large areas of epoxy repairs at the base of the spillway were missing. An estimated 45 cu yd of debris was removed from the basin. At that time the District conducted an experiment to determine whether currents and eddies carried material back from the channel downstream of the stilling basin and over the end sill into the stilling basin. Approximately 5500 bricks of three different types were dropped from a boat into the river and were distributed downstream of the basin (Figure 149). Six days after the placement of the bricks, the

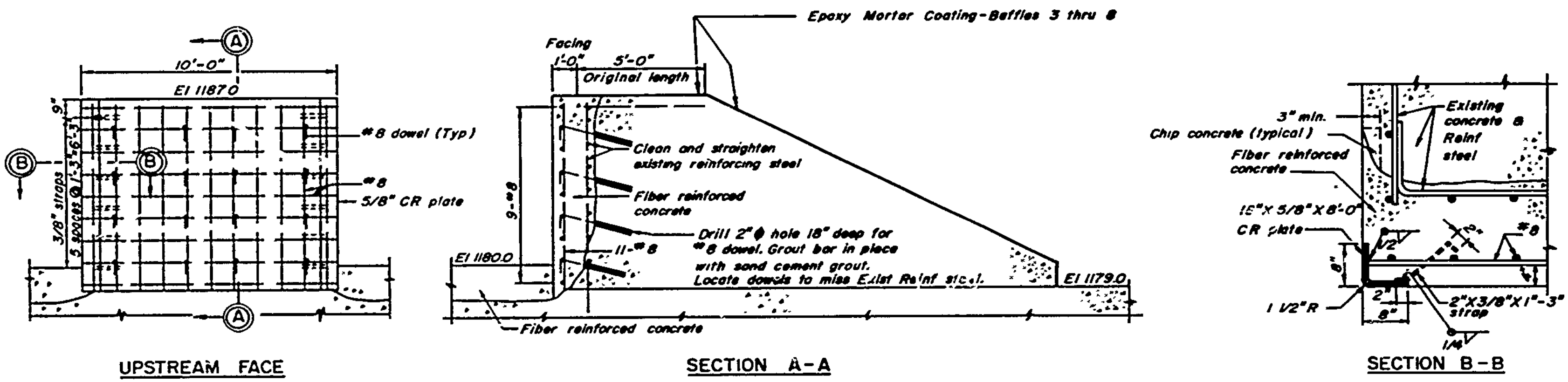


Figure 147. Baffle repair details, Kinzua Dam

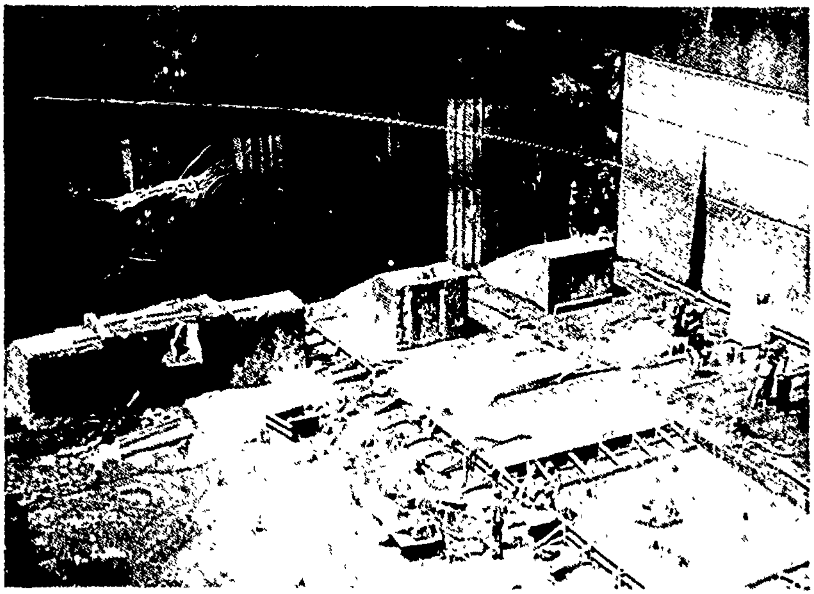


Figure 148. Baffle and basin slab repair,
Kinzua Dam

stilling basin was inspected by two Pittsburgh District scuba divers who found innumerable smooth pieces of all three types of brick in various sizes.

262. In April 1975, additional concrete erosion was noted on five baffles and in the floor area between and downstream of the baffles. Trenches around some baffles had approximate maximum depths of 4-12 in. The stilling basin floor upstream of the baffles contained several areas of erosion in the recently completed fiber-reinforced concrete overlay. Numerous No. 8 dowels were exposed in vertical heights from about 1 to 13 in., thus indicating depths of concrete erosion in these areas to be approximately 5-17 in. Additional areas of epoxy repairs at the bottom of the spillway were missing. Approximately 45 cu yd of debris was removed from the stilling basin. Included in this debris were numerous pieces of all three types of brick previously deposited in the river. The original bricks had been worn or broken into small pieces of various sizes and had been rounded smooth. This proved beyond any question that scouring material was being brought into the stilling basin from areas downstream of the end sill.

263. Downstream of the end sill and a pile of debris at the right training wall, the divers located a trench in the riverbed rock adjacent

Table 2

Kinzua Stilling Basin - Cost of Repairs

Description	Quantity	Unit Price	Amount
Mobilization & Demobilization			
Stage I	Job	Sum	$ 55,000
Stage II	Job	Sum	35,000
Cofferdams			
Stage I	Job	Sum	221,927
Stage II	Job	Sum	206,361
Cofferdams Fill			
Stage I	14,300	$ 10.00	143,000
Stage II	10,790	8.00	86,320
Cofferdam Seal - Concrete Filled Sand Bags			
First 1250 bags	1,250	25.00	31,250
Over 1250 bags	1,803	25.00	45,075
Cleaning Stilling Basin			
Stage I	Job	Sum	20,000
Stage II	Job	Sum	25,000
Remove & Replace Training Wall Railing	Job	Sum	30,000
Rock Fill	85 cy	50.00	4,250
Free Draining Fill (Access Road)	406 cy	8.00	32,528
Not Used			
Wet Sand Blast	2980 sy	5.00	14,900
Chip Concrete	2316 sf	20.00	46,320
Drill & Grout 2"-diam Holes			
a. Stilling Basin Floor	4300 lf	12.00	51,600
b. Baffle Faces	216 lf	15.00	3,240
Drill & Grout 2-1/2"-diam Holes for Anchors	200 lf	15.00	3,000
Epoxy Bonding Compound	480 gal	100.00	48,000
Fill Concrete	101 cy	100.00	10,100
Fibrous Concrete			
a. Floor	1150 cy	210.00	241,500
b. Baffles	39 cy	350.00	13,650
c. Steel Wire Fibers	2930 cwt	25.00	73,250
d. Fibrous Concrete Floor	276 cy	190.00	52,440

(Continued)

Table 2 (Concluded)

Description	Quantity	Unit Price	Amount
Cement	11,407 cwt	$ 2.00	$ 22,814
Dowels	29,330 lb	0.50	14,665
Anchors & Reinforcing Steel	5,755 lb	1.00	5,755
Miscellaneous	5,240 lb	2.00	10,480
Not used			
Pipe Culvert 15"-diam CMP	190 lf	20.00	3,800
Drain Pipe Extensions			
a. 12 inches	20	50.00	1,000
b. Over 12 in.	232	60.00	13,920
Environmental Protection	Job	Sum	1,000
Special Connections	Job	Sum	40,000
Drilling 6" Concrete Cores	8 lf	75.00	600
Epoxy Mortar Coating of Additional Stilling Basin Areas - Stage I	Job	Sum	42,500
Furnish & Install Additional Wales & Tie Rods	Job	Sum	5,000
Furnish Special Sand for Fibrous Concrete			
Stage I	727 ton	4.50	3,271
Stage II	738 ton	4.50	3,321
Drilling 6" Concrete Cores	16 lf	36.50	584
Epoxy Mortar Coating of Additional Stilling Basin Areas - Stage II	Job	Sum	39,950
Stone Toe Excavation	70 cy	5.80	406
12" Stone Slope Protection	370 cy	33.00	12,210
			$1,714,987

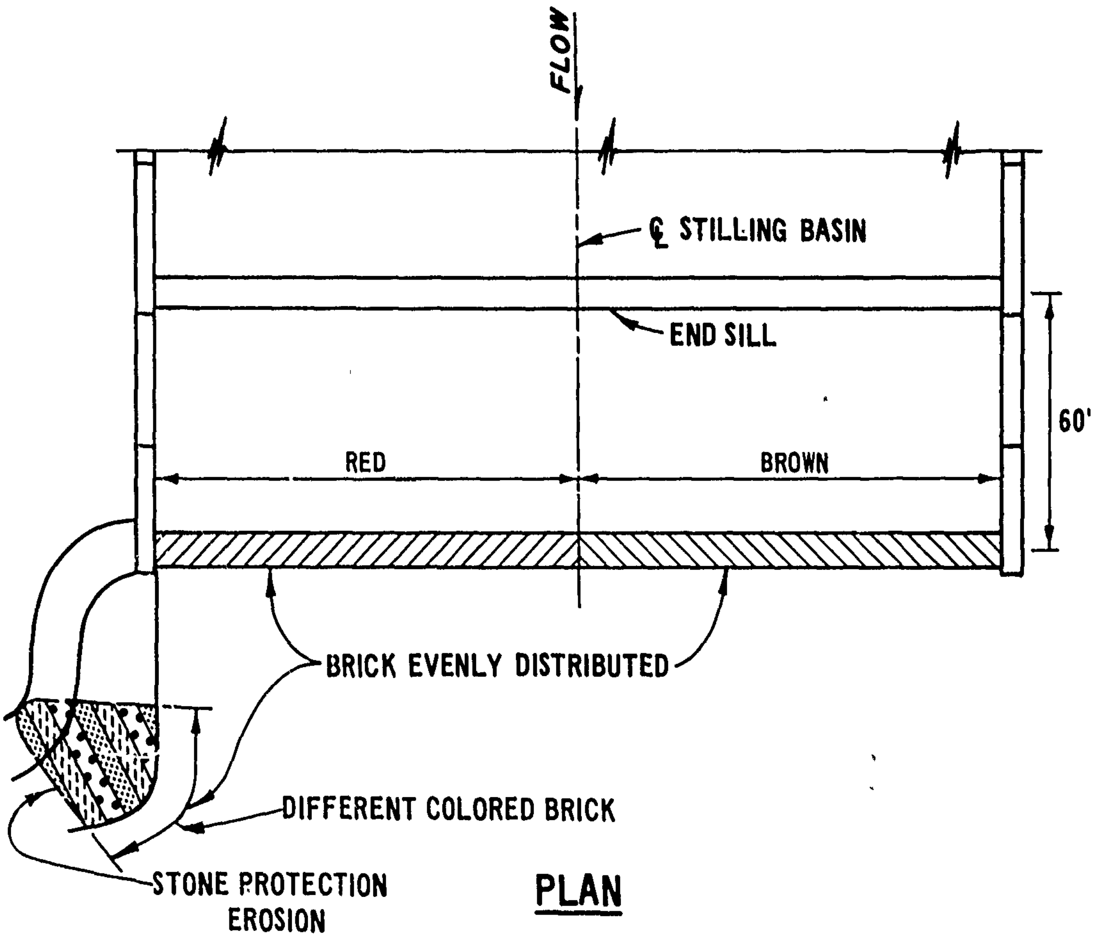


Figure 149. Experiment to determine source of stilling basin debris, Kinzua Dam

to and parallel with the right training wall during the May 1975 inspection. This trench, which was about 10 ft wide and 2-3 ft deep, extended downstream beyond the end monolith 26 in a meandering direction. About five or six new dished holes in the concrete were located in the second and third rows of floor slabs downstream between sluices 7 and 8. The largest of these new holes was about 2 ft in diameter and had a maximum depth of about 14 in. Approximately 60 cu yd of debris was removed from the stilling basin. The Pittsburgh District hydraulics section recommended that all sluices be operated symmetrically, despite any objections from the pumped-storage power plant representatives. This policy was placed in operation.

264. The stilling basin was next inspected in July 1975 and the general condition was reported to be about the same as in the previous inspection. An estimated 3 cu yd of debris was removed from the basin.

265. Additional concrete erosion in the stilling basin was noted during the September 1975 inspection, but the rate of deterioration had decreased considerably. There was increased erosion of the original concrete flooring downstream of the baffles, exposing approximately 90 rebars. It was estimated that the two piles of debris found in the stilling basin contained about 20 cu yd, which was approximately the estimated amount of debris left inside the stilling basin after the cleanout in May 1975, as the crane could not reach the center area between the training walls. This material was removed in November 1975, and the basin was reported to be clear of debris. As a result of sluice experiments, a table outlining the sluice operating procedure for a range of outflow was prepared.

266. No piles of debris were found on the stilling basin floor during the January 1976 inspection. The only debris was located in some existing scour holes. The scour holes located upstream of the baffles in the fiber concrete went down to or continued into the original concrete below. Many of the No. 8 dowels were exposed from about 1 in. up to a maximum of about 18 in. Soundings were taken along the deepest parts of the 10-ft-wide trench in the bedrock adjacent to the right training wall. The approximate elevation at the deepest part was 1177.7. There was no undercutting of the two downstream moholiths 25 and 26, which had a final grade between el 1173.3 to 1171.3. Additional tests of various symmetrical sluice operations with different gate openings were conducted. The power plant added a series of heat lamps to the electrical equipment installed on top of the roof to help prevent the insulators from becoming covered with ice.

267. No piles of debris were found on the stilling basin floor in either the April or September 1976 inspection. The only debris was located inside existing scour holes in the fiber concrete overlay, mainly

in the first three floor slabs downstream from and between sluices 3 and 8. Similar conditions were noted at the last inspection, but the debris was not removed from the basin. The debris consisted of smooth stones, ranging from about 1-in. diameter to about fist-sized pieces of epoxy that had broken away from repairs to the lower portion of the downstream face of the spillway, and a few riprap stones, the largest measuring 8 by 6 by 18 in. long. These riprap stones had apparently been thrown into the stilling basin by visitors at the project. In the first row of floor slabs downstream from and in front of sluices 6, 7, and 8, the concrete floor surface had deteriorated to a greater extent in September 1976 and had become more wavy than was reported previously. With this exception, there appeared to be no additional concrete deterioration of the stilling basin floor, the baffles, and the end sill since the inspection of September 1975. The conditions downstream of the end sill appeared to be about the same as those found during the inspection of January 1976.

268. A diver inspection in June 1977 indicated the fiber concrete in the basin slab had eroded to a maximum depth of 36 in. During a similar inspection in October 1978, divers removed approximately five wheelbarrow loads of miscellaneous debris, most of which appeared to have been tossed into the basin by visitors. Although a number of the No. 8 dowels were still exposed, this was the first inspection that failed to locate any sheared dowels within the basin since the repair. Major erosion of the fiber concrete was concentrated in the first two rows of slabs immediately downstream of the spillway. The deepest erosion (42 in.) occurred in the second floor slab downstream of sluice 7. The next deepest erosion (29 in.) was located in the second floor slab downstream from and to the right of sluice 5. The next inspection is scheduled for the Spring 1979.

Tionesta Dam

269. Tionesta Dam is located on Tionesta Creek 1.25 miles upstream of the junction of the creek with the Allegheny River at Tionesta,

Pennsylvania, about 78 miles northeast of Pittsburgh, Pennsylvania. The main features of the project are an earth-fill dam, saddle spillway, conduit, and stilling basin.

270. The stilling basin (Figures 150 and 151) is a diverging type with a parabolic drop and dentations. The basin is 185 ft long and the width increases from 17.5 ft at the tunnel outlet to 81.73 ft at the extreme downstream end. That section of the basin from the tunnel outlet to a point 85 ft downstream is paved with concrete approximately 6 ft thick. The final 100 ft of the basin was unpaved but the sides consist of heavy gravity walls deeply embedded into the rock. The paved

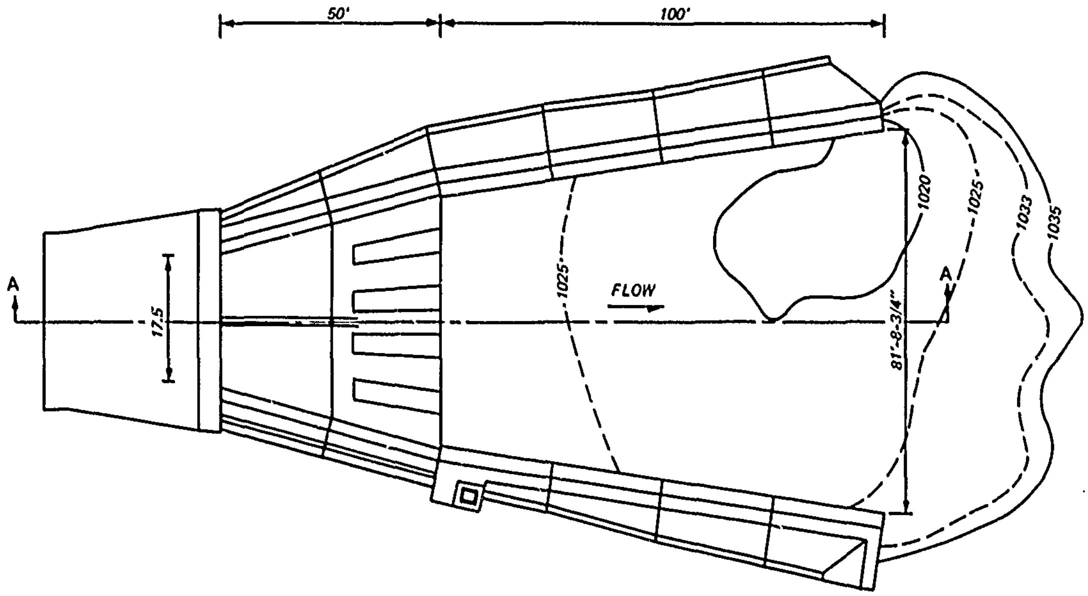


Figure 150. Stilling basin plan, Tionesta Dam

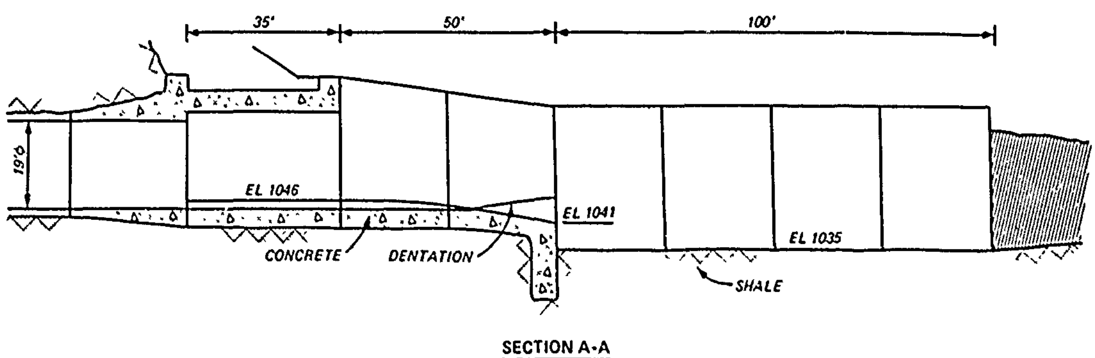


Figure 151. Stilling basin profile, Tionesta Dam

section has a concrete cutoff at its downstream end with dentations about 20 ft in length projecting 5 ft above the top of the concrete.

271. The concrete section ranges from el 1046.0 at the tunnel outlet to el 1041.0 at the end of the parabolic curve. Its width varies from 17.5 ft at the tunnel to 52.54 ft at the end of the cutoff and dentations. The remaining 100 ft of the stilling basin floor at el 1035.0 consisted of a medium hard silt shale with no major seams or fractures and was left unpaved. The concrete used in the construction of the stilling basin slab and walls had a 28-day compressive strength of 3000 psi. The rock in the stilling basin floor at el 1035.0 was of such good quality that the elevations of the toe trench bottom for the gravity walls and cutoff end sill were raised from the original el 1020.0 to a maximum elevation of 1024.7.

272. The maximum flow through the outlet works with the pool at spillway crest would reach 16,100 cfs. This would produce a velocity of 58 ft/sec in the tunnel and a velocity of 23-27 ft/sec at the end of the stilling basin (100 ft downstream of the dentations). A velocity of 40 ft/sec was used in the design of the dentations.

273. Concrete placement in the stilling basin began in November 1938 and the basin was put into operation in July 1939 when the stream was diverted through the outlet works tunnel.

274. After nearly 30 years of service, large holes were observed in the stilling basin floor in the unpaved rock area. The eroded area varied from only about a foot (el 1034.0), a short distance downstream of the paving, to as much as 16 ft at the downstream end of the stilling basin (Figure 150). The erosion was a little more severe on the left side of the basin than on the right with undercutting of the sidewalls. A decision was made in March 1968 to restore the stilling basin floor as nearly as practicable to its original condition.

275. The work was done during the low-water period. The gates of the outlet works were closed, and a small wooden dam was placed at the downstream end of the tunnel to collect leakage, which was pumped to a point downstream of the stilling basin. A small dike was placed across Tionesta Creek downstream of the stilling basin to keep out water from

downstream sources. The deep holes in the rock portion of the stilling basin were pumped out by submersible pumps. Approximately 700 cu yd of loose material was removed from the eroded portion of the stilling basin; the bulk of the material was in the hole on the left side of the stilling basin floor.

276. After all loose material was removed from the dewatered stilling basin by clamshell and the surface was cleaned, concrete and rock were placed in the eroded portion to el 1033.0. Finally, a 2-ft-thick concrete cap containing no large rock was placed over the stilling basin (Figure 152). This cap required four placements totaling

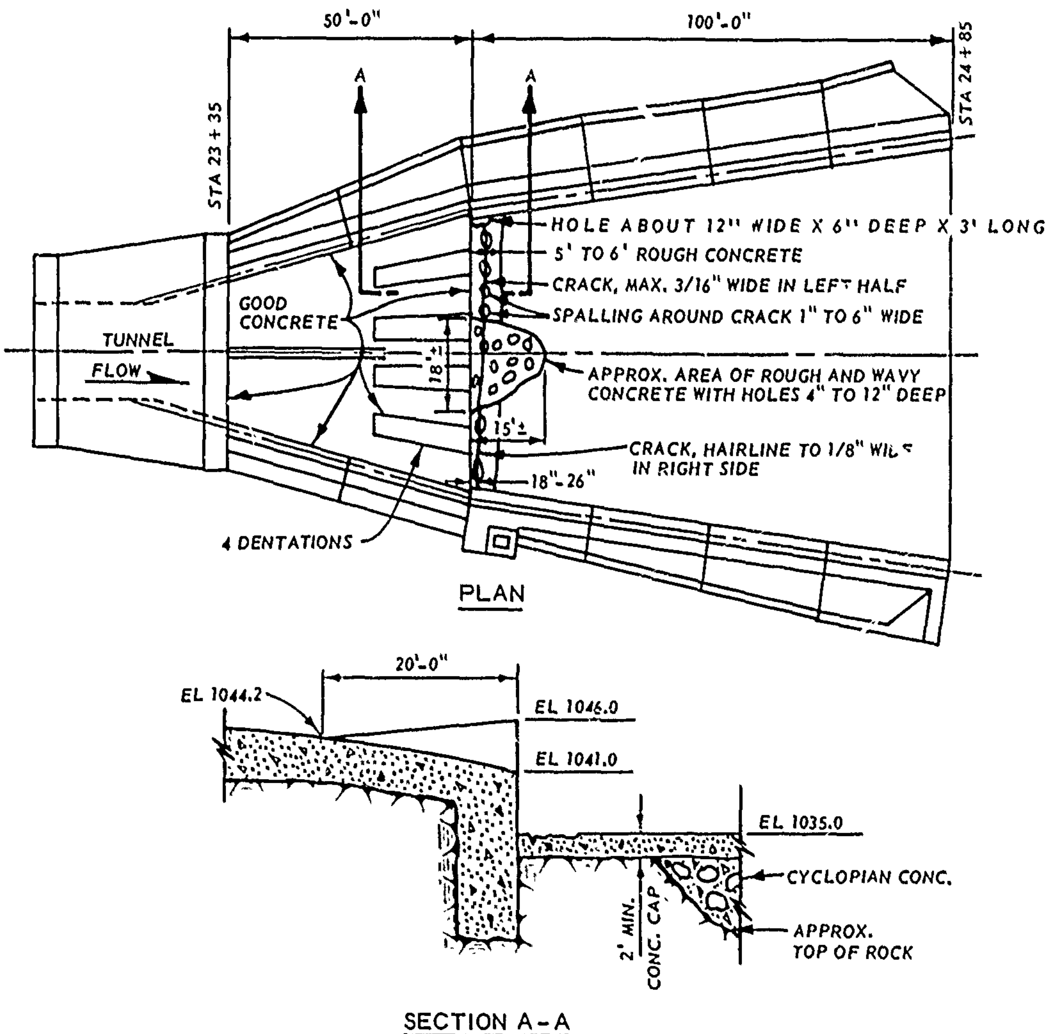


Figure 152. Results of 1975 stilling basin inspection, Tionesta Dam

692 cu yd. In addition, 7 cu yd of concrete was used to repair the downstream end of the right training wall where a considerable amount of erosion had taken place. Conventional concrete with a 28-day strength of 3000 psi was used. Of the total 3215 cu yd of concrete used, 227 cu yd was high, early strength. Repairs were made between 20 May and 12 June 1968 by Pittsburgh District personnel and locally hired labor at a cost of nearly $126,000.

277. The stilling basin was pumped down to about 18 in. of water in July 1973. The concrete dentations and the original concrete slab at the upstream end were in good condition. The new section was rough immediately downstream of the dentations, extending from 3 in. to 5 ft downstream with several holes about 6 in. deep and two holes near the center line almost 12 in. deep. The rest of the floor was smooth but there was some scattered rock on the surface, and most of it was concentrated near the downstream end of the left training wall. The rock did not appear to be moving around in the basin, and the total accumulation was small. The stilling basin was not dewatered during the second periodic inspection in September 1974, but a diver's inspection was made on 16 July 1975 by two Pittsburgh District scuba divers. The concrete of the upstream level and the four dentations was found to be in good condition. Immediately downstream of the dentated section, the concrete was rough for a distance of 5 to 6 ft downstream (Figure 152). Near the center of this rough section there was a triangular-shaped area of rough and wavy concrete about 18 ft in width, extending approximately 15 ft downstream. This area contained several circular-shaped holes approximately 4-6 in. deep and two large holes with a maximum depth of 12 in., and a crack in the new concrete was noted across the stilling basin between 18 and 26 in. downstream of the dentations. The eroded areas were in the same part of the stilling basin where the original rock had experienced relatively light erosion prior to the repairs.

278. A diver inspection in May 1978 indicated that the concrete was in generally good condition. There had been a slight increase in the erosion immediately downstream of the dentations to a maximum depth

of 14 in. The condition of cracks noted in previous inspections was unchanged.

San Gabriel River

279. Drop structure data for the San Gabriel River between Santa Fe and Whittier Narrows Dams in Los Angeles County, California, is shown in Figure 153. A reinforced concrete structure with a straight drop of approximately 14 ft, the rectangular stilling basin ranges in width from 275 to 400 ft and in length from 41 to 45 ft. Built in 1961 for a design discharge velocity of 17 fps, the structure suffered abrasion-erosion damage that was repaired with epoxy in 1970 by Los Angeles County Flood Control District personnel. This damage was caused by cobbles trapped in the basin grinding the concrete in the basin corners. It was suggested that a sloping end sill would preclude trapping cobbles. No flows have been reported since completion of the repairs.

Folsom Dam

280. Folsom Dam is located on the American River, approximately 2 miles upstream of Folsom, California. The dam was built by the Corps of Engineers and was transferred to the Bureau of Reclamation for operation on 14 May 1965. The dam (Figure 154) is a concrete gravity structure 340 ft high and 1400 ft long. Normal river regulation is maintained by two tiers of four outlets each controlled by 5- by 9-ft slide gates. The outlets consist of rectangular conduits through the dam that exit on the spillway face and discharge into the spillway stilling basin. Spillway releases are made through five radial gates located near the crest of the dam. Three additional gates for extremely large flood releases are located to the left of the main spillway, and release flow to a flip bucket on the downstream face of the dam.

281. Although model tests revealed that the flip bucket energy dissipator was adequate as far as the safety of the dam was concerned,

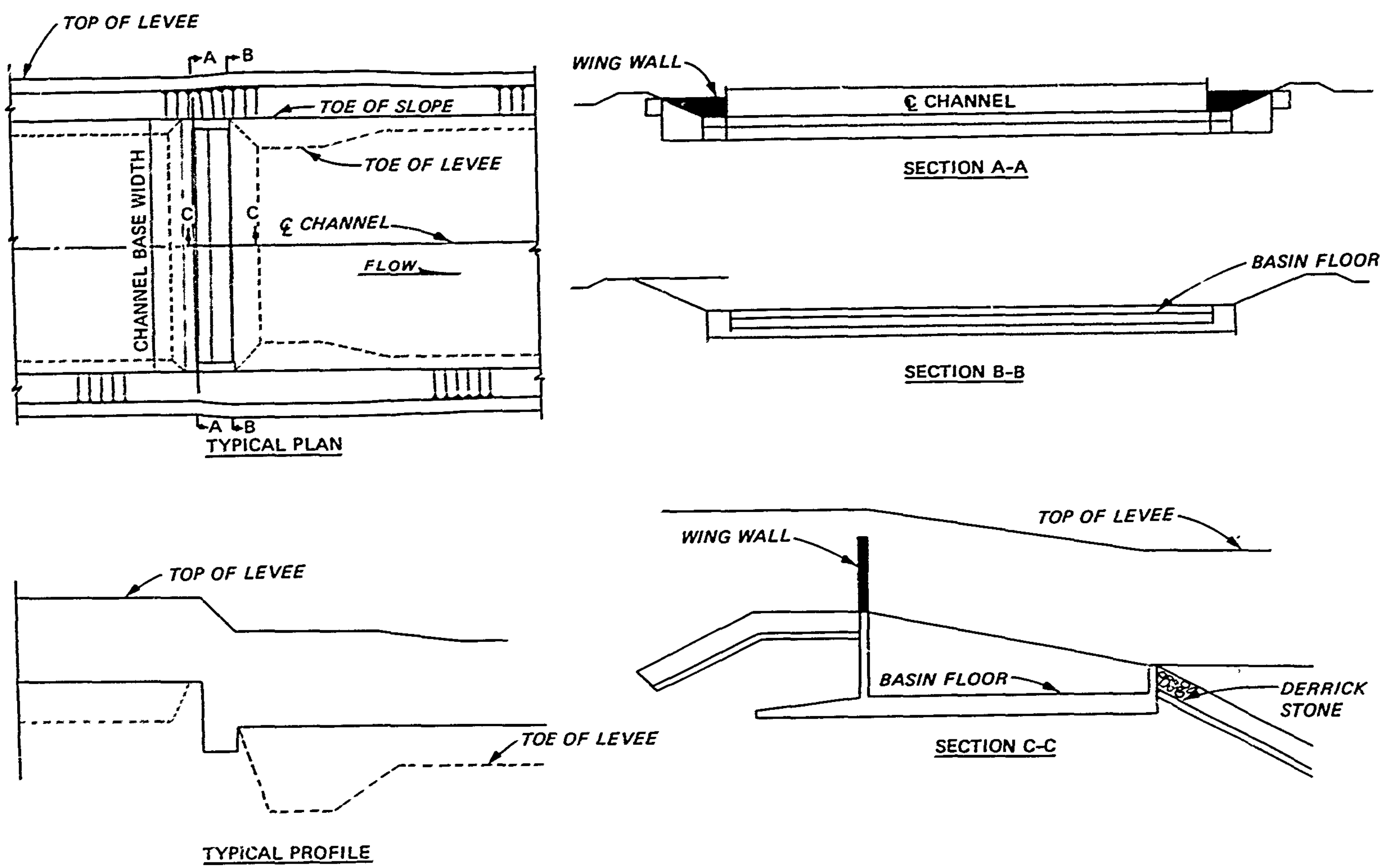


Figure 153. Drop structure data, San Gabriel River

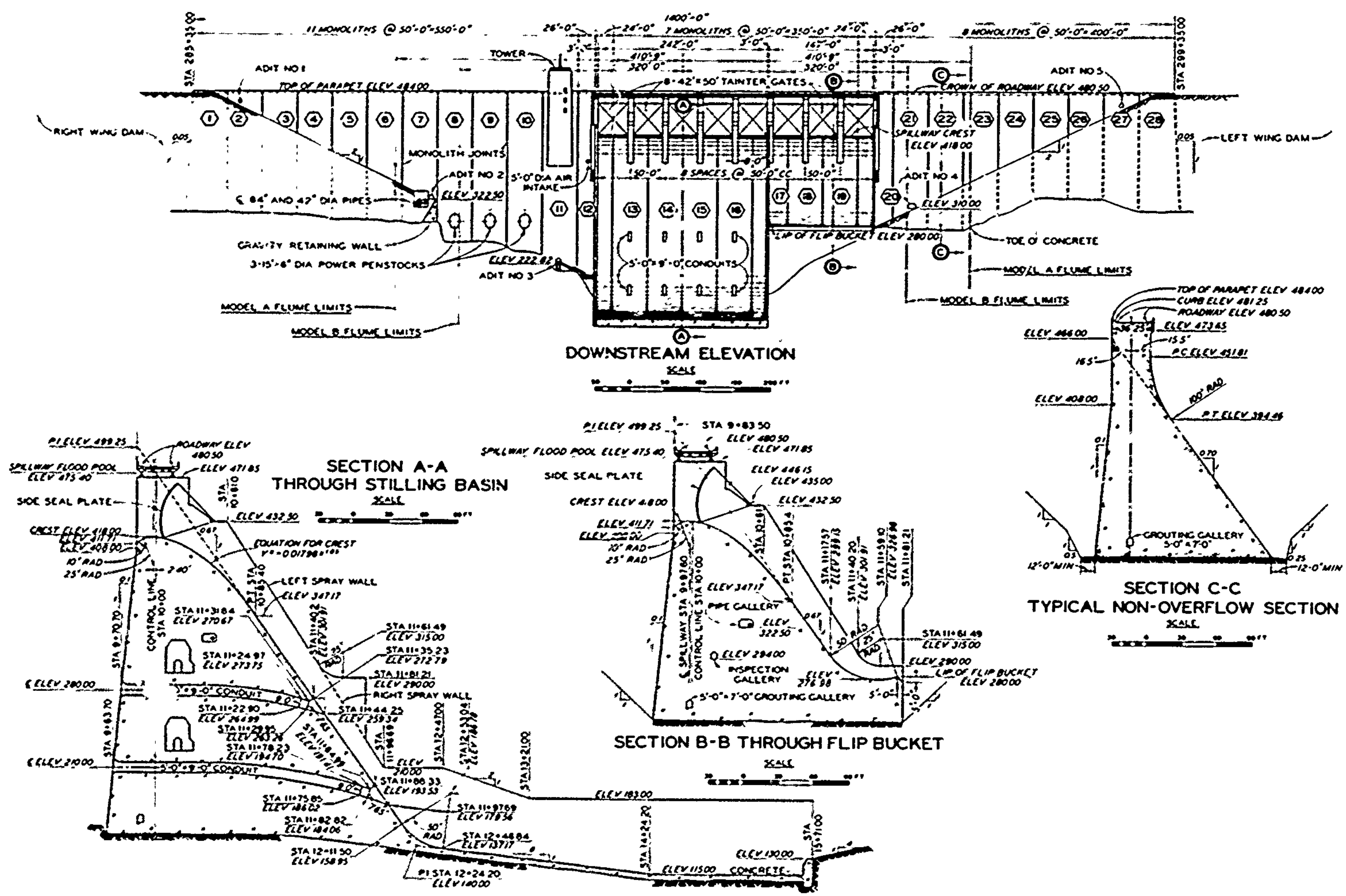


Figure 154. Elevation and sections, Folsom Dam (from WES 1953a)

the stilling basin type was adopted for construction downstream from the five gate bays adjacent to the right abutment because it would afford greater safety to the powerhouse and reduce possible maintenance costs after completion of the dam (WES 1953a). The extra cost of the stilling basin was considered warranted in view of the increased energy dissipation secured. The flip bucket energy dissipator was constructed downstream from the three gates adjacent to the left abutment; however, flow is confined to the five spillway bays discharging into the stilling basin, except for extreme emergencies.

282. Based on the model test results, a stilling basin was selected (Figure 154). This type basin resulted in good hydraulic jump action for discharges up to 200,000 cfs and fair jump action for discharges from 200,000 to 567,000 cfs. With the bed of the exit area molded to natural configuration, velocities immediately downstream of the end sill reached a maximum of 57.4 fps for a discharge of 300,000 cfs. Maximum velocities for discharges of 115,000 and 200,000 cfs were 16.0 and 35.2 fps, respectively. With a simulated scour hole, approximately 400 ft in length and a maximum depth of 32 ft, downstream of the end, maximum bottom velocities were only 12.4 fps and were upstream in direction.

283. The spillway face and downstream end of the outlets incurred considerable cavitation damage during passage of flood releases in 1955, 1963, and 1964 (Isbester 1971). The 1963 flood passed over the spillway only. The most extensive damage occurred in 1955 and 1964 with simultaneous operation of the spillway and outlet works. Outlets 1 and 5 were closed during flood passage to minimize spray on the adjacent power plant.

284. Underwater inspections of the stilling basin were made in 1965, 1968, and 1969. In each instance, objectionable deposition of debris and concrete erosion was noted. The rather extensive erosion damage resulted from a "ball mill" type of grinding action of rocks believed to be drawn back into the basin from downstream of the end sill. After the 1965 inspection, the basin was unwatered and cleaned. A siphon extension pipe was installed to facilitate dewatering. The basin was

unwatered and cleaned again in 1970, and erosion along the contraction joint at Station 12+25 was repaired with epoxy resin.

285. An underwater inspection was made in 1974 to evaluate the 1970 repair, assess the progression of concrete erosion, and estimate the amount of objectionable debris present in the stilling basin. Maximum releases into the stilling basin between 1970 and 1974 ranged from 5,000 to 50,000 cfs with an average of 23,600 cfs.

286. Concrete erosion was evident over virtually the entire floor of the stilling basin. Reinforcing steel mats were intermittently exposed over much of the left 75 ft of the basin beginning about Station 12+45 and continuing to the downstream sill. An estimated 60 percent of this area showed exposed rebars, with up to 4 or 5 in. of clearance beneath some of the bars. A number of the bars were found to be broken and bent with one loose end. A number of rocks to 20-in. maximum size were found to be scattered generally over the left one fourth of the basin. Though the largest of these showed no evidence of tumbling, rocks up to approximately 14-in. maximum size showed definite tumbling characteristics. About 8 cu yd of gravel and cobbles in a 10-in.-deep layer were noted near the left wall between Stations 13+40 and 14+50; otherwise, deposition consisted of scattered gravel and cobble up to 12 in. in size. Total quantity of deposition was estimated to be 12 cu yd. The epoxy repairs made along the contraction joint at Station 12+25 in 1970 were generally in good condition and appeared to be holding up well; however, some of the longitudinal joints crossing this joint had deteriorated severely. The longitudinal joint 71 ft from the right wall had a local up to 14 in. deep in the Station 12+25 region.

287. Localized scour holes were prevalent along both the longitudinal and transverse contraction joints, particularly between Stations 12+21 and 14+71. At the intersection of the joint along Station 12+21 and the longitudinal joint 71 ft from the right wall, a dish-shaped hole approximately 4 ft by 8 ft by 18 in. deep was noted.

288. The end sill and the riprap downstream of the end sill were both in excellent condition. The siphon flange was covered with

corrosion and will require considerable cleaning before it is suitable for reuse. The threaded holes in the flange will have to be cleaned with a tap. Several large rocks up to 3/4 cu yd were found along the end sill about 50 ft from the right wall.

289. Team members who had participated in the previous examinations concluded that the erosion of the concrete in the floor of the stilling basin had progressed considerably, particularly over the left side of the basin and along the upstream longitudinal joints. Unfortunately, a lack of reference points precluded determining any quantitative measurement of the erosion. It should be noted that the area of major erosion and deposition of debris was downstream of outlets 1 and 5.

290. Design drawings show the stilling basin floor slab to be a minimum of 5 ft thick with a single mat of reinforcing steel located at the center of the slab. While the presence of the exposed steel might indicate that up to 30 in. of concrete has been lost, the original contact line at the walls shows that the overall loss is probably nearer 12 in. It was suggested that representative datum points be established to determine the loss of concrete and to provide a reference for measuring future progression of the erosion. It was also suggested that methods to prevent rocks from entering the stilling basin from the auxiliary spillway be investigated since this appeared to be the source of many of the rocks found in the basin.

Pine Flat Dam

291. Pine Flat Dam is a straight gravity concrete dam with a crest length of 1,820 ft, a maximum height of 429 ft, and a concrete volume of 2,150,000 cu yd, and is located on the Kings River 26 miles east of Fresno, California. Construction of the dam and appurtenances was started in 1947 and the bucket was built in 1950. The project began operation in February 1954. An overflow spillway near the center of the dam controls releases above el 916.5 by means of six tainter gates. The five upper level conduits have an ogee vertical curve and discharge into the overflow spillway (Figure 155). The five lower level conduits are horizontal and discharge through the spillway monoliths below the flip

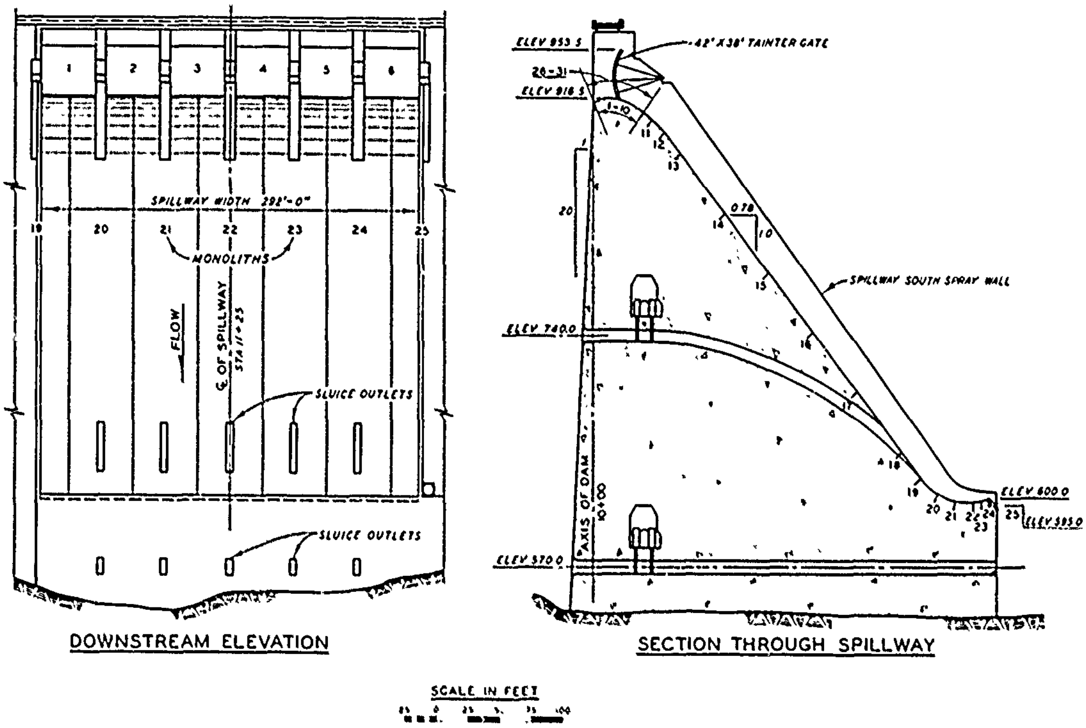


Figure 155. Spillway and sluices, Pine Flat Dam
(from WES 1953b)

bucket. The design discharge of the spillway is 145 fps. Model tests (WES 1953b) indicated that a flip bucket with a radius of 50 ft, terminated 20 deg above horizontal, was adequate to protect the toe of the dam against harmful erosion.

292. Erosion in the conduits was first noted during the first regular inspection of the sluice conduits in October 1952. At that time, sand and silt were deposited upstream of the dam. In addition, a local slide on the left abutment had deposited boulders and some construction debris immediately upstream of the entrances to the lower sluices. It was concluded that the erosion in all the lower conduits was caused primarily by the sluicing of loose rock and construction debris through the conduit. Prototype tests (Dawsey 1959) were conducted in June 1956 to observe and evaluate the hydraulic performance of the spillway crest and flip bucket.

293. During the past several years some small areas on the spillway section below the upper conduits have eroded and the overall surface

of the concrete has been roughened (Figure 156). Also the lip of the flip bucket has been eroded slightly, with most of the erosion occurring on the downstream end of the lip. Different areas of this erosion were repaired with calcium aluminate cement mortar* in 1968, 1970, and 1972.

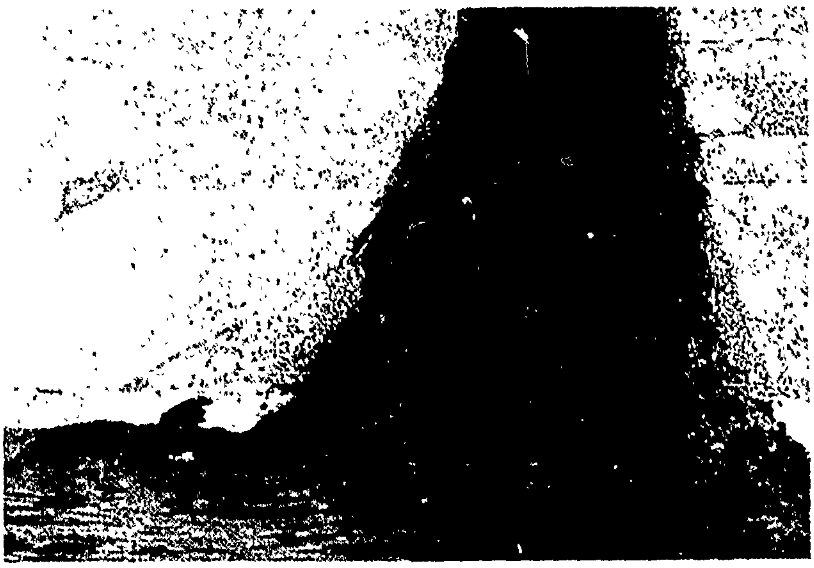


Figure 156. Concrete condition below conduit, Pine Flat Dam

After loose concrete was removed by hand, one part high alumina cement mixed with four parts sand and a small amount of water was trowelled into the eroded area. The condition of a typical repair on the lip where a large quantity of flow occurs is shown in Figure 157.

Coyote Dam

294. Coyote Dam is an earth embankment with a reinforced concrete outlet works on the East Fork, Russian River near Ukiah, California. The outlet works includes an intake structure, a single conduit (12.5-ft diameter), and an outlet chute consisting of a drop structure and

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* This type of mortar has been found to be a most economical and very effective method of controlling erosion in the conduits and on the flip bucket. The mortar is applied on a continuing basis as needed by dam maintenance crews.

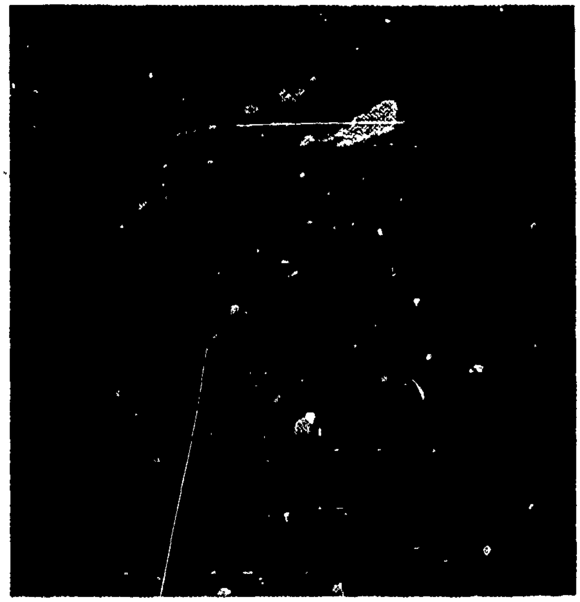


Figure 157. Condition of repair after 1-1/2 years service at spillway flip, Pine Flat Dam

stilling basin. The drop structure flares from a width of 16 ft at the outlet to 50 ft at the stilling basin. The rectangular hydraulic jump stilling basin (Figure 158) is 50 ft wide and 63 ft long with a single row of baffles and a vertical end sill. The stilling basin was built in 1959 for a design discharge of 65 fps.

295. In 1969, erosion of the end sill was repaired using concrete proportioned for 4000 psi compressive strength at 28 days. The new concrete was bonded and anchored to the existing sill (Figure 159). In an effort to prevent future erosion and loss of downstream riprap, an additional row of baffles was constructed (Figure 158). A 2-in. saw cut was made in the basin slab to outline the new baffles and the inscribed concrete was chipped out to a depth of 2 in. Anchor bars were embedded in the existing concrete with a nonshrink grout and the baffles were reinforced (Figure 160). During the repair work the contractor diverted a flow of 300 cfs around the site and kept the basin dry by pumping.

296. The structure was inspected in April 1973, and no significant damage was noted. The stilling basin was dewatered for a periodic inspection in November 1977. No significant erosion damage was noted,

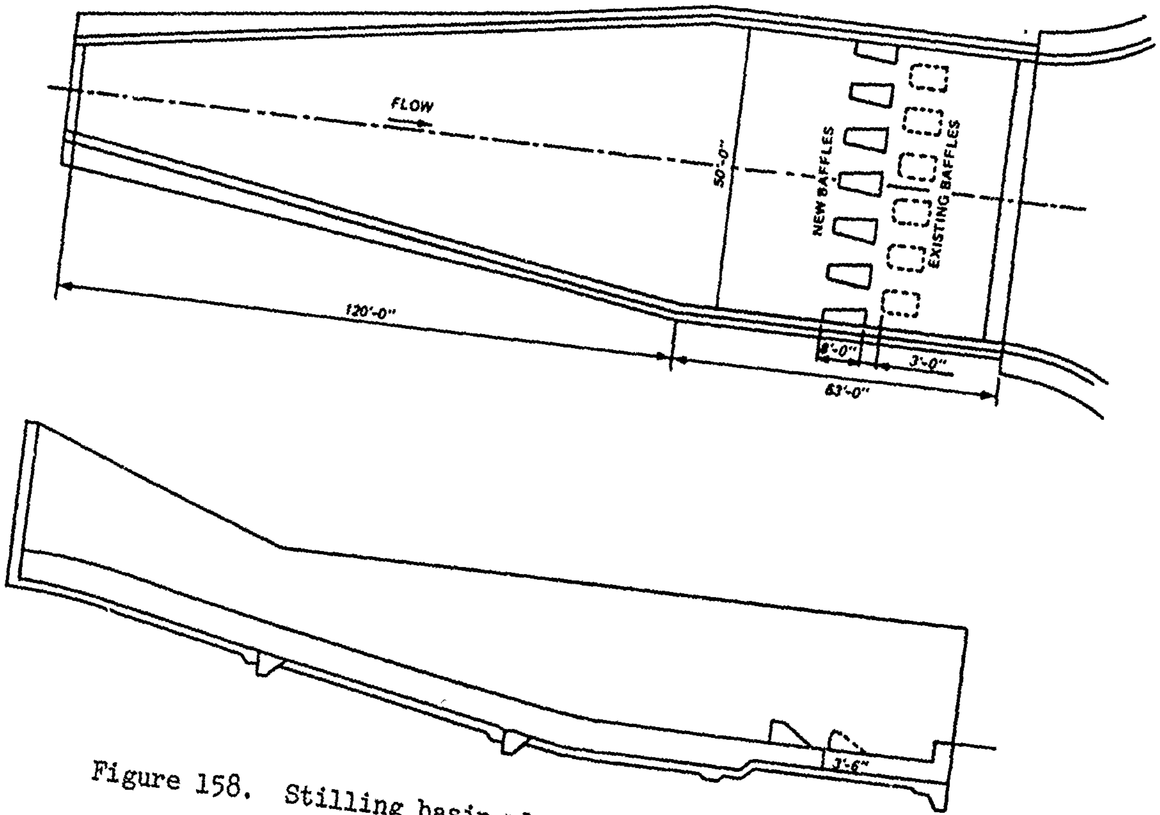
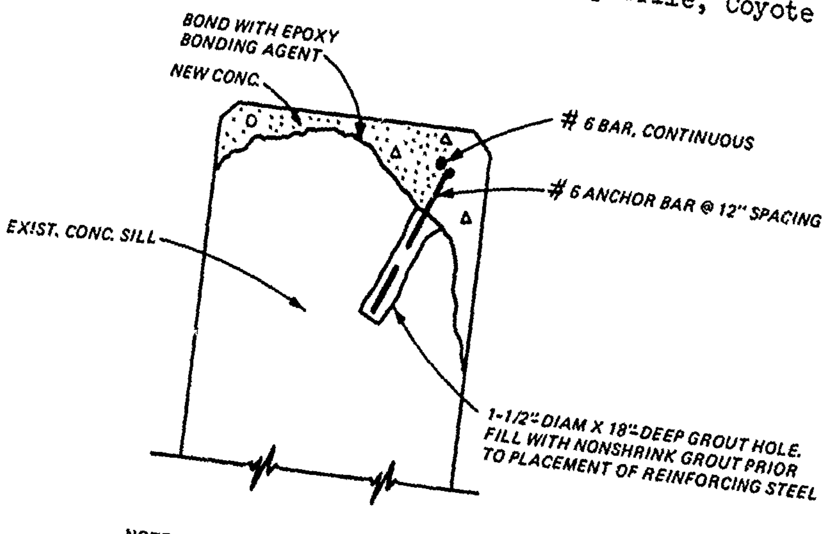


Figure 158. Stilling basin plan and profile, Coyote Dam



NOTE: NEW REINFORCING STEEL PLACED ONLY IN AREAS WHERE EXISTING STEEL IS EXPOSED BY EROSION.

Figure 159. End sill repair, Coyote Dam

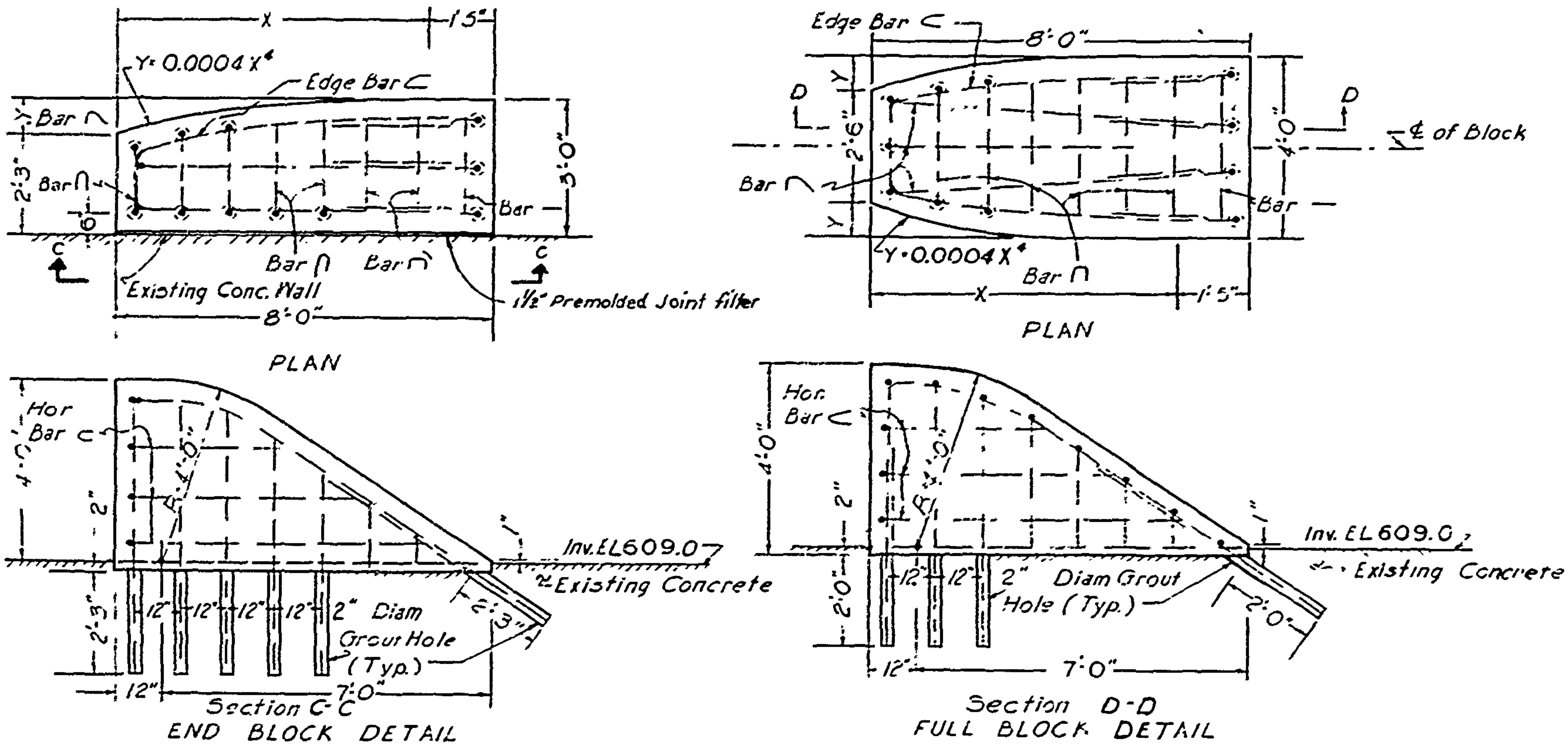


Figure 160. Baffle block details, Coyote Dam

although there were several rocks in the basin ranging in diameter from 2 to 10 in. A diver inspection of the area around the intake structure indicated a deposit of rocks, which is assumed to be the source of the rocks found in the stilling basin. It is believed that these rocks were flushed from the basin by unusually high discharges during 1978.

Conchas Dam

297. Conchas Dam is located in San Miguel County in northeastern New Mexico on the South Canadian River just below its confluence with the Conchas River. The main portion of the dam is a straight concrete gravity structure with a maximum height of 235 ft from the foundation to the top of the roadway and crest length of 1250 ft. The ogee crest spillway (Figure 161), 340 ft in overall length, consists of five 60-ft bays separated by 10-ft wide bridge piers, which are partially streamlined at the downstream end. The six sluicing conduits are 4 by 5 ft in cross section, and each is controlled by a pair of hydraulically operated slide gates. The stilling basin (Figure 162) consists of a paved concrete apron 127-1/2 ft in length; one row of stepped baffles, 8 ft in height, located 31 ft upstream from the end of the apron; and a solid, stepped end sill, 12 ft in height. The stilling basin was completed in April 1939 using class "B" concrete.

298. When the basin was dewatered in September 1944, pronounced erosion was noticeable in only four areas on the floor of the stilling basin (Figure 163). The most noticeable erosion, designated Area 1, occurred between the end sill and the baffle adjacent to the north training wall. The maximum depth of erosion in this area was 1 ft 6 in. at one point on the north side where the floor mat of reinforcing steel was exposed for about a 1-ft length. In general, however, the depth of erosion was from 9 to 12 in. in a roughly elliptical-shaped trough bounded by the baffle, training wall, and end sill, and extended about 18 in. south of a line extending from the south side of the baffle. The wear in this area also extended into the downstream face of the baffle and into the training wall as much as 6 in., exposing the vertical reinforcing

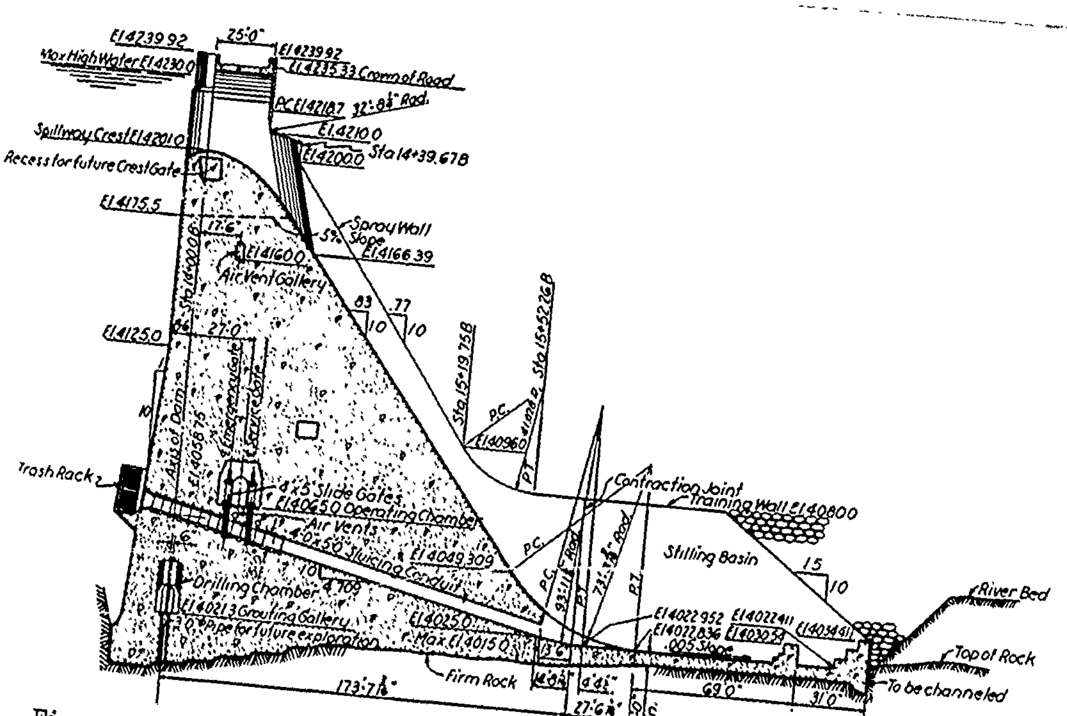


Figure 161. Section through sluiceway, Conchas Dam (from U. S. Army Engineer Office, Caddoa)

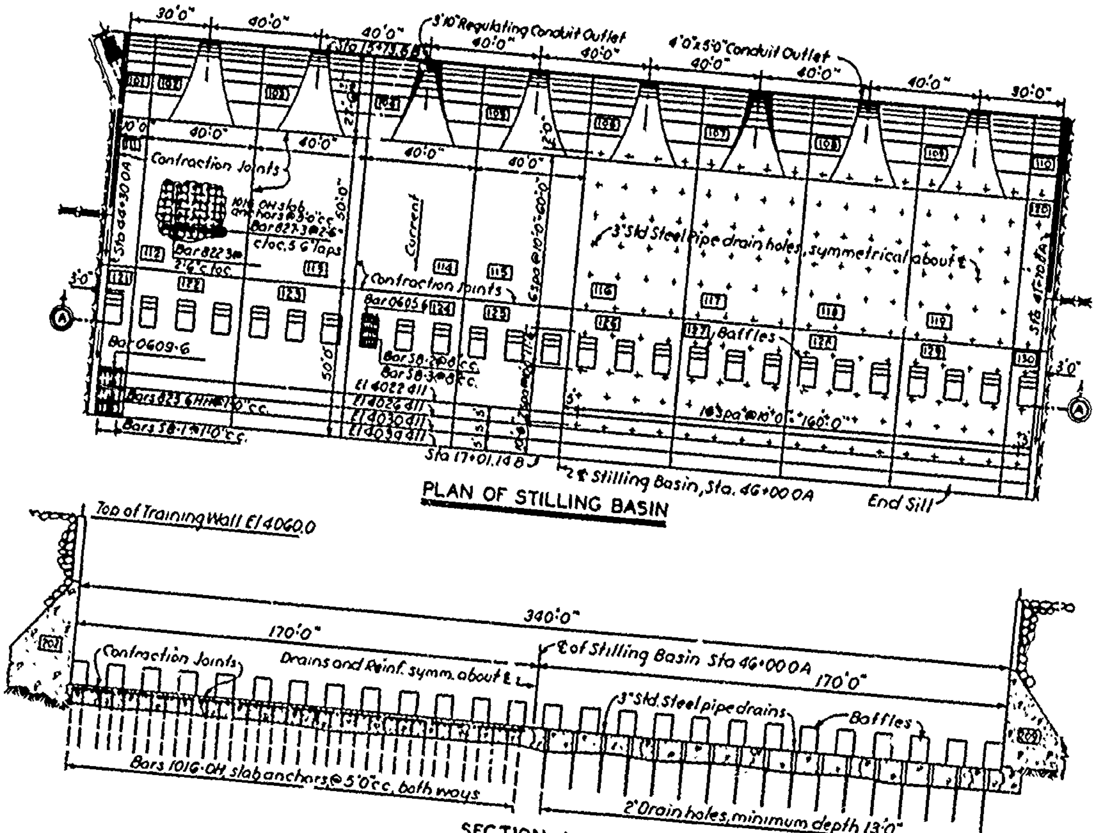


Figure 162. Stilling basin plan and section, Conchas Dam (from U. S. Army Engineer Office, Caddoa)

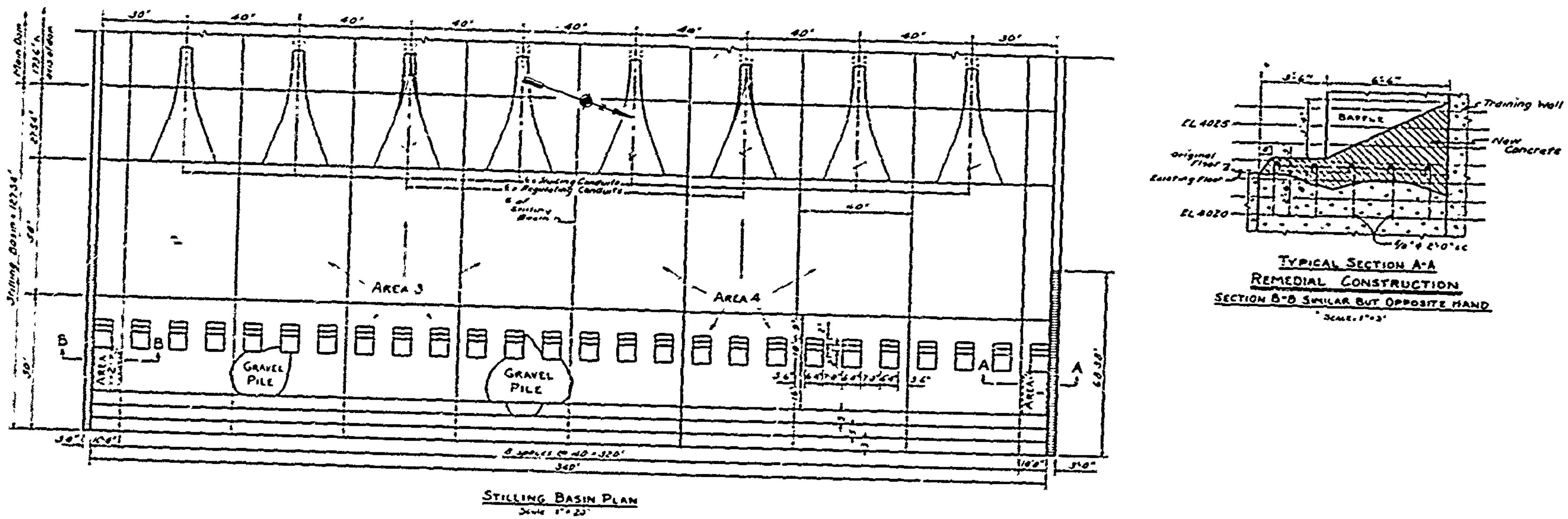


Figure 163. Stilling basin inspection and repair, Conchas Dam

bars in the baffle. Area 2, between the end sill and the baffle adjacent to the south training wall, had a maximum depth of wear of about 12 in. and was eroded in a manner very similar to but less extensive than the first area described above.

299. Areas 3 and 4 lie between the toe of the dam and the baffles, directly downstream from the regulating conduits in monoliths 12 and 13. The erosion in each of these areas ranged from slightly more than 3 in. 15 ft upstream from the baffles and in line with the conduits to virtually none at the toe of the dam, at the midpoints of adjacent monoliths, and at the upstream face of the baffles. All other portions of the floor of the stilling basin were in excellent condition, and only the top mortar was worn away and a little aggregate exceeding 1/2 in. in size was exposed.

300. Evidence of erosion on the sides, downstream faces, and tops of the baffles was not perceptible except for slight wear on the lower 12 in. of all baffles and on the downstream face of the end baffles as previously described. The upstream face of all baffles showed signs of erosion; the most pronounced erosion occurred on the eighth baffle from each training wall. These two baffles are located in the monoliths downstream from the regulating conduits. The least erosion of the upstream face of the baffles occurred on those two adjacent to the center line of the stilling basin.

301. There was no evidence of serious wear on the upstream face of the end sill. Slight erosion was noticeable, however, with the most extensive being directly downstream from the baffles that showed most wear. The top of the end sill was free of erosion with the exception of a portion approximately 75 ft long midway between the training walls where about 3/4 in. of mortar and aggregate was worn away.

302. The major portion of the stilling basin was virtually free of sedimentation. Two piles of gravel and rock were found between the baffles and the end sill (see Figure 163). The largest pile, containing approximately 60 cu yd of material, was located directly downstream from monolith 13 and covered almost the entire area between the monolith joints and between the baffles and the end sill. The other pile,

directly downstream from monolith 11, and containing about 25 cu yd, covered the south half of the area between the monolith joints and between the baffles and end sill.

303. Prior to the inspection, water had been discharged irregularly over the spillway and through the sluicing and regulating conduits. The releases in acre-feet, by water years ending each first of October, are shown in the following table:

Table 3

Water Releases at Conchas Dam

1940-1944

Unit	Discharge, acre-feet 1940	1941	1942	1943	1944	Total
Spillway	0	377,025	305,500	0	22,925	705,450
Conduit M-10	353	48,991	117,091	16,124	16,439	198,998
Conduit M-11	350	39,636	111,204	16,143	17,396	184,729
Conduit M-13	367	53,490	146,400	15,442	18,740	234,439
Conduit M-14	350	51,206	145,385	15,470	17,706	230,117
Conduit M-16	372	39,511	117,224	15,430	16,387	188,924
Conduit M-17	371	48,890	120,058	15,459	15,274	200,052
Conduit M-12	352	3,023	14,478	125	10,865	28,843
Conduit M-15	413	4,464	14,444	80	10,769	30,170
Power Unit	2,441	4,654	4,979	4,764	2,428	19,266
Yearly Total	5,369	670,890	1,096,763	99,037	148,929	
Combined Total						2,020,988

304. The erosion of the areas between the end baffles and the end sill was believed caused by eddy action. These eddies were attributed to the inability of water to flow on both sides of the baffles, thus causing "dead" spots behind the baffles. Similar but somewhat less defined eddies account for the erosion in the areas downstream from the regulating conduits. The regulating conduits, discharging virtually no water or, at most, much less water than the adjacent sluicing conduits,

could not maintain a discharge velocity in the lower reaches of the stilling basin comparable to that caused by the sluicing conduits. This difference in discharge caused a partially "dead" or slow spot in the discharge downstream from each regulating conduit with the resulting eddy action scouring the floor only slightly but over an extensive area.

305. The rock and gravel piles in the south half of the stilling basin downstream from the baffles are located on the approximate edges of the eddies described above. Deposits composed of rocks and gravel from the downstream river bed apparently had been swirled upstream by the eddy action and deposited when they approached the outer edge of the eddy where the velocity was dissipated. The absence of these deposits in the north portion of the stilling basin, however, indicates that the topography of the downstream river bed had considerable effect upon the degree of turbulence of the flow. Observation of the basin under conditions of discharge symmetrical about the center line of the stilling basin indicated that considerably more turbulence did occur in the south side of the stilling basin.

306. Inasmuch as the operation of both the spillway and the conduits was to be minimized upon completion of the irrigation facilities being constructed at the time, the only remedial action deemed practical was removal of the rock and gravel deposits and renovation of the areas between the end baffles and the end sill. The latter was accomplished by placing concrete between each end baffle and the end sill (see Figure 163).

307. The stilling basin was dewatered in 1974, and inspection indicated that the basin and repair work done in 1945 was in excellent condition. As a part of the 1974 effort, two sumps were built into the floor of the stilling basin to facilitate future dewatering and inspection. There were no discharges through the stilling between 1974 and 1977. Future inspections are contingent upon operation of the project.

Somerville Dam

308. Somerville Dam is located at river mile 20.0 on Yegua Creek about 2 miles south of Somerville, Texas. The spillway (Figure 164) is

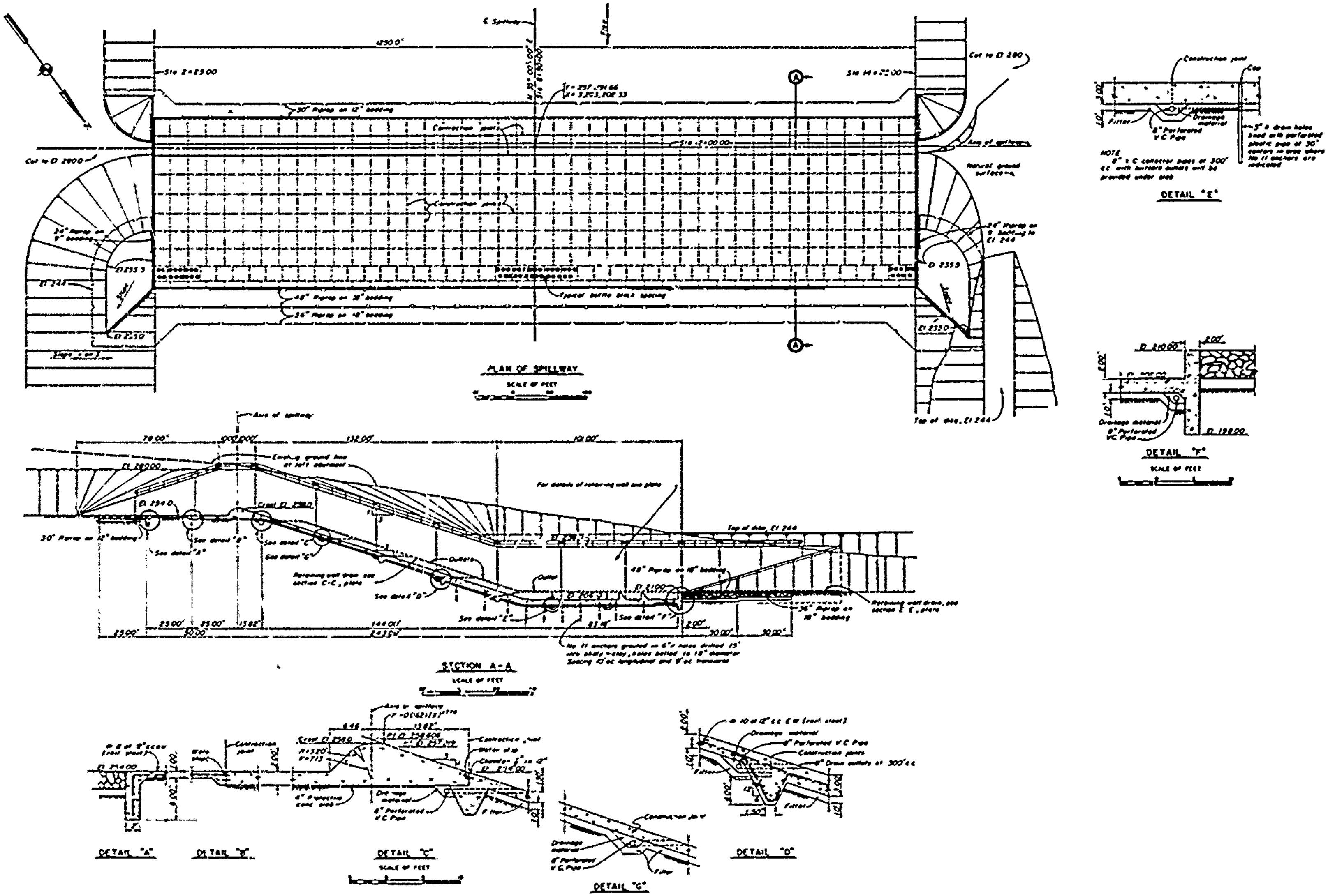


Figure 164. Spillway plan, section, and slab details, Somerville Dam

a 1250-ft uncontrolled ogee weir with the crest at el 258.0. A straight grade chute extends downstream to the horizontal stilling basin floor at el 206.0. The rectangular stilling basin has double rows on 4-ft-square baffles. Both the baffles and end sill are 4 ft high. Spillway concrete was designed for 3000 psi compressive strength at 28 days. The capacity of the spillway is 289,400 cfs at maximum design water surface. Construction of the dam began in June 1962 and deliberate impoundment began in January 1967.

309. During the periodic inspection in August 1968, it was observed that a 6- by 8-ft area in a corner of a chute slab panel had surface spalling about 1 in. deep, apparently from a poor finish. The spalled area was at about the midlength and midwidth of the chute. Project personnel first repaired the area by applying an epoxy compound as a bonding agent for a portland cement fill against the existing concrete. But the patch developed a hollow ring, indicating a separation. Shortly before the September 1970 inspection, project personnel made a second repair. Loose and poor quality concrete was broken out to a depth of 12-14 in., and edges were vertically saw-cut from 1 to 2 in. Broken surfaces of existing concrete were painted with epoxy and the fill concrete was prepared with an iron aggregate mortar.

310. At the September 1970 inspection, it was noted that the repair appeared well placed; however, a transverse hairline crack had developed near the middle of the repair.

Waco Dam

311. Waco Dam is located at river mile 4.6 on the Bosque River on the north edge of Waco, Texas. The Waco spillway (Figure 165) is an ogee weir controlled by fourteen 40- by 35-ft tainter gates separated by 8-ft piers. The length between abutments is 664 ft and the crest elevation is 465.0. A straight grade chute transitions down to the stilling basin floor at el 369.0. The horizontal basin is 664 ft wide and 192.53 ft long. Two rows of baffles 10 ft high by 8 ft wide by 10 ft long and an end sill 10 ft high provide energy dissipation. Concrete

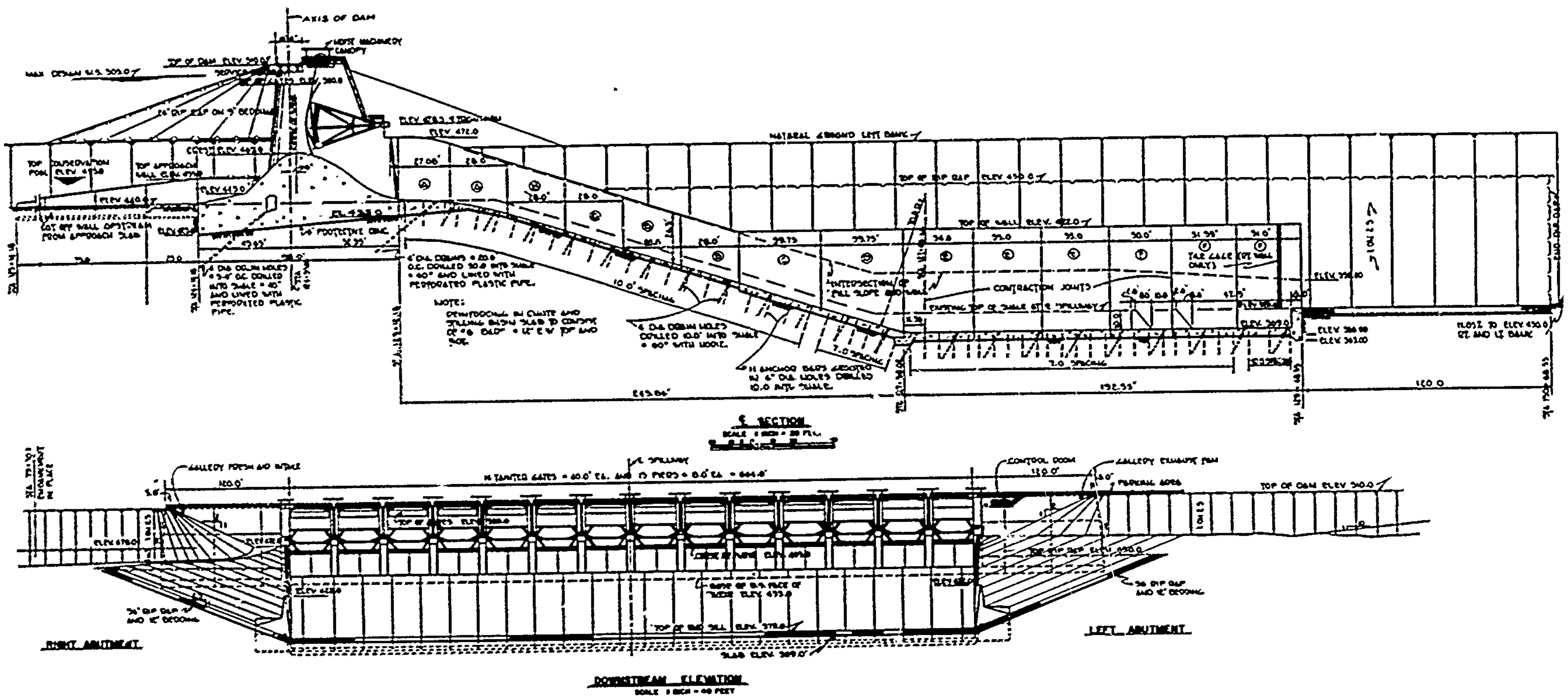


Figure 165. Downstream elevation and section, Waco Dam

design strength was 3000 psi at 28 days. Construction of the dam began in June 1958 and deliberate impoundment began in February 1965. The spillway capacity is 563,300 cfs at maximum design water surface and 458,000 cfs at top of flood control pool.

312. As a result of damage to the original slab during construction, the contractor patched the downstream edge of the weir section at the right side of the fourth gate bay from the right abutment. The patch was 22 ft long with an average width and depth of 40 and 18 in., respectively. During releases of flood water in 1968, this patch was loosened and partially torn out.

313. Using a paving breaker, project personnel removed the remains of the old patch. The edges of the existing concrete had been sawed to provide a vertical bonding surface for the original patch. The patch area was chipped, washed, and cleaned of all loose and foreign material. No. 3 reinforcement bars at 12 in. center to center each way were grouted into the existing concrete. Existing concrete was painted with Sonobond bonding agent before placing the new patch material. This was a mixture of two parts nonshrink iron aggregate grout and one part by weight of 1/4- to 3/8-in. crushed stone. The patch was water cured for 10 days. In 1970 the patch was reported to be in very good condition and holding tight.

Bull Shoals Dam

314. Bull Shoals Dam, completed in November 1951, is located on the White River in north central Arkansas approximately 10 miles west of Mountain Home, Arkansas. The dam is a straight concrete gravity structure (Figure 166) and has a top length of 2256 ft and a maximum height of 284 ft. At crest el 667 (Figure 167), flow over the spillway is controlled by 17 tainter gates 40 ft wide by 28 ft high. The spillway design discharge and velocity are 556,000 cfs and 127 fps, respectively. Release of floodwaters when the pool is below spillway crest is accomplished by 16 sluices 4 ft wide by 9 ft high. The sluices terminate in the stilling basin where the sluice invert elevation is 437.5.

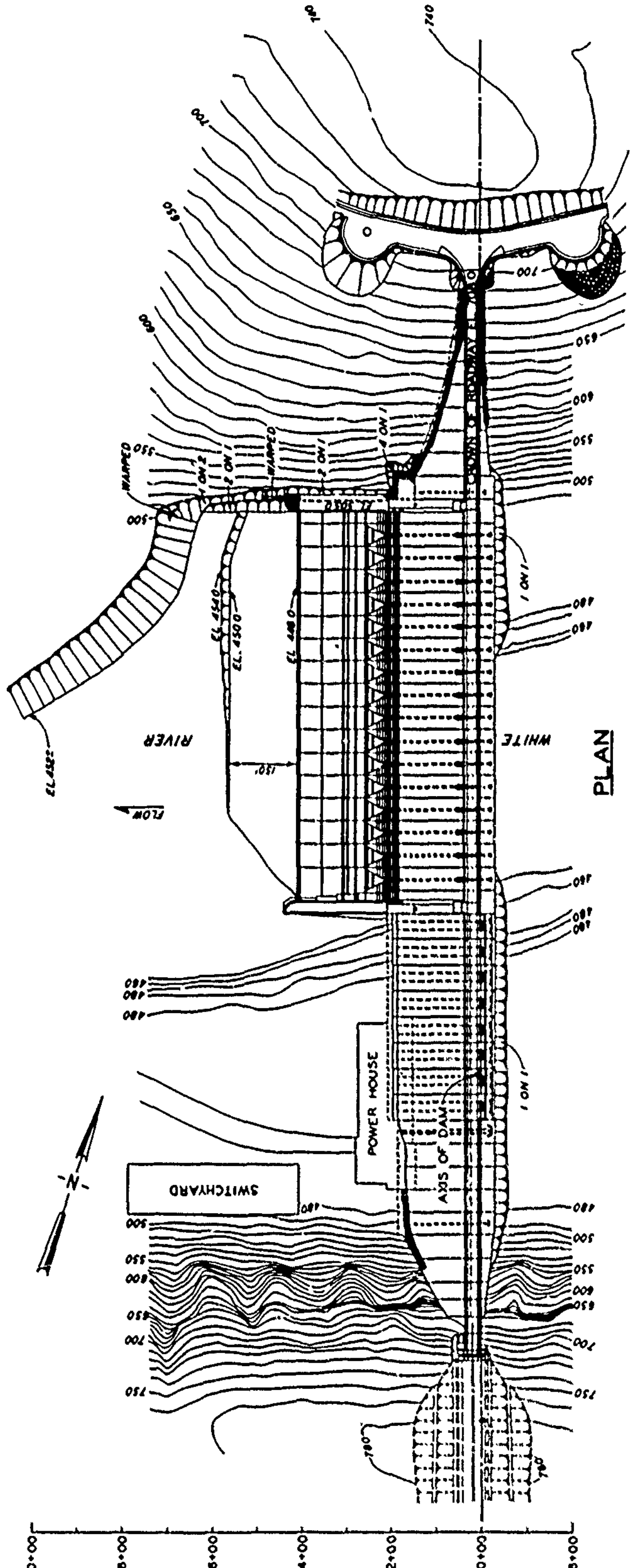


Figure 166. General plan, Bull Shoals Dam (from WES 1958)

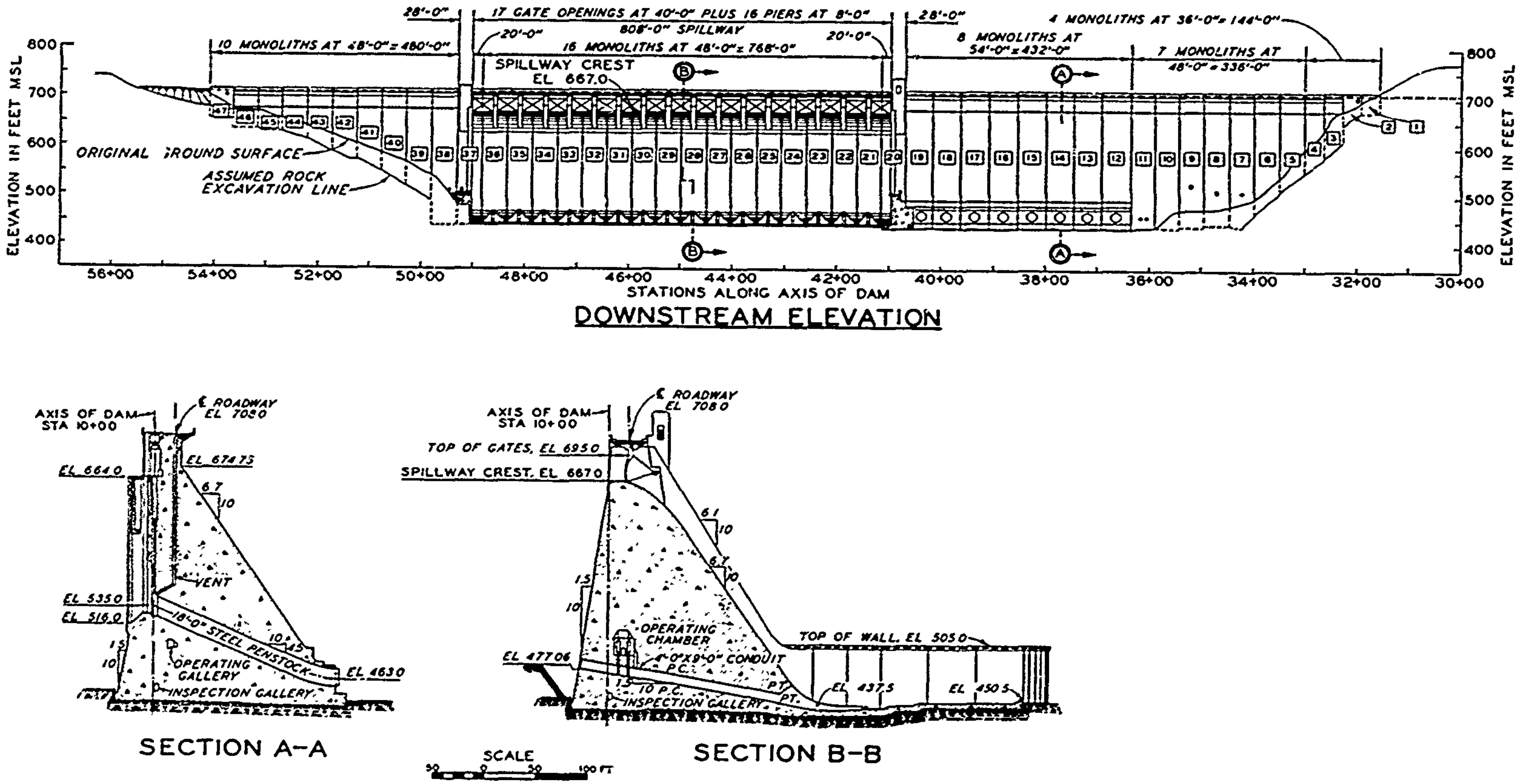


Figure 167. Downstream elevation and sections, Bull Shoals Dam (from WES 1958)

Discharge for one sluice, pool el 667, is 3500 cfs.

315. The stilling basin finally adopted for construction was based on the results of model studies conducted at WES (1947) in 1944-45. The elliptical-step stilling basin (Figure 168) is 808 ft wide (the full width of the spillway section) and 203.3 ft long with an end sill 4 ft high. The aggregates used in the stilling basin concrete were manufactured from a material consisting of approximately 80 percent sandy dolomitic limestone and about 20 percent sandstone. Class "B Exterior" concrete with 3-in. MSA was placed in the stilling basin. The water-cement ratio ranged from 0.56 to 0.59. The cement consisted of about 25 percent natural cement containing an interground air entraining agent and about 75 percent Type II portland cement. Accelerated freezing and thawing tests conducted prior to construction indicated that the concrete would have satisfactory resistance to natural freezing and thawing. However, no tests for resistance to abrasion and cavitation erosion were conducted. Compressive strengths at 7 and 28 days averaged approximately 2000 and 3600 psi, respectively. Pertinent data about the stilling basin concrete, placed during the period September 1948 to June 1949, have been reported in detail by the U. S. Army Engineer District, Little Rock (1953).

316. Second-stage diversion through the conduits began in August 1949. Operation continued with all conduits fully open, except for a few short periods of closure for each conduit, until filling the reservoir began in July 1951. The maximum pool elevation during this period was about 573 on 24 February 1951, and the corresponding discharge was about 2650 cfs from each conduit. The velocity of this flow through the conduit was about 74 fps. In May 1950, a flood with a peak discharge of about 178,000 cfs overtopped the incompleted monoliths in the spillway section. The top elevation of most of the spillway monoliths was 482, and the others were at higher elevations. The maximum pool elevation during the flood was 524.1. It was estimated that the nappe struck the apron upstream from the upstream step.

317. During the reservoir filling period (July 1951 - October 1952), the sluices were operated at various gate openings from 0.2 ft

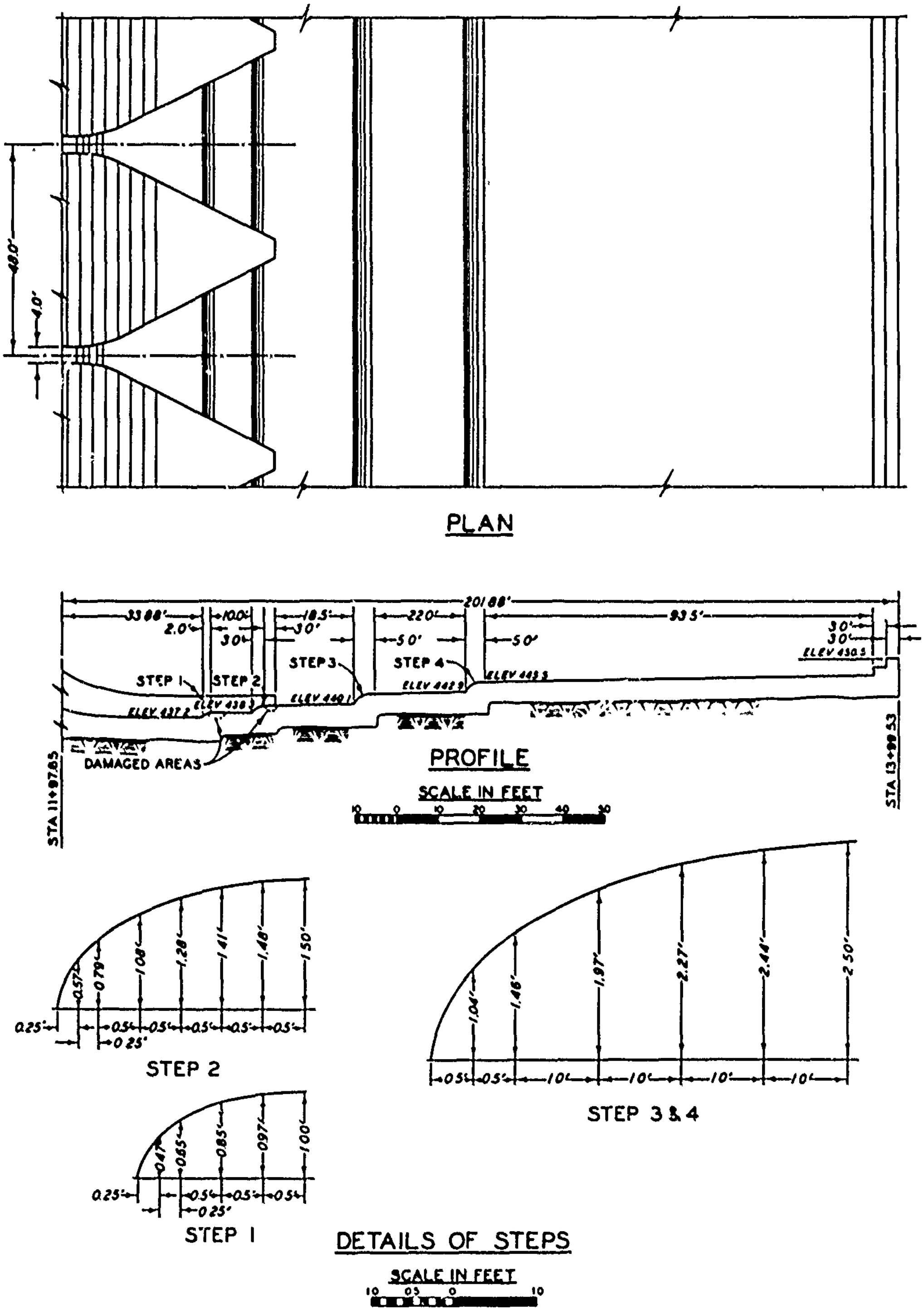


Figure 168. Typical sluice portals and stilling basin step details, Bull Shoals Dam (from WES 1958)

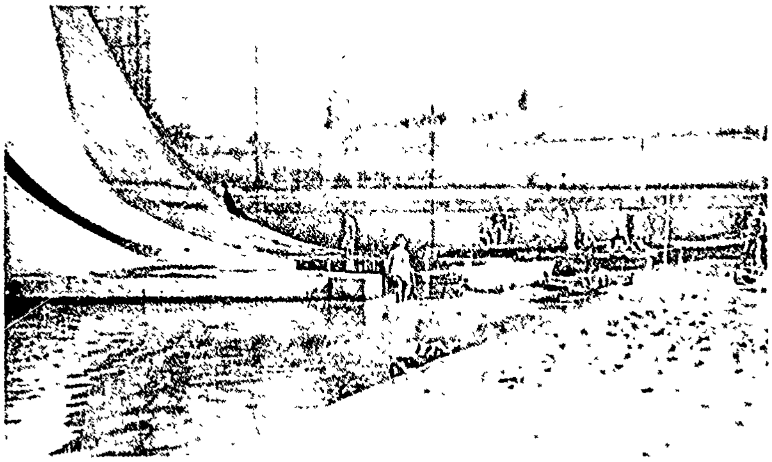
to fully open at operating heads of 70 to 190 ft. During this period, the pool rose from a minimum elevation of about 524 to a maximum of about 640. It was noted during the operation of one or two conduits under conditions of relatively high pool elevation and relatively low tailwater elevation that the jets were confined by large eddies in the stilling basin and tended to shoot over the end sill.

318. In June 1952, the stilling basin was inspected by using an underwater viewing device and by probing, and possible damage to the basin was indicated. The basin was subsequently unwatered using an earth cofferdam constructed downstream of the stilling basin.

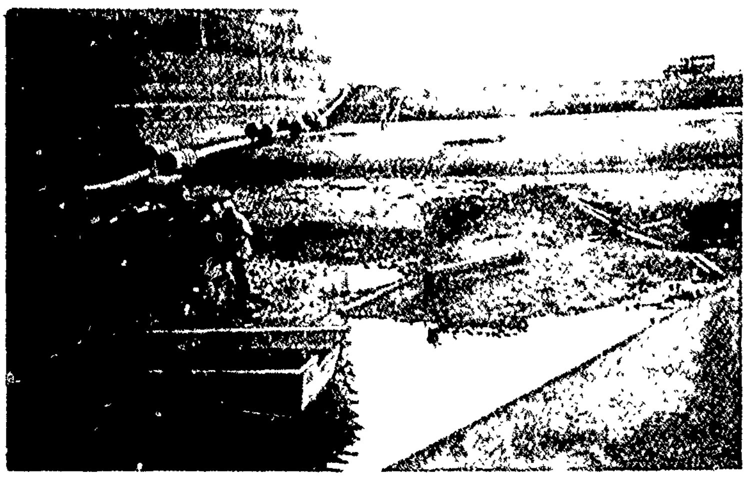
319. Inspections of the unwatered stilling basin in October and November 1952 revealed deposits of sand, gravel, timber, and miscellaneous scrap steel. The largest deposit (Figure 169) was in monolith 21, and most of the pieces of rock and steel deposited in the basin were rounded by abrasion (Figure 170). Areas of eroded concrete and exposed reinforcing steel were present on or immediately downstream from the first elliptical step below 12 of the 16 conduit outlets and on or immediately downstream from the second step below 6 of the conduit outlets. Locations of the eroded areas are shown in Figure 171 and typical examples are shown in Figures 172 and 173.

320. The volumes of the material eroded from these areas and a summary of the flow data for the period 1 August 1951 to 15 July 1952 are presented in Table 4. Between 15 July 1952 and 6 October 1952, there were very small discharges from conduits operated singly at gate openings of 0.4 ft to 1.2 ft.

321. Damage occurred downstream from only those conduits which had been operated during this period, but only a very general relationship was found between the eroded volume and the flow data (Figure 174). The erosion on the first steps ranged in depth to a maximum of about 3.5 ft. The exposed reinforcing steel appeared to have failed because of bending fatigue as there were no "necks" characteristic of tension failures. The downstream ends of many longitudinal reinforcing bars appeared to have been "peeled" out of the 3-in. concrete cover, leaving rough grooves about 2 in. wide, 4 in. deep, and 1 ft long. On the top



a. View toward left training wall



b. View toward end sill

Figure 169. Sand and debris deposits downstream from monolith 21, Bull Shoals Dam (from Little Rock District 1953)

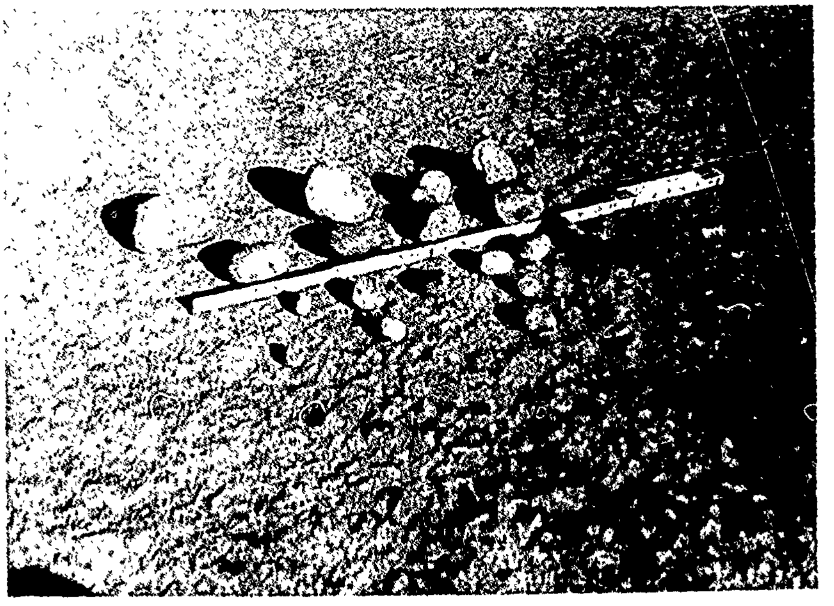


Figure 170. Typical abraded aggregate from debris deposits, Bull Shoals Dam (from Little Rock District 1953)

Table 4

Volume of Eroded Material and Flow Data Summary

Conduit Number	Total Eroded Volume cu yd	Total Discharge acre-ft	Total Time Operated days	Conduit Gate Opening ft	Total Horsepower days 1000's
21	42.0	7,350	21	0.4 to 0.5	100
22	14.5	417,570	224	0.5 to 9.0	5,020
23	0	0	0	0	0
24	8.5	230,400	36	9.0	3,180
25	6.5	108,800	17	9.0	1,590
26	11.0	300,800	47	9.0	5,570
27	0	17,040	45	0.5 to 1.0	50
28	11.5	371,200	58	0.4 to 9.0	4,580
29	13.0	185,600	22	9.0	1,730
30	8.5	160,000	25	9.0	1,780
31	10.0	140,800	22	9.0	2,140
32	6.5	224,000	35	9.0	2,220
33	0	0	0	0	0
34	19.0	224,000	35	9.0	4,390
35	0	5,800	91	0.5 to 1.0	150
36	8.0	218,400	52	0.5 to 9.0	3,860
	159.0	2,611,760			

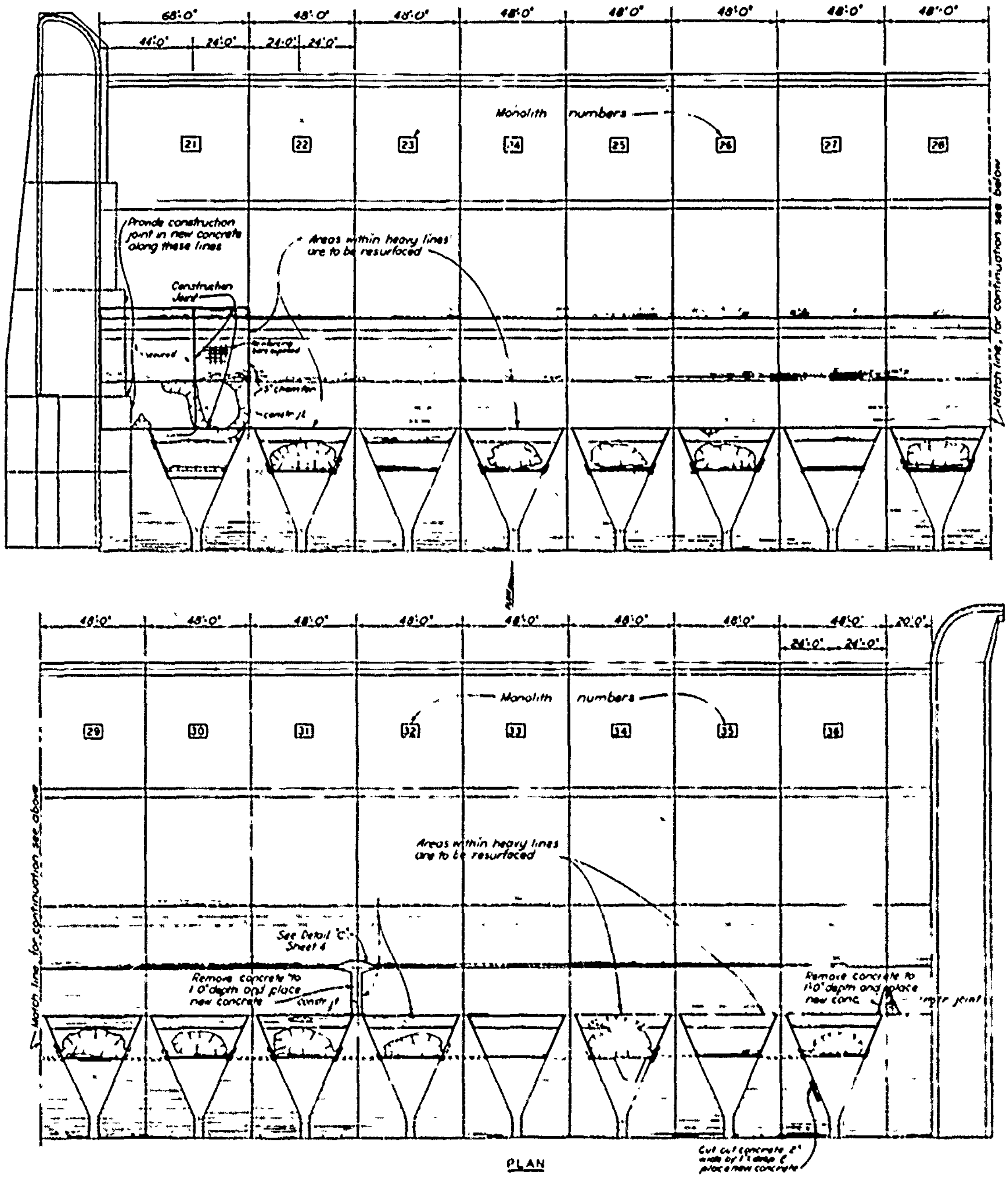
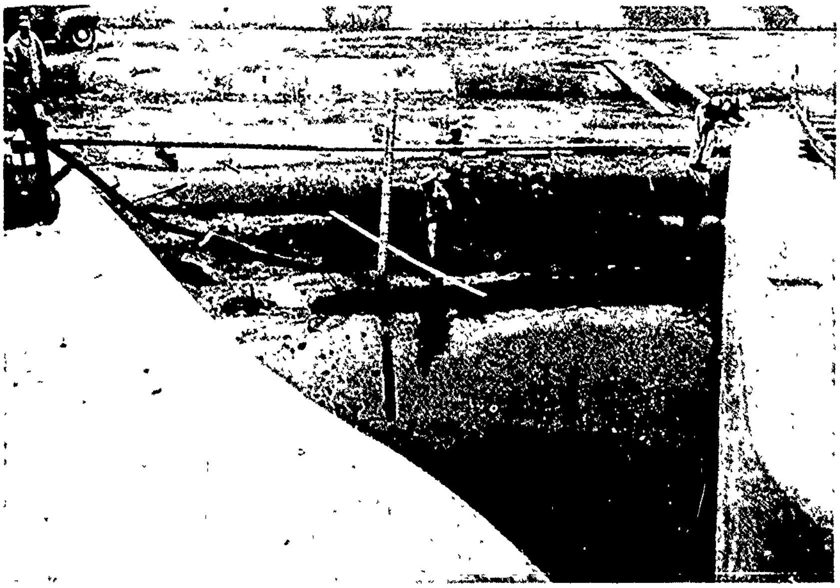
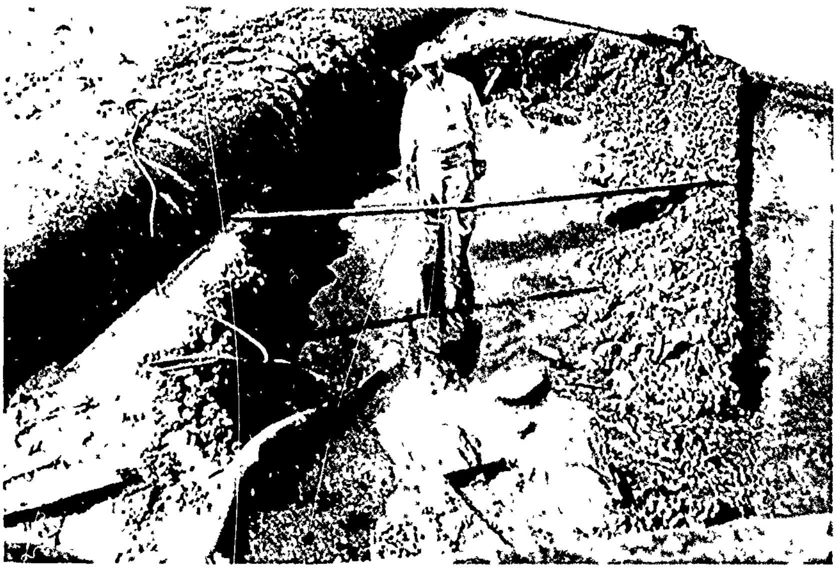


Figure 171. Areas of erosion and plan of repairs, Bull Shoals Dam (from Little Rock District 1953)



a. Downstream view



b. View parallel to steps

Figure 172. Erosion of first step and face of second step downstream from monolith 22, Bull Shoals Dam (from Little Rock District 1953)

Figure 173. Eroded first and second steps downstream of monolith 34, Bull Shoals Dam (from Little Rock District 1953)

of the first step downstream from conduit 35 and on the top of the second step downstream from several conduit outlets, there was small-scale pitting of the concrete. The location and nature of this pitting on the elliptical surfaces of the steps indicated that the initial stages of the damage were probably caused by cavitation. Following the initial damage, the direct impact of the jet upon the rough surface and the increased cavitation apparently caused the relatively rapid destruction of the steps. There was some evidence of impact abrasion on the faces of the steps downstream from the eroded areas. On the apron between and downstream from the steps, there was surface erosion that varied from slab to slab and indicated differences in the resistance of the concrete to abrasion. The large amount of damage downstream from conduit 21 in comparison with the relatively small amount of discharge is attributed to two factors: (a) the abrasive action of the gravel and debris previously eroded from other parts of the stilling basin and recirculated in this damaged area by eddy action, and (b) the relatively lower resistance to erosion of the monolith 21 apron concrete, as evidenced by the abrupt change in depth and extent of erosion at the apron joint between monoliths 21 and 22. The edges of the dividing walls between the

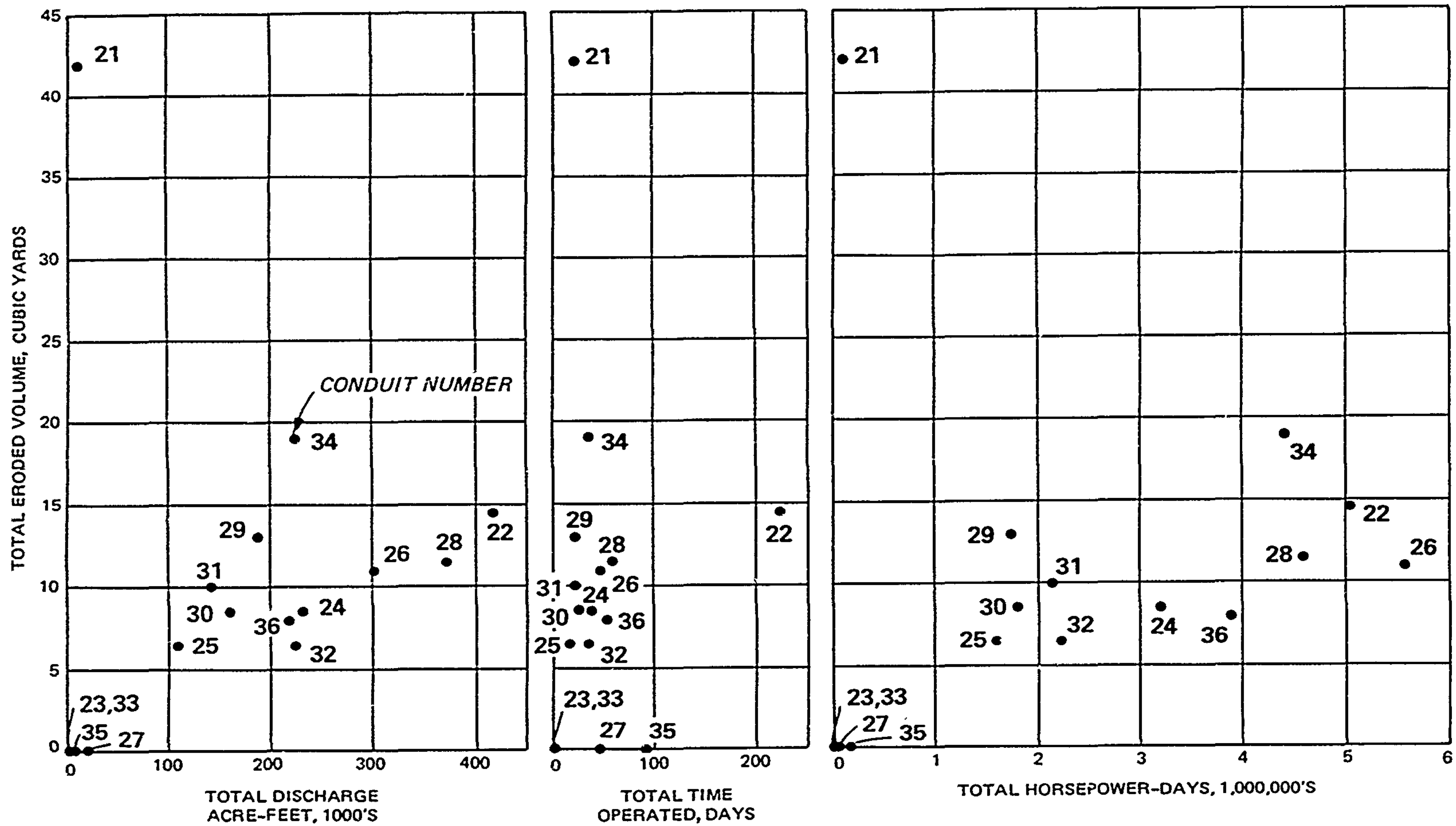


Figure 174. Relationships between erosion and flow, Bull Shoals Dam (from Little Rock District 1953)

conduit outlets were in good condition, indicating that there was no impact damage caused by the overfall nappe of the May 1950 flood.

322. A temporary repair of the basin while it was dewatered included cutting back the damaged steps and replacing them with smooth, upward sloping slabs. The slabs replaced the two upstream steps at conduit outlets 22 and 24-36 and all four steps at conduit outlet 21 (Figure 175). The undamaged steps at conduit outlet 23 were to be retained for testing purposes.

323. Concrete in the stepped areas was broken with paving breakers and was excavated by hand. Anchors were grouted into holes drilled on 4-ft centers each way and reinforcing mats were welded to the anchors (Figures 176 and 177).

324. After considering the extent of damage to the stilling basin concrete, it was deemed desirable to make repairs using concrete with a greater resistance to abrasion. Based on the theory that the use of fine aggregate manufactured from limestone tended to produce concrete with lower abrasion resistance, a natural siliceous sand as fine aggregate was used in the repair concrete. The sand was composed of about 83 percent subangular, slightly frosted, dense particles of quartz, about 10 percent of feldspar, and about 7 percent of other miscellaneous minerals, including chert, sandstone, mica, and granite. About 5 to 10 percent of the chert particles (less than 0.4 percent of the material) consisted of fibrous chalcedony, a mineral known to cause a deleterious reaction with the alkalies in portland cement. The coarse aggregate used in the repaired section of the stilling basin, obtained from stockpiles of the Flippin quarry, was identical in composition to that used in the original construction in the stilling basin.

325. Mixture proportions for 1 cu yd of the class "A Exterior" repair concrete were as follows:

Material	SSD Weight, lb
Portland cement, Type I	517
Fine aggregate	1239
Coarse aggregate (3/4 in. - No. 4)	1070
Coarse aggregate (1-1/2 in. - 3/4 in.)	1212
Water	212

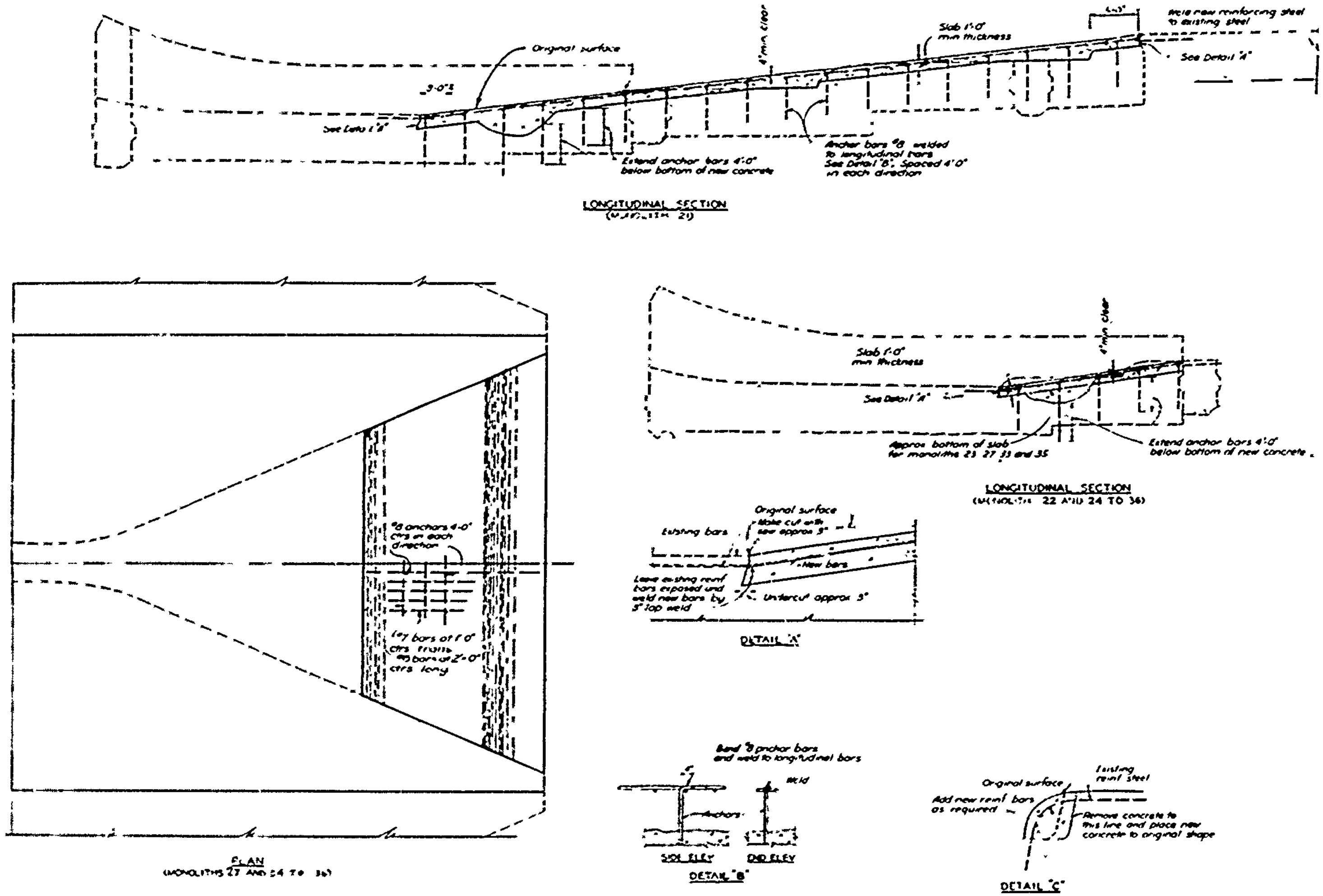


Figure 175. Details of stilling basin repairs, Bull Shoals Dam (from Little Rock District 1953)

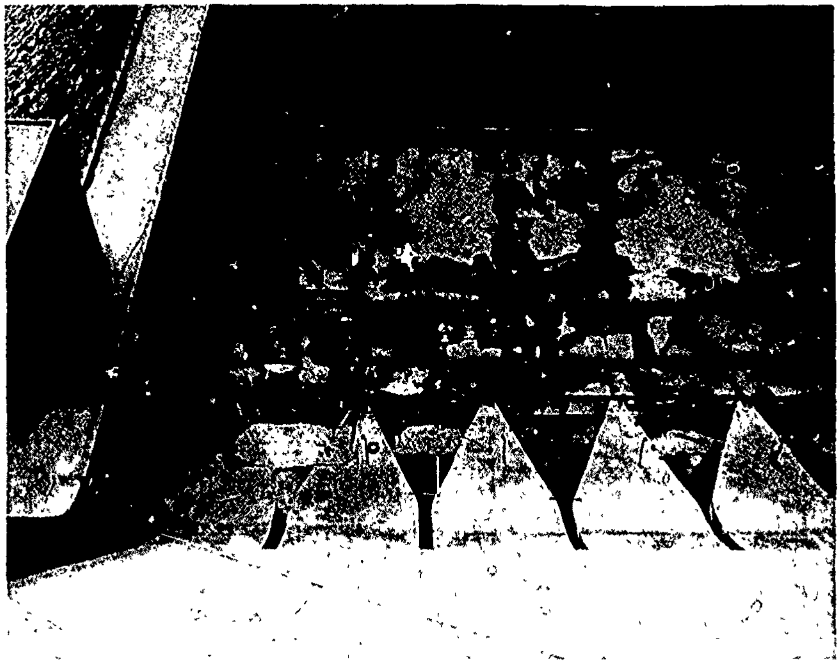


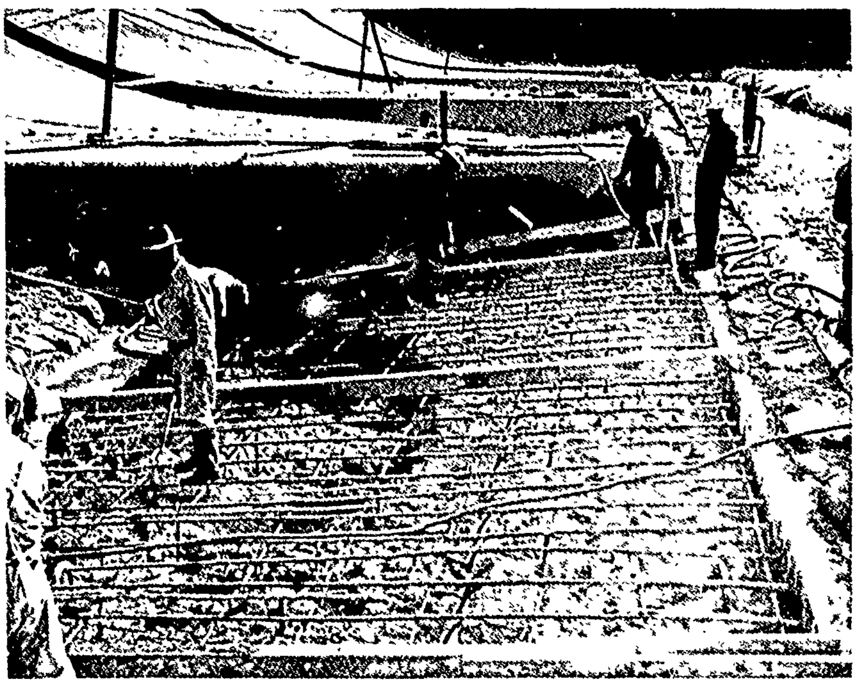
Figure 176. Repair operations downstream from monoliths 21-25, Bull Shoals Dam (from Little Rock District 1953)

No air entraining admixture was used. Average compressive strengths at 7 and 28 days were 3630 and 5820 psi, respectively. All concrete surfaces were given a hard troweled finish. Typical completed repairs are shown in Figure 178. The repairs were made by contract from November 1952 to January 1953 at a cost of $67,689, excluding dewatering, which was accomplished by hired labor at a cost of $11,947.

326. Although the original model studies of the stilling basin had not revealed the existence of any large negative pressures that might have caused cavitation, it was decided to install 16 pressure cells in the stilling basin in conjunction with the repair work. Manufacture and installation of the cells are described in detail by Campbell (1954).

327. Prototype tests, conducted on 28 January 1953, disclosed that pressure fluctuations downstream from the top of the first step, as originally constructed, were in the cavitation range for operations at half-gate opening; at full-gate opening, pressure cells at this position were pulled out of the concrete and no reading was possible. Cells located just downstream from the end of the temporary ramp covering the first two steps recorded pressures that were consistently negative.

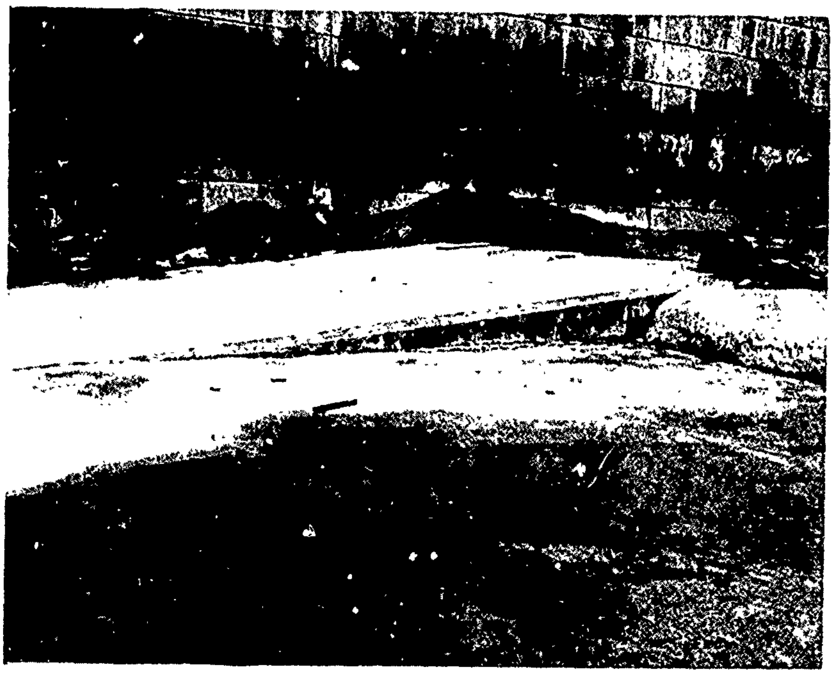
a. Concrete removal



b. Immediately prior to placing new concrete

Figure 177. Repair operations downstream of monolith 26, Bull Shoals Dam (from Little Rock District 1953)

a. Slab replacement of first and second steps, monolith 22



b. Slab replacement of four steps downstream from monolith 21

Figure 178. Completed repairs, Bull Shoals Dam
(from Little Rock District 1953)

Pressure fluctuations into the cavitation range also were noted near the beginning of the sloping ramp where positive pressures were expected. Consequently, additional model tests were considered desirable to further investigate the causes of damage to the prototype and to determine a satisfactory method of making permanent repairs to the stilling basin.

328. Tests were conducted during the period June 1953 to May 1954 (WES 1958) on a 1:12-scale conduit model and a 1:50-scale section model of the spillway. An upward-sloping ramp, replacing the first three steps of the stilling basin, proved most feasible as the basic scheme for permanent repairs to the prototype. Large and rapid pressure fluctuations still occurred at the upstream end of this ramp for conduit discharges but average pressures were positive. The existing 4-ft-high end sill was found inadequate for dissipating energy from conduit flows when the smooth floor ramp was installed. Raising the end sill height to 10 or 12 ft caused retention of deeper tailwater in the basin and resulted in improved stilling action. The higher end sills produced an adequate hydraulic jump under spillway discharge, but were somewhat less effective than the low end sill in preventing erosion in the exit channel. Baffles provided improved stilling action for spillway flows, but were of negligible value for conduit discharges. When used in conjunction with a 10- or 12-ft-high end sill, 8-ft-high baffles were found to be the most efficient.

329. Based on the results of these tests, a contract was awarded in June 1954 for construction of one row of vertically faced baffles, 8 ft in height (Figure 179). Also, the end sill was increased in height to 12 ft, and a 19.5-ft-long horizontal apron terminated by another 2-ft-high sill was provided downstream for protection of the exit area during periods of low flow. Total cost of the contract was $294,350.

330. The stilling basin was inspected by a scuba diver in August 1968. The concrete in the repaired section of the stilling basin floor extending downstream from the outlet sidewalls exhibited erosion estimated to range from 3/4 in. to as much as 2-3 in. deep. The deeper scars occurred along monolith joints. The erosion of the original concrete,

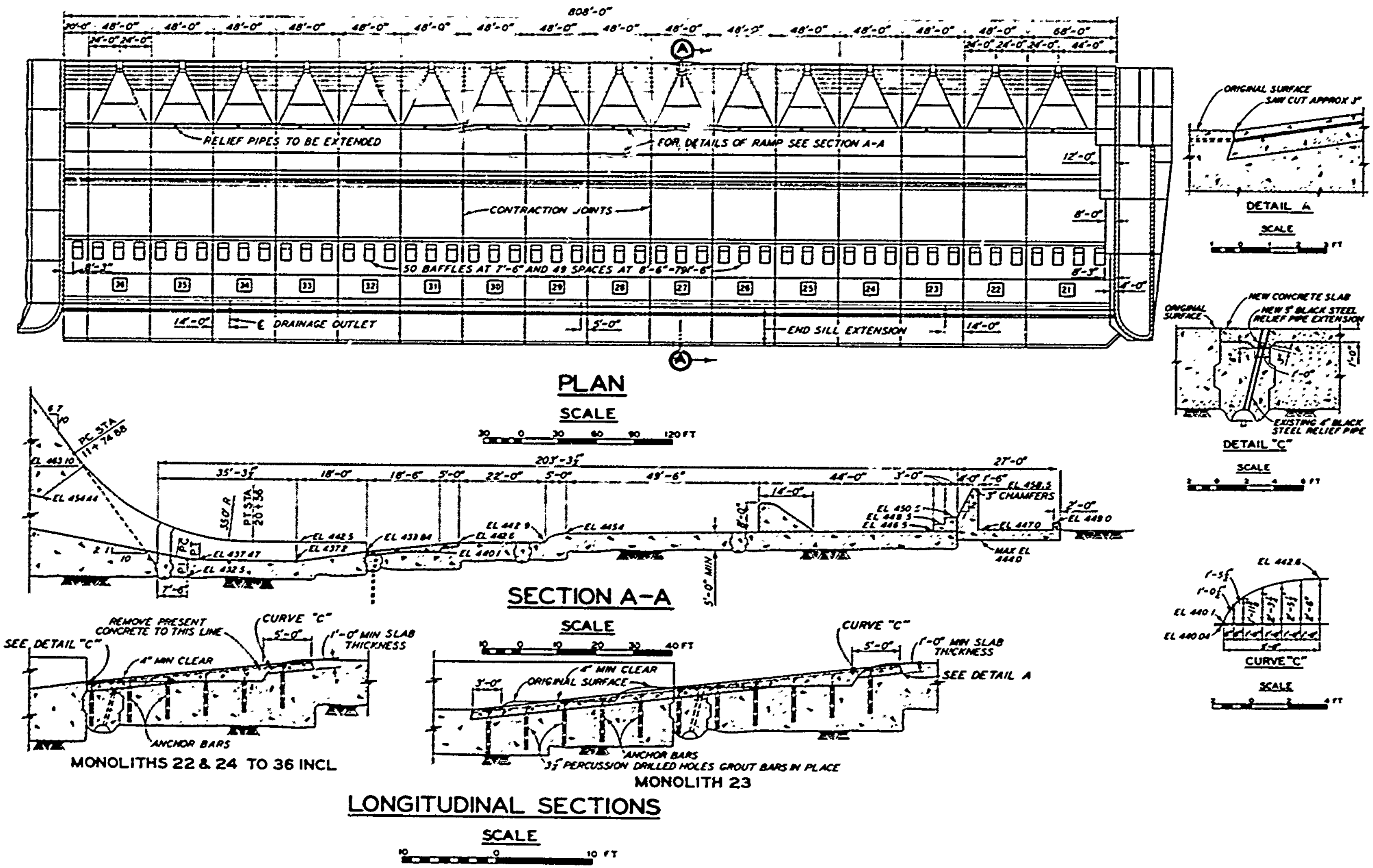


Figure 179. Modifications to stilling basin, Bull Shoals Dam (from WES 1958)

while ranging in depth similar to that of the repaired section, occurred in a more widespread pattern across the floor. The mortar appeared to have been removed from around the coarse aggregate, leaving it dislodged or protruding. A large collection of this aggregate was piled up on the floor of monolith 21. The aggregate was well rounded as a result of rolling and agitating across the stilling basin floor. The degradation noted along monolith joints was greatest at the intersection of the joints with expansion plugs and ogee sections and along the joints where they extended between baffles. Erosion was also common across the stilling basin along expansion plugs where surface irregularities were left during construction.

331. It was concluded that the erosion of the repaired sections of the stilling basin floor was minor in all areas except in monoliths 21 and 22 where pitting as much as 3 in. deep was observed. Reinforcing steel was exposed in a small section of monolith 21. The junction between new and old concrete showed signs of raveling. In general, the condition of the stilling basin as a whole was considered satisfactory. However, it was suggested that the areas in monoliths 21 and 22 upstream from the baffles should be observed frequently.

332. Based on the results of this inspection, the recommendation was made to study the feasibility of erecting low-height sills extending from the conduit sidewalls to the end of the stilling basin to prevent debris and eroded concrete and concrete aggregates from collecting into a large pile, as observed on the floor of monolith 21. For stilling basins as long as that at Bull Shoals, four low-height sills would control the migration of this debris and would reduce costs of repair work.

333. A diver inspection in October 1978 indicated that the concrete was in essentially the same condition as previously reported in the 1968 inspection.

Nimrod Dam

334. Nimrod Dam, completed in April 1940, is located on the Fourche La Fave River in the western part of Perry County, Arkansas, about

29 miles south of Russellville, Arkansas. The dam is a straight concrete gravity-type structure with a crest length of 1012 ft and a maximum height of 97 ft. A 22-ft-wide roadway extends across the top of the dam. The ungated spillway section is located across the natural stream channel and is flanked on both sides with nonoverflow sections extending to the abutments. The overflow section, crest el 373, has an overall length of 198 ft and is surmounted by six piers (Figure 180). Spillway design discharge velocity was 77 fps. Seven 6- by 7.5-ft, gate-controlled conduits through the base of the dam provide for the regulated release of floodwaters. These conduits have a total capacity of 17,300 cfs with the pool at spillway crest. In addition, there are two conduits with 60-in. Howell-Bunger valves installed in their discharge ends, which provide close regulation of low flow discharges.

335. The reinforced concrete stilling basin below the overflow section of the dam is 88 ft long with a 30-ft apron extending beyond the end sill (Figure 181). One row of 6-ft baffles, two division walls 14 ft high, and a stepped end sill 4 ft high are provided. The low division walls were provided to improve basin action when a small number of conduits are discharging by preventing the formation of destructive eddies and accompanying contraction of the outflowing stream. The basin floor slab (minimum thickness 3 ft), end sill, and divider walls were constructed using class B concrete. The baffles were constructed using class A concrete.

336. The stilling basin and conduits were dewatered and inspected in 1952, 1959, and 1966. The inspections indicated minor erosion and cracking in the conduits. During the 1952 inspection, localized erosion up to 5 in. in depth was found in the conduits. Consequently, until they were repaired in 1973, conduits 2, 3, 5, and 6 were not used except for routine flushing of all conduits. The 1959 inspection revealed a small area with reinforcing steel exposed in conduit 2. The 1966 inspection team recommended that the stilling basin be repaired while it was dewatered for the next inspection.

337. The stilling basin was dewatered for inspection and repair in October 1973. The concrete surfaces in conduits 1, 4, and 7 were in good

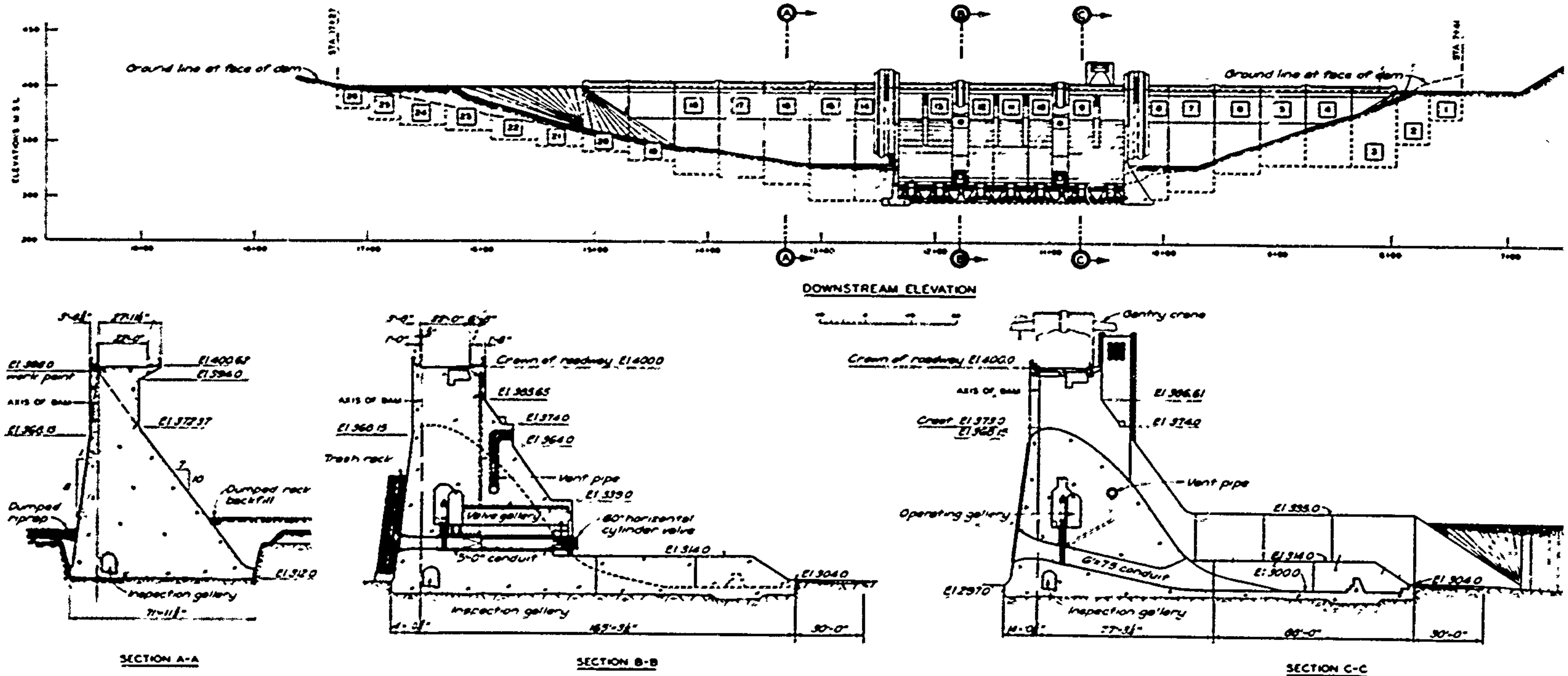


Figure 180. Downstream elevation and sections, Nimrod Dam

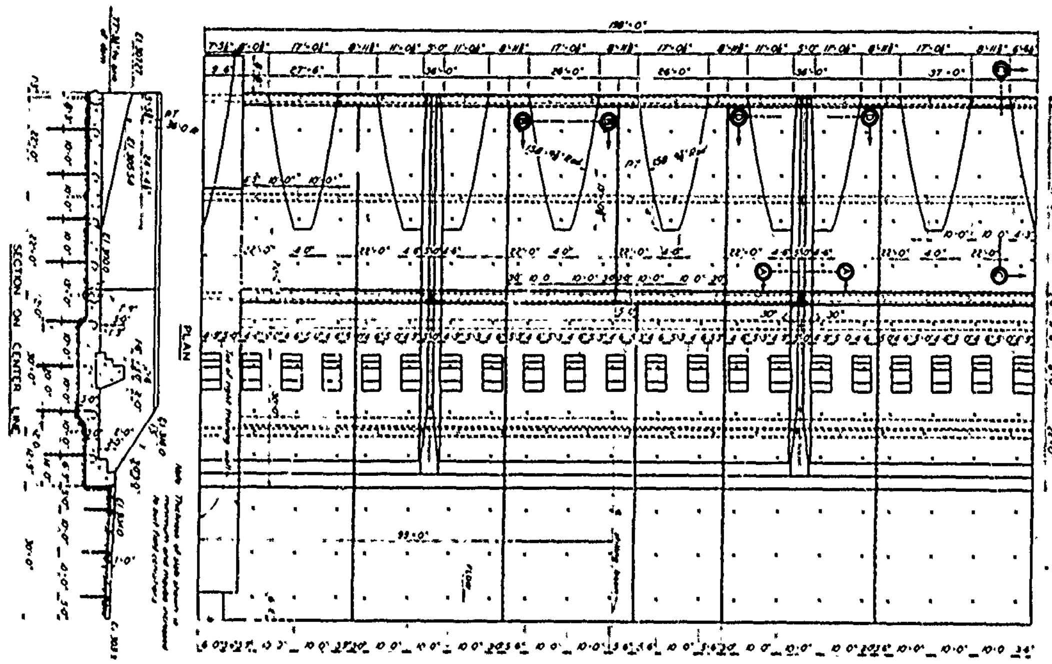


Figure 181. Stilling basin plan and section, Nimrod Dam

condition with no evidence of erosion. The remaining conduits all exhibited some concrete erosion. In each case the erosion began approximately 3 to 5 ft upstream of the stilling basin-dam construction joint and extended 15 to 25 ft downstream from the construction joint in the conduit outlet. The damage at the ends of each conduit is summarized in the following.

a. Conduit 2 was eroded to a depth of 8 in. in the center and reinforcing steel was exposed for a length of 4 ft and a width of 8 in., exposing two crossbars. Erosion was sloped from both sides and ends, extending over 20 ft in length.

b. Conduit 3 had erosion extending 18 ft in length and 7 ft in width and sloping to a depth of 8 in. in the center. No reinforcing steel exposed.

c. Conduit 5 had similar erosion to conduit 3 with the eroded area extending about 22 ft in length. Depth of erosion in the center was up to 4 in.

d. Conduit 6 was similar to conduits 3 and 5 with erosion extending about 24 ft in length and to a depth of 4 in. In addition to the erosion, approximately 25 cu yd of sandstone boulders stacked up to a 4-ft height and covering an area 30 ft long was found near the toe of the conduit slope. This material consisted of creek boulders (not cobbles) of irregular shape and size up to 200 lb. The probable place of origin was from upstream because some material may have worked down from in front of the conduit where top of rock is above the invert in conduit 7 and 5 ft below the invert in conduit 6.

338. After partial dewatering of the entire bay, sandbags were placed in the exit of each conduit and a small pump was used to finish dewatering. The construction joint leaked water along the floor and sidewalls, making complete drying of the entire eroded area difficult. After sandblast cleaning and drying with a butane burner, two-component epoxy was spread on the eroded areas and allowed to become tacky. Concrete consisting of one part Type I portland cement, two parts sand, three parts crushed rock (3/4-in. MSA), and water to desired consistency was then placed. Upstream edges were feathered and the downstream edges squared off about 1 in. in height. The concrete was allowed to set at least 4 hr prior to flooding. Dewatering and repair of the stilling basin was accomplished by personnel from the Little Rock District at a total cost of $9045.

339. An inspection in September 1978 indicated a small amount of concrete erosion in the stilling basin. Some of the baffles had rounded corners. There was no exposed reinforcement. The repairs appeared to be performing satisfactorily and it was concluded that the general condition of the stilling basin was good.

Table Rock Dam

340. Table Rock Dam, completed in August 1958, is a dual-purpose flood control and power project located on the White River in Taney and Stone Counties, Missouri, 8 miles southwest of Branson, Missouri. The 1602-ft-long dam is a concrete gravity structure with a maximum height of 261 ft above the foundation. The concrete structure, flanked on each side by embankment sections, includes an overflow spillway, crest el 896, with nonoverflow sections on each side. The intake and penstocks for supplying water to the generating units are located in the nonoverflow

section to the left of the spillway. The spillway (Figure 182) has 10 tainter gates 45 ft wide by 37 ft high for passage of extreme flood flows. Spillway design discharge velocity is 125 fps. Four flood-control conduits, each 4 ft wide by 9 ft high, in the overflow section of the dam pass flood releases when the pool elevation is below the spillway crest.

341. The concrete stilling basin (Figure 183) is a conventional type basin with a horizontal apron, two rows of baffles, and an end sill. The width (531 ft) is the same as that of the overflow section. The sides are formed by the left and right training walls. The length of the basin is 251 ft. The 5-ft-thick apron is firmly anchored into a rock foundation. Hydrostatic pressures are reduced by drain holes and relief pipes in the apron. Each baffle is about 9 ft 2 in. wide, 14 ft long, and 12 ft high. The baffles are placed on 21-ft, 4-in. alternate lateral spacing, and the two rows are 65 and 93 ft from the downstream end. The end sill across the entire basin width is 7.5 ft high and 12 ft long.

342. A major flooding of the White River occurred from May to July 1957 prior to completion of the dam. When this occurred, the non-overflow portions were at final grade and the overflow portion, with the exception of monoliths 18, 19, and 20, was at spillway crest el 896. Monolith 19 was the low block (el 877.5) at this stage of the construction and was overtopped by the pool from 24 May to 15 July 1957. The maximum amount of overtopping was 18.5 ft. Construction of the four flood-control conduits had been completed and the slide gates were open for the entire period. Total outflow was about 26,000 cfs, of which about 12,000 cfs was passed through the four conduits and about 14,000 cfs was passed over the three low monoliths. Tailwater elevation was 716, which is about 6 ft higher than normal with four conduits discharging. The gates of the two conduits located in monoliths 17 and 18 were opened full (9-ft opening), and the gates in the conduits in monoliths 16 and 19 were set at 8-ft openings.

343. When the gates in monoliths 16 and 19 were opened full, loud noises that sounded like metal pounding on metal were heard (WES 1959b). The noises occurred at irregular intervals of 1-2 minutes in the

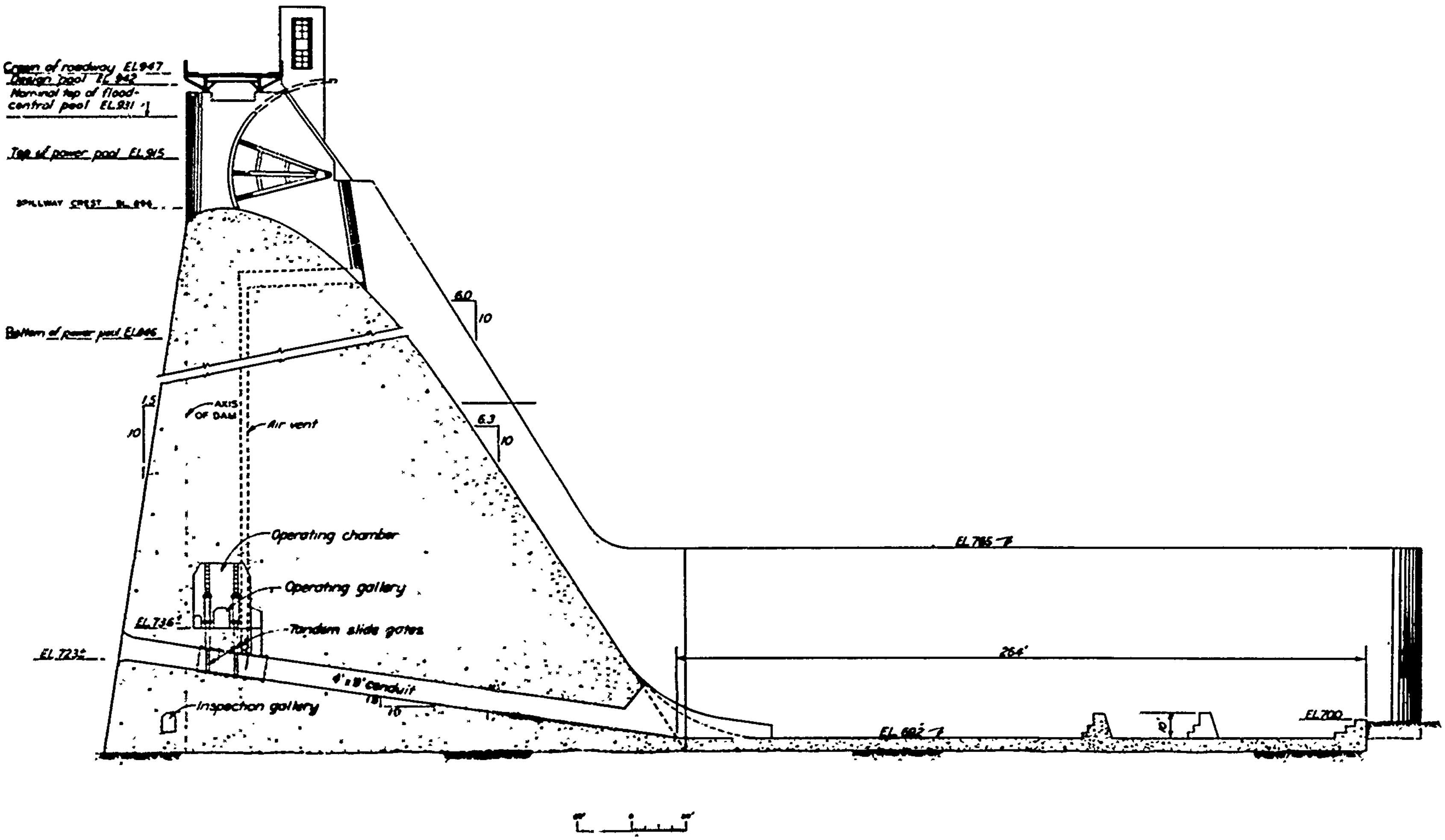


Figure 182. Section through sluiceway, Table Rock Dam (from Little Rock District 1954)

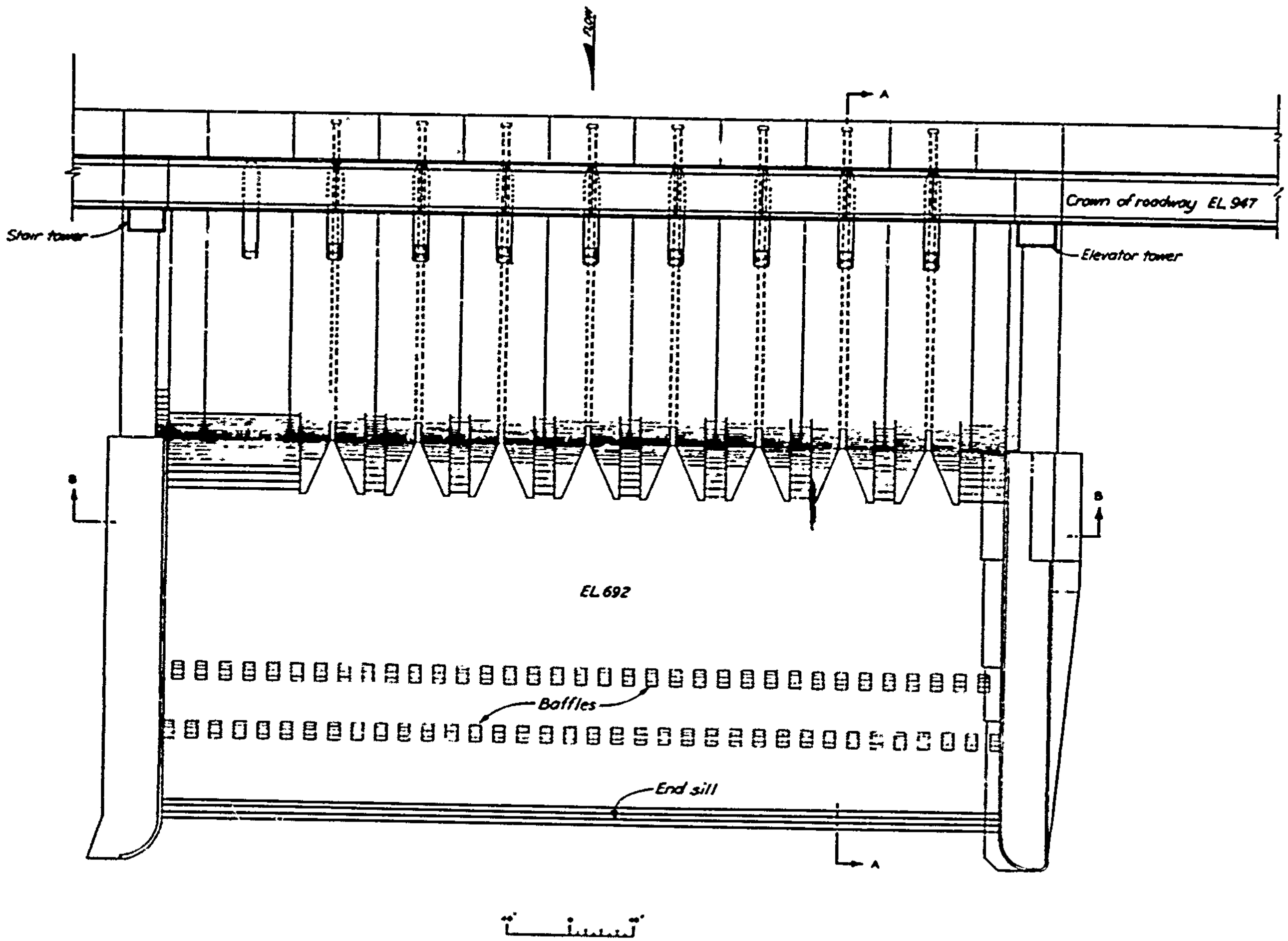


Figure 183. Plan of spillway and stilling basin, Table Rock Dam (from Little Rock District 1954)

monolith 16 conduit, and at regular intervals of about three per second in the monolith 19 conduit. The outlets of both conduits were submerged by eddy flow in the stilling basin and flow over the low monolith. The closure of both gates a few tenths of a foot eliminated the noise.

344. The action observed at the prototype conduit outlet during the high tailwater condition was reproduced in a 1:12-scale model. The undesirable pressure fluctuations observed are discussed in WES (1959b). Remedial measures determined by the model tests were not implemented because the high tailwater causing the pressure fluctuations would not occur after construction was completed.

345. The stilling basin and flood-control conduits were initially inspected in December 1957. During the period of uncontrolled flow, construction materials were carried into the stilling basin and resulted in damage to some of the concrete surfaces. Damage included two eroded areas in the conduits in both monoliths 16 and 19. Both eroded areas in monolith 16 were about 4 in. in diameter and 3 in. deep; they were located at each upper corner of the outlet. The eroded areas in monolith 19 occurred on both sidewalls at the conduit outlet. These areas were about 2 ft high by 4.5 ft long and had a maximum depth of 4 in. and an average depth of 1.5 in. One baffle had exposed reinforcing steel. The erosion on the sidewalls of the conduit outlet in monolith 19 was attributed to cavitation.

346. The stilling basin was again unwatered for inspection and repair in November 1959. During this inspection, an eroded area was noted at the outlet of the conduit in monolith 19 near the stilling basin slab on the right side of the flared wall. Project personnel repaired this area by removing loose particles and filling the void with epoxy resin concrete. The eroded areas in monolith 16 noted during the 1957 inspection were repaired in the same manner.

347. The eroded areas in monolith 19 noted in 1957 were repaired by removing concrete to a 4-in. minimum depth and filling the void with epoxy resin concrete. The edges of the patched areas were saw cut to ensure a smooth edge. The baffle with exposed reinforcing steel was repaired by anchoring reinforcement bars to the exposed steel and

then patching with epoxy resin concrete.

348. The horizontal scars on the upstream row of baffles opposite the sluiceways were not considered serious and therefore were not repaired. These scars are typically one to three horizontal grooves 4-7 in. wide and up to 4 in. deep. When more than one groove occurs on the face of the concrete, they are approximately parallel. Most of the grooves tend to be deeper and wider on the right front side (steep abutment side) of the baffles than on the powerhouse side. These grooves may have been inflicted by 3- or 4-in.-diameter pipes swept across the stilling basin during the 1957 flood. The appearance of the grooves indicates that the velocity of the water caused the pipes to vibrate against the concrete, thereby etching horizontal grooves on the upstream faces of the baffles.

349. The stilling basin was unwatered and inspected for a third time on 29 and 30 October 1968. The damaged baffles appeared to be the same as they were in 1959. Patches on the splitter block and sluiceways were intact and in good condition except for one, which was dislocated, on the right flared wall of monolith 19. The eroded area was approximately 6 by 2 ft and was considerably worse than it had been at the 1959 inspection. This damaged area was patched during the 11 April 1972 inspection, using the 1959 repair specifications as guidelines. A dislocated patch was also found near the basin floor in monolith 19 and was repaired. The cost of dewatering and repairs made in 1972 by project personnel was $4875.

350. The 1972 inspection team recommended that in the future the stilling basin be flooded in such a manner to prevent material in earthen cofferdams from being washed back into the basin. They stated that this could be accomplished by allowing the basin to gradually flood with seepage water or by cutting the cofferdam when a release is not being made through the powerhouse turbines. The next inspection of the stilling basin is scheduled for October 1982.

Oologah Dam

351. Oologah Dam is located at river mile 90.2 on the Verdigris

River, 25 miles northeast of Tulsa, Oklahoma. It consists of a 4000-ft-long rolled earth-fill embankment, a concrete gate tower controlling flows through two 19-ft-diameter concrete conduits, a gated saddle spillway, and a saddle dike. Construction of the dam was accomplished in two stages. The first stage, consisting of the embankment, outlet works, and uncontrolled saddle spillway, was completed in 1964. It provided for flood control and a minimum amount of conservation storage. The gated spillway and saddle dike were built during the second stage of development and were completed in 1969. The conservation pool was then raised 30 ft to provide the navigation storage and additional water supply. The ultimate conservation pool elevation of 638.0 was reached in July 1972.

352. The outlet works stilling basin is shown in plan and section in Figure 184. The stilling basin receives the discharge from the two 19-ft-diameter conduits. The basin is a rectangular, hydraulic jump box type structure, 114 ft long and 110 ft wide. A flared transition section containing a parabolic drop from exit invert el 560.0 to the apron el 549.0 connects the stilling basin to the exit portals of the conduits. A center wall divides the stilling basin into halves so the conduits can be operated together or separately with equal dissipating action. The top of this center wall was set at el 583.5, which corresponds to the tailwater elevation with only one conduit operating at maximum capacity. Tops of the training walls are at el 593.0, which is the tailwater elevation with both conduits operating at maximum capacity. The velocity in the stilling basin at maximum discharge is about 60 fps. The past 13-year average discharge through the conduits was approximately 2,600 cfs. A maximum discharge of 29,500 cfs was recorded in October 1975. Frequent discharges of 15,000 to 20,000 cfs have been made since the pool was raised to its ultimate conservation level in 1972. The stilling basin concrete was designed for a minimum 28-day compressive strength of 3000 psi and a 3-in. maximum aggregate size. During the stilling basin repair, cores were taken at several locations, and all tests indicated a strength in excess of 4000 psi and in most cases the strength exceeded 5000 psi.

353. The first inspection of the stilling basin, made in November

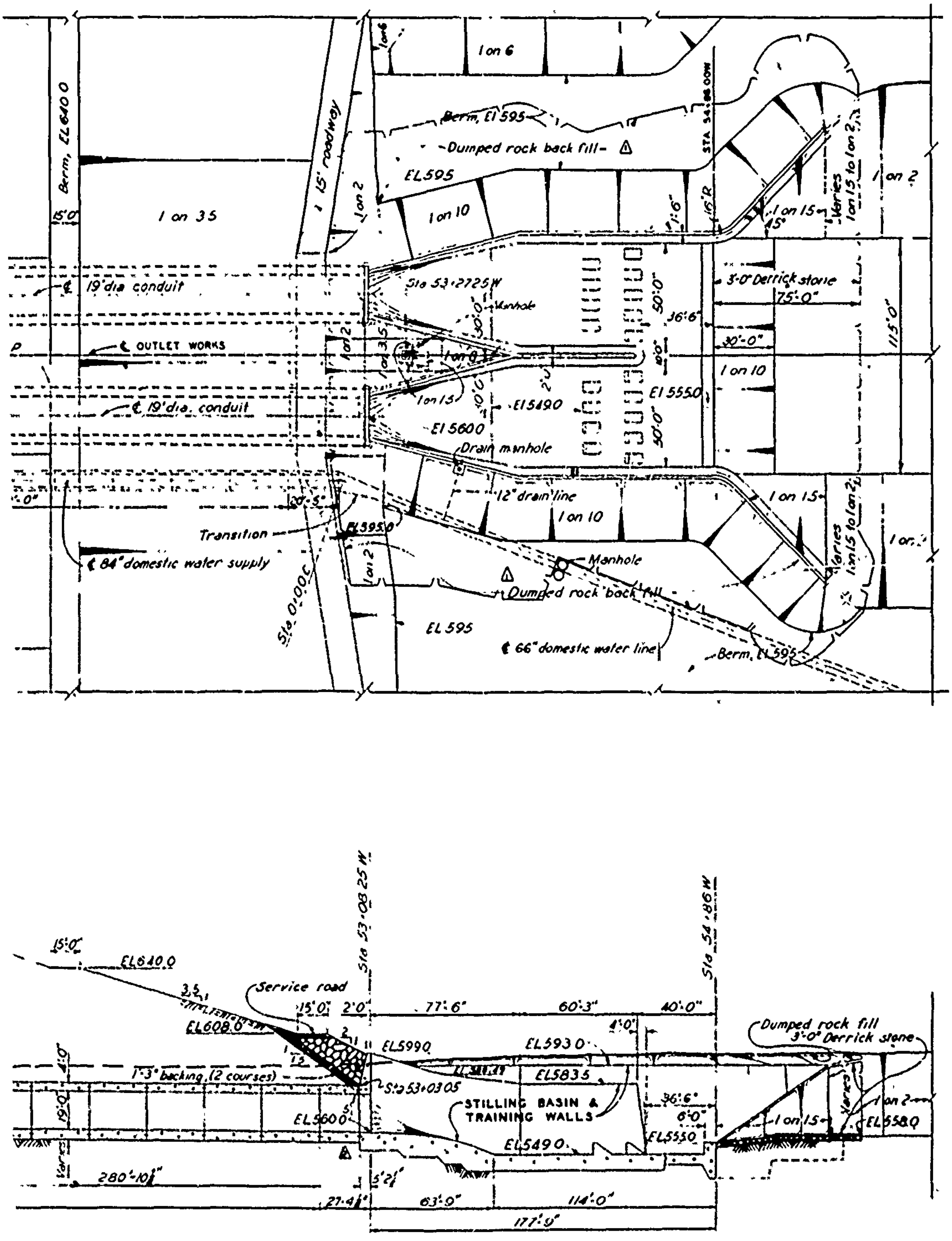


Figure 184. Outlet works stilling basin, Oologah Dam

1970, revealed only minor areas of scour on the floor and three baffles with cavities deep enough to require repair. Repair were made by installing anchor bolts and filling the cavities with epoxy mortar. The second inspection of the basin in September 1975 revealed that a large amount of damage had occurred since the first inspection and presumably since the pool was raised in 1972. In the right basin, the erosion of the floor was deep enough (17-in. maximum) to expose and remove reinforcing steel from two separate layers (Figure 185). In comparison, the left basin lost very little reinforcement but most of the top layer was exposed (Figure 186). Most of the baffles had exposed reinforcing but the erosion was generally confined to about 18 in. at the base (Figure 186). It was concluded that erosion of the stilling basin concrete resulted from debris trapped in the basin.

354. The stilling basin was dewatered in June 1976 to conduct repairs. Basically, the dewatering scheme consisted of constructing an earth cofferdam about 300 ft downstream of the end sill and pumping the basin dry. However, several items had to be accounted for: (a) water quality releases of 18 cfs had to be maintained in the downstream channel; (b) the city of Collinsville, Oklahoma, was pumping about 1 mgd from a pump well adjacent to the stilling basin, and the intake into this well is through the right stilling basin wall; and (c) leakage around the gates and valves in the gate tower resulted in a small but continuous flow of water into the stilling basin. After the basin was dewatered, a minor problem resulted from seepage through weep holes and joints in the walls from the backfill. The seepage was intercepted and channeled into pipes where it was carried out of the basin by gravity flow.

355. Very little material had to be removed to prepare the surface of the old concrete to be overlaid. Near the toe of the parabolic drop a saw cut was made, and some concrete was removed to avoid a featheredge of the overlay. In the right basin where some reinforcing bars had to be replaced, concrete was removed from the ends of the old bars to provide sufficient lap distance for the new bars. Where holes were to be patched in the baffles and walls, the areas were saw cut to avoid featheredging the epoxy mortar. Other than these areas, only loose or unsound concrete

a. Loose reinforcement removed from right basin

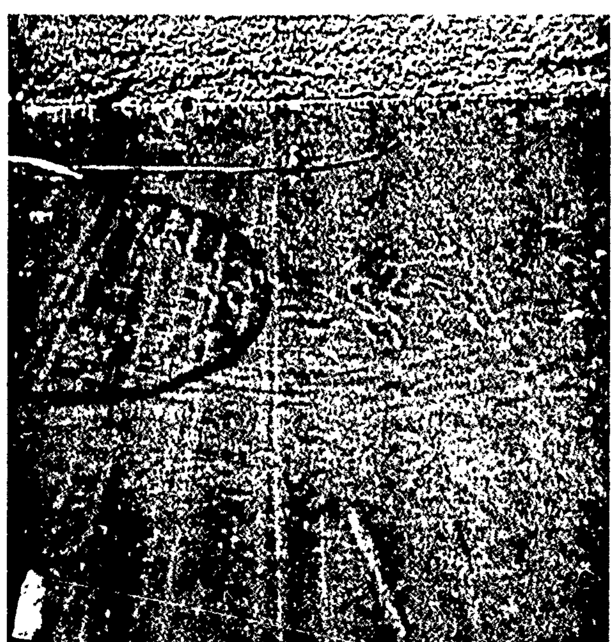
b. Stilling basin after removal of loose reinforcement

c. Floor of right basin. Note broken rebars

Figure 185. Right stilling basin damage, Oologah Dam

a. Typical baffle damage

b. Severe baffle damage



c. Floor of left basin. Note exposed rebars

Figure 186. Left stilling basin damage, Oologah Dam

had to be removed. Other preparation necessary prior to placing the overlay included grouting No. 4 anchors into the old concrete, extending the slab weep holes (Figure 187), replacing No. 11 bars in the right basin, installing temperature reinforcement for the overlay concrete, and cleaning the old concrete surface by sandblasting.

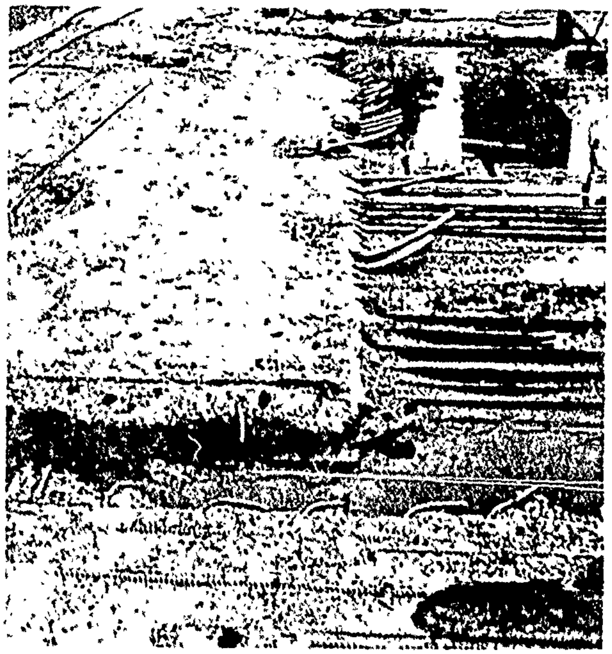
356. An 18-in.-thick concrete overlay was added on the basin floor between the toe of the parabolic drop and the downstream edge of the baffles (Figure 188). From that point to the end sill the thickness of the overlay was reduced to 6 in. This provided adequate depth for the formation of the hydraulic jump and provided a collection basin for debris. A 45-deg batter was added on the upstream face of the end sill (Figure 188) in order to facilitate removal of debris by higher discharges. The baffles received a 1-ft overlay on the upstream face and top (Figure 189). All overlays were anchored to the old concrete with both steel anchors and with epoxy grout. A concrete slab was added around the outside of the stilling basin walls to prevent the rock fill from being washed into the basin by high discharges.

357. Conventional concrete (3/4-in. MSA) with an average compressive strength of 5950 psi was used in the repair. The mixture proportions for 1 cu yd of concrete were as follows.

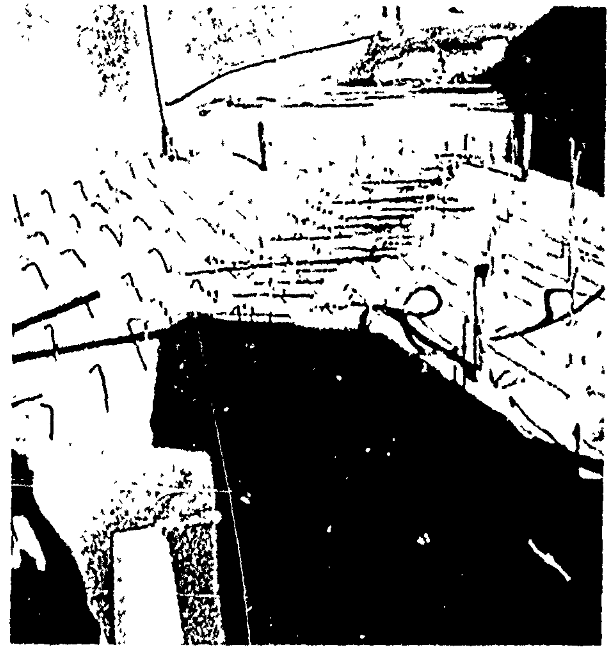
Material	Amount
Cement	658 lb
Coarse aggregate, crushed limestone	1730 lb
Fine aggregate, natural sand	1260 lb
Water	31 gal.
Water-reducing admixture	20 oz.
Air entraining admixture	2 oz.

Reinforcing steel for the overlay conformed to American Society for Testing and Materials (ASTM) A 615 (1978), grade 40. Two component epoxy resin systems were used.

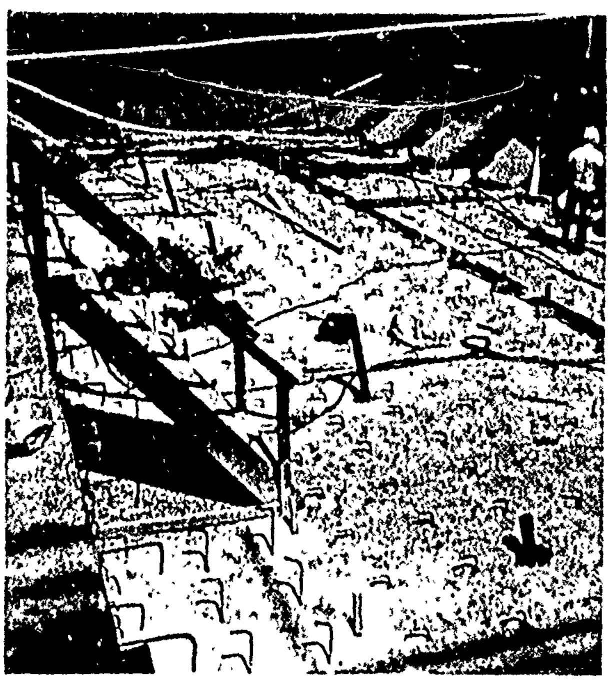
358. After the stilling basin slab had been prepared, a placement was formed and sandblasted. The placement was blown clean and a vacuum cleaner was used to pick up dust, sand, etc., from holes, cracks, and corners. The prepared concrete was kept dry until the epoxy grout had



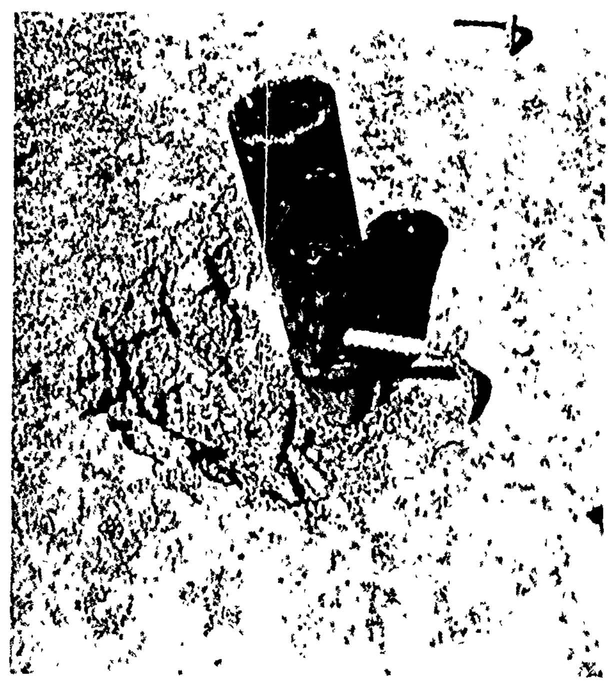
a. Floor of basin during preparation for overlay. Note holes drilled to receive anchors



b. Floor of basin with anchors and drain extensions in place

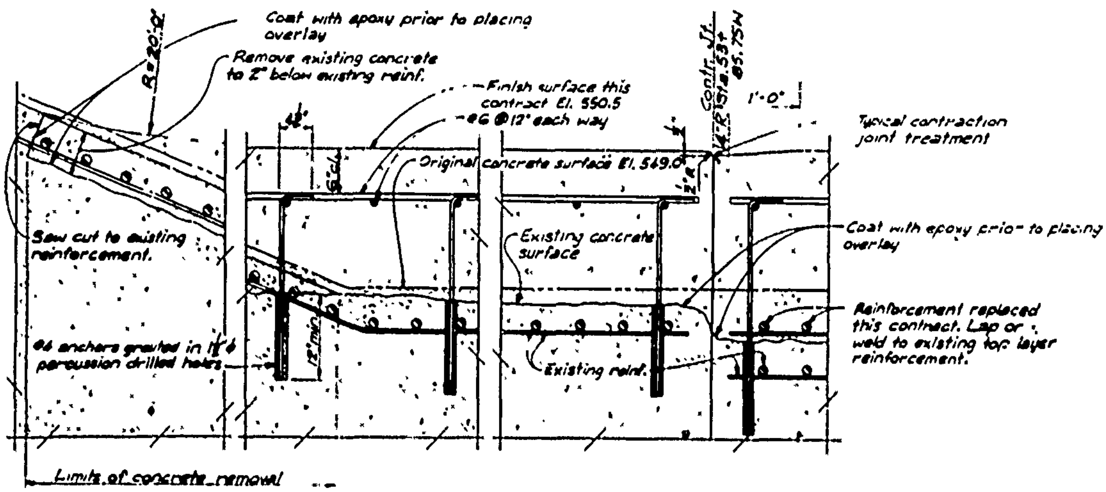


c. Floor and end sill being prepared for overlay

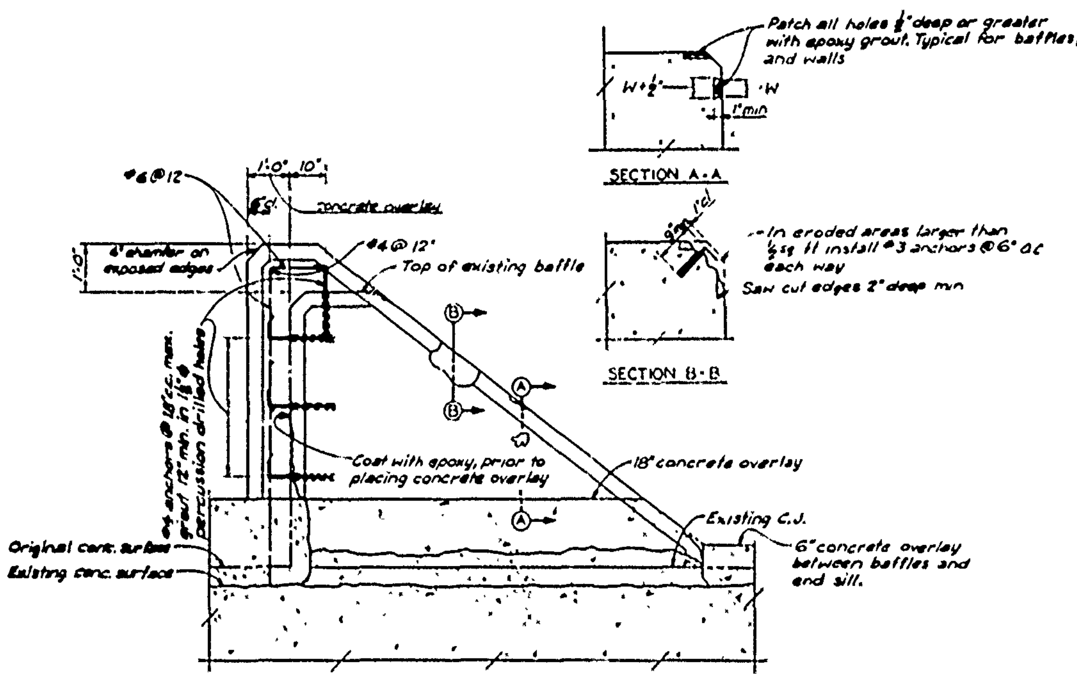


d. Weep holes from foundation drains extended through overlay

Figure 187. Stilling basin repair, Oologah Dam



a. Slab repair detail



b. Baffle repair detail

Figure 188. Repair details, Oologah Dam

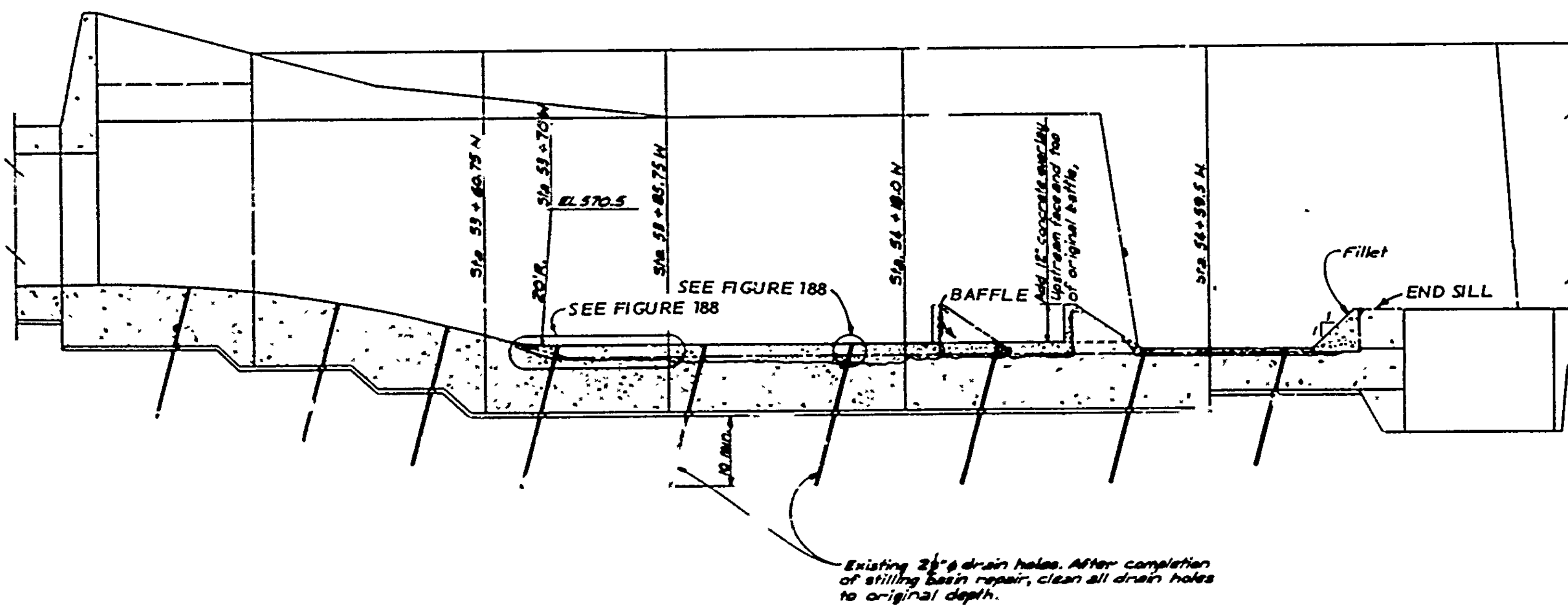


Figure 189. Stilling basin repair, Oologah Dam

been applied. On the 6-in. overlay placement, the reinforcing mats were made up but not secured in the placement until the epoxy grout had been applied. The reinforcing in the 18-in. overlay placement was secured in place and every other bar was removed as the epoxy was applied, then replaced.

359. The two components of a high-modulus epoxy grout were mixed together in a 5-gal plastic bucket for 3 minutes, using a variable speed 1/2-in. drill with a paddle recommended by the epoxy manufacturer. The epoxy was spread over the prepared concrete surface using a 5-gal plastic bucket with 1/4-in. holes on 1/2-in. centers drilled in the bottom. The epoxy was then scrubbed into the surface with a stiff-bristled push broom. A stiff-bristled hand brush was used to apply epoxy on areas not accessible to the broom. Workers wore golf shoes while spreading the epoxy to keep from tracking it. The epoxy was kept above 60°F before being mixed. At lower temperatures, the epoxy became thicker and difficult to spread. One gallon of epoxy was spread over approximately 50 to 60 sq ft. It was allowed to set for 30-60 minutes, depending on temperature, until it became very tacky. This setup time prevented the epoxy from losing bond to the old concrete during the placement and vibration of the fresh concrete. If the epoxy started to become gummy before concrete could be placed on it, another coat was applied. On a few small areas where existing construction joints were making water, the free water was removed and epoxy was applied immediately on the damp surface. The epoxy seemed to adhere to the surface.

360. The concrete was delivered to all placements by concrete buggies, and it was vibrated and finished in the conventional way. The overlay concrete on the basin floor was cured with a curing compound. After all placements had been made for the slab overlay, the baffle overlays were placed. The baffles were cleaned, epoxy was brushed on, reinforcing was placed, and the preassembled forms were set in place and secured. The overlay on the baffles and end sill, which was to receive a protective coating, was moist cured for a minimum of 7 days.

361. Within 48 hours after the protective coating was applied, the new concrete was lightly sandblasted to provide a hard, clean surface.

Any remaining dust was blown off and the surface was kept dry for 24 hours prior to applying the coating. The epoxy primer was mixed according to the manufacturer's directions and was applied to the concrete surface with a short-knap roller. The air temperature was in the low 50's and the concrete temperature about 40°F during the application. The concrete was covered and the air temperature kept above 50°F for a minimum of 24 hours. The epoxy coating was then applied in a similar manner and protected from cold weather as necessary.

362. Three baffles were designated for testing with protective coating material furnished and applied by WES personnel. Two polyurethane coatings were used, and each of the three baffles received a different prime coat. The test baffle coatings were applied in a similar manner to the other coatings except that the polyurethane top coats were applied with a sprayer. The stilling basin was kept dry for 4 days following completion of the coatings to allow the epoxy to cure.

363. No serious problems were encountered during the repair conducted between June and December 1976, and the total contract cost was $273,000. Perhaps the most persistent problem was keeping the basin dry enough to apply the designated methods and materials. Close inspection, good quality control, and numerous materials tests indicate that a quality repair job was obtained.

364. The stilling basin was dewatered for inspection of the repairs in March 1979. In general, the 18-in. slab overlay was in good condition. In each side of the basin there was an area of slight erosion approximately 10-15 ft wide extending the length of the overlay. Each area, located directly downstream of the conduits, exhibited general erosion of 3/8 to 1/2 in. deep. Maximum erosion, along construction joints (Figure 190), was approximately 3/4 to 1 in. deep. Although there was a small amount of debris upstream of the baffles in the left side of the basin, the majority of the debris was downstream of the baffles where the thickness of the overlay was reduced to 6 in. to provide a debris trap (Figure 191). General erosion within the debris trap appeared slightly more severe than in the remainder of the basin. There was no erosion of the end sill.

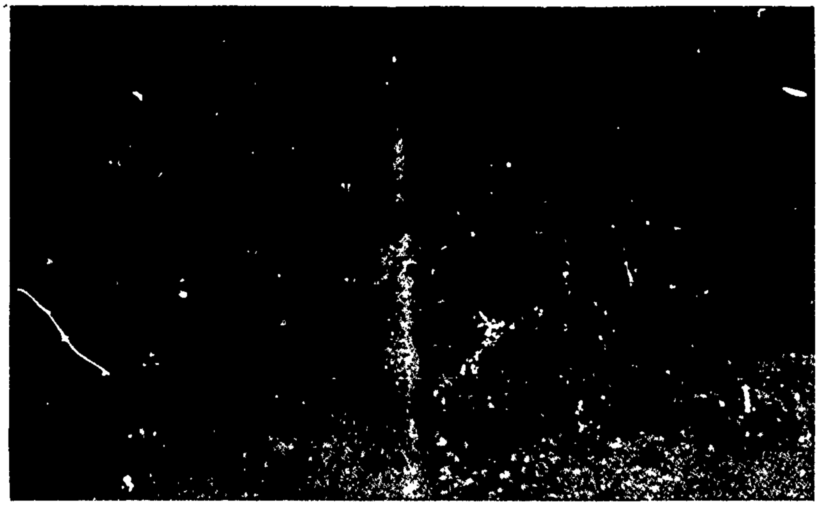
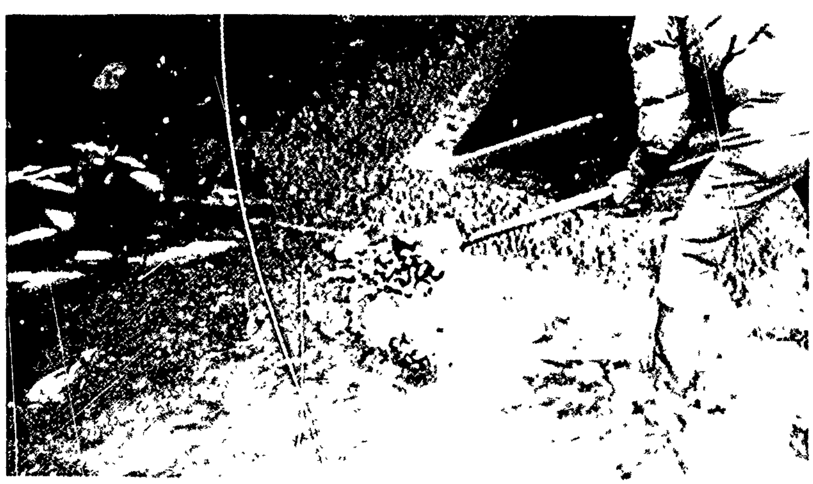


Figure 190. Erosion along construction joints, Oologah Dam

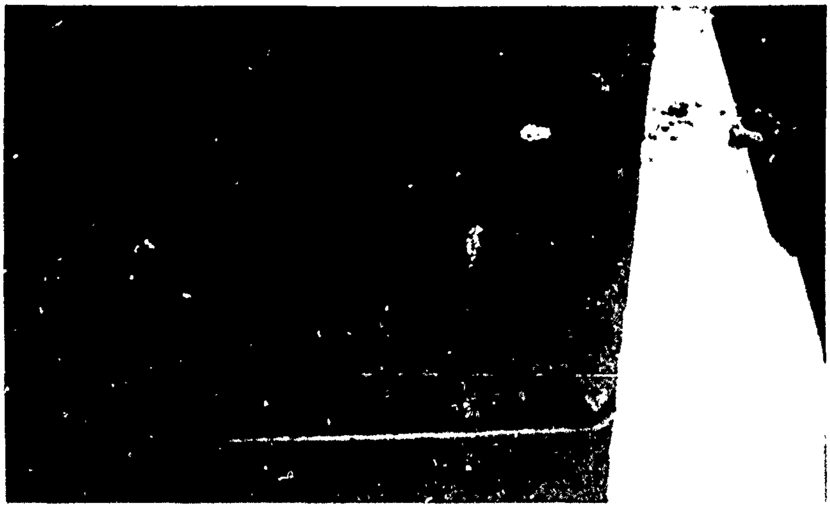
a. Debris upstream of the baffles



b. Removal of debris from the debris trap

Figure 191. Typical debris in the stilling basin, Oologah Dam

365. In general, all coatings on the baffle overlays sustained some damage, particularly on the lower one third of the baffle (Figure 192). The polyurethane coatings exhibited many small blisters and a few areas of bond failure between the coating and the primer.



a. Epoxy coating damage

b. Polyurethane coating damage

Figure 192. Typical damage to coatings on baffle overlays, Oologah Dam

Webbers Falls Dam

366. Webbers Falls Lock and Dam is a unit of the McClellan-Kerr Arkansas River Navigation System. It is located on the Arkansas River at navigation mile 368.9, about 5 miles northwest of Webbers Falls, Oklahoma. The project consists of an embankment, lock, spillway, and powerhouse. The embankment is a rolled earth-fill section about a half-mile long, extending from the left abutment to the river channel. It has a minimum height of 84 ft above the streambed. The spillway extends across the left half of the existing river channel, with the powerhouse in the right half. The spillway is a gated, concrete, ogee-type structure. The weir is surmounted by twelve 50-ft-wide by 41-ft-high tainter gates. The gates are separated by eleven 10-ft-wide intermediate piers that also support a 5-ft-wide personnel bridge. The lock is a 30-ft normal lift, Ohio River type with a culvert and port filling system and side-outlet discharge. The lock is located in the left bank of the channel with excavated approach channels. A general plan of the project is shown in Figure 193.

367. Construction of the $83 million project was divided into two principal phases. The first phase, begun in January 1965, was the construction of the embankment, lock, and the major portion of the spillway. With the completion of the first phase, the river was diverted through the lock during second-phase construction consisting of the completion of the spillway and the first stage of the powerhouse. The project became operational for navigation in December 1970. The second-stage powerhouse was constructed under a separate contract, and the last of the three units was placed on line in November 1973.

368. The spillway stilling basin is shown in plan and section in Figure 194. Constructed in 96 independently reinforced blocks, the basin is 710 ft wide and about 100 ft long. The basin does not contain baffles but was constructed with a 6-ft-high end sill at the downstream end. The basin slab is stepped down from el 421.0 at the left to el 410.0 at the right to follow the dip of the limestone foundation. For most of the basin the minimum slab thickness is 4 ft. The basin is

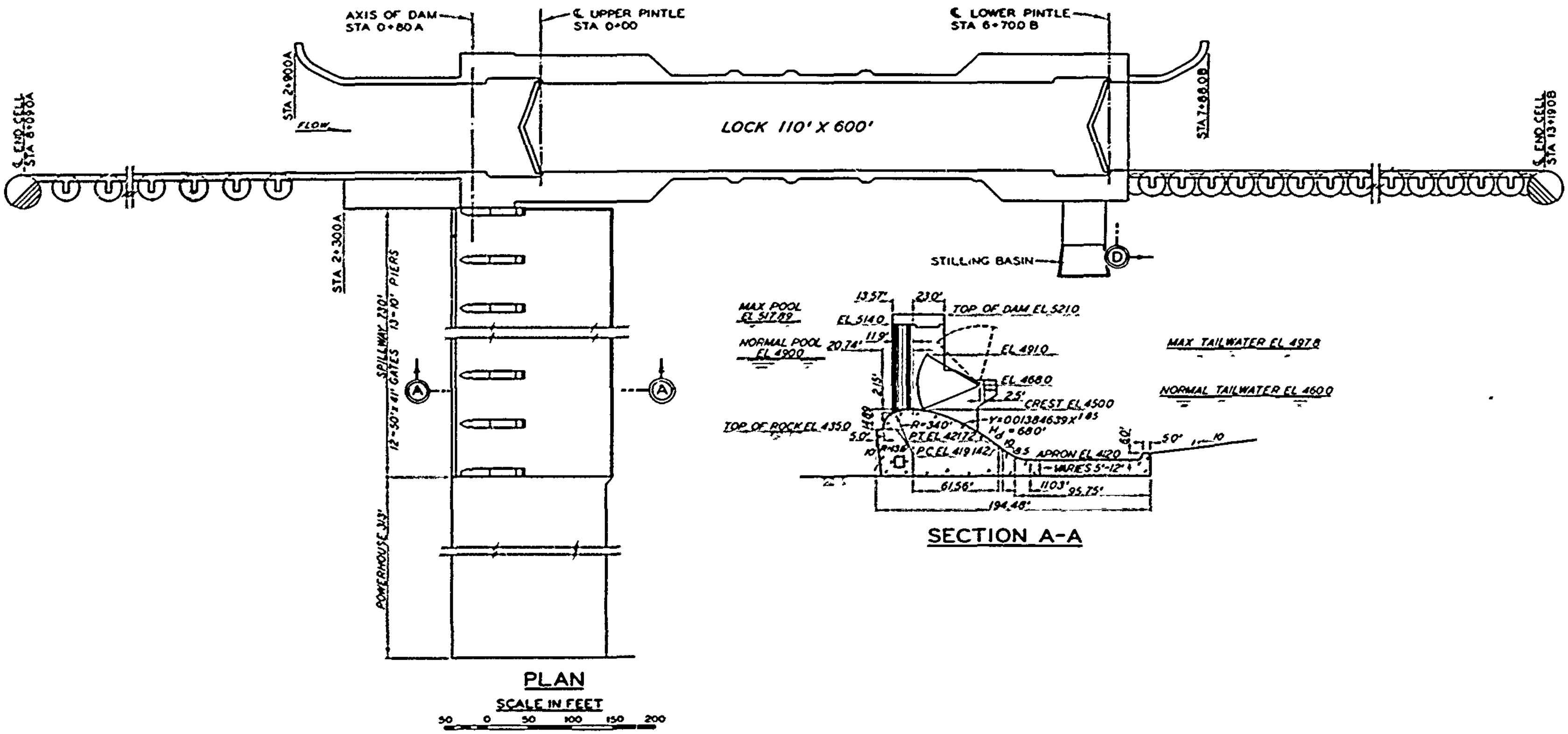


Figure 193. General plan of lock and dam, Webbers Falls

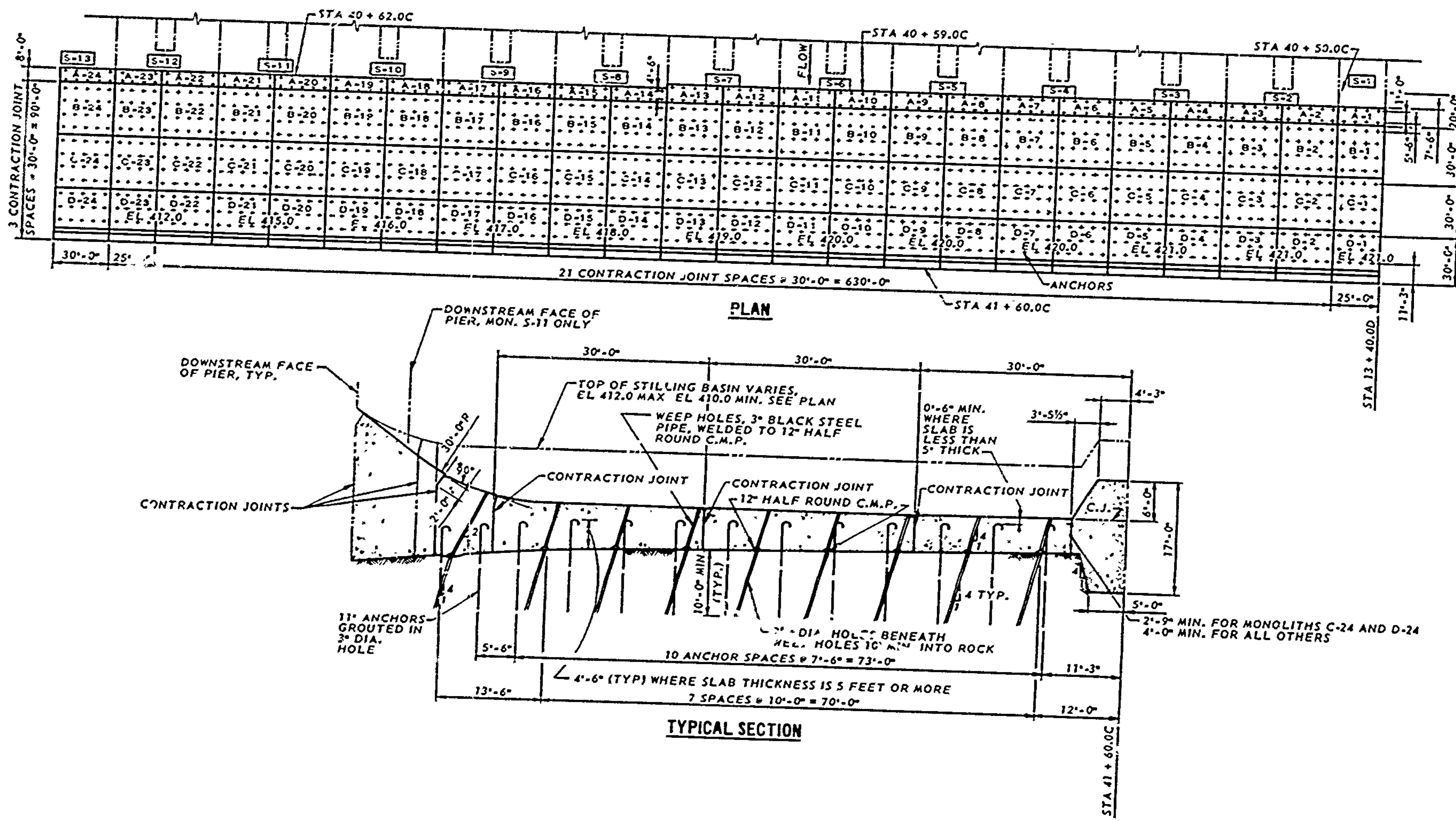


Figure 194. Stilling basin plan and section, Webbers Falls

anchored to the foundation with No. 11 bars grouted 10 ft into rock. Reinforcement for the basin slab consists of No. 6 bars at 12-in. spacing each way near the top face for shrinkage and temperature stresses. The stilling basin concrete was designed for a minimum 90-day compressive strength of 3000 psi. The maximum size aggregate used was 3 in. for all concrete except the top 2 ft where the maximum size was limited to 1-1/2 in. In conjunction with the recent repair work, 15 cores were taken from the D row of slabs (Figure 194) where the damage occurred. Thirty-four test samples were cut from these cores, and compression tests indicated a strength ranging from 2255 to 4334 psi with an average of 3316 psi. The actual strength was probably somewhat higher since the wobble of the 45-ft-long kelly turning the core barrel produced cores slightly less than 6-in. in diameter. Also, some of the cores contained 3-in. aggregate.

369. The capacity of the spillway at maximum pool elevation is 1,518,000 cfs. The tabulation below lists the five largest discharges through the spillway prior to the repair work since the project was placed in operation. These figures represent 24-hr average discharges. The velocity associated with discharges of this magnitude is about 30 fps.

Date	Discharge, cfs
November 1974	159,000
April 1973	141,200
March 1973	137,200
October 1973	115,000
June 1974	105,800

370. The erosion of the stilling basin slab was discovered during a routine underwater inspection in October 1974. This was the first inspection of the basin since the project went into operation. The inspection was made by Army divers from Fort Belvoir, Virginia. The damage was generally confined to the D slabs (Figure 194) and to the end sill itself. Damage ranged from slight erosion in the D-1 and D-2 slabs and in the D-15 through D-24 slabs to severe erosion in the D-3 through D-14 slabs. In slabs D-3 through D-14 the floor was trenched from the toe of the end sill upstream 2-4 ft with an average depth of

1-2 ft. The trenched area was pockmarked with holes 6-12 in. deeper than the adjacent area. A large hole on the D-5, D-6 joint measured 4 ft deep. The end sill had erosion damage over its entire length up to a depth of 4 ft. Reinforcing steel was damaged or exposed on most of the D slabs and end sill. Some rock was eroded downstream of the end sill but the total extent of erosion could not be determined. The exact cause of the stilling basin damage is unknown, but the inspection revealed debris in the basin, especially large loose rocks that had been ground smooth, indicating that the erosion could have been started by a grinding action between the rock and concrete. The inspection also revealed areas with pitted surfaces, which indicated possible cavitation.

371. Although the erosion in the basin was severe enough in places to warrant attention, it was not considered a serious threat to the integrity of the structure. However, it was decided to fill the deepest holes with concrete placed by tremie, and 21 cu yd was placed. This concrete was tested under normal operating conditions for several months and then examined by a diving team. Had this concrete failed to hold up, a board of consultants was to be convened to determine the next course of action. However, the concrete was still intact and cores of the area indicated a good bond with the old concrete. Placements were then made in other eroded areas.

372. Conventional concrete placed with a 10-in.-diameter tremie pipe was used to fill the eroded areas. The concrete was designed for a cement content of 752 lb/cu yd and a water content as required to obtain a 5- to 8-in. slump with a maximum water-cement ratio of 0.45. During the placing operation, a 5-1/2- to 7-1/2-in. slump was found to be best for this type operation. The mixing water was heated to about 150°F to offset the effect of the cold river water. Mixture proportions for 1 cu yd were as follows:

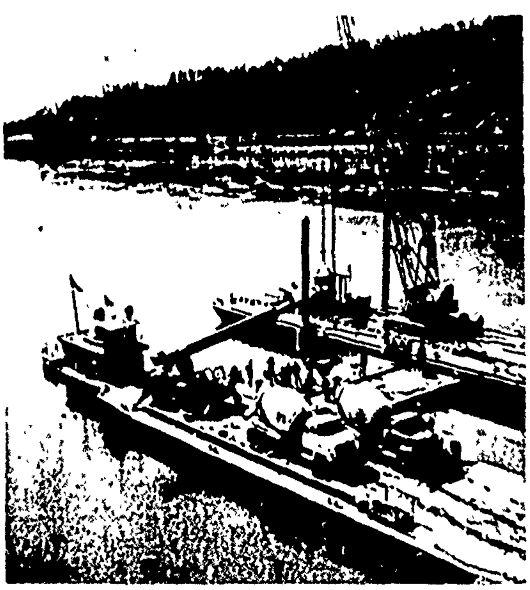
Material	Quantity
Portland cement, Type II	752 lb
River gravel, 1-in. MSA	1567 lb
Natural sand	1279 lb
Water	40 gal
Air-entraining admixture	7 oz

373. Several barges lashed together formed a work platform over the area to be repaired. Prior to the placement of concrete, the damaged area was air cleaned of silt and debris. The concrete was conveyed to the area by loading the transit mix trucks on a barge (Figure 195) at a point about half a mile below the dam. Because of the length of time required to get from the batch plant to the work site, the cement and some water were withheld from the mix until the trucks had been loaded on the barge and were approaching the work area. A placement was made by transferring the concrete from the truck into a 1-cu-yd capacity bucket. The bucket was positioned by crane over a 1-cu-yd-capacity hopper attached to a 10-in. ID tremie pipe. A "rabbit," composed of a greased polyethylene bag filled with burlap bags, was placed in the top of the tremie prior to placing concrete in the hopper (Figure 196). The bottom of the tremie pipe was positioned by divers and rested on the surface of the existing concrete in the hole. When the fresh concrete forced the rabbit to the bottom of the pipe, the pipe was raised approximately 1 ft to eject the rabbit and permit the flow of concrete. This process was repeated each time it was necessary to move the tremie pipe. The depth of the water in the stilling basin during the repair was about 40 ft. The water velocity was 3 or 4 fpm due to releases through the adjacent power plant. No releases were made through the spillway for 7 days following the concrete placement. A total of 43 cu yd was placed during two different periods. Twenty-one cu yd was placed in March 1975: 7 cu yd in slabs D-5 and D-6 and 14 cu yd in slabs D-7 through D-10. After this repair proved resistant to heavy flows, an additional 22 cu yd was placed in October 1975. Seven cu yd was placed on the D-6 and D-7 slabs and 15 cu yd in the D-4, D-5 area.

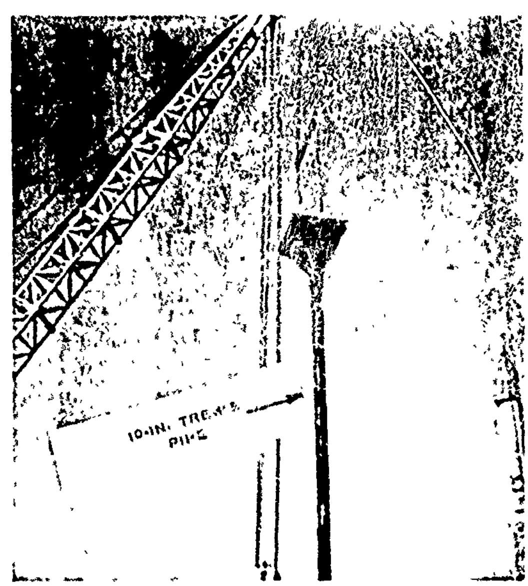
374. All work was done by Tulsa District personnel at a total cost of about $32,000 including surveys. A summary of the expenses involved is given in the following tabulation.

a. Work area showing equipment arrangement

b. Transit mixers carried to work site by barge

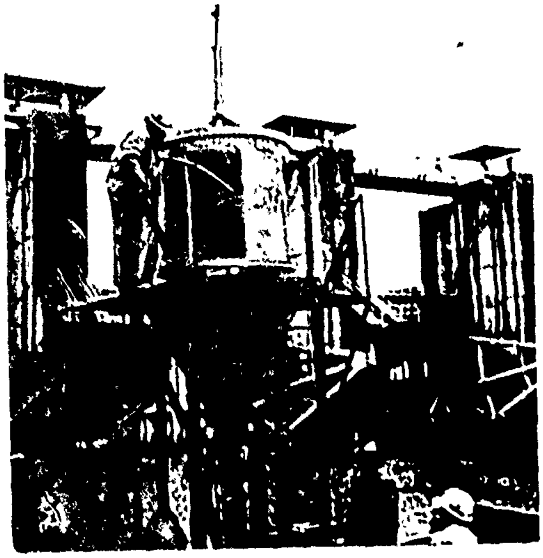


c. Placement in progress



d. Hopper and tremie pipe handled by crane

Figure 195. Repair operations, Webbers Falls Dam



a. Concrete being discharged into hopper

b. Workmen adjusting tremie pipe

c. A "rabbit" forced the water out of the tremie pipe ahead of the concrete

d. Army divers assisted in the repair work

Figure 196. Repair operations, Webbers Falls Dam

Item	Labor	Materials and Equipment
Surveys	$ 8,840	$ 8,550
Fleet	4,590	5,520
Engineering and Supervision	2,800	--
Concrete	--	1,720
Total	$16,230	$15,790

375. In conjunction with the repair work, several cores were taken from D slabs, including one through the tremied concrete placed in March. A sample from this core showed a strength of 5250 psi (at about 60 days) and good bonding between the new and old concretes. In July 1975, a diving team made a cursory inspection of the repaired areas, and the areas were in good condition even though they had experienced high flows. On the basis of this inspection, the decision was made to make the October placement. The following summer another inspection was made of the area, and both placements seemed to be in good condition. However, no large releases have been made since the October 1975 placement. At the present time, no further concrete placements are anticipated.

376. It was found that the project's hydrographic surveying equipment, used for plotting profiles of the navigation channel, could be adapted for plotting the concrete surface in the stilling basin. It is planned to use this type survey after periods of heavy releases to monitor any further scour.

PART III: DISCUSSION

377. Of the 31 repair projects described herein, follow-up inspections of the repairs have been reported in 23 cases. The repair materials and techniques used in these cases have been in service for various lengths of time and have been exposed to different operational conditions as well as different levels and durations of flow. This makes any comparison of the relative merits of the various systems difficult at best; however, a number of general trends did emerge.

Materials

378. The resistance of steel plate to abrasion erosion is well established; however, it must be sufficiently well anchored to the underlying concrete to resist the uplift forces and vibrations created by flowing water. Welding of anchor systems as nearly flush with the plate surface as possible appears more desirable than raised bolted connections. In any case, the ability of the anchor system, including any embedding material, to perform satisfactorily under the conditions of exposure, particularly fatigue, should be evaluated in detail during design of the repair.

379. After a 2-year exposure, the epoxy mortar at Enid had excellent bond to the fill concrete and generally good abrasion resistance. The rate of maximum erosion in the stilling basin floor was approximately one half of that occurring prior to repair. The baffles were in excellent condition with no evidence of erosion. After approximately 7 months of limited discharges, an estimated 25 percent of the epoxy mortar at Dworshak had failed probably due to workmanship, weather conditions, and lack of sufficient curing during construction. After over a 4-year discharge exposure at Pomona, no erosion damage to the epoxy mortar was visible; however, there were several areas of cracking and loss of bond attributed to improper curing and thermal incompatibility with existing concrete. Epoxy mortar failed within a year under the severe abrasion conditions existing at Kinzua prior to the adoption of a symmetrical

sluice operation policy. Under comparable conditions at Upper St. Anthony Falls, both epoxy mortar and epoxy concrete performed satisfactorily in spite of adverse placing conditions. The performance of the epoxy concrete was particularly impressive. The epoxy-resin concrete patches at Table Rock have exhibited satisfactory erosion resistance; however, there were problems with the bond between the patch and existing concrete, and some patches were dislocated. Epoxy mortar patches in the baffles and walls at Oologah have performed excellently. The epoxy used to repair a contraction joint at Folsom appeared to be in generally good condition after 4 years.

380. After 4 years exposure, the high-strength, reinforced concrete overlay containing abrasion-resistant aggregate at Pomona exhibited only minor general erosion of about 1/8 in. with maximum depths of 1/2 in. High-strength, reinforced concrete containing an abrasion-resistant fine aggregate and given a hard-troweled finish was used in the repair at Bull Shoals. After 25 years, the repaired section experienced minor concrete erosion except for two monoliths where erosion up to 3 in. deep was observed. The deepest erosion occurred along monolith joints between baffles and at the intersection of the joints with expansion plugs and ogee sections. During the same period, erosion of the original concrete, while of similar depth, occurred in a more widespread pattern across the basin. At Oologah the high-strength, reinforced concrete overlay outside the debris trap was in generally good condition after more than 2 years. A relatively small area of the basin floor exhibited general erosion of 3/8 to 1/2 in. with maximum erosion along construction joints approximately 3/4 to 1 in. deep. After an 8-year exposure, the high-strength concrete used to repair the end sill at Coyote showed no significant erosion damage.

381. In an effort to obtain concrete with the greatest possible resistance to erosion and cavitation, the water-cement ratio of the reinforced concrete repair at Bonneville was not allowed to exceed 0.40 by weight. Also, forms were lined with absorptive liners or vacuum mats in an attempt to harden the concrete surfaces by reducing the effective water-cement ratio. Diver surveys 4 and 11 years after repair showed

increasing but not critical erosion of the repaired baffle deck and only minor erosion on the pier noses and the solid downstream baffle. Eighteen years after the repair an inspection noted no significant changes from the previous inspection. An inspection of the reinforced, class "A" concrete repair at Center Hill after a 26-year exposure indicated three relatively small areas of erosion. Two of these areas were approximately 30 sq ft each with maximum erosion to 3- and 6-in. depths. The smallest area, approximately 8 sq ft, had the deepest erosion, 8 to 10 in.

382. The initial inspection at Nolin, less than a year after repair, indicated approximately 4 tons of rock in the stilling basin. Under these very severe exposure conditions the 5000-psi unreinforced concrete overlay had eroded to maximum depths of 8 in. The location of this erosion behind the baffle attached to each wall coincided with the location of the original erosion. Under comparable exposure conditions at Upper St. Anthony Falls, erosion had progressed completely through the conventional concrete (approximately 6 in.) within 2 years. After 5 years the conventional concrete (3000 psi) repair at Tionesta had several holes in a relatively small area immediately downstream of the dentations eroded to maximum depths of nearly 12 in. An inspection 10 years after the repair indicated that the area of erosion increased and the maximum depth of erosion had increased to 14 in. Also, a crack had formed across the basin in the unreinforced concrete immediately downstream of the dentations. The condition of the conventional concrete repair at Conchas was described as excellent, after nearly 30 years in service. After 5 years exposure the bonded conventional concrete repair at Nimrod appeared to be performing satisfactorily.

383. Periodic use of calcium aluminate cement mortar has been found to be an economical and effective method of controlling erosion in the conduits and on the flip bucket at Pine Flat. Epoxy bonding of an iron aggregate mortar resulted in a well-bonded repair at Somerville; however, a transverse hairline crack has developed near the middle of the repair. A similar repair at Waco was reinforced and water cured for

10 days. After 2 years this patch was reported to be in very good condition.

384. No erosion of the exposed fiber grout at Old River has been reported. However, the location of this material on a 1 in 7 slope at the end sill should significantly reduce the effects of abrasion forces. The fiber concrete in the discharge laterals at Upper St. Anthony Falls was not subjected to the abrasive effects of waterborne rocks, and erosion in these areas was negligible. In comparison, fiber concrete in the test section, which was exposed to abrasion by rocks, exhibited considerable erosion. After 2 years, erosion had progressed completely through the fiber concrete (approximately 6 in.) and into the original concrete. Under the severe erosion conditions at Kinzua, the bonded and anchored fiber concrete overlay exhibited erosion ranging from 5 to 17 in. within a year after repair. Subsequent inspections indicated continued erosion in the fiber concrete repair area with maximum depths of 42 in. after approximately 4 years exposure.

385. After 7 months exposure to limited discharges at Dworshak, the reinforced fiber concrete (both polymer impregnated and nonimpregnated) was in generally good condition. There were several areas in the center of the basin with erosion up to an inch deep. Joints and open cracks in the fiber concrete had localized erosion up to about 1 in. deep. The polymer-impregnated side of the basin was described as being probably a little better than the nonimpregnated side. A diver inspection 1 year after repair indicated large areas of "grooves" near the center of the basin. These grooves were oriented in the direction of flow with maximum depths of 2-3 in. in both the impregnated and nonimpregnated fiber concrete.

386. Guniting was used to repair shallow, unformed areas of erosion on the lower portion of the spillway at Bonneville. No erosion of the repair was reported after 18 years exposure. High-strength shotcrete mortar was used in the repair of the stilling basin walls at Dworshak. After a 7-month exposure to limited discharges, the walls exhibited less than 1 in. of surface erosion. However, there were

several areas at the junction between the floor and wall with erosion up to 3 in. deep.

387. Both preplaced-aggregate concrete and pumped concrete were placed underwater at Chief Joseph. After significant flows for a year, the condition of the preplaced-aggregate concrete was described as excellent. At the same time, pumped concrete surfaces were described as being in good condition with only minor surface damage noted. An inspection 8 years after the repair indicated that both surfaces were generally very smooth with no erosion having occurred since the initial inspection. During the same period the rate of erosion in the unrepaired section was described as minimal. Approximately 18 months after the stilling basin at Ice Harbor was cleaned of debris and repaired with tremie concrete, a diver inspection indicated that both the repaired and original concretes were in good condition with no further erosion. No accumulations of debris were reported, and 10 years after the repair, soundings of the basin indicated no serious erosion. A core taken through the tremie concrete repair at Webbers Falls indicated good compressive strength and bond to the original concrete. A diver inspection 4 months after the repair indicated that the tremie concrete was in good condition even after experiencing high flows.

Revised Configuration

388. The steel modules anchored to the stilling basin slab and the top of the end sill at Old River essentially create a sloping end sill. Apparently, any debris discharged through the structure is being flushed from the basin over this fillet, since diver inspections have reported the basin to be free from debris. When Pomona was dewatered 4 years after repair, approximately 1 cu yd of rocks was found in the stilling basin. An inspection at Oologah 2-1/2 years after repair revealed a similar situation. In each case it is difficult to determine if these relatively small amounts of debris were influenced by the addition of a sloping end sill. Replacing part of the elliptical steps within the stilling basin at Bull Shoals with a smooth ramp created additional

hydraulic problems. The addition of a sloped concrete section between the baffles adjacent to the training walls and the end sill at Conchas has apparently eliminated debris collection and subsequent erosion in this area.

389. A debris trap was provided in several stilling basins by either reducing the overlay thickness or eliminating the repair overlay in an area between the downstream baffles and the end sill. At both Pomona and Oologah the debris trap appeared to function well. In each case most of the debris was found in the trap adjacent to the raised overlay slab. The circulatory currents within the stilling basin at Kinzua appear to have negated any effect of the debris trap. Following the adoption of symmetrical discharges, the small amounts of debris present in the stilling basin were located in areas of erosion throughout the basin.

390. Based on a hydraulic model study, the shape and size of the original upstream row of baffles at Bonneville was revised with the result of a significant reduction in negative pressures. Also, a solid baffle was added over the existing second row of baffles. Diver surveys conducted 4 and 11 years after repair indicated increasing but not critical erosion of the repaired baffle deck behind the upstream baffles and only minor erosion on the pier noses and the solid baffle. A new erosion pattern in the basin floor indicated a different scouring flow, probably due to the solid baffle. Subsequent inspections over the next 7 years indicated no increased erosion in either the repaired or original sections of the basin.

391. At Nolin the end sill was raised 12 in., end walls were added, and a 50-ft section of the outlet channel was paved in an effort to prevent riprap around the basin exit area from entering the basin. However, a diver inspection less than a year after repair indicated that approximately 4 tons of rock was in the stilling basin. This rock, piled up to a depth of 15 ft, apparently entered the basin from downstream. Rock piled up to 18 in. deep, similar in size and composition to that in the basin, was found on the slab downstream from the stilling basin. The top of the training walls at Oologah are the same elevation

as the tailwater with both conduits operating at maximum capacity. Under these conditions, rock fill adjacent to the walls apparently entered the basin; therefore, a concrete slab was provided around the stilling basin walls. There was a minimum amount of debris in the stilling basin when it was dewatered for inspection of the repair.

392. As a temporary repair of the stilling basin at Bull Shoals, the two upstream steps at 14 of the 16 conduit outlets were replaced with smooth, upward-sloping slabs. Prototype tests following the repair disclosed that pressure fluctuations on the ramp were in the cavitation range. Consequently, additional model tests were initiated to determine a satisfactory method of making permanent repairs to the stilling basin. Based on these tests, a row of baffles was added to the basin floor and the end sill was increased in height from 4 to 12 ft. Also, a 19.5-ft-long horizontal apron terminated by another 2-ft-high end sill was provided downstream of the existing end sill. After 25 years in service, the repaired sections experienced minor erosion in all areas except monoliths 21 and 22 where erosion up to 3 in. deep was observed. A large collection of coarse aggregate, well rounded as a result of rolling across the basin floor, was located on the floor of monolith 21. It was suggested that the feasibility of erecting four low-height sills extending from the sidewalls of selected conduits to the end sill be evaluated as a means of controlling the migration of this debris, thus minimizing abrasion erosion.

393. The square corners of the sluice outlets at Center Hill were eroded by the combined action of cavitation and impact of large boulders. The outlet edges were reformed on an 8-in. radius at the bottom of the opening, and after 26 years exposure no further erosion has been reported.

394. In an effort to prevent future erosion and loss of downstream riprap, an additional row of baffles was constructed at Coyote. No significant damage of the area was noted after 8 years exposure although there were several rocks in the basin. A diver inspection of the area around the intake structure indicated a deposit of rock, which was assumed to be the source of the rocks found in the stilling basin.

Operations

395. Model tests of the stilling basin at Pomona verified that severe separation of flow from the one sidewall and eddy action within the basin occurred for discharge and tailwater conditions common to the prototype. The strength of the eddy was sufficient to circulate stone in the model, and it was apparent that the abrasion damage observed in the prototype stilling basin was a result of this type of flow condition and the presence of rock and debris. The model also indicated that the eddy within the basin was capable of generating considerable reverse flow from the exit channel into and along one side of the basin with the potential to transport riprap from the channel into the basin. Based on the model tests, it was recommended that the most practical solution was to provide a wearing surface to the area of greatest wear, a depression at the downstream end of the basin for trapping debris, and a sloped face on the existing end sill. However, flow separation and eddy action were not eliminated by that modification and continued erosion can be expected if debris is present in the basin. Therefore, it was recommended that a fairly large discharge, so as to create a good hydraulic jump but not create any eddy action, be released periodically to flush debris from the basin. Guidance as to the discharge and tailwater relations required was developed.

396. Because of the proximity of a pumped-storage power plant on the left abutment and problems from spray, especially during the winter months, the right side sluices at Kinzua were used most of the time. This usage caused a circulatory current that carried debris back into the stilling basin from downstream and over, the end sill, which is below streambed level. This was verified by an inspection of the stilling basin in which innumerable smooth pieces of all types of brick were located 6 days after the bricks were placed in the river downstream. Under these conditions an average of 50 cu yd of debris was removed from the basin during each of three inspections within the 7 months following completion of repairs. At this point a policy of symmetrical sluice operation was initiated, and based on prototype experiments, a

table outlining sluice operating procedure for a range of outflow was prepared. Subsequent to the adoption of this revised sluice operation policy, a minimum of debris has been removed from the basin and the rate of erosion has decreased.

397. Erosion in the areas downstream from the regulating conduits at Conchas was attributed to eddies within the stilling basin. The regulating conduits, discharging virtually no water, or at most much less water than the adjacent sluicing conduits, could not maintain a discharge velocity in the lower reaches of the stilling basin comparable to that caused by the sluicing conduits. This difference in discharge caused a partially "dead" or slow spot in the discharge downstream from each regulating conduit with the resulting eddy action causing slight erosion of the floor over an extensive area. The piles of debris in the south half of the stilling basin downstream from the baffles were located on the approximate edges of these eddies. It appeared that the debris was composed of rocks and gravel from the downstream riverbed that had been swirled upstream by the eddy action. The deposits occurred when the debris approached the outer edge of the eddy where the velocity was dissipated. The absence of these deposits in the north portion of the stilling basin indicates that the topography of the downstream riverbed had considerable effect upon the degree of turbulence of the flow. Observation of the basin under conditions of discharge symmetrical about the center line of the stilling basin indicated that considerably more turbulence did occur in the south side of the stilling basin.

398. The majority of stilling basin repair operations require dewatering of the basin. In some cases it is extremely difficult and expensive to dewater a structure to make repairs under dry conditions. In fact, the data reported herein indicates that the cost of dewatering averages over 40 percent of the total repair cost. In an effort to reduce the cost of dewatering, sumps/siphon pipes have been added to some stilling basins during the repair operation. The new stilling basin end walls at Nolin are designed to support a closure at the end of the basin. Also, a concrete equipment pad adjacent to the stilling basin will permit a mobile crane to place the closure for more expeditious

dewatering of the basin. When a repair is completed, the stilling basin should be flooded in such a manner to prevent material from temporary access roads, cofferdams, etc., from being washed back into the basin.

PART IV: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

399. Depths of erosion damage to the 31 structures reported herein ranged from a few inches to approximately 10 ft. In general, this erosion damage resulted from the abrasive effects of waterborne rocks and other debris being circulated over the concrete surface during construction and operation of the structure. In most cases the presence of debris and subsequent erosion damage was the result of one or more of the following: (a) construction diversion flow through constricted portions of the stilling basin; (b) eddy currents created by diversion flows or powerhouse discharges adjacent to the basin; (c) construction activities in the vicinity of the basin, particularly those involving cofferdams; (d) nonsymmetrical discharges into the basin; (e) sufficient flow separation and eddy action within the basin to transport riprap from the exit channel into the basin; and (f) topography of the outflow channel.

400. A variety of materials and techniques have been used in the repairs reported herein with varying degrees of success; the degree of success has generally been inversely proportional to the degree of exposure to those conditions conducive to erosion damage. In many instances, materials have been used in prototype repairs with limited or no laboratory evaluation of their effectiveness in the particular application. This survey shows a definite need for laboratory evaluations, particularly erosion resistance, of repair materials prior to their use in prototype repairs costing millions of dollars. Consequently, a new underwater erosion test for evaluating the resistance of concrete subjected to the abrasive action of waterborne particles has been developed and a testing program has been (Liu, in preparation) initiated.

Recommendations

401. It appears that given appropriate flow conditions in the

presence of debris, all of the materials described herein are susceptible to some degree of erosion. No one material has demonstrated a consistently superior performance advantage over alternate materials. Pending the results of additional laboratory tests and field inspections to evaluate repairs, it is recommended that conventional concrete of the lowest practical water-cement ratio containing the hardest coarse aggregate economically available should be used for repair and in new construction of structures subjected to abrasion erosion damage (OCE 1978). Regardless of the material, construction joints are generally the areas most susceptible to abrasion erosion; therefore, they should be given appropriate attention in design and construction.

402. The majority of the materials currently being used in repair of stilling basins requires dewatering of the basin. In some cases, dewatering a structure completely for repairs is difficult. Problems associated with dewatering should be recognized in the design of both new structures and repairs to existing structures, and features to alleviate these problems (sumps, supports and guides for temporary closures, slope of outflow channel, etc.) should be incorporated into the design where feasible.

403. Since the average cost to dewater a structure is a significant percentage of the total repair cost and the placement of concrete underwater does not appear to adversely affect its abrasion resistance, further investigation of underwater construction materials and techniques (tremie concrete, precast concrete, adhesives suitable for application underwater, etc.) applicable to repair is recommended.

404. Improvements in materials should reduce the rate of concrete damage due to erosion. However, until the adverse hydraulic conditions that caused the original damage are minimized or eliminated, it will be extremely difficult for any of the materials currently being used in repair to perform in the desired manner. Prior to major repairs, model studies of the existing stilling basin and exit channel should be conducted to verify the cause(s) of erosion damage and to evaluate the effectiveness of various modifications in eliminating undesirable hydraulic conditions. If the model test results indicate that eliminating

the undesirable hydraulic conditions is impractical, provisions should be made in design of the repair to minimize future damage. To the extent practicable, the following measures should be taken:

a. The design should include provisions (debris traps, low division walls, etc.) for minimizing circulation of debris.

b. Avoid baffles that are connected to basin training walls, or considering their susceptibility to erosion, avoid appurtenances such as chute blocks and baffles altogether.

c. Based on model tests, design the exit configuration (shape and height of end sill, training wall flare, shape of exit channel, etc.) to maximize flushing of the stilling basin and to minimize chances of debris from the exit channel entering the basin.

405. In existing structures, control releases so as to avoid discharge conditions where flow separation and eddy action are prevalent. Substantial discharges that can provide a good hydraulic jump without creating eddy action should be released periodically in an attempt to flush debris from the stilling basin. Guidance as to discharge and tailwater relations required for flushing must be developed through model/prototype tests. Periodic inspections should be required to determine the presence of debris in the stilling basin and the extent of erosion.

406. In addition to providing a state-of-the-art report on repair of erosion-damaged structures within the Corps of Engineers, the information reported herein is being used to develop direction and depth for directly related research activities to extend current knowledge in the area of maintenance and rehabilitation of civil works structures. Therefore, it is recommended that results of additional stilling basin repairs and inspections be compiled and analyzed as they become available to allow updating of this report within 3-5 years.

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