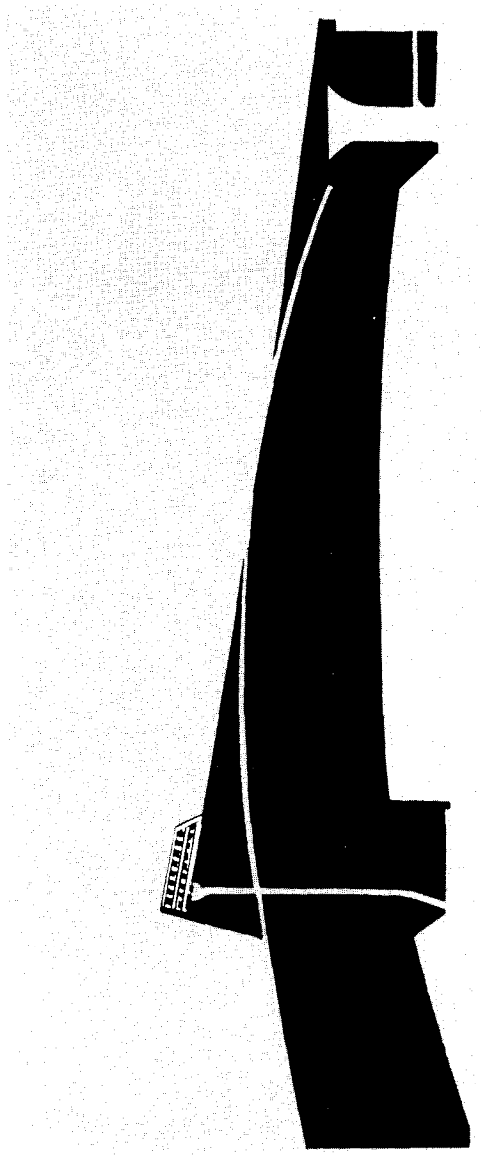
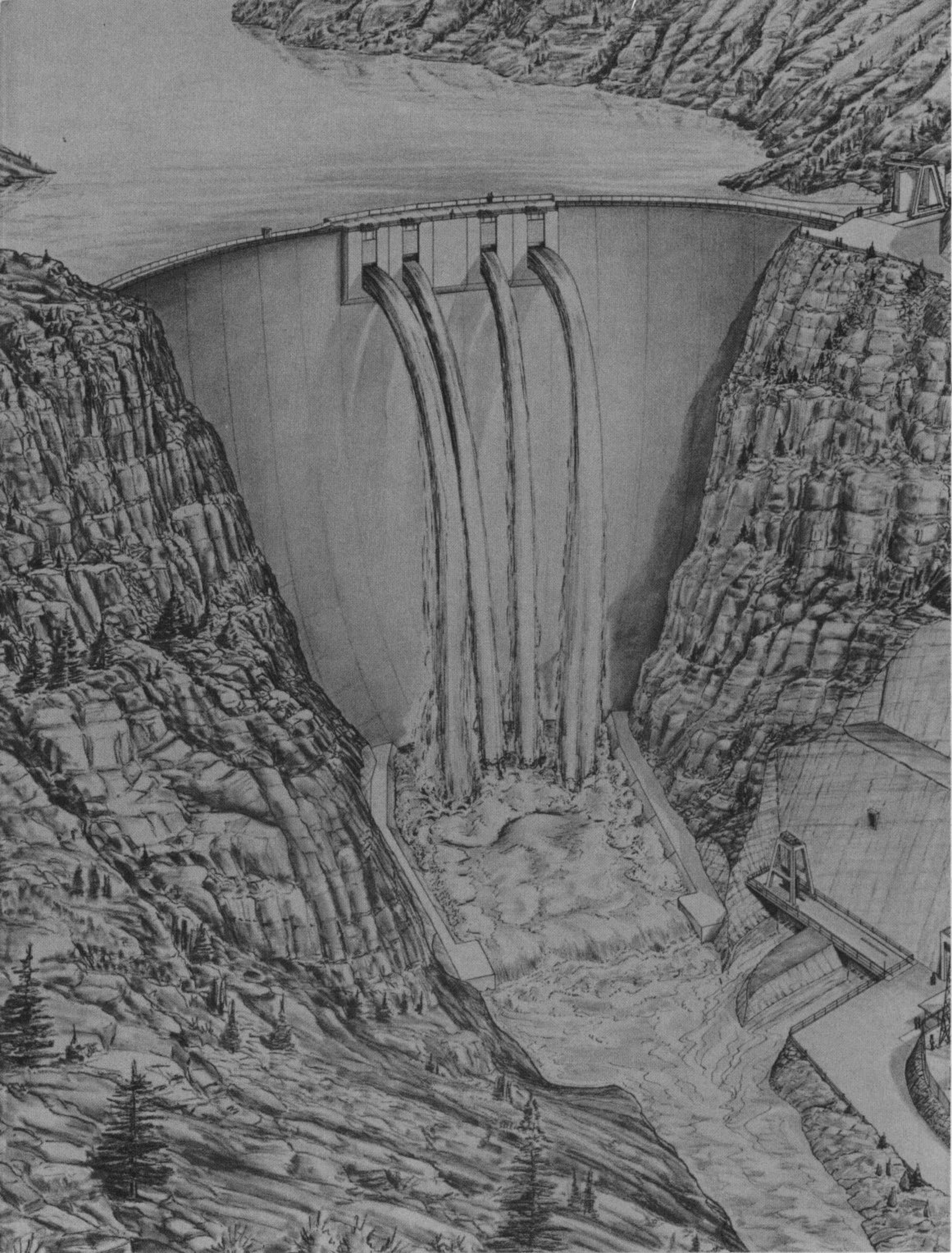
A WATER RESOURCES TECHNICAL PUBLICATION
ENGINEERING MONOGRAPH NO. 37



# Hydraulic Model Studies for Morrow Point Dam

UNITED STATES DEPARTMENT
OF THE INTERIOR
BUREAU OF RECLAMATION



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# Hydraulic Model Studies for Morrow Point Dam

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United States Department of the Interior • Stewart L. Udall, *Secretary*

BUREAU OF RECLAMATION

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***In its assigned function as the Nation's principal natural resource agency, the Department of the Interior bears a special obligation to assure that our expendable resources are conserved, that renewable resources are managed to produce optimum yields, and that all resources contribute their full measure to the progress, prosperity, and security of America, now and in the future.***

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# Preface

THIS MONOGRAPH presents the results of hydraulic model studies on the Bureau of Reclamation's Morrow Point Dam and Powerplant on the Gunnison River in southwest Colorado. Both the dam and powerplant structures are unique in the Bureau's 64-year experience in the development of water resources in the western United States. Morrow Point Dam will be the Bureau's highest concrete thin-arch dam with double curvature. The 120,000-kilowatt powerplant at Morrow Point will be the first underground power installation to be constructed on a Bureau of Reclamation project. Construction work started on this dam and powerplant in June 1963.

At Morrow Point the river is confined to a narrow channel cut deep in the rock-walled canyon of the Gunnison. Geologic and design studies established the structural and economic advantage of a high thin-arch concrete dam at this site. Below the dam, the rock-confined channel and the established location of the powerplant tailrace placed a practical limitation on the size of the stilling basin which serves to dissipate the energy of high velocity discharges from the spillway and the outlet works.

Laboratory model studies were initiated to investigate the hydraulic characteristics of the preliminary design of the outflow structures for Morrow Point Dam and Powerplant. Operation of the model indicated that discharges from the outlet works caused unfavorable flow conditions in the stilling basin. Further design studies and tests on modifications to the model developed the alternate and the recommended design described in this report. The size and capacity of the outlet works were significantly reduced. The design of the spillway was changed from an uncontrolled overflow-type to a submerged orifice-type with gate controls. Working together the designers and the laboratory engineers and technicians created a working model which provided the required maximum design outflow with acceptable flow characteristics in the limited space of the stilling basin and tailrace channel.

The free fall, orifice-type spillway at Morrow Point Dam will be a Bureau of Reclamation first. Water flowing through the four submerged orifices in the top central part of the dam will fall more than 350 feet—about twice the height of Niagara Falls—into the stilling basin at the toe of the dam. This spillway will have a maximum capacity of 40,000 cubic feet per second and each opening will be controlled by a fixed-wheel gate. The outlet works will consist of one 4- by 4-foot conduit through the center of the dam near the base. Flow through the conduit will be controlled by slide gates. The maximum capacity of the outlet works will be 1,500 cubic feet per second. Discharges will be directed into the stilling basin.

Morrow Point Dam and Powerplant are major features of the Curecanti Unit, one of the four initial storage units of the Bureau of Reclamation's Colorado River Storage Project. This project is a large Reclamation undertaking to develop the water and land resources of the Upper Colorado River Basin—a vast area embracing 110,000 square miles in the States of Colorado, Utah, Wyoming, New Mexico, and Arizona.

The primary purpose of the Curecanti Unit is to develop the potentials of water storage and hydroelectric power along the 40-mile section of the Gunnison River below the town of Gunnison and above the Black Canyon of the Gunnison National Monument. The Gunnison River is a major tributary of the Colorado River.

This report, describing the application of hydraulic model studies to the design of the Morrow Point Dam and Powerplant, will interest engineers concerned with the design of large water control structures. It may also be useful to the engineering departments of colleges and universities.

The model studies described in this report were made in the Bureau of Reclamation laboratories, which are a part of the Office of Chief Engineer in Denver, Colo. They represent primarily cooperative team effort of the design engineers specializing in concrete dams and the hydraulics research specialists and technicians of the laboratory.

Engineers of the Structural and Architectural and the Mechanical Branches of the Chief Engineer's Office contributed several helpful suggestions and valuable assistance on specific phases of the studies. The photography in the report was by personnel of the Office Services Branch. During the course of these studies, many foreign and domestic visitors observed the model.

The source document for this publication is the Bureau of Reclamation's Laboratory Report No. Hyd.-557, "Hydraulic Model Studies of Morrow Point Dam, Spillway, Outlet Works, and Powerplant Tailrace," issued by the Hydraulics Branch, Division of Research, April 1, 1966. Some of the photos and detailed data on the model studies which are part of the source document were determined to be extraneous to the publication and are not included therein.

Included in this publication is an informative abstract and list of descriptors, or keywords, and "identifiers". The abstract was prepared as part of the Bureau of Reclamation's program of indexing and retrieving the literature of water resources development. The descriptors were selected from the *Thesaurus of Descriptors*, which is the Bureau's standard for listings of keywords.

Other recently published Water Resources Technical publications are listed on the inside back cover of this monograph.

# Contents

# FIGURES

# Introduction

MORROW POINT DAM, located on the Gunnison River, 22 miles east of Montrose, Colo., figure 1, is one of the key features of the Curecanti Unit of the Colorado River Storage Project. The other features include Blue Mesa Dam, upstream from Morrow Point, and Crystal Dam, downstream from Morrow Point.

Morrow Point Dam, the Bureau of Reclamation's highest double-curvature, thin-arch concrete dam, contains the Bureau's first orifice-type freefall spillway, and its first underground powerplant, figures 2 through 4. The dam varies in thickness from 12 feet at the crest to 52 feet at the base and rises 465 feet above the foundation. It

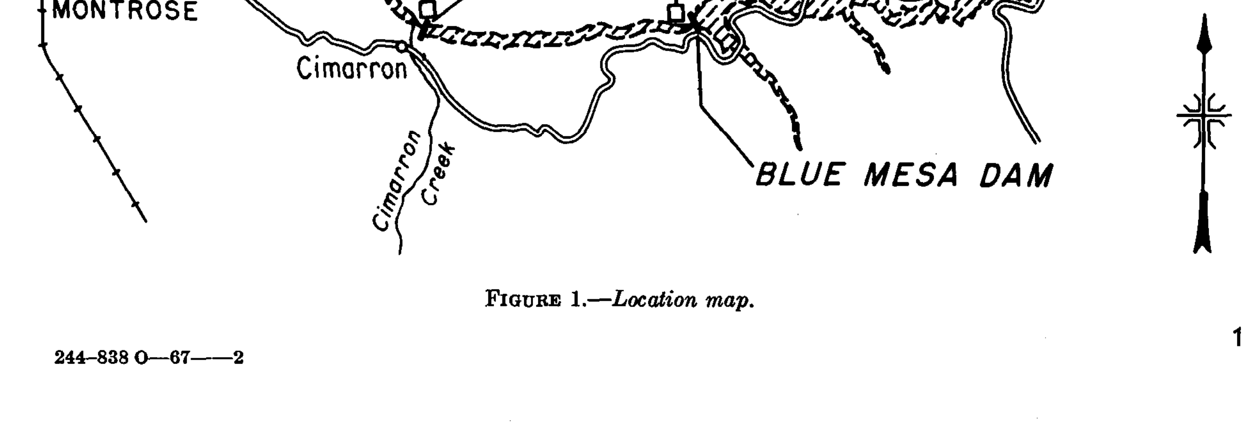


FIGURE 1.—*Location map.*



244-838 O—67——2

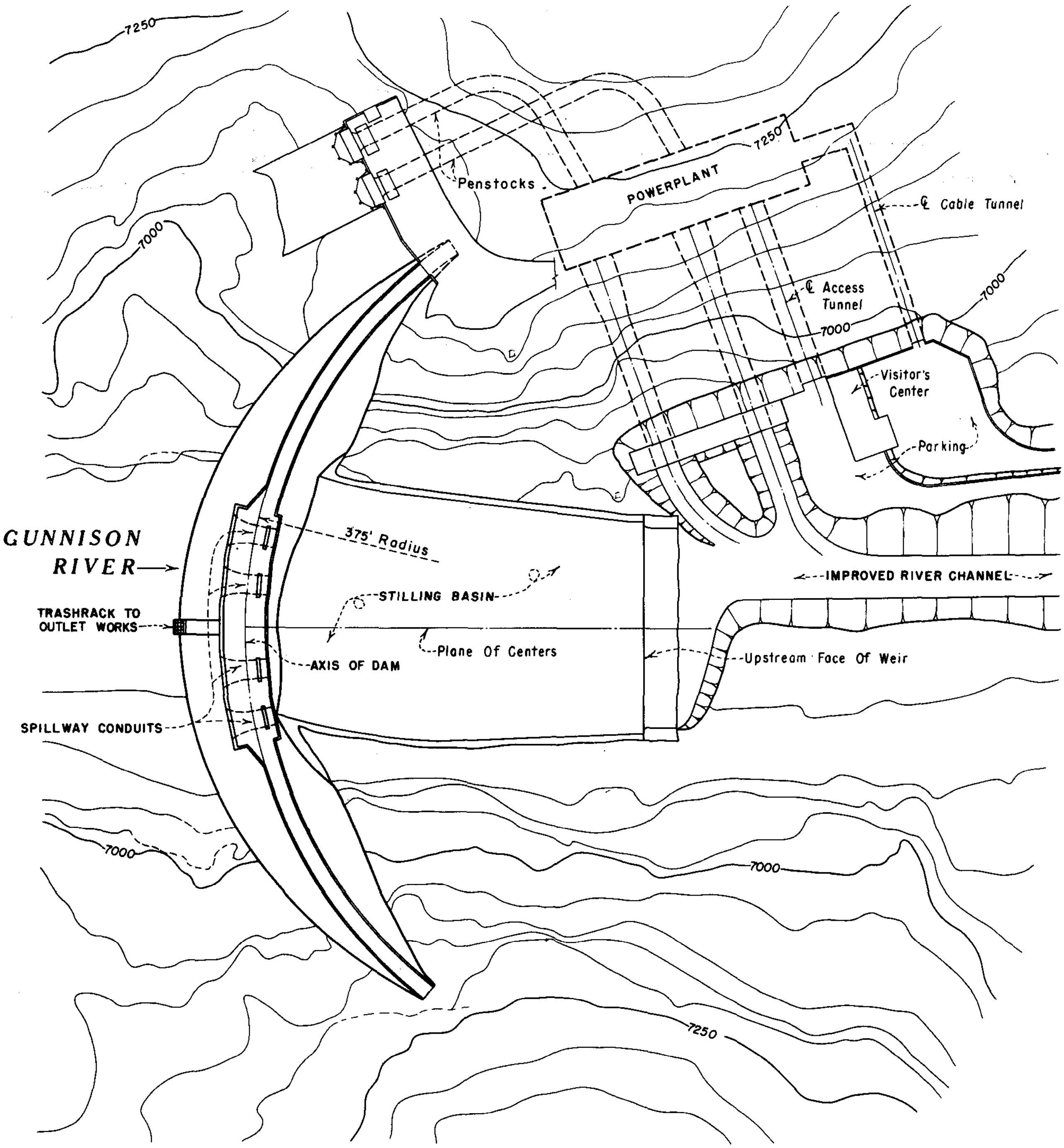


FIGURE 2.—*Plan of Morrow Point Dam and Powerplant.*

contains 360,000 cubic yards of concrete and has a crest length of 720 feet. The reservoir has a storage capacity of 117,000 acre-feet, extending upstream to within one-half mile of Blue Mesa Dam.

The spillway includes four 15- by 15-foot openings controlled by fixed-wheel gates, as shown on figure 5, with a capacity of 40,000 cubic feet per second. The discharge jets fall freely, approximately 350 feet to a stilling basin at the base of the

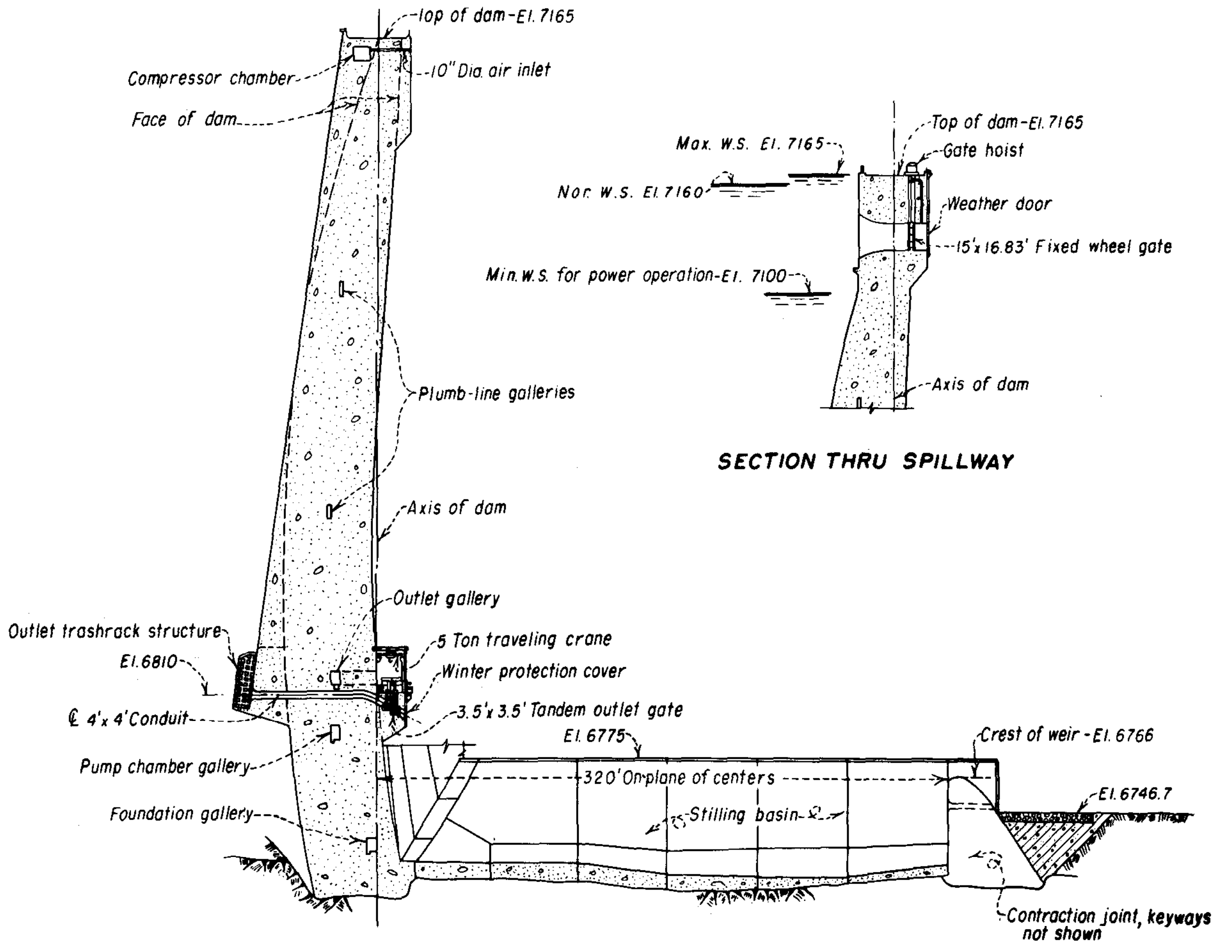


FIGURE 3.—*Section through the dam and stilling basin.*

dam. The basin, formed by a 65-foot-high weir, is lined with 5-foot-thick concrete and is about 180 feet wide. The weir is 320 feet downstream from the dam axis.

The outlet works near the base of the dam consists of a 4- by 4-foot steel-lined conduit controlled by 3.5- by 3.5-foot tandem slide gates, as shown on figure 6. The outlet works discharges up to 1,500 cubic feet per second. Model studies of the outlet works gates are described in a separate report.

The underground powerplant, figures 2 and 4, located in the left canyon wall, contains two 60,000-kilowatt generators and discharges up to 5,200 cubic feet per second through two tailrace channels into the river. The river channel is improved for a distance of approximately 1,200 feet downstream from the powerplant tailrace channels.

The preliminary design of the spillway and outlet works, which provided a basis for the first model tests, consisted of a free overflow spillway at the dam crest with a discharge capacity of 19,500 second-feet, and a two-conduit, slide-gate-controlled, low-level outlet works with a discharge capacity of 19,000 second-feet. The spillway discharge fell almost vertically into the stilling pool and the outlet works discharged into the pool at an angle of 15° below horizontal. The length of the stilling basin was limited by the location of the powerplant tailrace channels.

Although many modifications were tested, the outlet works arrangement was found to be unsatisfactory. The energy of the high-velocity outlet works efflux was not effectively dissipated in the stilling basin, resulting in excessive impact on the upstream face of the weir and undesirable flow turbulence and spray at the weir. The preliminary design was therefore abandoned without testing the overflow spillway. An alternative design was proposed that combined the spillway and outlet works discharges into a free-falling

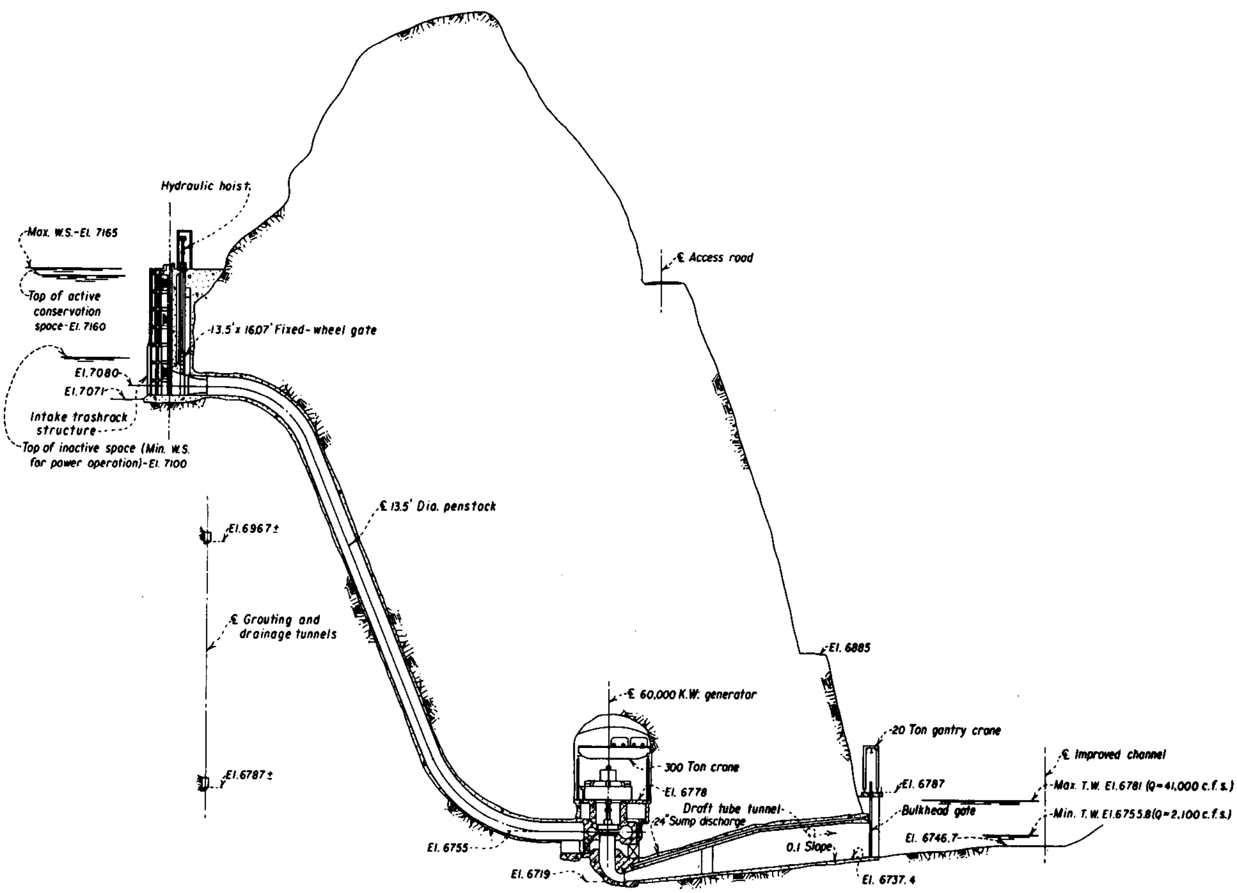


FIGURE 4.—*Section through the powerplant.*

spillway discharge through four gate-controlled outlets near the crest of the dam. The hydraulic model studies proved this design to be satisfactory. In the preliminary design the top of the parapets on the dam and the maximum water surface were elevation 7174. The normal water surface elevation was 7160. In the recommended design the maximum water surface was elevation 7165 and the normal water surface remained at elevation 7160.

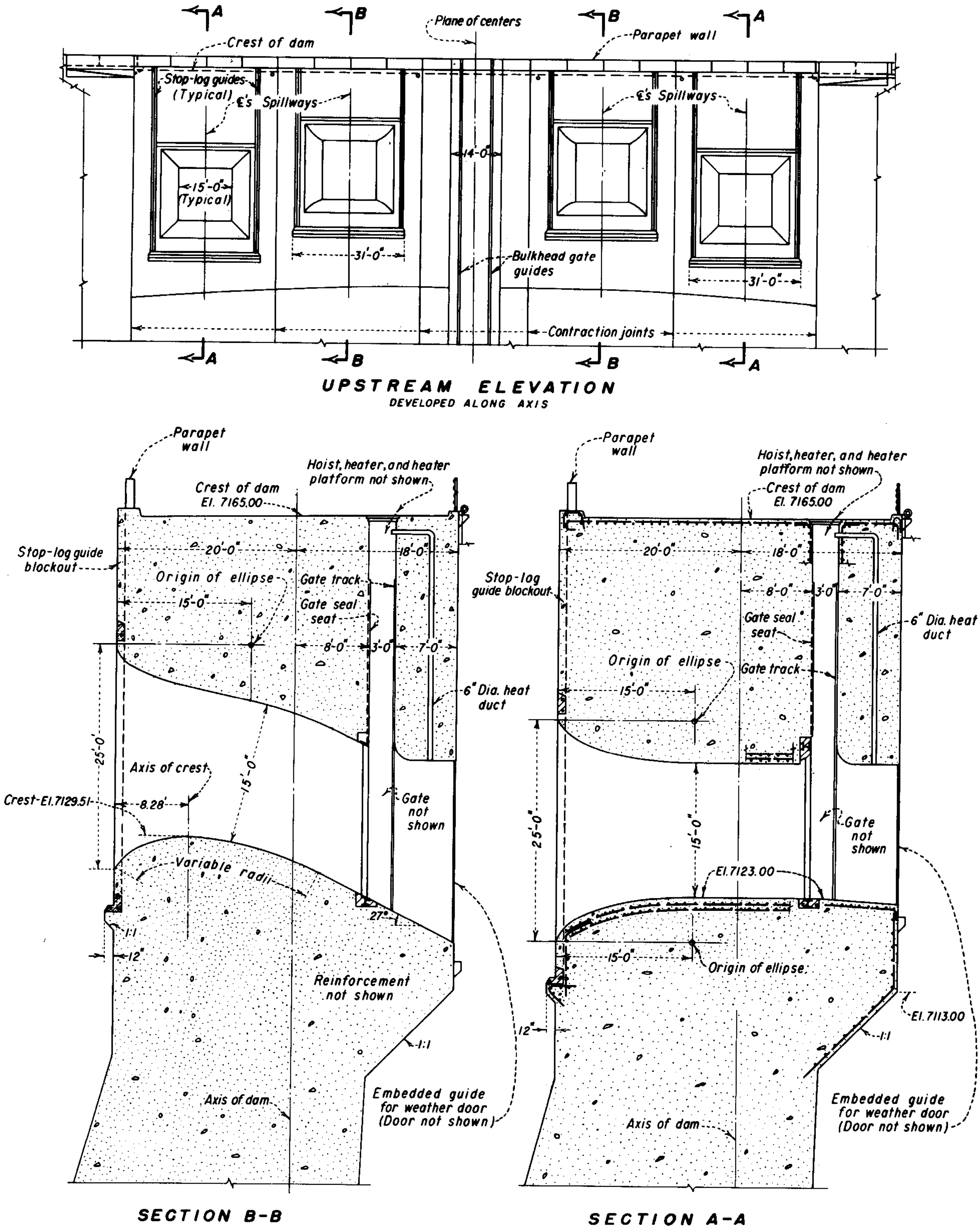


FIGURE 5.—*Elevation and sections of spillway conduits.*

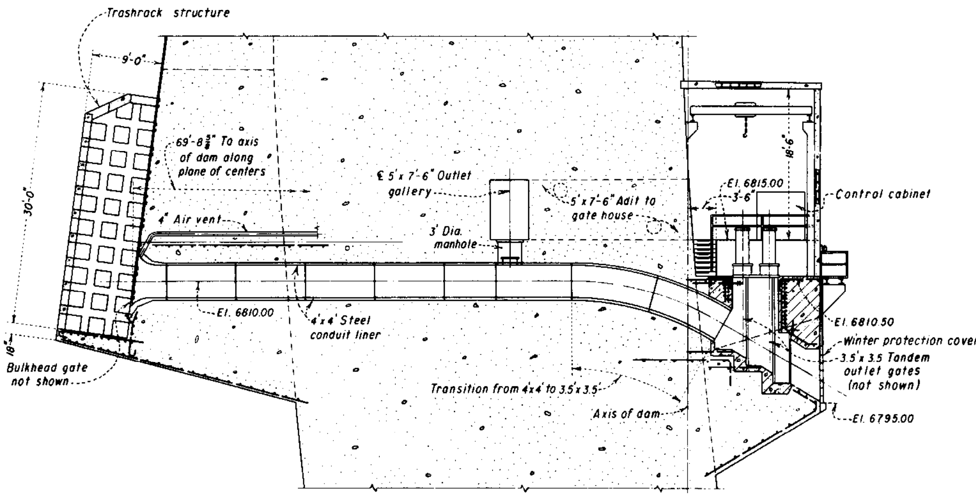


FIGURE 6.—*Section through the outlet works.*

# The Model

THE PORTIONS OF THE PRELIMINARY DESIGN simulated in the original 1:24[1] model included the slide-gate-controlled outlet works, the stilling basin, and the weir. The outlet works discharge was supplied through a pressure tank connected directly to the laboratory water supply system. Only the control gates were built; the upstream guard gates were not included in the model. The stilling basin topography was formed with concrete and the weir was formed with concrete screeded to sheet metal templates.

Following abandonment of the preliminary design, a headbox was included in which the spillway conduits and a portion of the upstream face of the dam were installed. The spillway conduits and bellmouth entrances were fabricated with sheet metal and included piezometers for pressure measurement. The fixed-wheel gates controlling the spillway flow were simulated by slide gates with correctly proportioned gate slots. The low-level outlet works was added later, with water supplied directly from the headbox. Discharge through the outlet works was controlled with a slide gate.

[1] The model was constructed to a scale of 1 dimensional unit equal to 24 units on the prototype. The prototype is the full-scale structure.

Piezometers were included in the stilling basin floor and the upstream face of the weir for determination of impact pressures. Final revisions included improved stilling basin topography, the powerplant tailrace channels, and a portion of the improved river channel.

Water was supplied to the spillway and outlet works through a recirculating system by centrifugal pumps. The combined spillway and outlet works discharges were measured by permanent volumetrically calibrated venturi meters. A portable centrifugal pump supplied the powerplant discharge, which was measured by a portable orifice meter. All discharges and elevations shown in this report are for the prototype. All dimensions are for the prototype unless otherwise noted.

The tailwater was controlled with an adjustable tailgate and the tail water elevation was measured by a staff gage in the improved river channel. Tailwater elevations were set and carefully controlled according to the curve of figure 7.

Special instrumentation consisted of pressure transducers, which were used to record the variation in impact pressure on the stilling basin floor and weir face and to more closely examine the pressures in the spillway conduits and bellmouth

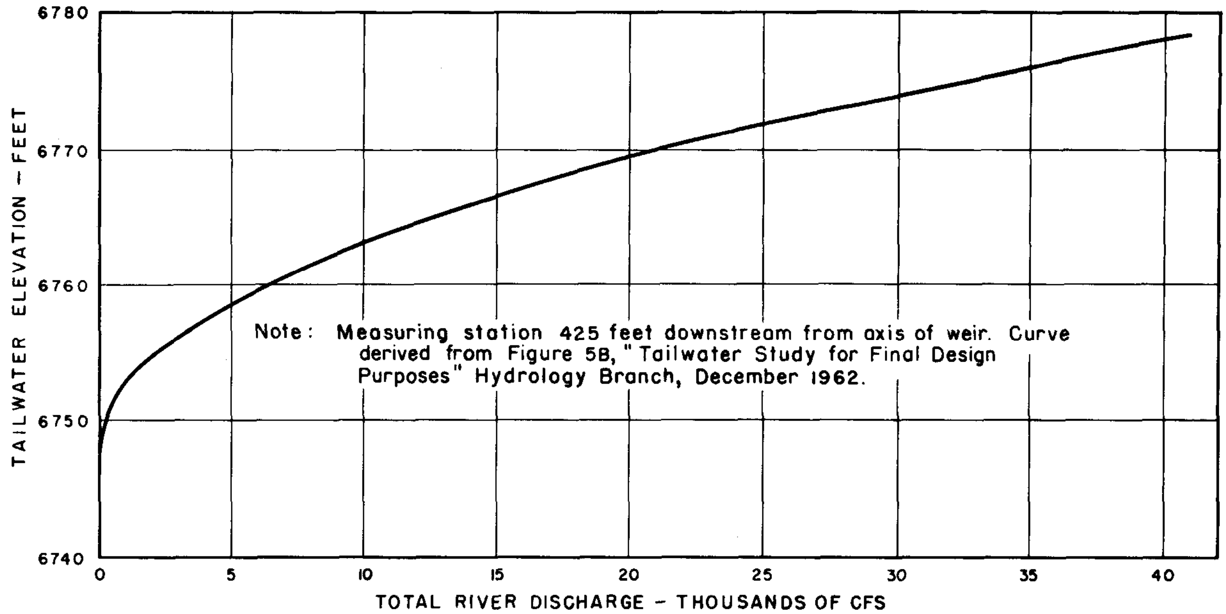


FIGURE 7.—*Design tailwater curve for discharges below the dam and powerplant.*

entrances. The transducers were connected to a six-channel direct writing oscillograph. The oscillograph records were analyzed with the aid of a short electronic digital computer program, which converted oscillograph deflections directly to prototype pressure heads.

# The Investigation

THE INITIAL PHASE OF THE investigation determined the hydraulic characteristics of the preliminary design. Operation of the model outlet works, consisting of two high-head, slide-gate-controlled conduits near the bottom of the dam, caused undesirable flow conditions in the stilling basin. Following abandonment of this design, the model aided in the development of the alternate scheme for the spillway and the outlet works. Testing of the flow characteristics of the alternate design provided the information which developed the recommended design.

## The Preliminary Design

The preliminary design for Morrow Point Dam included an outlet works and a spillway. The outlet works consisted of two 7- by 10.5-foot conduits, located at elevation 6800 near the base of the dam and controlled by high-head slide gates, which discharged at an angle of 15° below horizontal. The maximum outlet works design discharge was 19,000 cubic feet per second with the reservoir at the top of the parapet, elevation 7174. At normal reservoir elevation 7160 the outlets were designed to discharge approximately 18,600 cubic feet per second. The high velocity jets (approximately 130 feet per second) discharged into a stilling basin formed by a weir located about 220 feet downstream from the axis of the dam. The preliminary spillway was 132 feet wide with the crest at elevation 7162; its discharge capacity was 19,500 cubic feet per second.

The early model configuration included only the outlet works and stilling basin, as shown on figure 8. The spillway was to be added and tested after completion of the outlet works investigation.

Operation of the preliminary design model at the maximum discharge is shown in figure 9. The high-velocity jets were not effectively diffused in the stilling basin and struck the upstream face of the weir with great force; there was practically no energy dissipation in the basin. The initial tests indicated that the stilling basin was too short; therefore, the weir was moved downstream approximately 100 feet (320 feet downstream from the axis of the dam) and the tests were repeated. Flow conditions were improved, but the flow appearance and energy dissipation were still unsatisfactory for the maximum discharge. Even at 50 percent gate opening, the flow conditions were undesirable. Increasing the downward tip angle of the conduits



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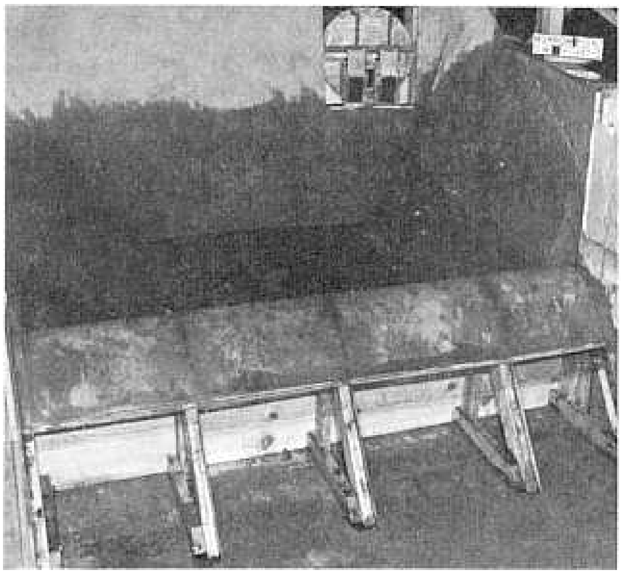


FIGURE 8.—*The preliminary model. Looking upstream at the weir, stilling basin, and outlet works.*

from 15° to 24° did not improve the energy dissipation or flow appearance.

A 24-foot-wide horizontal baffle was attached to the top of the upstream weir face, as shown on figure 10, in an attempt to improve the energy dissipation and confine the turbulence to the stilling basin. Again, little improvement was noted in the flow conditions.

These preliminary observations indicated that a longer stilling basin was needed for adequate energy dissipation. However, the weir could not be moved farther downstream because it would interfere with the powerplant tailrace, and the powerplant location could not be changed because of economic and geologic considerations. Other modifications of the preliminary design to achieve satisfactory operation, such as widening or deepening the basin, were considered impracticable. Therefore, the preliminary design of the outlet works and the overflow spillway was abandoned and an entirely new concept was proposed.

## The Alternate Design

The alternate design consisted of four 15-foot-square conduits with their inverts at elevation 7123. All four conduits discharged horizontally. The maximum design reservoir elevation was lowered from elevation 7174 to elevation 7165. Thus, the centers of the fixed-wheel gates controlling the discharge were submerged 34.5 feet. The stilling basin design, including the weir, remained the same as at the completion of tests on the preliminary design. The four conduits, which will hereafter be referred to as the spillway, were designed to have the same capacity as the combined total discharge of the preliminary spillway and outlet works, or 38,500 cubic feet per second. In reality, the model indicated that the spillway had a discharge capacity of approximately 40,000 cubic feet per second at the maximum reservoir elevation.

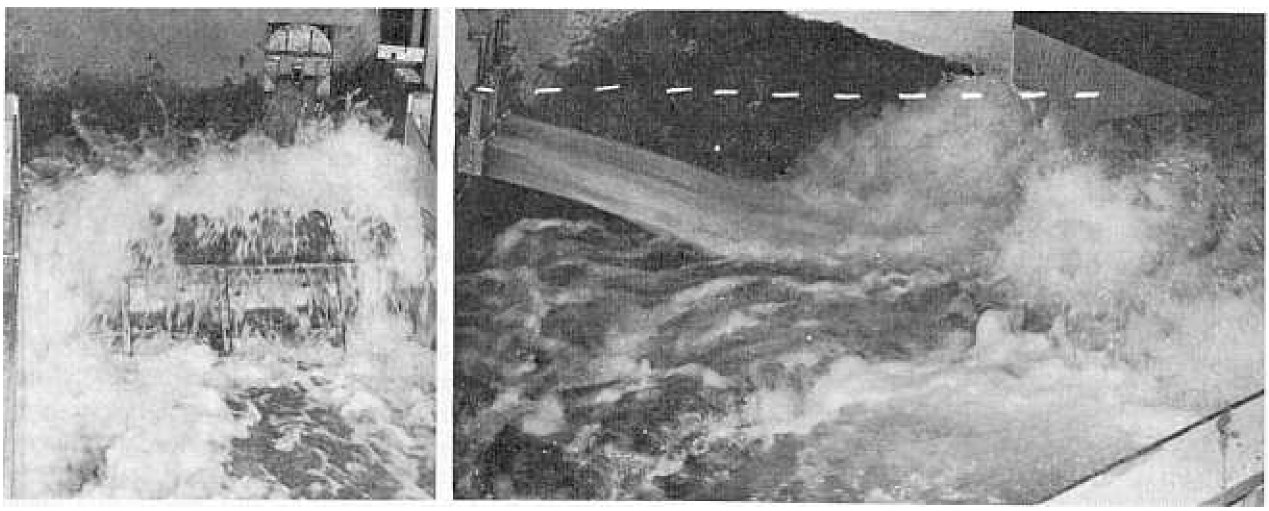


FIGURE 9.—*Testing the preliminary model. Maximum discharge of 19,000 cubic feet per second, views looking upstream and from the side.*

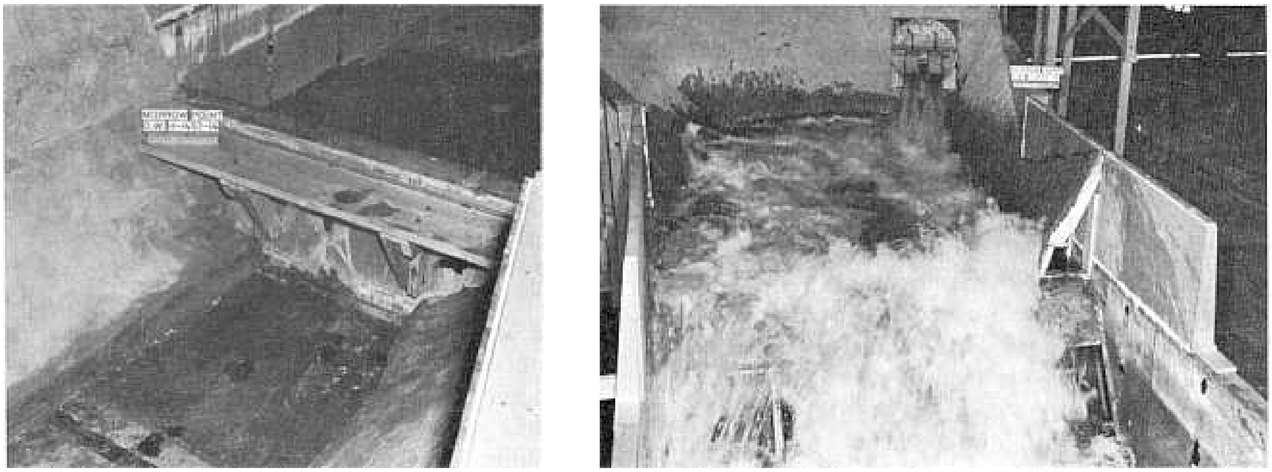


FIGURE 10. Weir with horizontal baffle on upstream face, and view showing test with maximum discharge of 19,000 cubic feet per second (preliminary design).

Figure 11 shows discharge from the spillway with all four gates equally open to provide maximum capacity and one-half capacity outflows, and the corresponding stilling basin flow conditions. The jets penetrated through the pool to the floor of the basin with a strong downstream velocity along the basin floor. Flow conditions at the weir were similar to those observed for

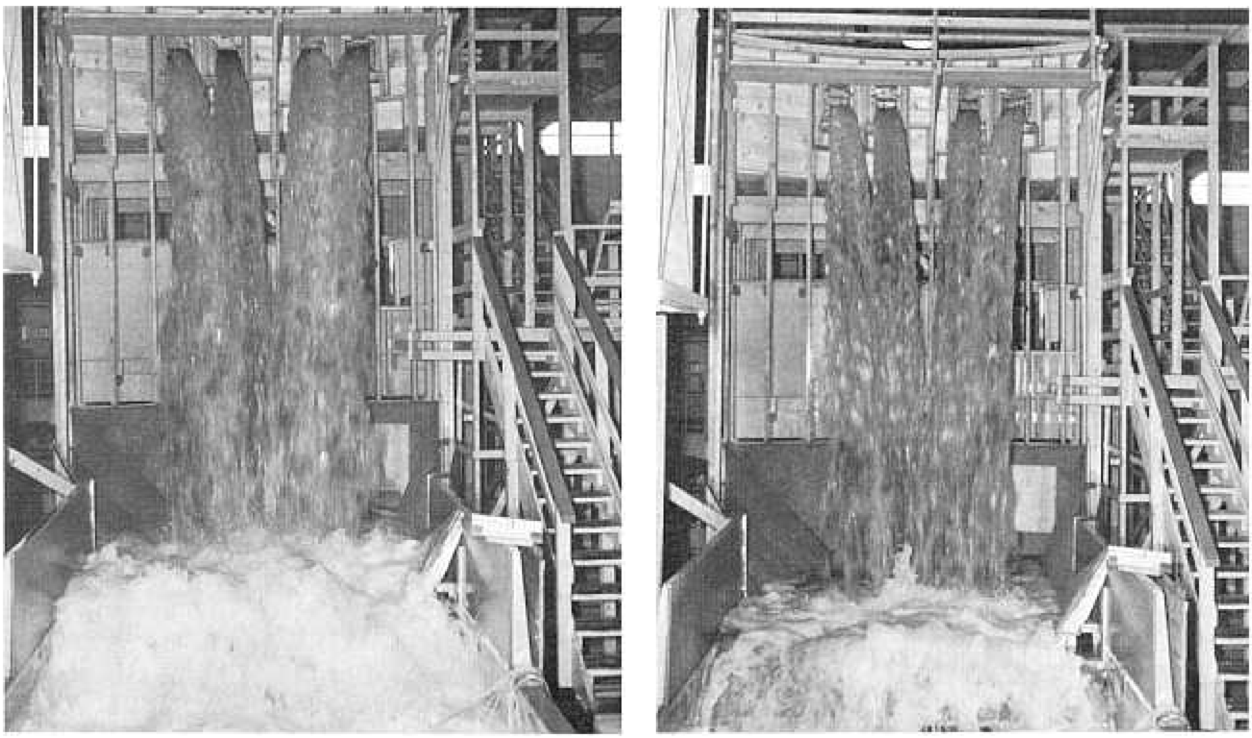


FIGURE 11.—Test on spillway with horizontal conduits of the alternate design. Left view: Reservoir elevation 7165, gates fully open (Q=40,000 cubic feet per second). Right view: Reservoir elevation 7160, gates half open (Q=20,000 cubic feet per second).

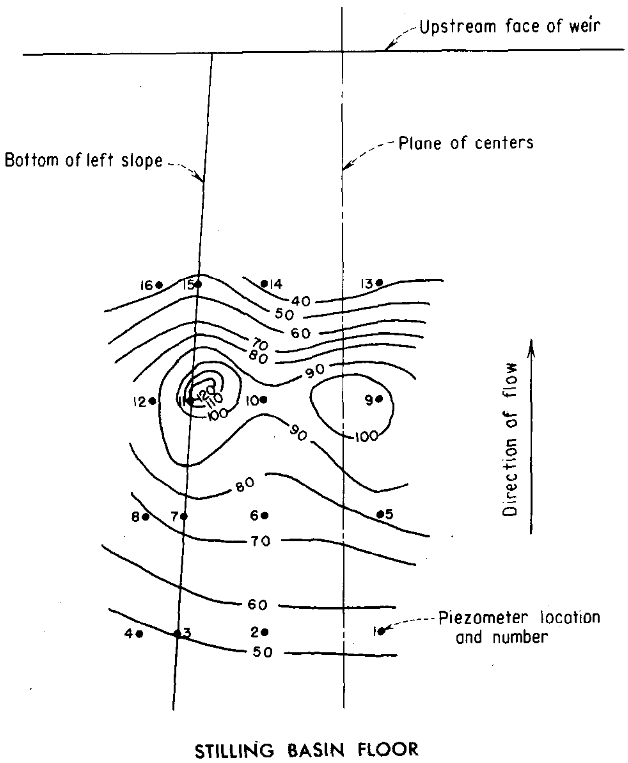
the preliminary design, but the flow was more evenly distributed across the width of the basin and weir.

## Impact Pressure Tests

The maximum impact pressures on the upstream face of the weir and on the stilling basin floor were measured by open-tube water manometers connected to piezometers. The results of these tests are shown on Figure 12. The piezometer locations and the contours of equal pressure (in feet of water for the prototype) are plotted for this and all subsequent impact pressure tests. All pressures are referenced to the piezometer opening. The maximum pressure on the floor was about 120 feet of water, occurring near the bottom of the left slope of the stilling basin (Piezometer 11) about 75 feet upstream from the weir. Pressures on the upstream face of the weir varied from about 40 feet of water near the corbel to a maximum of approximately 80 feet of water near the center of the weir base (Piezometer 20). Piezometer coverage was limited to the left half of the basin and weir, assuming that the pressure distribution would be essentially symmetrical about the centerline. Later tests included more complete coverage and a more accurate determination of maximum pressures by using electronic transducers.

## Revision of the Inside Spillway Conduits

Figure 13 shows operation of the spillway with approximately one-half the maximum discharge through either the two inside conduits or the two outside conduits. This operation indicated that the undesirable flow conditions at the weir were due primarily to discharge from the two inside gates. Deflecting hoods were placed on the downstream side of the inside conduits to test this premise. The hoods deflected the inside jets downward, causing them to strike the stilling basin water surface 75 to 100 feet upstream from the zone of impact of the outside jets. Figure 14, when compared with figure 13, illustrates the improved stilling basin flow conditions. Figure 15, compared to figure 12, shows the reduction in impact pressure on the stilling basin floor and



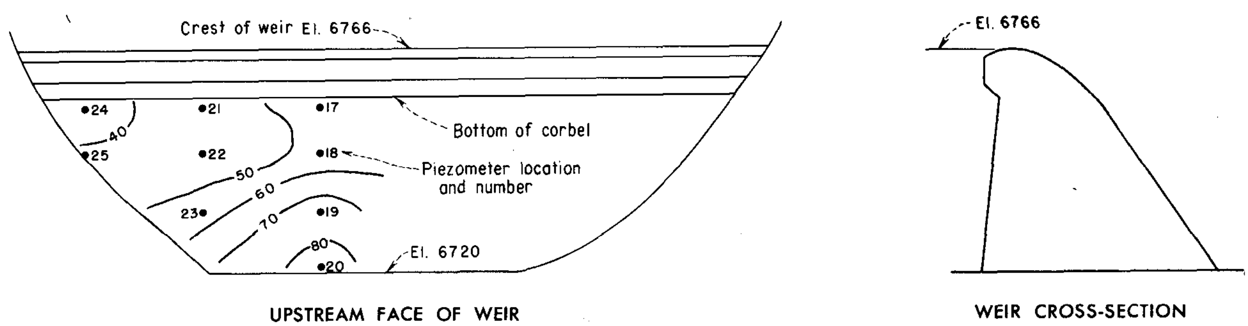




FIGURE 12.—*Maximum manometer impact pressures on weir and stilling basin floor, measured in feet of water for the prototype (alternate design, Q=40,000 cubic feet per second).*

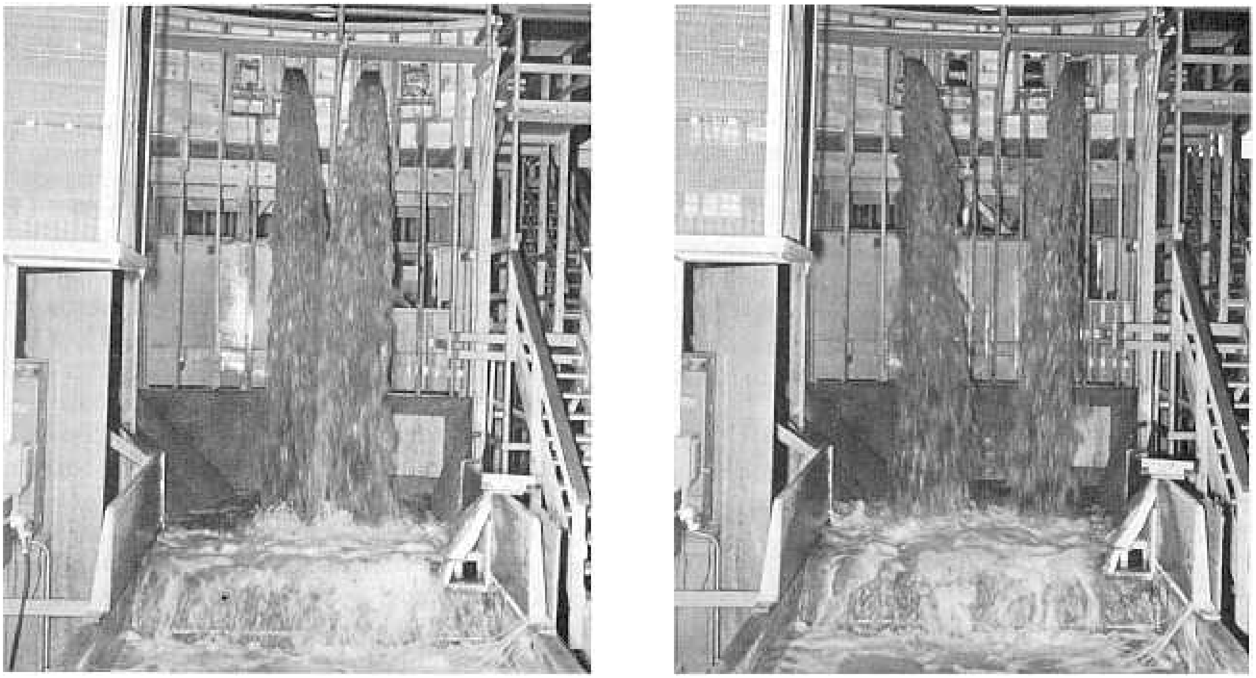


FIGURE 13.—*Test on spillway with horizontal conduits of the alternate design. Reservoir elevation 7160. Left view: inside gates fully open. Right view: outside gates fully open (approximate Q=20,000 cubic feet per second in each view).*

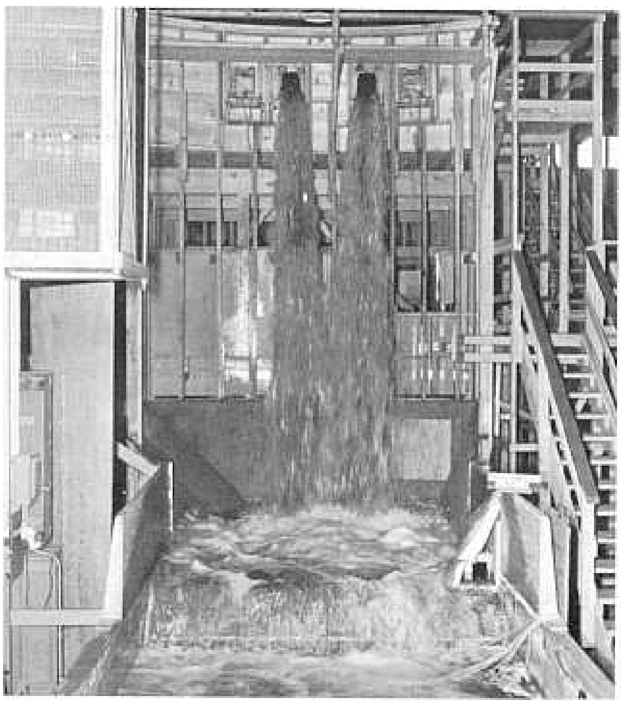


FIGURE 14.—*Test on spillway of alternate design with deflecting hoods on inside conduits. Reservoir elevation 7160, inside gates fully open (approximate Q=19,600 cubic feet per second).*

weir with the deflecting hoods installed. Maximum impact pressures on the stilling basin floor were reduced from 120 feet to about 70 feet of water, and the maximum impact pressure on the weir was reduced from 80 feet to about 50 feet.

Although the deflecting hoods resulted in satisfactory flow conditions, their inclusion in the recommended design was impractical because of structural design problems. The hoods also slightly reduced the spillway discharge capacity.

A practical method to move the impact zone of the inside jets upstream and make use of a large area of the stilling basin was to tip the inside conduits downward. Computations using the path of projectile equations showed that a tip angle of 27° would cause the inside jets to strike the pool in approximately the same location as observed with the deflecting hoods. Stilling basin flow conditions were greatly improved with the tipped conduits, figure 16, and were considered satisfactory.

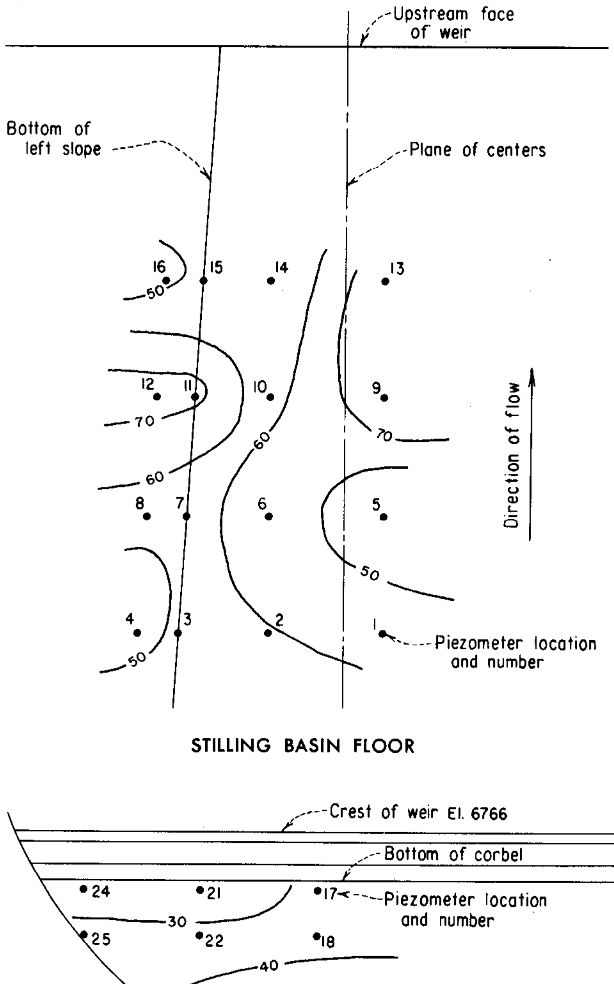
Figure 17 shows the magnitude of maximum impact pressures on the weir, obtained with electronic pressure transducers and a direct writing oscillograph. The pressures indicated are instantaneous absolute maximums, and therefore cannot be directly compared with those of figures 12 and 15, which were recorded with water manometers. A concentration of higher pressures was noted under the weir corbel for several combinations of gate operation. At the maximum spillway discharge of 40,000 cubic feet per second, figure 17, a pressure of about 60 feet of water occurred immediately under the corbel near the center of the weir. Since this force near the top of the weir would significantly increase the over-turning moment of the weir, the corbel was removed in later studies when the model was revised to include the powerplant tailrace, outlet works, and improved downstream river channel.

## Modification of Bellmouth Entrances

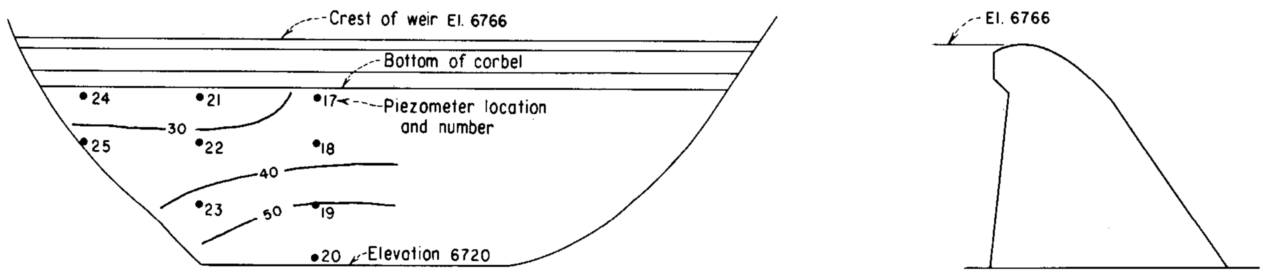
The original alternate design, with the four spillway conduits horizontal, included symmetrical, elliptical bellmouth entrances, designed according to Corps of Engineers data.[2] With the two inside conduits tipped downward, it became necessary to modify the bellmouth entrance shapes of the two inside conduits. The inside bellmouths were modified to include an approximate ogee crest shape on the conduit bottom, with the roof a combination of an elliptical curve and a reverse curve approximately paralleling the downstream portion of the ogee section. The sides remained elliptical as before. Figure 5 shows the recommended configurations. The model was changed to include one revised bellmouth and the upstream face of the dam was simulated for the two right conduits, as shown on figure 18. Sixty-eight piezometers were installed in the two right conduits to determine the effects of the bellmouth shapes on the pressure distribution. Piezometer locations and pressure profiles for 100 percent gate opening are shown in figure 19. The pressures were obtained from water manometers, and in all cases the pressures exhibited negligible fluctuation when examined with electronic transducers.

Under maximum reservoir head subatmospheric pressures as low as approximately 6 feet of water occurred on the roof of the horizontal conduit. Pressures on the roof were above atmospheric from the entrance to a point about

[2] U.S. Corps of Engineers Technical Memorandum No. 2-428, Report No. 1, "Investigation of Entrance Flared in Four Directions", Waterways Experiment Station, Vicksburg, Miss., March 1956.




STILLING BASIN FLOOR




UPSTREAM FACE OF WEIR

WEIR CROSS-SECTION

FIGURE 15.—*Maximum manometer impact pressures on weir and on stilling basin floor, measured in feet of water for the prototype (alternate design, with deflecting hoods on inside spillway conduits Q=40,000 cubic feet per second).*

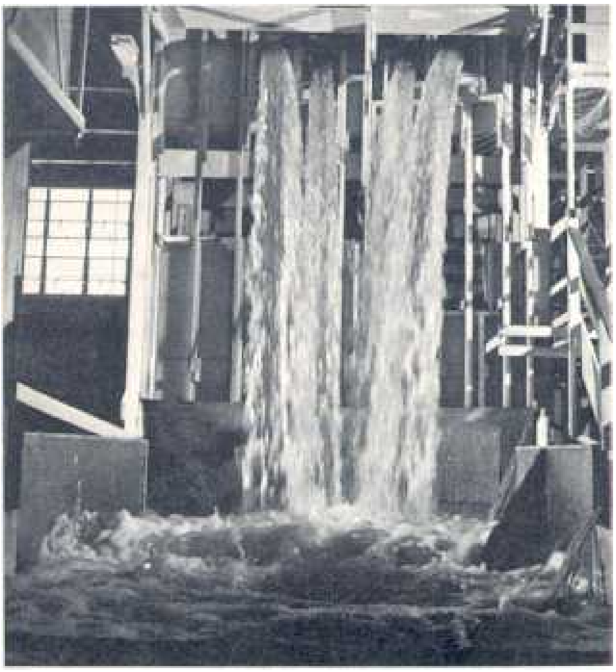


FIGURE 16.—*Test on spillway of alternate design with inside conduits tipped 27° downward. Reservoir elevation 7165 with gates fully open (approximate Q=40,000 cubic feet per second).*

7 feet downstream from the entrance and throughout the conduit floor. Subatmospheric pressures as low as 8 feet of water existed on the floor of the tipped conduit, between the gate and a point about 7 feet downstream from the entrance. Subatmospheric pressures as low as 5 feet of water occurred on the conduit roof between the gate and a point about 3 feet downstream from the entrance.

A pressure of 9 feet of water below atmospheric was recorded at Piezometer 66 in the horizontal conduit during one test run. Attempts to repeat this reading showed a subatmospheric pressure of only 3 feet of water. The former reading was apparently caused by separation of the flow from the conduit roof. It is expected that either condition could exist in the prototype. Slight subatmospheric pressures were observed on the floor of the horizontal conduit downstream from the gate (Piezometers 59–61); these low pressures apparently were caused by the slight divergence of the floor and the tendency of the jet to separate from the invert.

Pressures were also recorded with the reservoir at normal elevation to determine the effect of reduced upstream head on the pressure magnitude. This would not be a prototype condition, since full gate opening is required only at the maximum reservoir elevation. The results of this test are not shown but indicated a pressure distribution similar to that of figure 19.

The manometer pressure tests on the spillway conduits showed that if the gates were closed slightly (approximately 1 percent), so that the bottom of the leaf controlled the flow, pressures at all points upstream from the gates were above atmospheric.

Modifications to the upstream vertical face of the spillway section of the structure were tested in the model in an attempt to improve the pressure distribution in the outside bellmouth entrances. One modification moved the vertical corners of the structure farther from the spillway entrances. Another trial modification provided rounding of the vertical corners with a 4-foot radius. Neither of these modifications produced any noticeable change in the bellmouth pressures.

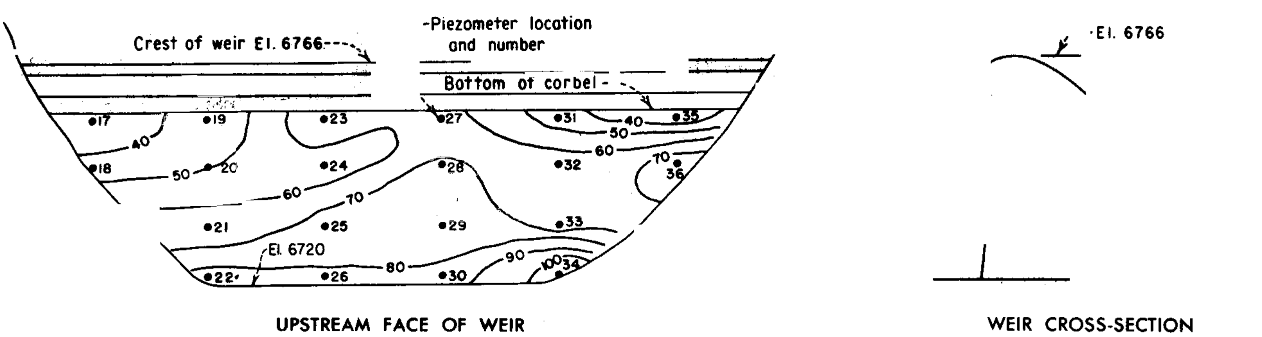



FIGURE 17.—*Maximum manometer impact pressures on the weir from discharges from spillway of alternate design, modified by tipping inside conduits 27° downward (reservoir elevation 7165 with all gates fully open, approximate Q=40,000 cubic feet per second).*

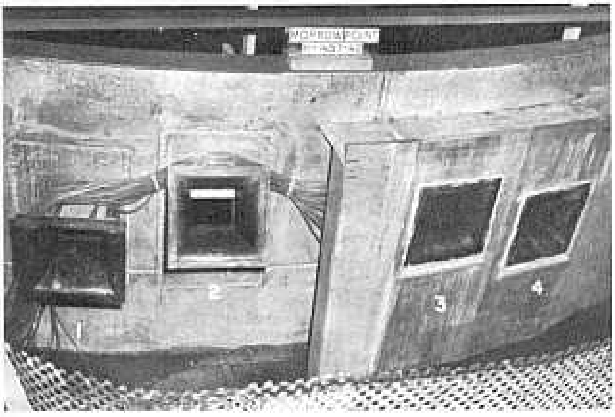


FIGURE 18.—*Upstream face of dam showing modifications to spillway entrances:*
1. *Original configuration. All four conduits horizontal.*
2. *First modification. Original bellmouth tipped 27° downward.*
*3 and 4. Recommended configuration. No. 3 conduit tipped 27° downward with modified bellmouth. No. 4 conduit horizontal with original bellmouth. Upstream face of dam installed.*

Therefore, no modification was made in the prototype design.

## The Recommended Design

Since model data indicated at this point that the alternate scheme for the spillway with the recommended bellmouth entrances would provide satisfactory operation, it was decided to include additional details to more accurately evaluate the hydraulic performance of the structure.

The recommended bellmouth entrance was installed in the left inside spillway conduit, and the upstream face of the dam was correctly simulated. The stilling basin topography was reshaped to correspond to the most recent field surveys. The weir was modified to eliminate the corbel, making the upstream face vertical, and was placed so that the vertical face was 320 feet downstream from the dam axis. The 3.5-foot-square outlet works was installed near the base of the dam to discharge into the stilling basin. The outlet works flow was supplied from the head box and controlled by a slide gate.

The powerplant tailrace channels and a portion of the improved downstream river channel were also included. The powerplant discharge was supplied by a portable centrifugal pump and was measured by a portable orifice meter.

The model, following this revision, simulated the recommended design of the outflow structures for Morrow Point Dam and Powerplant. A series of hydraulic tests was made on this model to determine the flow characteristics below the dam. Variations of discharges from the spillway, the outlet works, and the powerplant were tested independently and in combination. These tests resulted in minor revisions, which affected the recommended design for the stilling basin and weir, and the powerplant tailrace and river channel topography.

## Development of Design Details

*Stilling basin topography.*—The right spillway jet struck the topography near the intersection of the right bank and the pool water surface. Therefore, the right side of the basin was widened about 3 feet (prototype) so that the right jet would not strike the topography. The top of the lining at the bank was raised 10 feet (prototype) between the weir and a point 140 feet downstream from the dam axis on both sides of the stilling basin. From this point, the top of the lining sloped downward at 6:1 toward the dam until intersecting the original top of lining. The left side of the basin near the weir was made 3 to 6 feet narrower than before. These changes in the stilling basin topography were designed to improve the flow characteristics of discharges from the spillway and the outlet works.

*Design discharge requirements.*—Up to this time, the model operation had been based on the total capacity of the spillway with all gates wide open under maximum reservoir head, which resulted in a discharge of approximately 40,000 cubic feet per second. With an outlet works discharge of 1,500 cubic feet per second and a powerplant discharge of 5,200 cubic feet per second, the total river flow was approximately 46,700 cubic feet per second.

Since the actual inflow design flood peaks at about 41,100 or 5,600 cubic feet per second less than the above indicated capacity, it could be released as a spillway discharge of 34,400 cubic feet per second, plus 1,500 cubic feet per second through the outlet works and 5,200 cubic feet per second through the powerplant. Subsequent tests included runs at the maximum spillway capacity of 40,000 cubic feet per second, the design discharge of 34,400 cubic feet per second, and three-fourths, one-half, and one-

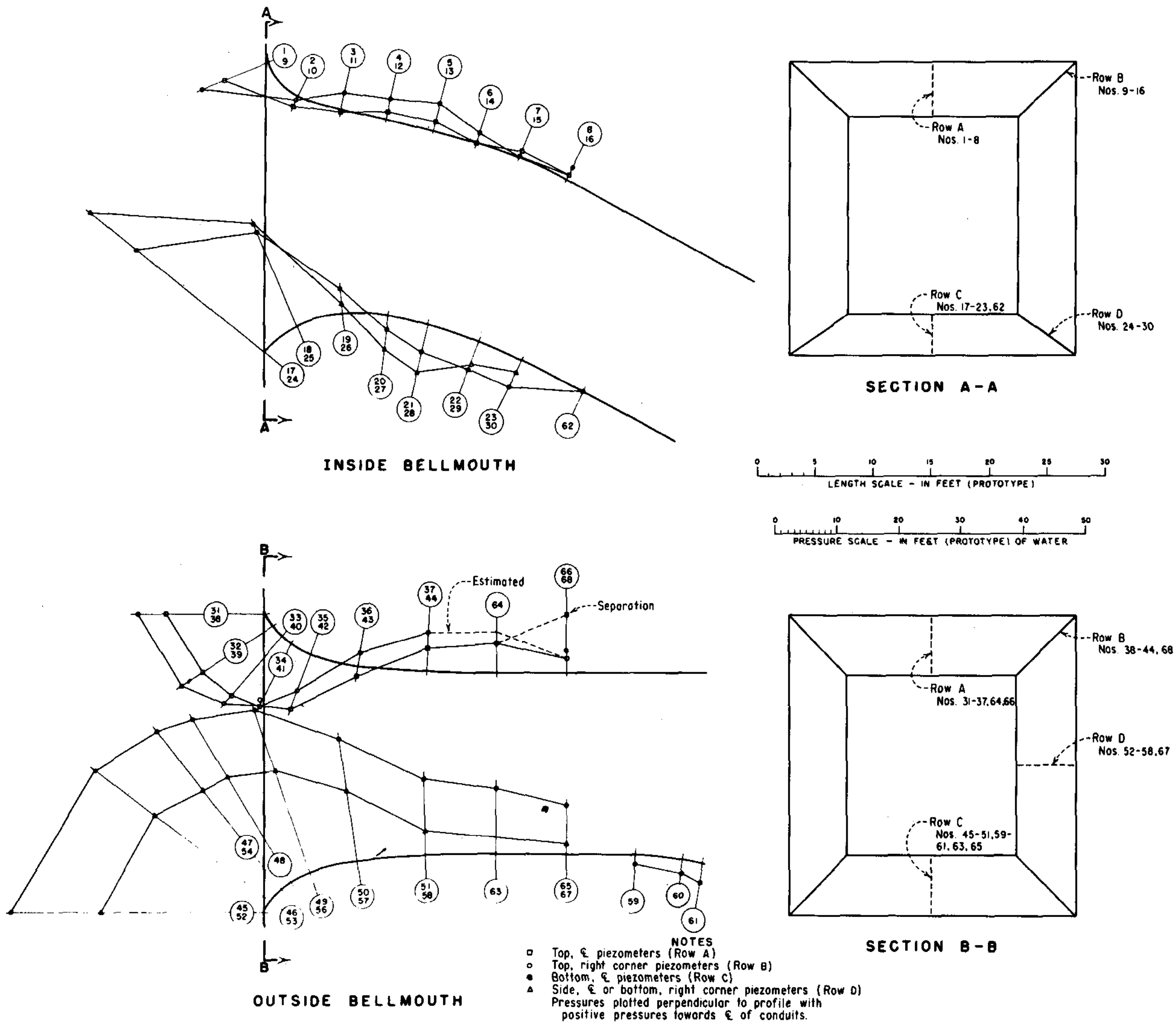



FIGURE 19.—*Water manometer pressures in spillway bellmouth entrances. Reservoir elevation 7165, gates fully open.*

fourth of the design discharge, with the outlet works and powerplant operating as described above. Another possible operating condition consisted of the total design inflow discharging through the spillway and outlet works, with the powerplant shut down. This condition corresponds to a spillway discharge of 39,600 cubic feet per second (41,100 minus 1,500 through the outlet works). This is nearly the same as the maximum spillway capacity and was therefore not included in the model test program.

*Modification of the powerplant tailrace.*—The extreme turbulence which occurred in the stilling basin for spillway discharges greater than approximately 26,000 cubic feet per second resulted in large waves and surges, which were carried downstream to the powerplant tailrace, as shown on figure 20. The topography surrounding the tailrace channels was subjected to sudden large impact forces because of these flow conditions. Also, there was a drawdown in the water surface at the downstream end of the left bank of the left tailrace channel during operation of the powerplant alone, as shown on figure 21. The tailrace topography was modified to alleviate the possibility of the rock being dislodged by these flow conditions and falling into the tailraces. The area between the tailraces was shortened and rounded to eliminate the thin dividing section, the right bank of the right channel was shortened and rounded and the left bank of the left channel was rounded. These

FIGURE 20.—*View showing flow in unmodified powerplant tailrace with all gates fully open (approximate discharges: spillway Q=40,000 cubic feet per second, outlet works Q=1,500 cubic feet per second, powerplant Q=5,200 cubic feet per second).*

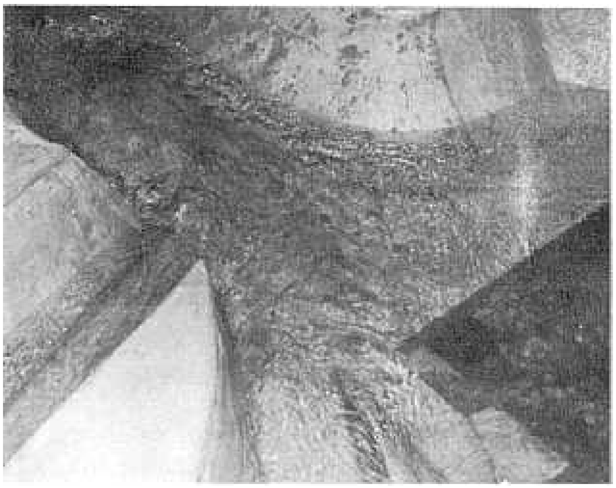


FIGURE 21.—*Flow in unmodified powerplant tailrace with discharge from powerplant only (Q=5,200 cubic feet per second).*

modifications of the tailrace channel are shown on figure 22.

The topography between the tailrace channels was further modified in an attempt to reduce wave action in the left tailrace channel near the visitors' center parking area. The elevation of the top of the topography was increased first by 5 feet and then by 10 feet. The 5-foot increase was ineffective, but the 10-foot increase reduced the

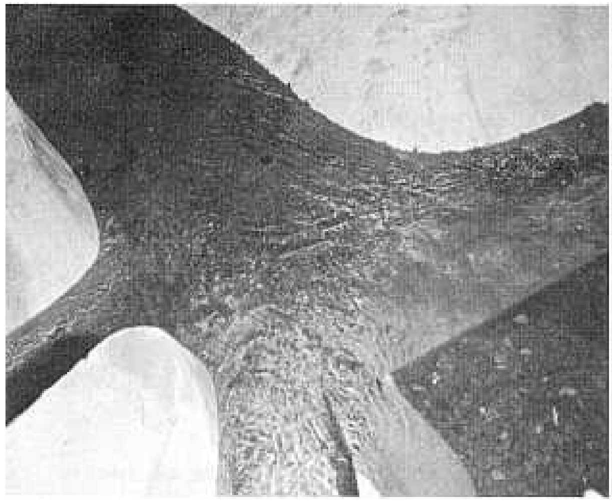


FIGURE 22.—*Powerplant tailrace channel showing improved flow after modification, powerplant discharge only.*

splashing during the maximum design discharge. A 4-foot-high floodwall around the visitors' center was also tested, as shown on figures 23 and 24. The floodwall protected the area from inundation for spillway discharges below approximately 26,000 cubic feet per second. At 26,000 cubic feet per second, minor infrequent splashing over the wall occurred. At a spillway discharge of about 31,000 cubic feet per second, large waves frequently overtopped the wall. For discharges above 31,000 cubic feet per second, the wall was ineffective.

Because of the infrequency of occurrence of the design discharge, it was decided not to attempt additional improvements to the stilling basin and

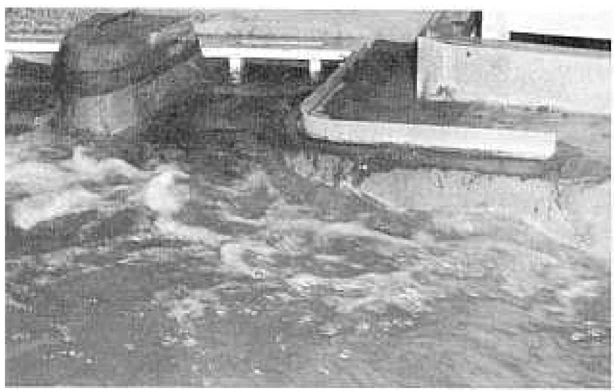


FIGURE 23.—*Test of floodwall protection around visitors' center (approximate discharges: spillway Q=26,000 cubic feet per second, outlet works Q=1,500 cubic feet per second, and powerplant Q=5,200 cubic feet per second).*

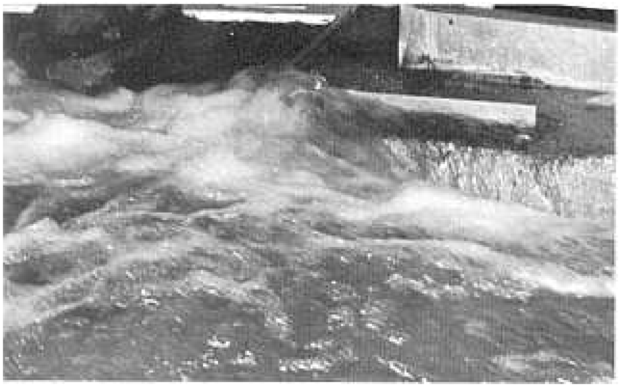


FIGURE 24.—*Test of floodwall protection around visitors' center (approximate discharges: spillway Q=32,900 cubic feet per second, outlet works Q=1,500 cubic feet per second and powerplant Q=5,200 cubic feet per second).*

tailrace channel flow conditions. Also, since the spillway and outlet works were capable of discharging the maximum design inflow, powerplant operation could be suspended during high spillway flows.

*Revision of the improved downstream channel.*—The relatively narrow entrance to the improved channel downstream from the weir acted as a control section, causing high tailwater immediately downstream from the weir and a drop in the water surface at the control section. The right channel bank, which is a talus slope in the prototype, was excavated in an attempt to alleviate the backwater condition. Figures 25 and 26 show flow conditions immediately downstream from the weir and in the stilling basin. The excavation resulted in the control section being moved farther downstream, causing some improvement, even though the backwater condition was not eliminated.

*Modification of the weir.*—To improve flow conditions at the weir, the piers on either side of the weir were removed and the stilling basin lining was extended downstream. The modified weir and basin are shown on figure 27. Although there was no apparent improvement in the flow conditions in the tailrace area, as indicated on figure 28, the stilling basin flow conditions were slightly improved, as shown on figure 29.

*Impact pressures on the stilling basin floor and the weir.*—Impact pressures were obtained on the weir and on the stilling basin floor of the recommended design, as shown on figures 30, 31, and 32. Pressure patterns on the weir and the floor were

FIGURE 25.—*Flow conditions downstream from the weir after excavation of talus slope on the right bank (approximate discharges: spillway Q=17,200 cubic feet per second, outlet works Q=1,500 cubic feet per second, and powerplant Q=5,200 cubic feet per second).*

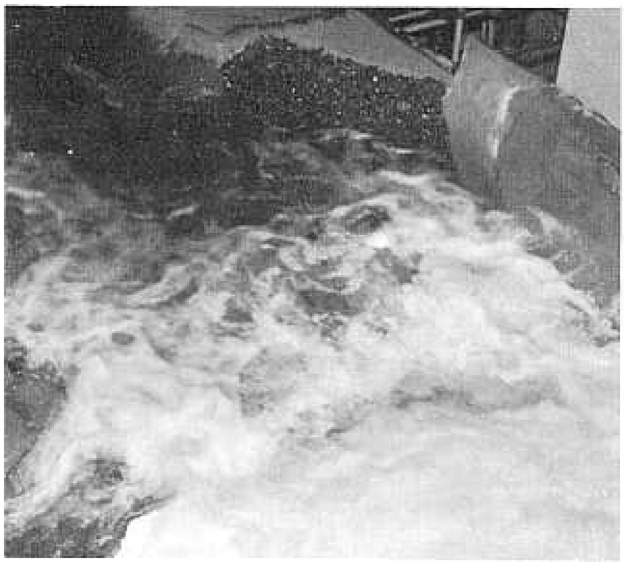


FIGURE 26.—*Flow conditions downstream from the weir after excavation of talus slope on the right bank (approximate discharges: spillway Q=34,400 cubic feet per second, outlet works Q=1,500 cubic feet per second, and powerplant Q=5,200 cubic feet per second).*

similar to those in tests made prior to revision of the stilling basin, the weir, and the powerplant tailrace channel.

The revised weir configuration, with the corbel and piers removed, showed substantially lower

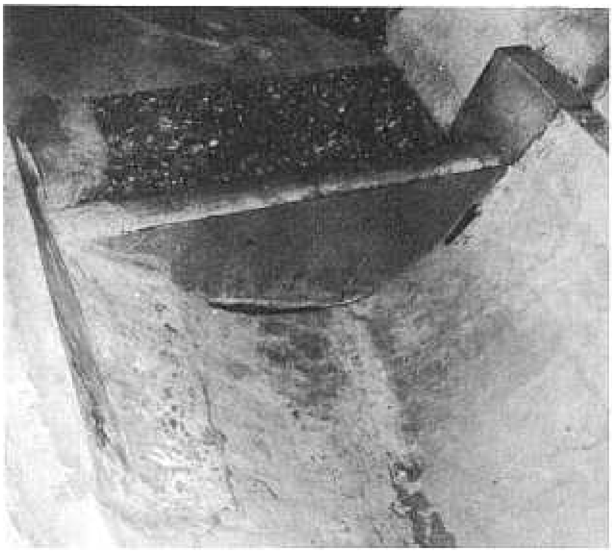


FIGURE 27.—*View downstream showing the piers removed from each side of the weir and extension of the lining downstream from the stilling basin.*

FIGURE 28.—*Flow conditions in the tailrace area following removal of the piers from the weir structure and extension of the lining (maximum design discharge: spillway Q=34,400 cubic feet per second, outlet works Q=1,500 cubic feet per second, and powerplant Q=5,200 cubic feet per second).*

pressures near the top of the weir, as shown on figure 30. Tests prior to removal of the corbel, shown on figure 17, would indicate that the corbel was the cause of the concentration of pressure high on the weir. Pressure fluctuations on the downstream face of the weir were relatively steady. The largest fluctuation

FIGURE 29.—*View showing flow characteristics in the stilling basin following removal of the piers from the weir structure and extension of the stilling basin lining. Maximum design discharge.*

was approximately 2 feet of water for the maximum design discharge. The average magnitude of pressures was approximately hydrostatic, i.e., due to tailwater depth alone.

The impact zones of the outside spillway jets on the stilling basin floor are clearly apparent in figures 31 and 32. The model piezometer distribution did not allow determination of the configuration of the impact zones of the inside jets, but the patterns would have been similar to those observed for the outside jets. Pressures under the right outside jet (left side of the figures) were higher than those under the left outside jet because the topography was closer to the water surface on the right side of the stilling basin.

Impact pressures were observed for various spillway discharges and, in general, were less for the revised stilling basin. However, in the initial tests, asymmetrical flow conditions included wide open gates (Q=20,000 cubic feet per second); while for the recommended design, the gate openings were limited to allow only one-half the design discharge (17,200 cubic feet per second). Therefore, the lower impact pressures observed in the final tests on the stilling

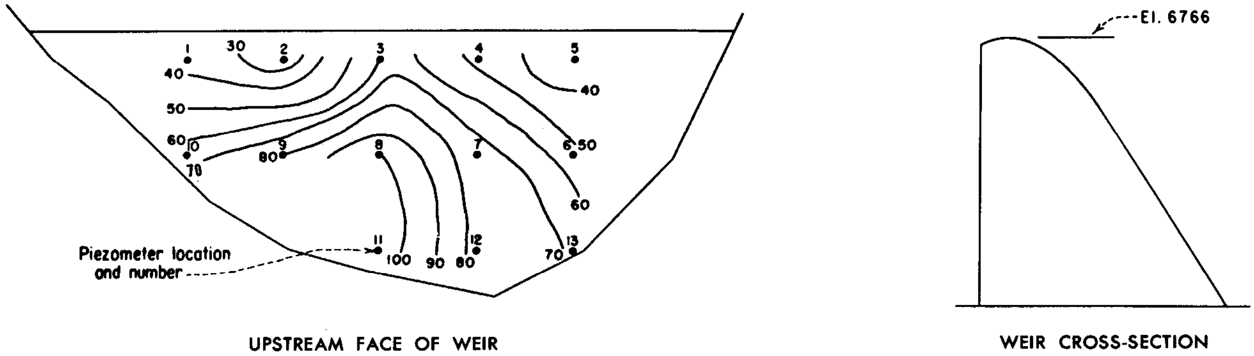



FIGURE 30.—*Maximum instantaneous impact pressures on the upstream face of the weir, following removal of the corbel and the piers. Reservoir elevation 7165 with spillway gates fully open. Spillway Q=40,000 cubic feet per second.*

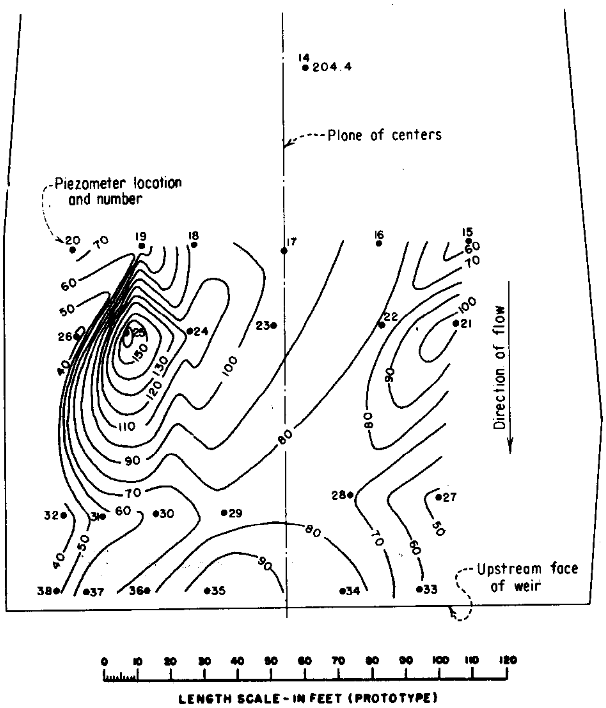



FIGURE 31.—*Maximum instantaneous impact pressures on the stilling basin floor. Reservoir elevation 7165 with spillway gates fully open (Q=40,000 cubic feet per second).*

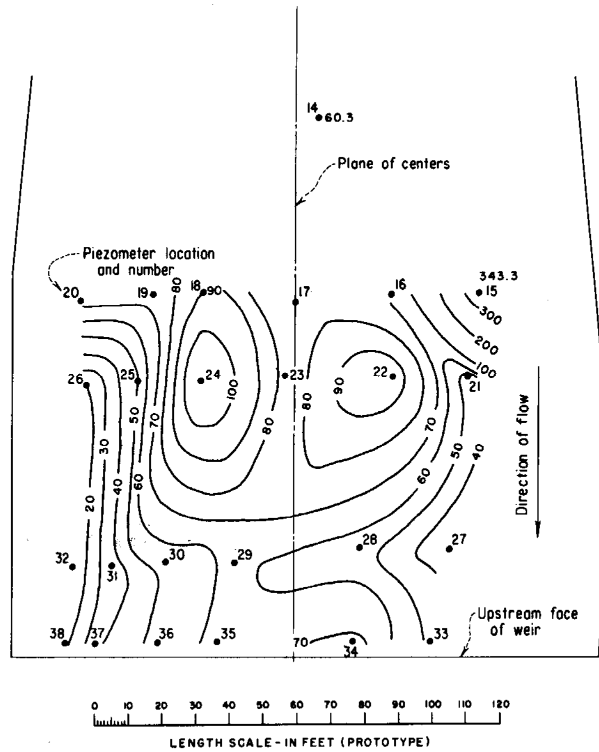



FIGURE 32.—*Maximum instantaneous impact pressures on the stilling basin floor. Reservoir elevation 7160 with outside gates open and inside gates closed (spillway Q=17,200 cubic feet per second).*

basin floor and the weir of the recommended design were partially due to the smaller discharge.

The probability of occurrence of various magnitudes of pressure at several representative piezometers on the basin floor and on the face of the weir were estimated from the oscillograph records. For example the absolute maximum pressure at Piezometer 24 on the stilling basin floor was approximately 110 feet of water for one-half spillway design discharge of 17,200 cubic feet per second, figure 32; however, a pressure with one-half the maximum magnitude (55 feet of water) occurred about 70 percent of the time, figure 33.

*Riprap stability test.*—The stability of the protective riprap on the channel floor immediately downstream from the weir was estimated by operating all gates symmetrically for a representative time interval at one-fourth, one-half, three-fourths, and full design discharge. After 2 hours

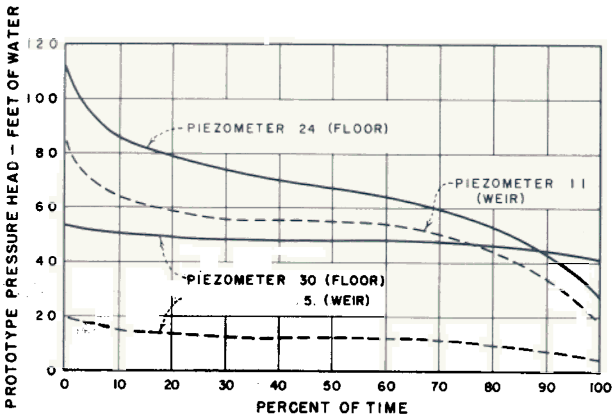



FIGURE 33.—*Frequency distribution of impact pressures on the stilling basin floor and weir. Data for operation with outside gates of spillway open, inside gates closed (spillway Q=17,200 cubic feet per second). Pressures measured above piezometer openings.*

(equivalent to about 10 hours prototype operation) at one-fourth the design discharge, very slight movement of the riprap was noted. No further movement was observed after 1 hour operation at either one-half or three-fourths design discharge. After 1 hour operation at the design discharge (34,400 cubic feet per second), several pieces were removed from the upstream left corner of the riprap layer, and material along the right side of the layer was washed toward the center of the layer. No material was observed to move into the tailrace. The rock used in the model had a maximum size of 2 to 3 inches, which is representative of the specified prototype material, with a maximum dimension of 5 feet.

*Movement of rock in the stilling basin.*—The possibility exists that rock from the cliffs above the stilling basin could fall into the basin and cause abrasive damage during operation of the spillway. A series of tests was made to determine movement of material in the basin for several spillway discharges. Material of the same size used in the riprap test was placed in the bottom of the stilling basin and sections were identified with white spray paint, as shown on figure 34. The model was operated for 1 hour for each of the discharges used for the riprap test, and for operation of the outlet works along with no spillway discharge.

Independent operation of the outlet works caused no movement of the rock material. After symmetrical operation of all gates at one-fourth design spillway discharge, the material remained relatively stable except in the impact zone of the jets. The material was piled immediately downstream from the impact zone, figure 35. Movement of the material increased as the discharge increased. The design discharge resulted in essentially complete removal of all material downstream from the impact zone, figure 36.

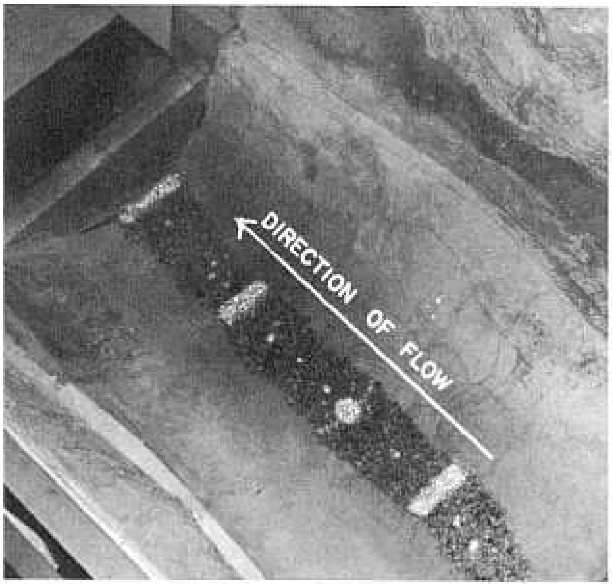



FIGURE 34.—*View of stilling basin pool prepared for test of rock movement due to spillway discharges.*

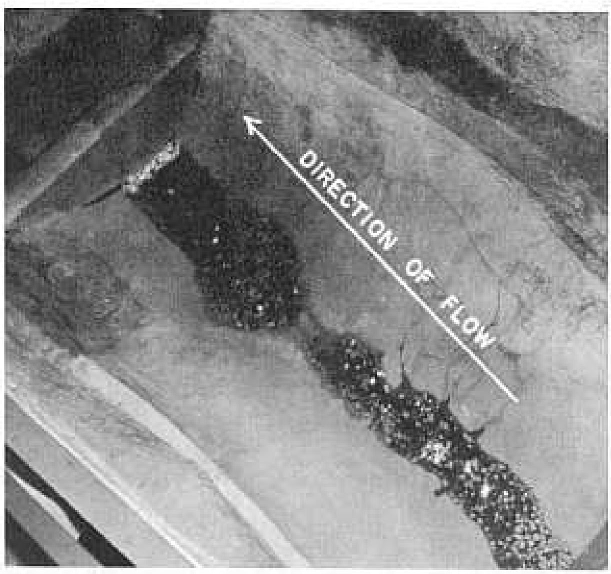



FIGURE 35.—*Evidence of rock movement in stilling basin pool after 1 hour at spillway Q=8,600 cubic feet per second.*


**Figure 36.**—*Evidence of rock movement in stilling basin pool after 1 hour at spillway Q=34,400 cubic feet per second. (Maximum design discharge for spillway.)*

**Figure 37.**—*View of area below the weir after 1 hour at spillway Q=34,400 cubic feet per second. (Maximum design discharge for the spillway.)*

Figure 37 shows a closeup of the tailrace and the amount of material deposited downstream from the weir. A small amount of material was deposited in the right tailrace channel. The tests indicated that material which falls into the basin will circulate during operation of the spillway with probable accompanying abrasive damage to the concrete lining.

*Reservoir flow conditions at the spillway entrances.*—Observations were made of flow conditions in the reservoir near the spillway entrances to determine possible adverse operating conditions. With the reservoir water surface at the maximum elevation (7165) and with all four spillway gates 100 percent open, discharge = 40,000 cubic feet per second, small vortices formed at the corners of the vertical section on the upstream face of the dam which contains the bellmouth entrances, as shown on figure 38. At times, these vortices became large enough to take small quantities of air. Smaller vortices formed over the inside bellmouth entrances, taking minute quantities of air. Strong circulation was apparent above the outside entrances, but no vortices formed. For 39,600 cubic feet per second, with the gates

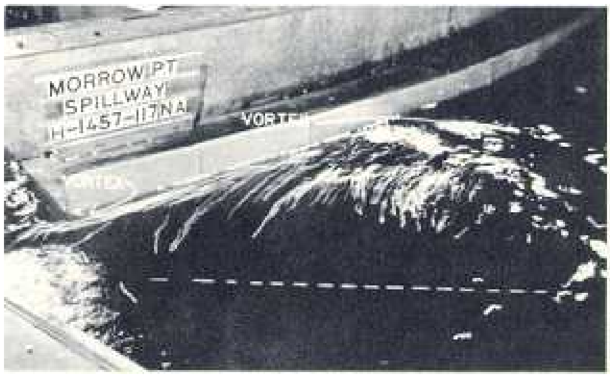



**Figure 38.**—*View of reservoir surface above spillway entrances, showing flow pattern with gates fully open. Reservoir at elevation 7165 (approximate spillway Q=40,000 cubic feet per second).*

slightly closed, conditions were similar to those described above.

At a spillway discharge of approximately 27,500 cubic feet per second, with the gates 100 percent open, the reservoir water surface was immediately below the top of the two inside entrances. The inside conduits flowed full for a short period of time, increasing the discharge, and the reservoir dropped so that a free water surface existed through the conduits. The lower reservoir elevation resulted in a decreased discharge and the reservoir elevation increased until the conduits again flowed full. This cycle was repeated and resulted in a periodic surging with a frequency of about one surge per second. Such a condition could cause undesirable flow conditions in the conduits and vibration in the gate mechanisms. With one inside gate closed about 25 percent, surging was eliminated in that conduit but continued in the other inside conduit. By closing one of the outside gates to increase the reservoir elevation, surging was stopped in both inside conduits when the water surface rose 2 to 3 feet (prototype) above the top of the entrances. Similar surging was noted in the outside conduits with the water surface near the top of the outside entrances, and was eliminated by a gate closure of about 25 percent. The surging condition, which was a function of the conduit geometry and a critical reservoir elevation, can be eliminated by reducing the gate opening or adequate submergence of the entrances. According to the prototype operating criteria, the conditions described above will not occur. The minimum submergence, measured from the top of the bellmouth entrances, will be about 10 feet for the inside conduits and about 17 feet for the outside conduits.

With either the two inside or two outside conduits operating alone with the gates 100 percent open and the reservoir at elevation 7160, vortex action similar to that described for the maximum spillway discharge was observed. At three-fourths maximum discharge, with the gates equally open and the reservoir at elevation 7160, as shown on figure 39, the vortex was somewhat stronger and larger than that observed for the maximum discharge because of the lower reservoir elevation. Vortex action diminished with a further decrease in discharge, until at one-half maximum discharge only a dimpled water surface was observed above the entrances.

*Spillway discharge capacity.*—Measurement of the discharge capacity of the recommended spillway for various conditions of symmetrical and asymmetrical operation showed that each conduit discharged a maximum of 10,000 cubic feet per second, whether operating alone or in various combinations with other conduits. These data also showed that the orientation of the conduits (horizontal or tipped downward) had negligible effect

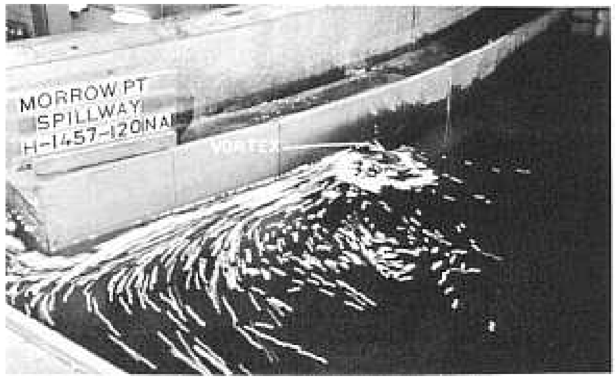



FIGURE 39.—*View of reservoir surface above spillway entrances, showing flow pattern with gates about three-fourths open. Reservoir at elevation 7160 (approximate spillway Q=25,800 cubic feet per second).*

on discharge capacity. Data for operation at partial gate openings, which would occur in the range between normal and maximum reservoir elevations, were not obtained. The model gates did not correctly simulate the prototype gates, and it was lt that such data would not be sufficiently accurate.

Photographs of operations of the recommended design are shown on figures 40 through 45. Colored movies (including slow motion) were also taken for future reference.

*Miscellaneous observations.*—The average elevation of the backwater on the downstream face of the dam was found to be 6780 for the design spillway discharge, with waves as high as elevation 6790. For one-half design discharge, the backwater stood at an average elevation of 6770. The downstream tailwater was set for the total river flow, including the outlet works and powerplant discharges.

Operation of the low-level outlet works was satisfactory either with or without the spillway operating.

The feasibility of calibrating the model weir was investigated to determine the possibility of using the prototype weir for discharge measurement. With only the outlet works operating, the stilling basin water surface was too rough to allow an accurate elevation reading. Also, the two 3-foot-square openings in the weir would not be easily plugged. Use of the prototype weir for this purpose was therefore considered impractical.

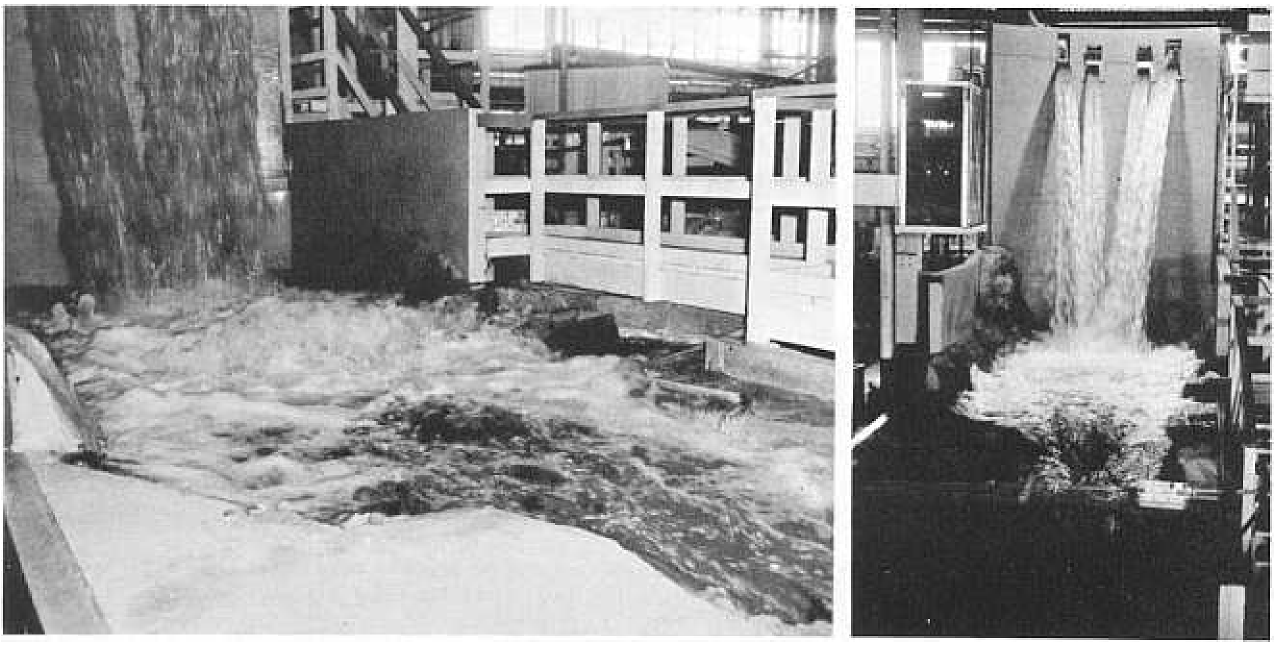


FIGURE 40.—*Operation of the recommended design with all gates fully open (spillway Q=40,000 cubic feet per second, outlet works Q=1,500 cubic feet per second, and powerplant Q=5,200 cubic feet per second).*

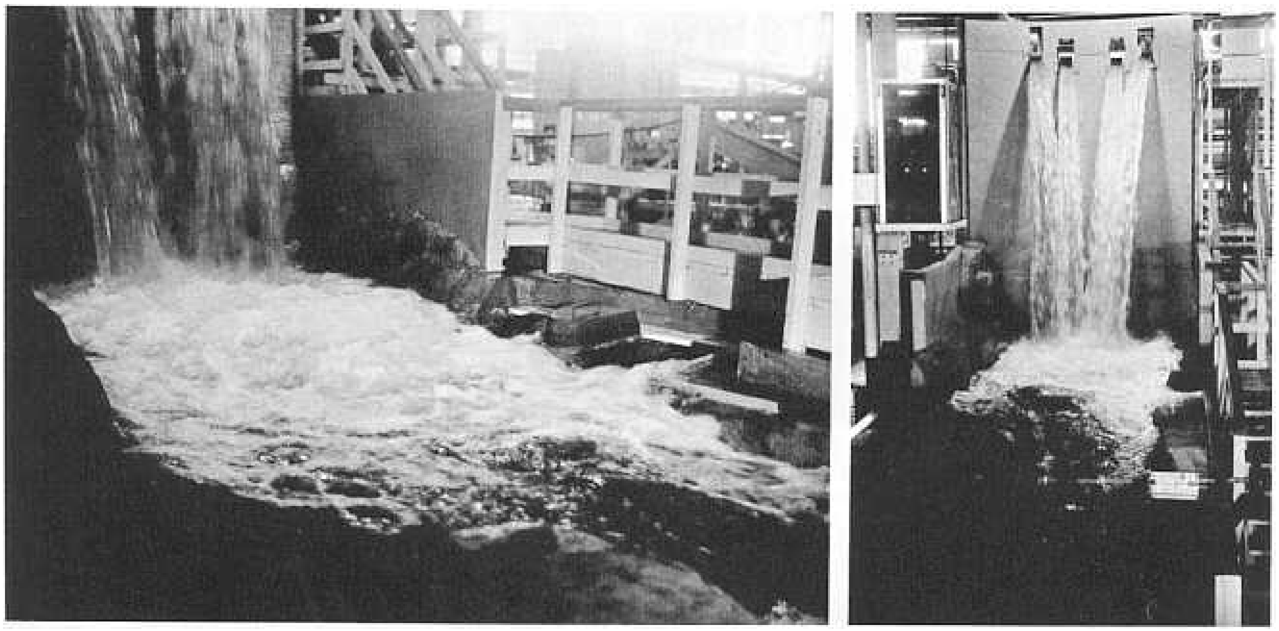


FIGURE 41.—*Operation of the recommended design with the maximum design discharge (spillway Q=34,400 cubic feet per second, outlet works Q=1,500 cubic feet per second, and powerplant Q=5,200 cubic feet per second).*

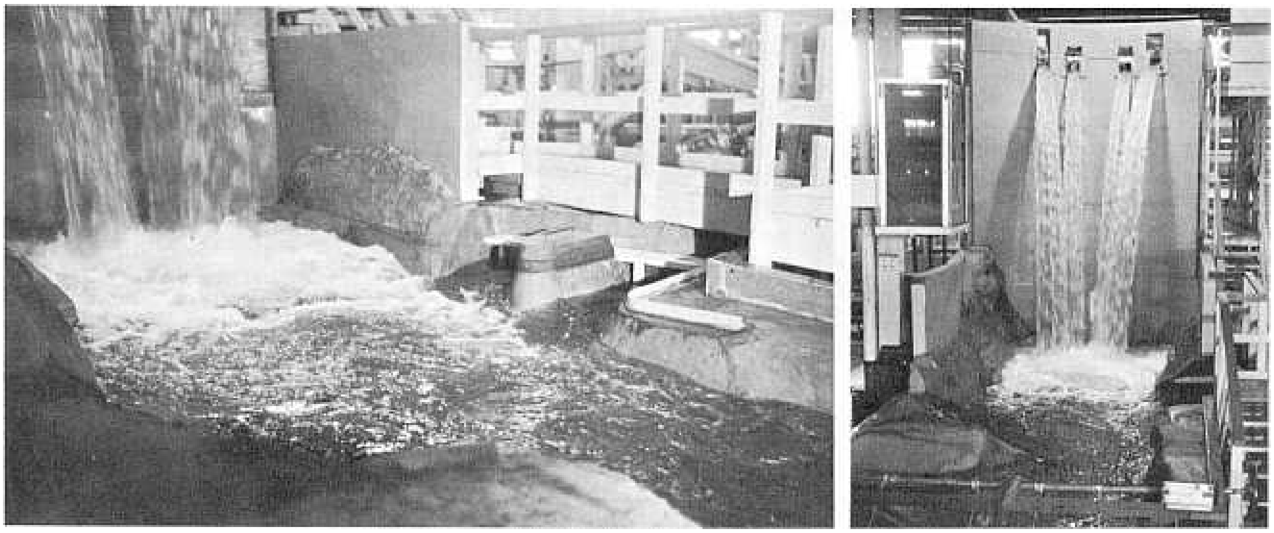


FIGURE 42.—*Operation of the recommended design with all gates open and the spillway gates set for one-half the maximum design discharge (spillway Q=17,200 cubic feet per second, outlet works Q=1,500 cubic feet per second, and powerplant Q=5,200 cubic feet per second).*

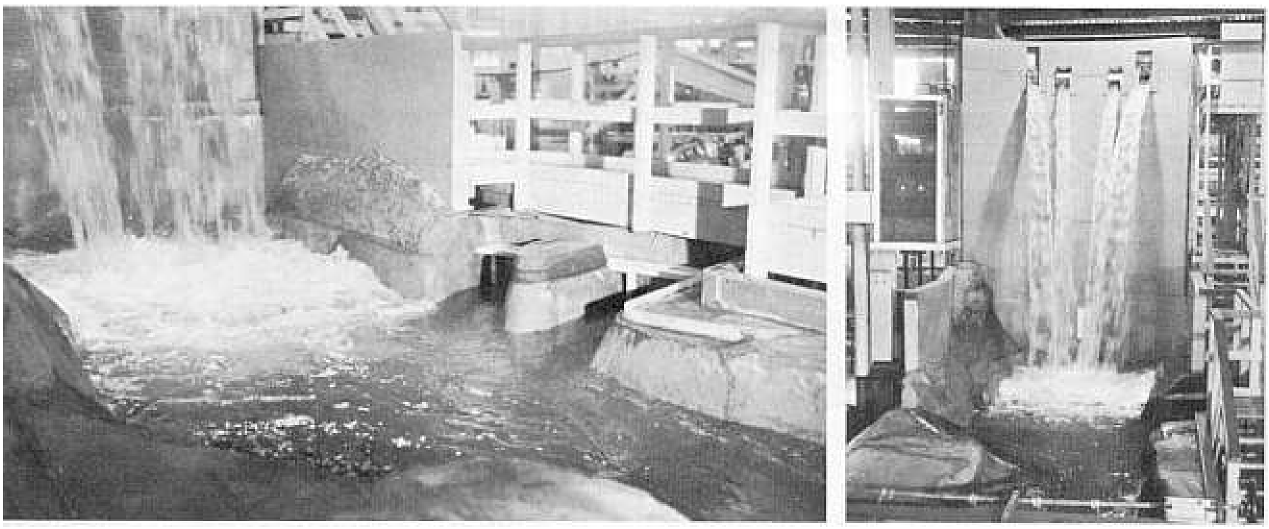


FIGURE 43.—*Operation of the recommended design with all gates open and the spillway gates set for one-quarter of the maximum design discharge (spillway Q=8,600 cubic feet per second, outlet works Q=1,500 cubic feet per second, and powerplant Q=5,200 cubic feet per second).*

FIGURE 44.—*Operation of the recommended design with powerplant discharge only (Q=5,200 cubic feet per second).*

FIGURE 45.—*Operation of the recommended design with discharges from the outlet works and the powerplant (outlet works Q=1,500 cubic feet per second and powerplant Q=5,200 cubic feet per second).*

TABLE 1.—*Metric equivalents of important quantities*

| Feature | English units | Metric units |
|---|---|---|
| Dam thickness at crest | 12 feet | 3.66 meters. |
| Dam thickness at base | 52 feet | 15.85 meters. |
| Dam height above foundation | 465 feet | 141.73 meters. |
| Volume of concrete | 360,000 cubic yards | 273,600 cubic meters. |
| Length at crest | 720 feet | 219.46 meters. |
| Reservoir capacity | 117,000 acre-feet | 144,319,500 cubic meters. |
| Reservoir length | 12 miles | 19.31 kilometers. |
| Spillway capacity | 40,000 cfs | 1,132 m$^3$/sec. |
| Spillway gate size | 15 feet square | 4.57 meters square. |
| Height of fall | 400 feet | 121.92 meters. |
| Stilling pool length | 320 feet | 97.54 meters. |
| Stilling pool width | 180 feet | 54.86 meters. |
| Weir height | 65 feet | 19.81 meters. |
| Lining thickness | 5 feet | 1.52 meters. |
| Outlet works capacity | 1,500 cfs | 42.45 m$^3$/sec. |
| Outlet works gate size | 3.5 feet square | 1.07 meters square. |
| Powerplant generation | 120,000 kilowatts | 120,000 kilowatts. |
| Powerplant discharge | 5,200 cfs | 147.16 m$^3$/sec. |

# Abstract

The preliminary design of an overflow spillway at the crest of the dam and a slide-gate-controlled outlet works near the dam base was abandoned because of undesirable flow conditions in the stilling basin resulting from high velocity efflux from the outlet works. An alternate design, consisting of four fixed-wheel gate-controlled outlets near the dam crest and a small outlet works near the base, was recommended following several modifications. A 1:24 scale model was used in developing the design of the spillway and to determine the hydraulic operating characteristics of the recommended free fall orifice-type spillway, slide-gate controlled outlet works, and tailrace channels of the underground powerplant. Comprehensive data were obtained concerning pressure distribution in the spillway conduits and bellmouth entrances, on the stilling basin floor, and on the stilling basin weir. Minor modifications were made to the topography in the tailrace area to improve flow conditions during large spillway discharges. The stability of riprap protection was determined, and tests were made concerning movement of material in the stilling pool. Operating characteristics of all features of the recommended design were observed.



DESCRIPTORS—*arch dams/ *spillways/ *outlet works/ *free fall/ *model tests/ hydraulic models/ manometers/ orifices/ tailrace/ water pressures/ weirs/ instrumentation/ piezometers/ pressure measuring equip/ discharges/ underground powerplants/ fixed wheel gates/ slide gates/ impact/ energy dissipation/ riprap/ research and development.

IDENTIFIERS—*bellmouths/ Morrow Point Dam, Colorado/ Curecanti Unit, Colorado/ Colorado River Storage Project/ Colorado.

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