UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

HYDRAULIC MODEL STUDIES OF THE SPILLWAY AND OUTLET WORKS OF ANCHOR DAM--OWL CREEK UNIT MISSOURI RIVER BASIN PROJECT

Hydraulic Laboratory Report No. Hyd. 289

RESEARCH AND GEOLOGY DIVISION

BRANCH OF DESIGN AND CONSTRUCTION
DENVER, COLORADO

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HYD 289

CONTENTS

LIST OF FIGURES

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Branch of Design and Construction
Research and Geology Division
Denver, Colorado
February 9, 1951

Laboratory Report No. Hyd. 289
Hydraulic Laboratory
Written by: W. P. Simmons, Jr.
Reviewed by: J. W. Ball
W. C. Case

Subject: Hydraulic model studies of the spillway and outlet works of Anchor Dam--Owl Creek Unit--Missouri River Basin Project.

PURPOSE

To determine the hydraulic performance of the uncontrolled spillway and the outlet works and to recommend any design changes needed to insure good operation.

CONCLUSIONS

1. An erratic, slow moving, upward current will exist in the reservoir upstream from the right training wall and will cause reservoir turbulence which will roughen the flow on the right side of the spillway. This disturbance will be small at low flows and appreciable, though not serious, at the maximum discharge (Figures 4A and C).

2. The best location for the spillway along the axis of the dam is the location selected for the preliminary design (Figure 2).

3. The 5-percent vacuum crest (5-percent of the maximum head of 10.5 feet) with a 6.67-foot corbel (crest overhang upstream) of the preliminary spillway design will operate satisfactorily (Figure 5).

4. The 0.20 slope of the preliminary spillway design is too steep to maintain acceptable control of the flow. Decreasing the slope to 0.37 gives appreciable improvement (Figures 5 and 6).

5. Splashing and vibration occurs with a 20-foot radius bucket and spillway slopes of 0.20 to 0.33. Use of the parabolic bucket developed during the model studies, at spillway slopes of 0.33 and 0.37, greatly improves these conditions (Figures 5 and 6).

6. The alternate 40-inch wide drain slots and normal bucket shape (Figure 6C) produce good directional control on the discharging water and reduce the scour in the river channel.

7. Piers that extend 5 feet into the reservoir are required at the right and left ends of the spillway entrance to produce acceptable flow conditions along the training walls (Figures 7F and 9).

8. Converging spillway training walls which narrow the spillway from an original bucket lip width of 80.5 feet to 70 feet are desirable to effect good flow conditions and construction economy (Figure 8D).

9. The recommended spillway design with the 5-foot extension piers, 5-percent vacuum crest 100 feet long, 0.37 slope, parabolic bucket with drain slots and a width at the lip of 70 feet, operates satisfactorily at all applicable heads.

10. The scour for the recommended design is centered in the river bed 165 feet downstream from the dam and extends to within 20 feet of the downstream face. No damage will be incurred by the berm at the toe of the dam or by the dam itself.

11. The discharge curve obtained during the model studies of the spillway is presented in Figure 14. The spillway discharges the designed flow of 13,500 cfs at the designed maximum head of 10.5 feet.

12. The preliminary design of the outlet works operates satisfactorily at all flows (Figure 13).

13. An alignment chart showing the discharge for one of the two 30-inch hollow-jet valves of the outlet works is presented in Figure 15.

RECOMMENDATIONS

1. Use the recommended spillway design on the prototype structure. This design includes the 5-foot extension piers, the 5-percent vacuum crest with a 6.67-foot corbel, a maximum spillway slope of 0.37, a parabolic bucket with wide drain slots at the exit, and training walls which converge to make the bucket 70 feet wide at the lip (Figures 5B and 8D).

2. Use the spillway location shown in the preliminary design (Figure 2).

3. Use the outlet works shown in the preliminary design (Figures 2 and 13).

4. Install during construction of the dam any equipment necessary for field tests to study the vibration, if any should occur, due to spillway operation. The design of this dam departs from conventional American practices and performance data will be valuable.

ACKNOWLEDGMENT

The recommended design of the Anchor Dam spillway is the result of cooperative efforts between the Concrete Dams Section of the Dams Division and the Hydraulic Laboratory of the Research and Geology Division.

INTRODUCTION

Anchor Dam is a thin arch concrete structure proposed for construction across the narrow canyon on the South Fork of Owl Creek, 34 miles west of Thermopolis, Wyoming (Figure 1). A 100-foot long uncontrolled overflow spillway near the center of the dam is provided to pass flood waters at a maximum rate of 13,500 cfs (Figure 2). An important consideration in the design of this spillway was to make it thin in cross section to fit the thin concrete dam. To accomplish this, the spillway crest was designed to operate at 5-percent vacuum at the maximum head of 10.5 feet. Water flows over this crest and then passes 91 feet down the steep spillway face to the trajectory bucket below. The bucket is situated 50 feet above the riverbed, and the water that leaves the bucket is projected about 140 feet downstream before falling to the river surface. An outlet works consisting of two 30-inch manually operated hollow-jet valves located at elevation 6345.0 about 17 feet to the left of the spillway bucket is provided to release water for irrigation and other use downstream (Figures 2 and 13). An additional 8-inch line is provided to release small discharges during the winter months and to fill or drain the two 30-inch outlet conduits. The trashrack structure and emergency slide gates are placed on the upstream face of the dam at the entrance to the 30-inch steel conduits.

The design of the spillway with the vacuum crest, the steep slope, the trajectory bucket situated 50 feet above the riverbed, and the outlet works 45 feet above the riverbed, represents a departure from conventional American practices. Therefore, hydraulic model studies were requested by the designing engineers. These tests were made to determine if the general design was practical, and, in the event that it was, to determine any detail changes necessary to insure proper operation.

INVESTIGATION

The Model

A 1:30 scale model was built with sufficient width to include 460 feet of the 500-foot wide dam, and long enough to include 210 feet of the reservoir above and 660 feet of the riverbed below the dam (Figure 3). The model was contained in a metal-lined wooden box 40 feet long, 14 feet wide, and 8.5 feet high at the reservoir end. The arch of the dam was built of wood and was covered with sheet metal to waterproof the structure. Metal lath was attached to this covering and cement mortar applied to produce the finished surfaces of the dam. The spillway was formed in smooth-finished mortar screeded to metal templates, and the spillway was made about twice the width of the expected final structure. This extra width permitted shifting the normal width spillway to determine the optimum location on the dam. Movable training walls and temporary bulkheads were used to block the extra width of the spillway (Figure 4). For the final test the model was revised so that only the desired spillway width was formed on the dam (Figure 11). Piezometers were placed in the spillway face and in the bucket so pressure measurements could be made on these surfaces. Two 1-inch pipes with model hollow-jet valves at the downstream ends were placed through the dam to represent the 30-inch outlets of the prototype structure (Figure 6C). The surface topography of the canyon in the reservoir above the dam and the bedrock topography of the river channel below the dam were formed in rough mortar. The model overburden material, which consisted of a mixture of sand and assorted gravels, was placed over the bedrock topography in the river channel to form the natural surface contours. Water was supplied to the model by the laboratory system through a regulating gate and a 12-inch-diameter pipe, and the rate of flow was measured with calibrated venturi meters. The depth of the water in the river channel was regulated by a tailgate at the end of the channel. A sand trap was provided downstream from the tailgate to retain the sand and gravel washed out of the riverbed. Point gages were used to measure the water-surface elevations in the reservoir and in the river channel.

Reservoir Approach Conditions

At the outset of the test program, turbulence was found in the reservoir immediately upstream from the right spillway training wall. Examination of the flow in the reservoir by means of dyes revealed that an erratic, slow moving, upward current was present and that part of the water from this current passed over the spillway to produce a rough water surface on the right third of the spillway (Figure 4A). Question immediately arose as to whether this turbulent reservoir condition was peculiar only to the model or if the condition would also occur in the prototype reservoir. The equivalent elevation of the floor that formed the reservoir bottom in the original model was 6390.0 (Figure 3). This provided a minimum equivalent depth of approach of 51 feet or about 5 times the maximum head on the crest. This depth appeared ample, but to make certain and to remove all doubt about the accuracy of the model results the floor of the reservoir was lowered to the equivalent elevation of 6315.0 (Figure 3). This provided a minimum equivalent depth of approach of 126 feet, or 12 times the maximum head on the crest. The revised model was operated first without the topography in the reservoir so the effect of the topography could be determined. Smooth flow was found to occur in the reservoir and over the spillway crest for this condition (Figure 4B). However, when the topography was built into the model again, the upward current in the reservoir and the turbulent flow on the spillway reappeared (Figure 4C). It was therefore concluded that the nature of the topography in the reservoir upstream from the dam, together with the placement of the axis of the dam within this topography, resulted in the upward current in the reservoir, and that this turbulence should occur in the prototype reservoir. The disturbance is small at low spillway discharges, but becomes appreciable, although not serious, at the maximum discharge.

Location of the Spillway on the Dam

An attempt was made to avoid the turbulent reservoir region by moving the spillway 50 feet to the left of its initial location. However, the reservoir turbulence shifted with the spillway, and rough flow continued to occur on the spillway. The spillway was then moved 34 feet further to the left but this was also unsuccessful. In the latter case, the turbulent area on the right side of the spillway was reduced, but rough flow appeared on the left side. In addition, the water leaving the spillway bucket struck the river channel at such an angle that excessive pounding and scour occurred on the riverbanks. Analysis of these factors revealed that the best overall performance was obtained with the spillway in the initial location, and accordingly, the spillway was placed in that location.

Design of the Spillway and Trajectory Bucket

The crest. Special consideration was given during the design of the spillway to make it thin in cross section to fit the thin arch of the dam. To partly accomplish this, a partial vacuum spillway crest (5 percent of the maximum head) was designed for operation for a maximum head of 10.5 feet. Model studies showed that in the regions where the reservoir disturbances were not present water flowed smoothly over this part of the crest and, at low discharges, continued down the steep face and through the bucket in an even sheet. However, at moderate or high discharges, the flow on the steep spillway took place with an erratic, wavy surface which produced noticeable vibration and heavy splashing in the bucket. To determine if the flow irregularities resulted from the forces exerted by the pressures of the partial vacuum crest a new crest was designed to operate at a minimum pressure of zero gage. This crest terminated in a maximum slope of 0.30 which was considerably flatter than the preliminary slope of 0.20. Water flowed over the new crest in much the same manner as over the original one, although there was somewhat smoother flow on the lower portion of the 0.30 slope. Visual observation of the water flow over the crest and spillway face established that the improved flow resulted from the decreased slope of the spillway and not from the new shape of the crest. Because no adverse flow was incurred and because the partial vacuum crest was better adapted to the dam, it was decided to use this crest for the prototype structure and in all subsequent model tests.

To complete the fitting of the spillway to the dam, a corbel was used with the spring point 6.67 feet upstream from the axis of the dam (Figure 5A). During the model tests, question arose as to whether the corbel was partly responsible for the erratic water surface over the crest. A 36-inch-high sheet-metal wall (equivalent to 90 feet) was therefor placed on the face of the corbel to form a spillway crest without a corbel (Figure 5A). No change was noted in the flow pattern over the crest, and it was concluded that the corbel of the preliminary crest design did not cause the uneven water surface over the spillway.

The maximum slope. In the preliminary design the steep portion of the spillway face had a slope of 0.20 and was coincident with the downstream face of the dam (Figure 5A). The model tests showed that the 0.20 slope was too steep for stable flow because the water passing down the spillway fell almost freely and little control was exerted by the spillway. (Figure 6A). An uneven, erratic sheet of water developed whose depth ranged from the equivalent of a few inches to 7.5 feet within a lateral distance of only 8 feet. To obtain positive control of the flow and hence more uniform water depth on the lower portion of the spillway, the slope was reduced from 0.20 (1 horizontal unit to 5 vertical units) to 0.27, 0.33, and finally to 0.37 (1.85 horizontal units to 5 vertical units) (Figure 5B). The latter slope exerted sufficient control to reduce the variations in the water depths normal to the spillway to 3 feet (Figure 6B) and at the same time did not require an excessive amount of additional construction. Further reductions in the slope were not feasible because of additional construction expense and because the flow conditions obtained with the 0.37 slope assured good spillway performance.

The bucket. The trajectory-type spillway bucket in the preliminary design used a 20-foot radius which was tangent to the maximum slope of the spillway and which continued downstream beyond the bucket invert to terminate in an upward slope of 4:1 (Figure 5A). Model tests showed that the abrupt turn at the entrance of the radial bucket, combined with the somewhat uneven sheet of water which entered the bucket, produced excessive splashing and vibration. It was apparent that a much longer radius was required at the bucket entrance to turn the water smoothly. To accomplish this without building any unduly large structure a parabolic shape was selected with the parabola tangent to the maximum slope of the spillway and the origin at the bucket invert. The parabola was continued beyond the invert to terminate on a 4:1 upslope (Figure 5B). This design turned the water smoothly and continuously through the bucket with almost no tendency for splashing and vibration, and the jet leaving the bucket was uniform and well directed (Figure 6B).

Since the bucket continues beyond the invert to the point where a 4:1 upward slope is obtained, a large pool of water would remain in the bucket if provisions were not made to remove it. Several wide drain slots were proposed which extended from just upstream of the invert to the downstream edge of the bucket on a downward slope of 12:0.25. A preliminary study of the slot effect on the performance of the bucket suggested that improved flow would result if 40-inch-wide slots were alternated with equal widths of the normal bucket profile across the full width of the bucket exit (Figure 6C). Model studies showed that this dentated lip provided good directional control on the discharging water and, in addition, doubled the length of the jet impact area on the riverbed, thereby reducing the scour in the channel.

Spillway entrance piers and training walls. Part of the water flowing over the dam approaches the spillway from the sides and must therefore turn at the entrance piers before passing down the spillway. The preliminary pier design (Figure 7A) turned the water so abruptly that a severe contraction took place and the water swept away from the training walls. The flow subsequently passed over the crest and down the spillway face where the slope increased so fast that the water could not spread back to the training walls. The flow therefor entered the bucket with a space equivalent to 6 feet between the walls and the main portion of the flow. Inside the bucket, the large turning force exerted on the water made it spread rapidly toward the walls, where it impinged and formed large fins with much spray and vibration (Figure 4C). Attempts were made to improve the flow around the piers by making them wider with more gradual rates of turning (Figures 7B, 7C, and 9A). Attempts were also made to keep the spillway walls in contact with the water by converging them and by curving them toward the center of the spillway to conform to the contraction of the flow (Figures 8B and C). However, only moderate flow improvements were obtained by these means.

An examination of the flow around the above piers showed that the water had no opportunity to recover from the turn before the rapid acceleration down the spillway took place (Figure 9A). If the piers were extended upstream into the reservoir, the water should be able to turn at the pier and make its recovery before the acceleration occurred. Accordingly, a series of round-nosed piers, which extended into the reservoir 3, 4, 5, 6, 7, and 10 feet, were placed one at a time at the right side of the spillway entrance (Figure 7D). Considerable draw-down occurred at the nose of the pier as the water flowed around the 1.75-foot radius, but a rapid recovery followed and the water passed over the spillway in contact with the full length of the training wall. The extent of the recovery made by the water was related to the distance the pier extended into the reservoir. The 3-foot extension barely permitted the recovery, while the 7- and 10-foot extensions caused the water to climb the walls. Between these extremes the 5-foot extension permitted good recovery with only a slight tendency for the water to climb the walls. With 5-foot extension piers installed on both sides of the spillway entrance the water remained in contact with the training walls the full length of the spillway and smooth flow existed in the bucket. The 3.5-foot width of the pier noses was later increased to 5 feet for structural reasons with no effect upon the flow conditions.

The tests in which the converging and curved walls were used (Figures 8B and C) suggested that the width of the bucket could be considerably decreased with no detrimental effect on the flow. A compromise between the tendency of the water to climb the walls and the construction savings obtained by narrowing the bucket led to the design where straight converging walls extended to a point 40.8 feet downstream from the axis of the dam (Figure 8D). From this point to the exit of the bucket, the walls curved on a 70-foot radius to become parallel at the bucket lip with a total bucket width of 70 feet, or 10.5 feet less than the preliminary design. By combining these converging walls with the 5-foot extension piers, acceptable flow was obtained at the piers and all along the training walls. The flow around the right pier at discharges of 5,000, 10,000, and 13,500 cfs is shown in Figures 9A, B, and C, respectively. The depth of water normal to the spillway on the left and right training walls and at the center of the spillway is shown in Figures 10A, B, and C, respectively, for the discharges of 5,000, 10,000, and 13,500 cfs.

Scour in the River Channel

The spillway and outlet works were purposely located high above the river to make their operation independent of the scour or the rock slides which may occur in the river channel. However, the scour was important because an earth berm with a top elevation of 6309.0 will be maintained at the downstream face of the dam and because of the possibility of the river channel being dammed by slides of the deep overburden on the right bank (Figure 2). The talus material of the prototype overburden was represented in the model by a mixture of sand and assorted gravels which was placed to the proper topographical contours on the bedrock of the model canyon walls to an equivalent elevation of 6365.0 (Figure 12A). Flows representing 2,500, 5,000, 10,000 and the maximum of 13,500 cfs were discharged successively into the river channel for periods of 30 minutes each. After each run, the contours of the scoured channel were laid out at appropriate intervals by means of white cotton cord and a photographic record made. The overburden was then replaced to the original contours and the next test begun. The operation of the spillway and the flow conditions in the river channel for equivalent flows of 5,000 and 13,500 cfs are shown in Figures 11A and B, while the scour patterns in the river channel resulting from these flows are shown in Figures 12B and C. The tail water near the dam was quiet and showed no tendency to remove material from the berm at any spillway discharge. Land slides occurred along the right riverbank when the deep overburden became saturated at the higher discharges. The slides caused a temporary rise in the tail water due to the partial damming of the channel. Immediately after the slide occurred the flowing water began to move the material downstream, and the tail water receded. The maximum elevation reached by the tail water was 6320.0. The model studies confirmed the designer's belief that the safety and efficiency of the structure would not be affected by the scour in the riverbed. No damage is expected to result further down the river because there are no property rights for several miles.

RECOMMENDED DESIGN

Spillway and Trajectory Bucket

The spillway design recommended for the prototype structure is as follows: The spillway is situated in the original location and uses the 6.67 crest corbel, and the 5-percent vacuum crest which terminates in a 0.37 slope (Figure 5B). The parabolic bucket is tangent to this slope and the bucket exit is formed by alternate drain slots and normal profile. Five-foot-wide piers which extend 5 feet into the reservoir are used together with the converging training walls which form a bucket 70 feet wide (Figures 7F and 8D).

Flow occurs without splashing or vibration and with a stable, though somewhat wavy, surface (Figures 11A and B). The water discharging from the bucket travels about 140 feet downstream before falling to the river surface. In the prototype structure considerable spray will accompany the flow on the spillway and through the bucket because of the high-velocity flow and wind action in the canyon. No damage is anticipated from this spray, however, because there are no power lines or other easily damaged structures in the vicinity.

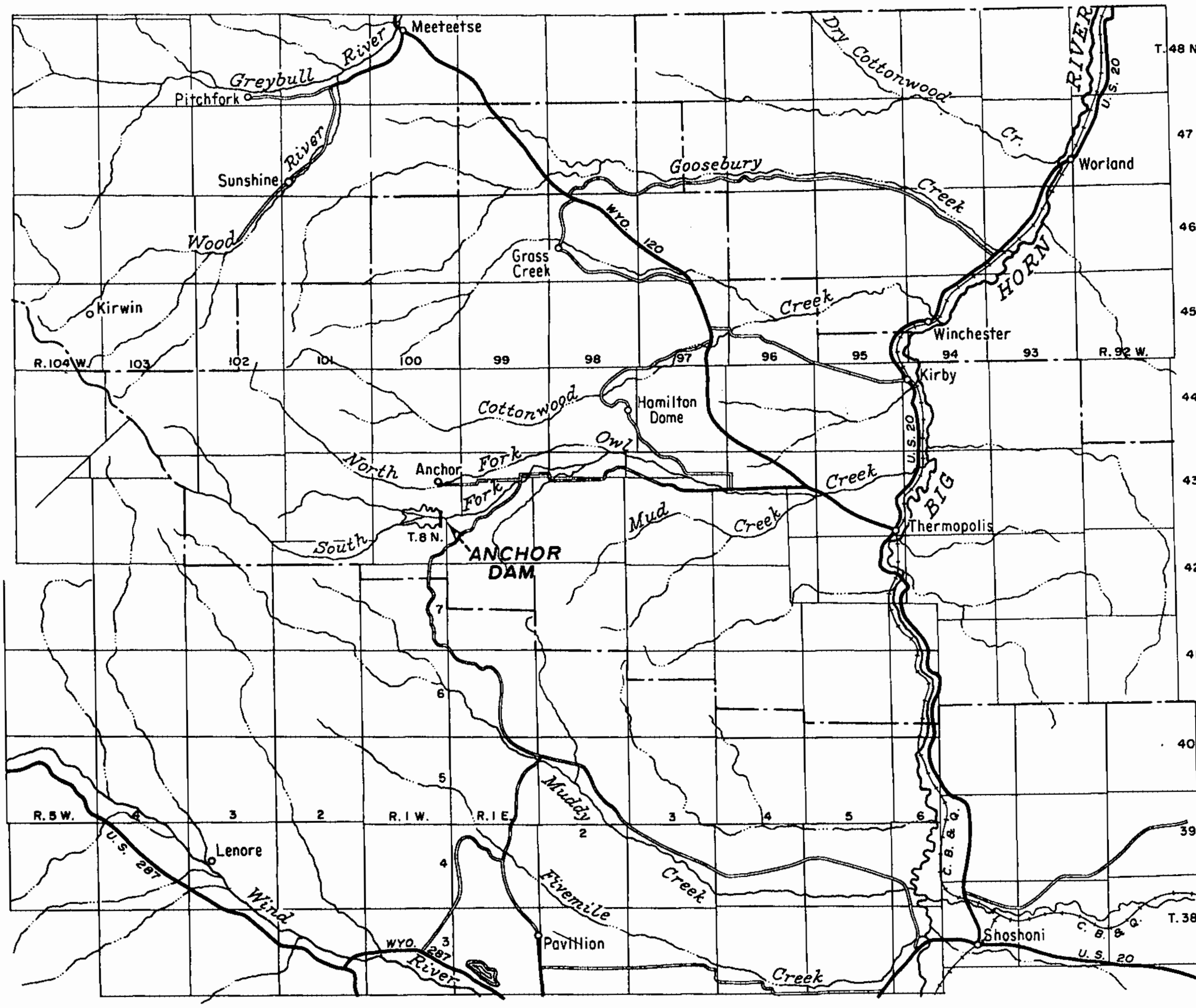
Outlet Works

The outlet works consisting of two manually operated 30-inch hollow-jet valves with the attendant trashracks, emergency gates, and piping provided in the preliminary design is recommended for the prototype structure (Figure 13). The valves are located at elevation 6345.0, 17 feet to the left of the spillway bucket and the water released by the valves falls 45 feet to the river channel below. The model tests of the flow conditions in the channel where the flow strikes the river showed that the position of the valves on the dam and the direction in which they discharged was satisfactory. Flows equivalent to 154, 210, and 253 cfs at reservoir elevations of 6380.0, 6410.0 and 6440.0 were released by the 100-percent open valves.

DISCHARGE CAPACITY CURVES--SPILLWAY AND OUTLET WORKS

A curve showing the relation of the spillway discharge to the head on the uncontrolled crest was obtained during the model tests (Figure 14). At the maximum design head of 10.5 feet (reservoir elevation 6451.5) the spillway discharge was 13,500 cfs, the maximum design rate. The relation of the discharge coefficient to the head on the crest is also presented in this figure.

The rate of discharge for one of the two 30-inch hollow-jet outlet works valves is presented as an alignment chart of total head one diameter upstream from the valve inlet versus valve opening (Figure 15). The relation of the discharge coefficient to the valve opening is also shown in this figure. The coefficient is based upon the total head on the valve centerline one diameter upstream from the valve inlet and upon the area of the conduit at the valve inlet. When both valves operate the total outlet works discharge may be obtained by adding the discharges of the valves.

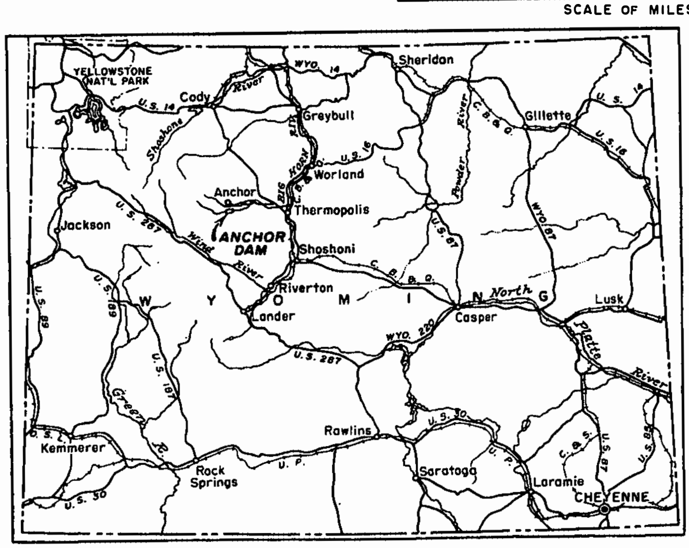


VICINITY MAP

10 0 10 20
SCALE OF MILES

EXPLANATION
VICINITY MAP ONLY

- PAVED
- IMPROVED
- UNCLASSIFIED



INDEX MAP

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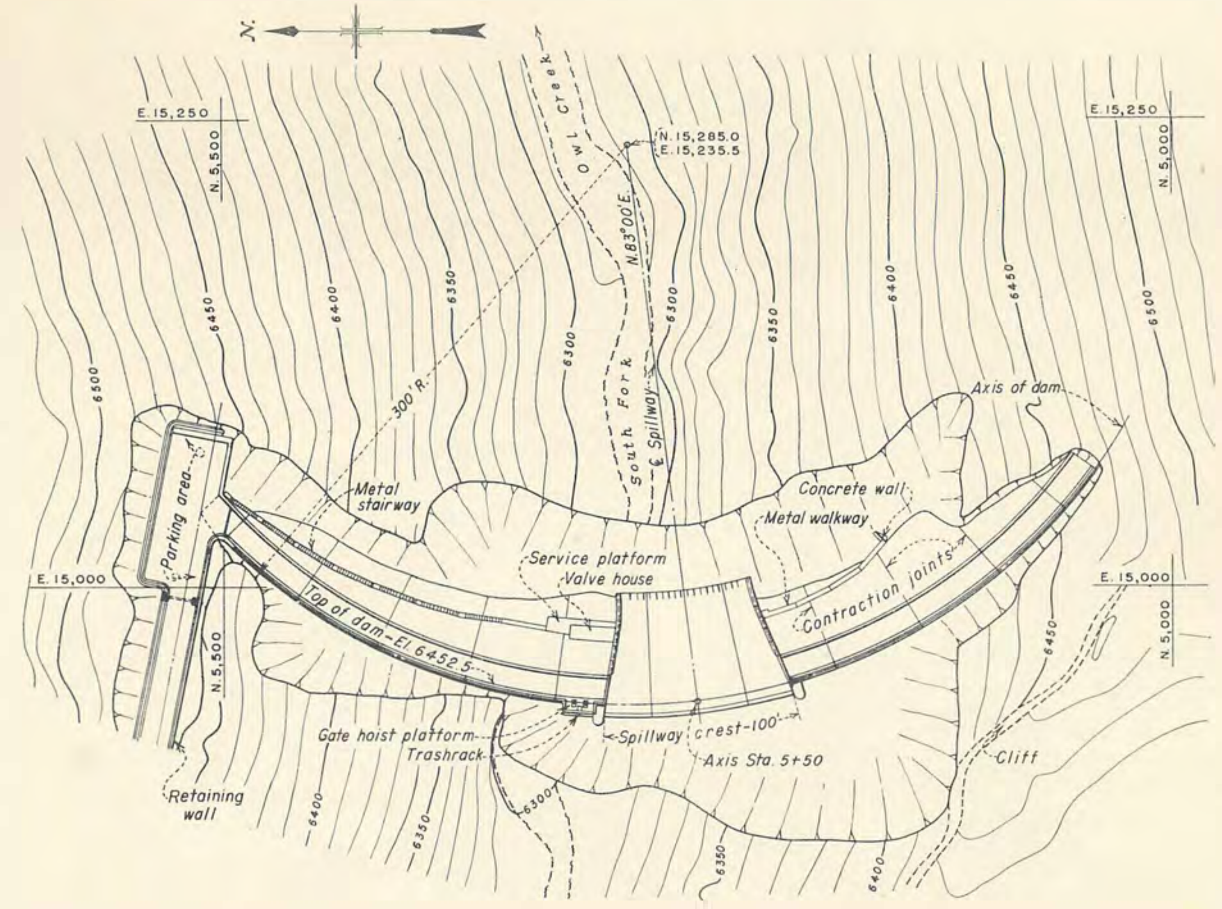
MISSOURI RIVER BASIN PROJECT
BIG HORN BASIN DIVISION - OWL CREEK UNIT - WYOMING

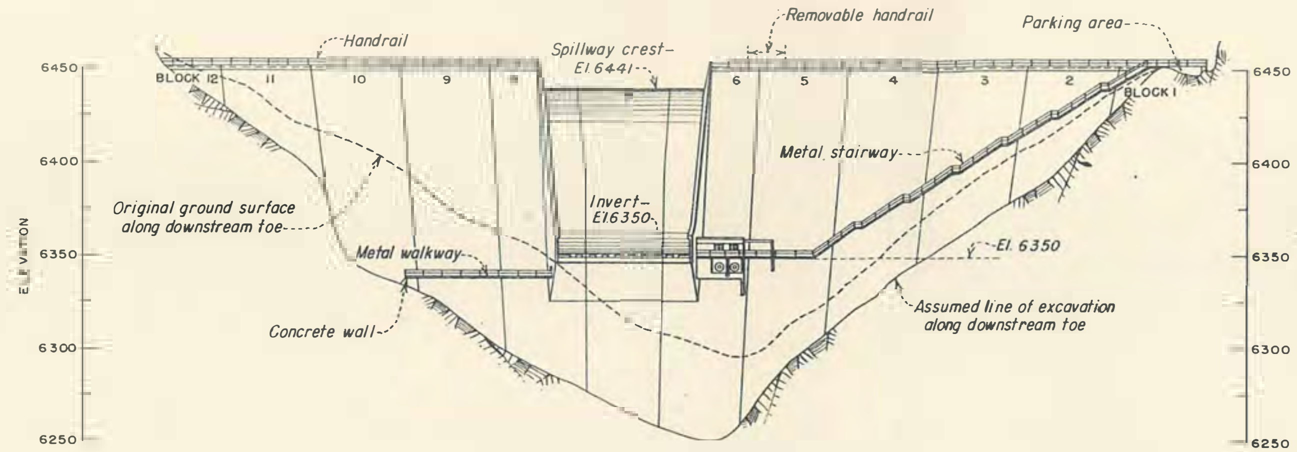
ANCHOR DAM
LOCATION MAP

DRAWN C.A.M. SUBMITTED T. B. Keener
TRACED B.F.W. RECOMMENDED W. H. Nalder
CHECKED S.E.A. J.J.H. APPROVED W. E. Blomgren, ACT. CHIEF ENGINEER

DENVER, COLORADO, MARCH 27, 1950. 339-D-106

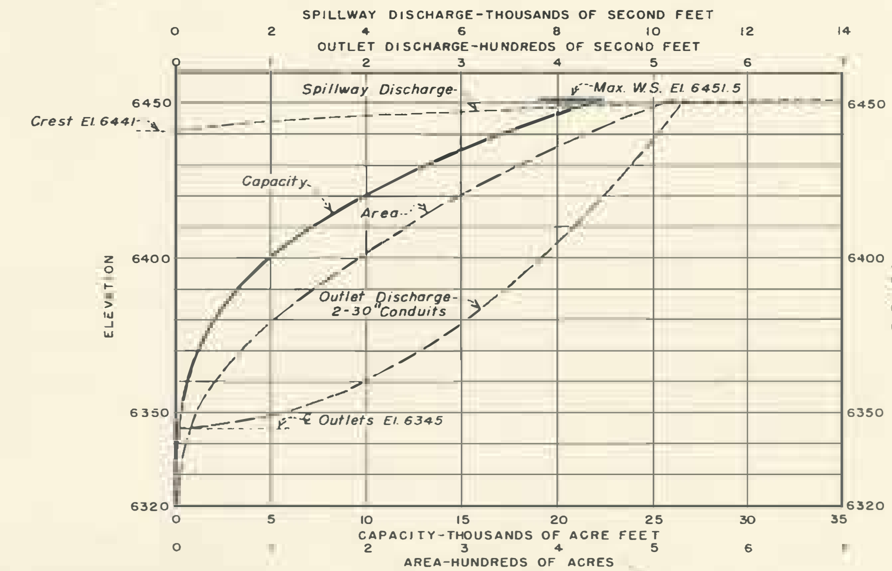


PLAN

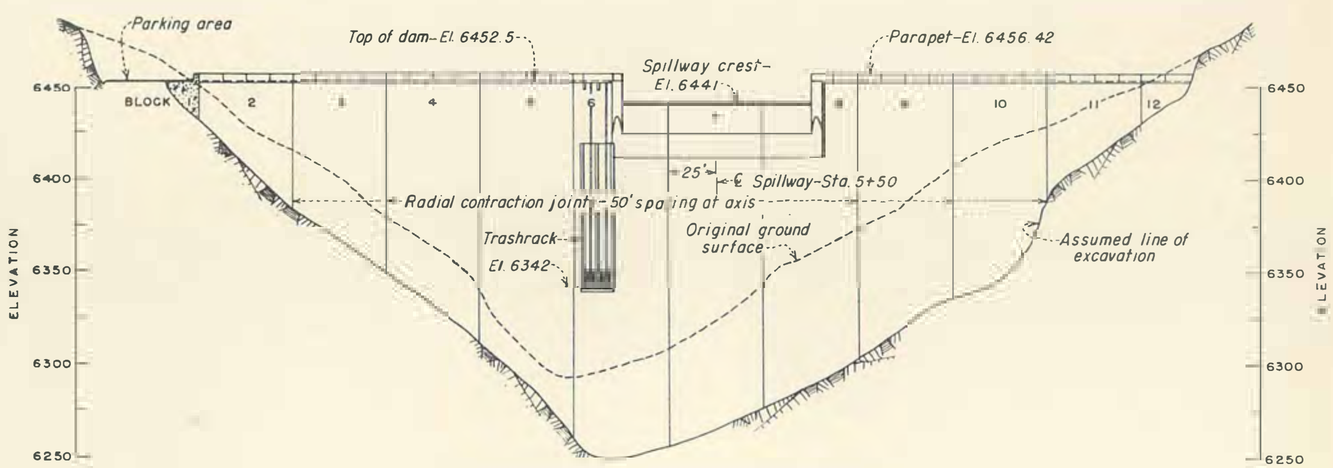


DOWNSTREAM ELEVATION
DEVELOPED

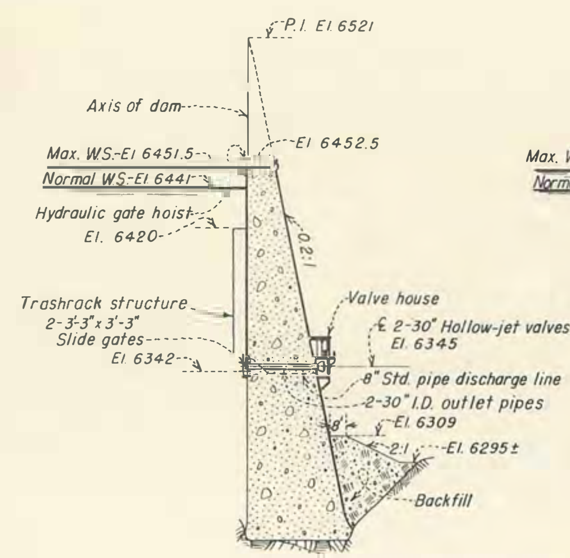
50 0 50 100 150
SCALE OF FEET



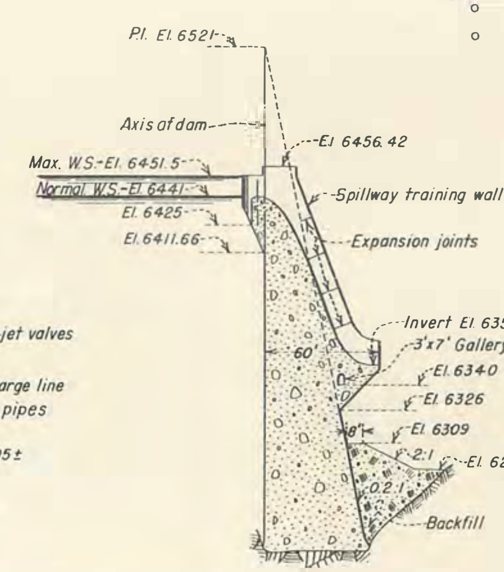
AREA, CAPACITY AND DISCHARGE CURVES



UPSTREAM ELEVATION ALONG AXIS
DEVELOPED



SECTION THRU OUTLET WORKS



SECTION THRU SPILLWAY

NOTE

Actual required foundations may vary widely from profiles and assumed excavation lines shown.

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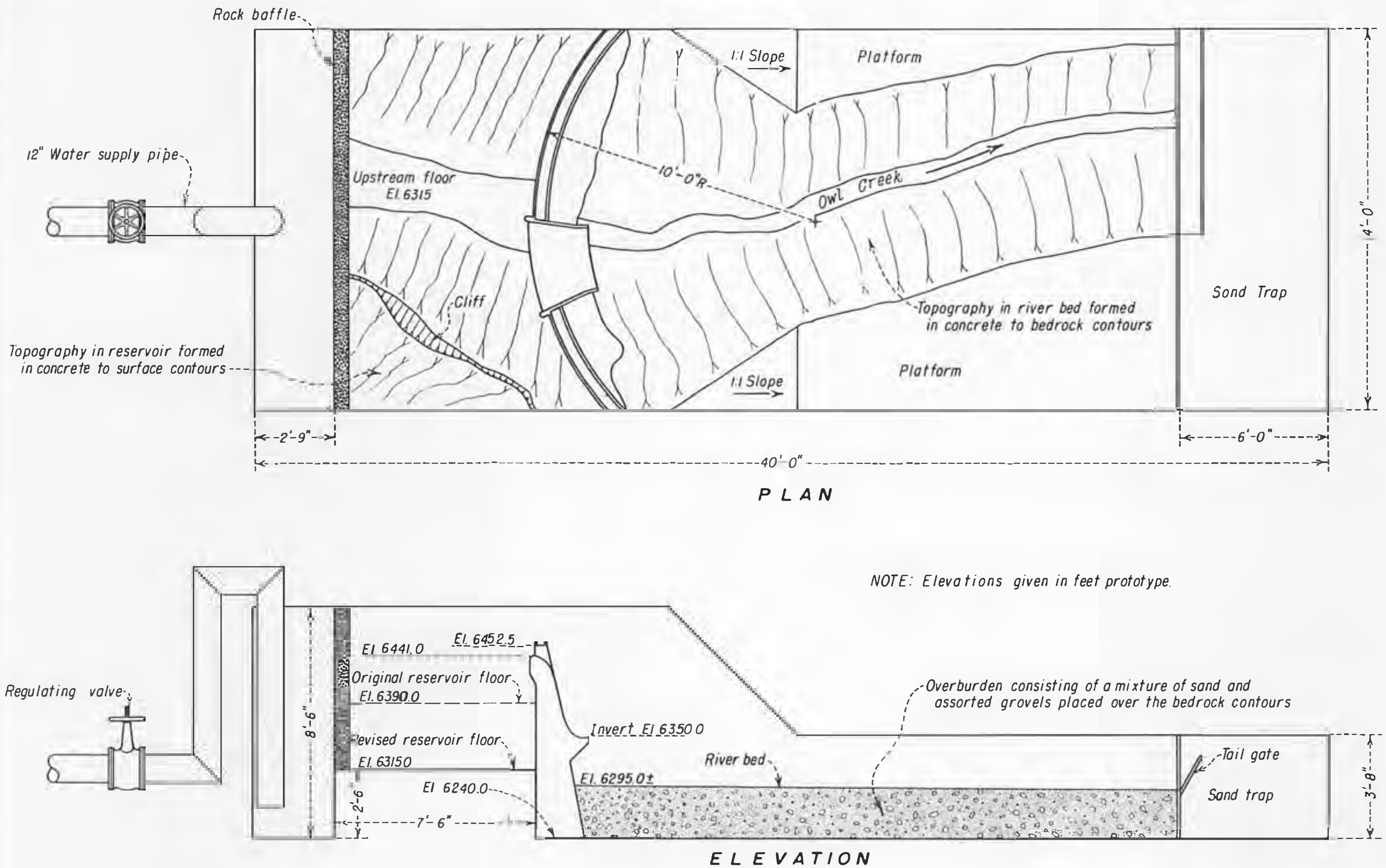
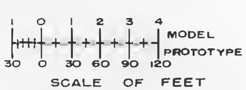
BIG HORN BASIN DIVISION-OWL CREEK UNIT-WYO.

ANCHOR DAM

PLAN, ELEVATIONS AND SECTIONS

DRAWN C.A.M.-G.W.O. SUBMITTED H.B. Keener
TRACED R.V.S. RECOMMENDED W. H. Nalder
CHECKED JK JJH APPROVED W. E. Blomgren CHIEF ENGINEER
DENVER, COLORADO, MARCH 27, 1950 339-D-110





ANCHOR DAM
SPILLWAY AND OUTLET WORKS
1:30 SCALE MODEL

FIGURE 3
REPORT HYD 289

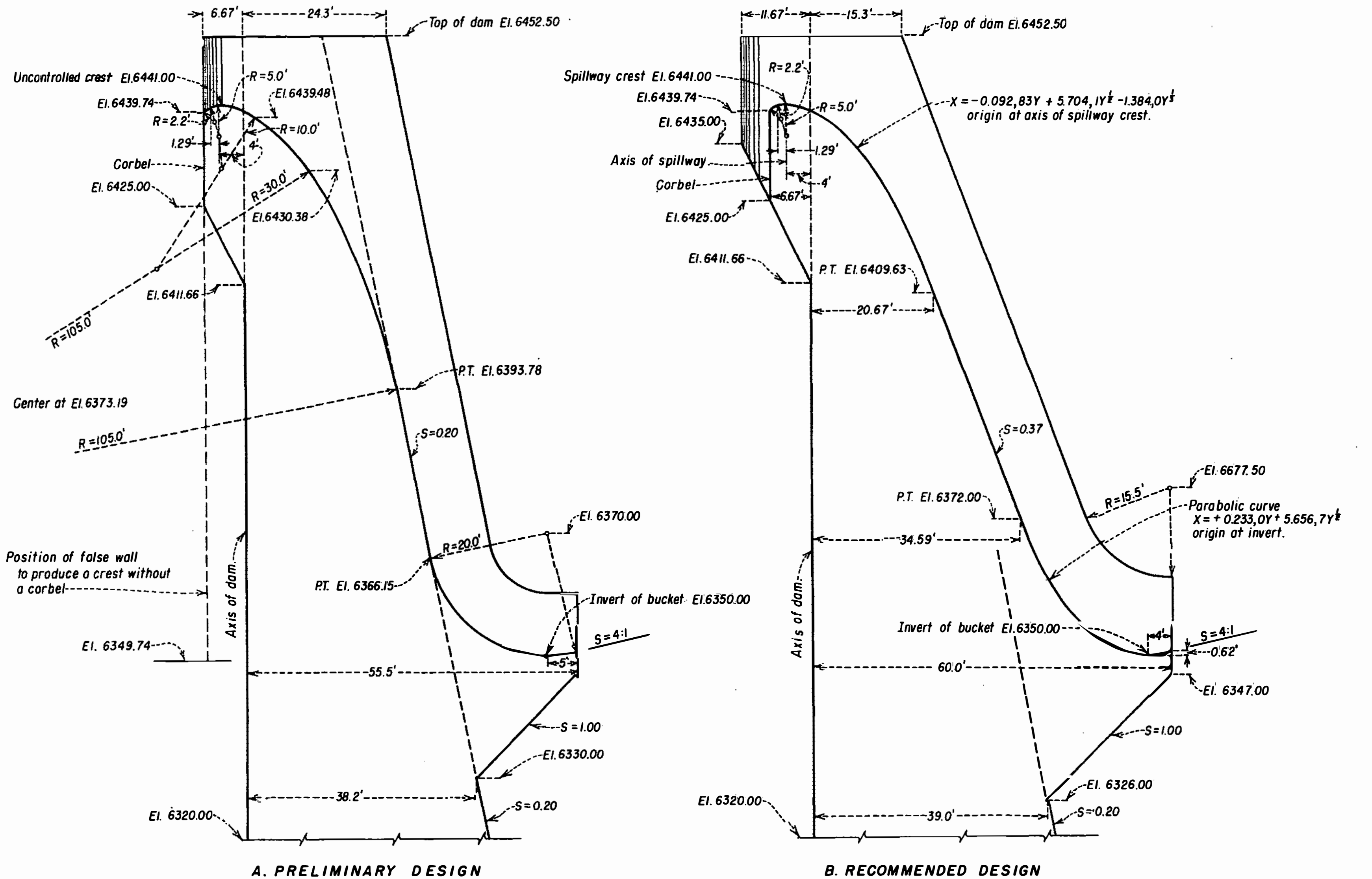
A. Equivalent elevation of the floor upstream from dam--6390.00. With topography in reservoir.

B. Equivalent floor elevation--6315.00. No topography in reservoir.

C. Equivalent floor elevation--6315.00. With topography in reservoir.

ANCHOR DAM

Effect of Turbulent Region in Reservoir on the Spillway Flow 13,500 cfs.
1:30 Scale Model



ANCHOR DAM

PRELIMINARY AND RECOMMENDED SPILLWAY DESIGNS

FIGURE 5
REPORT HYD 289

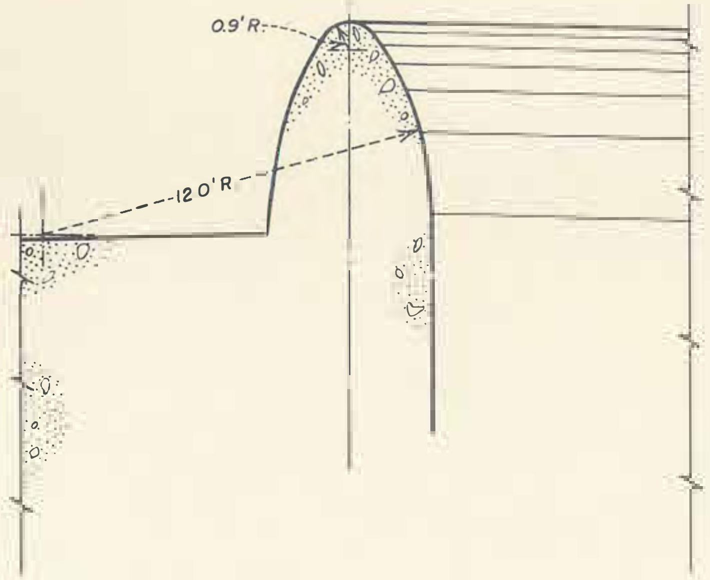
A. Spillway Face Slope = 0.20. 20-foot radius bucket.

B. Spillway Face Slope = 0.37. Parabolic bucket with the drain slots shown below.

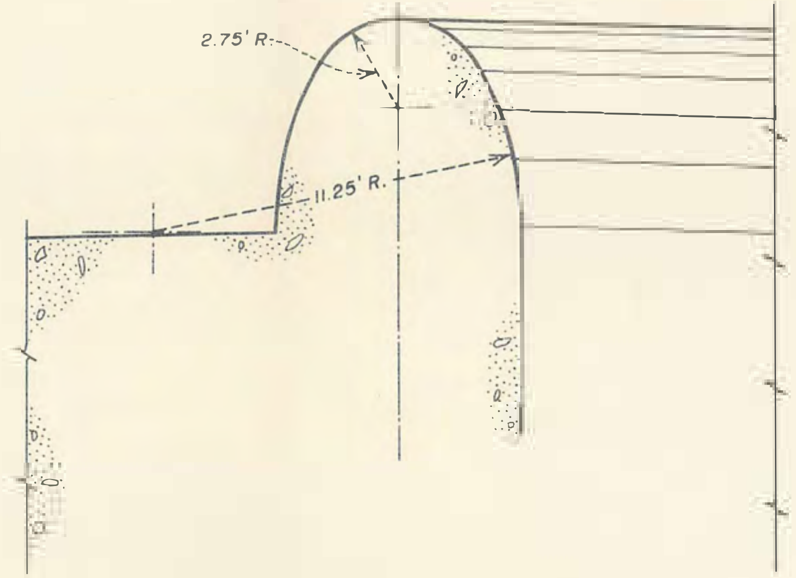
C. Drain slots alternate with equal widths of normal bucket profile to form dentated lip. Outlet works is at side of bucket.

ANCHOR DAM

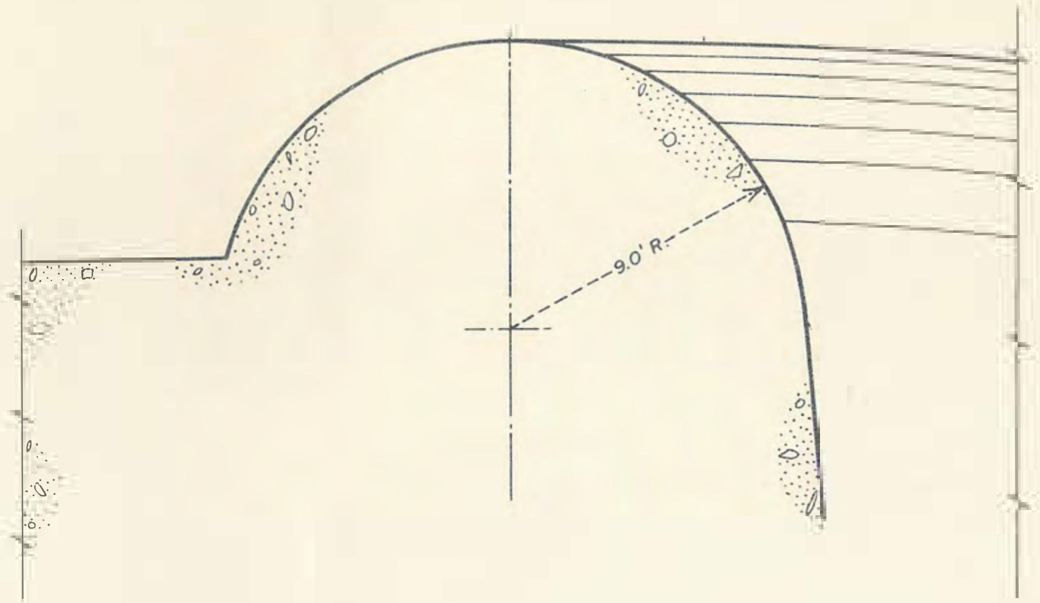
Effect of the Spillway Slope and the Bucket Shape on the Flow Over the Spillway and in the Bucket 13,500 cfs.



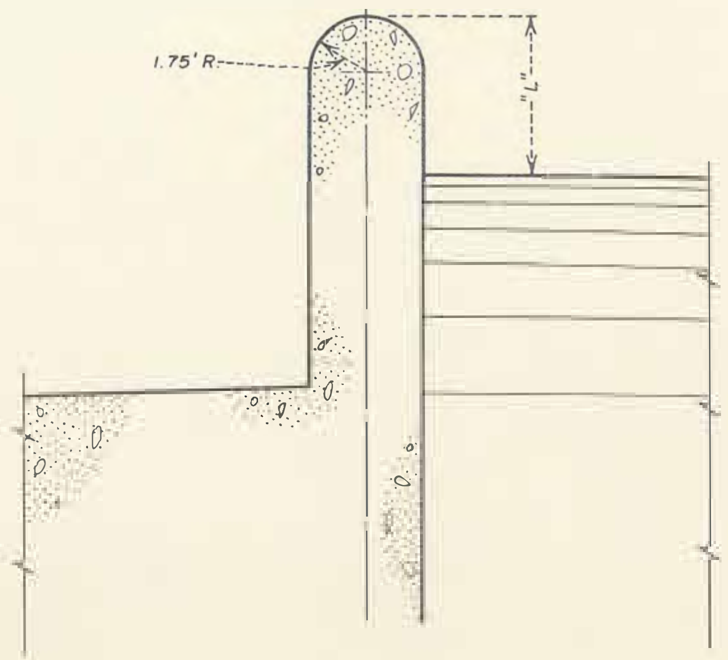
A. PRELIMINARY ELLIPTICAL DESIGN



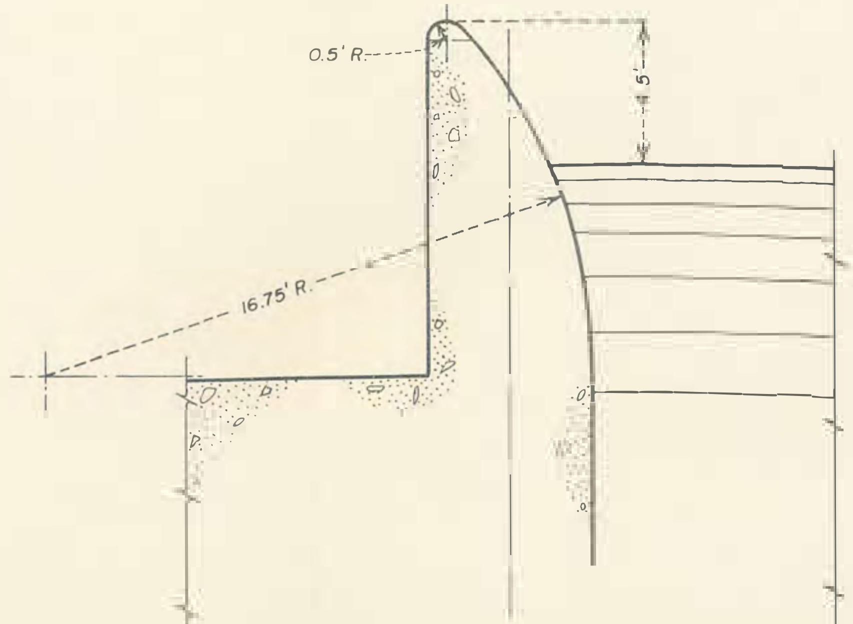
B. MODIFIED ELLIPTICAL DESIGN



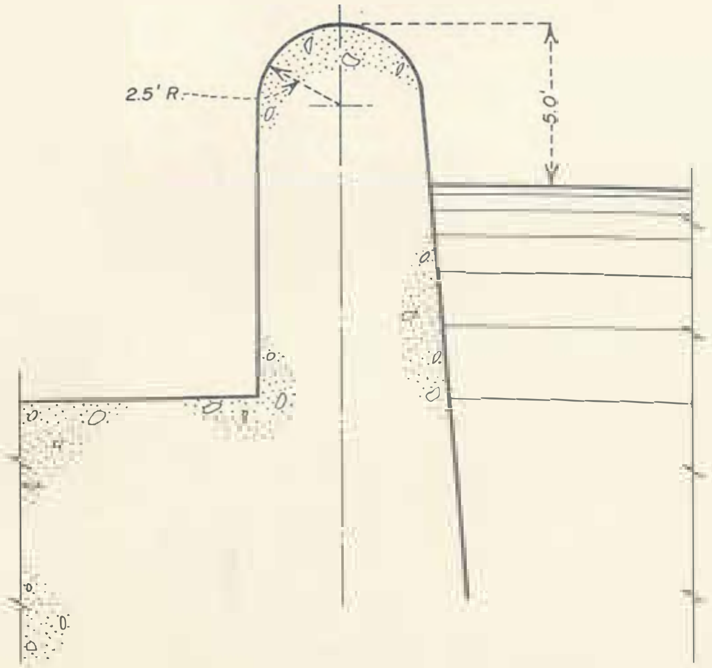
C. CIRCULAR DESIGN



D. EXTENSION PIER WITH ROUND NOSE
"L" = 3, 4, 5, 6, AND 10 FEET

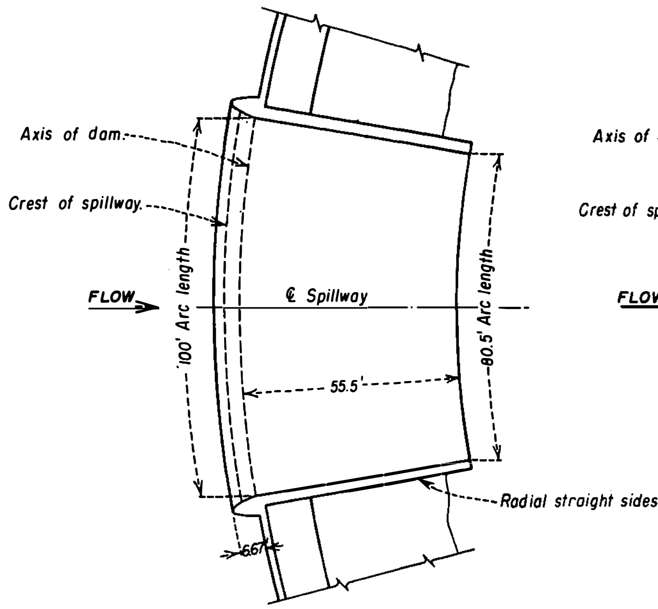


F. EXTENDED PIER WITH LONG RADIUS CURVE

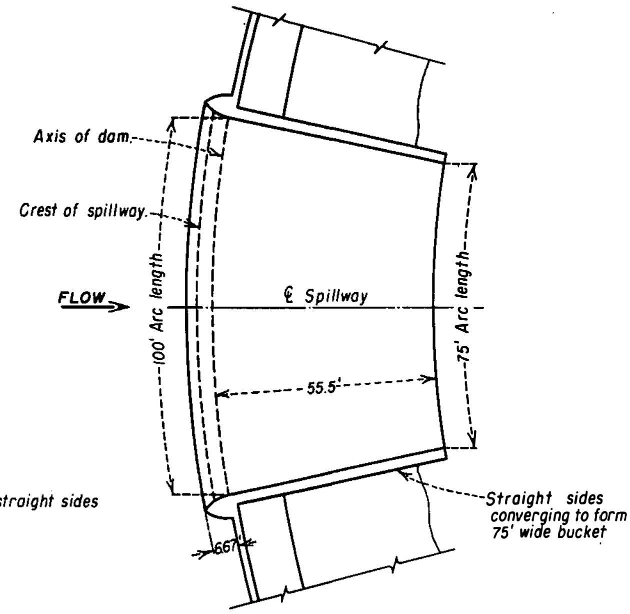


F. RECOMMENDED DESIGN

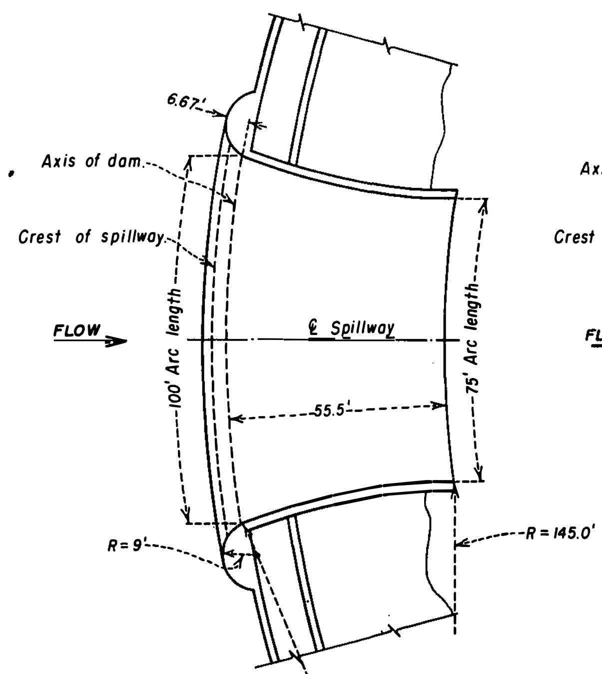
ANCHOR DAM
PIER NOSE DESIGNS TESTED IN 1:30 SCALE MODEL
PLAN VIEWS



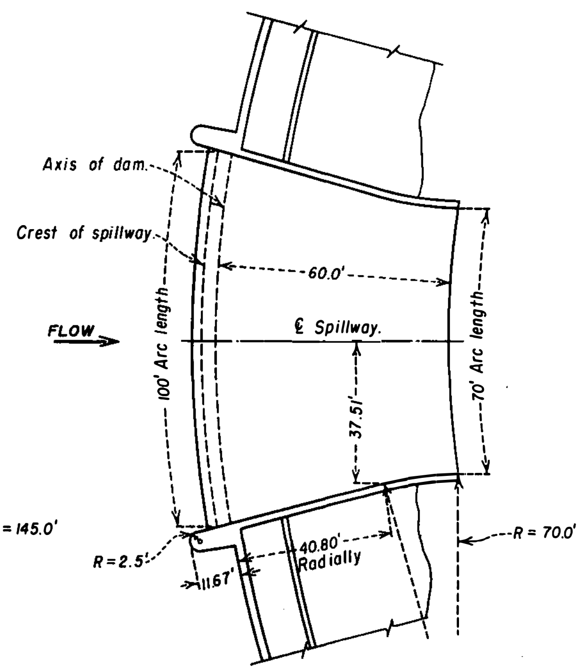
A. RADIAL TRAINING WALLS WITH AN 80.5' WIDE BUCKET
PRELIMINARY DESIGN



B. STRAIGHT TRAINING WALLS WITH A 75' WIDE BUCKET

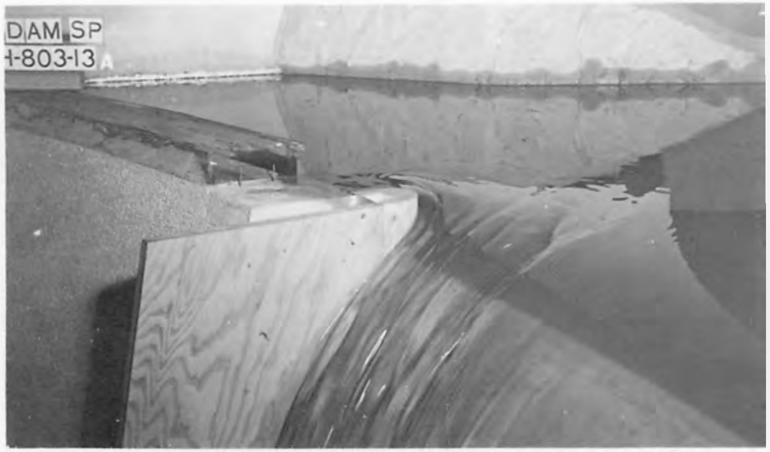


C. CURVED TRAINING WALLS OF A 145' RADIUS WITH A 75' WIDE BUCKET



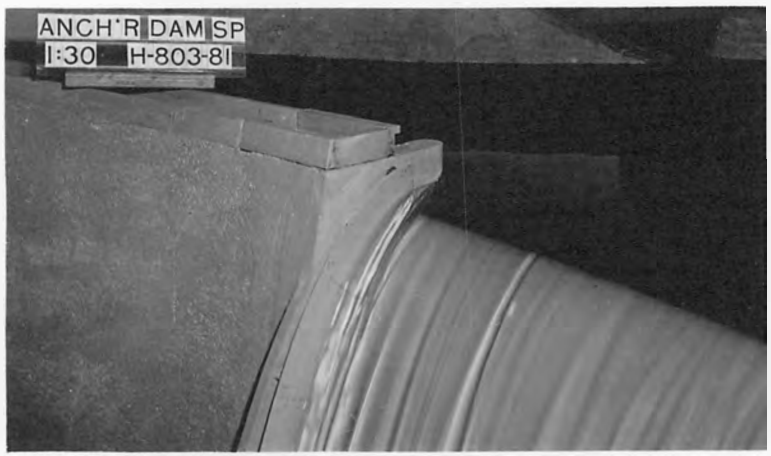
D. STRAIGHT TRAINING WALLS TERMINATING WITH 70' RADIUS CURVES TO FORM 70' WIDE BUCKET
RECOMMENDED DESIGN

ANCHOR DAM
TRAINING WALL DESIGNS TESTED IN 1:30 SCALE MODEL
PLAN VIEWS



A. Modified Elliptical Design - 13, 000 cfs.

B. Recommended design - 5, 000 cfs.

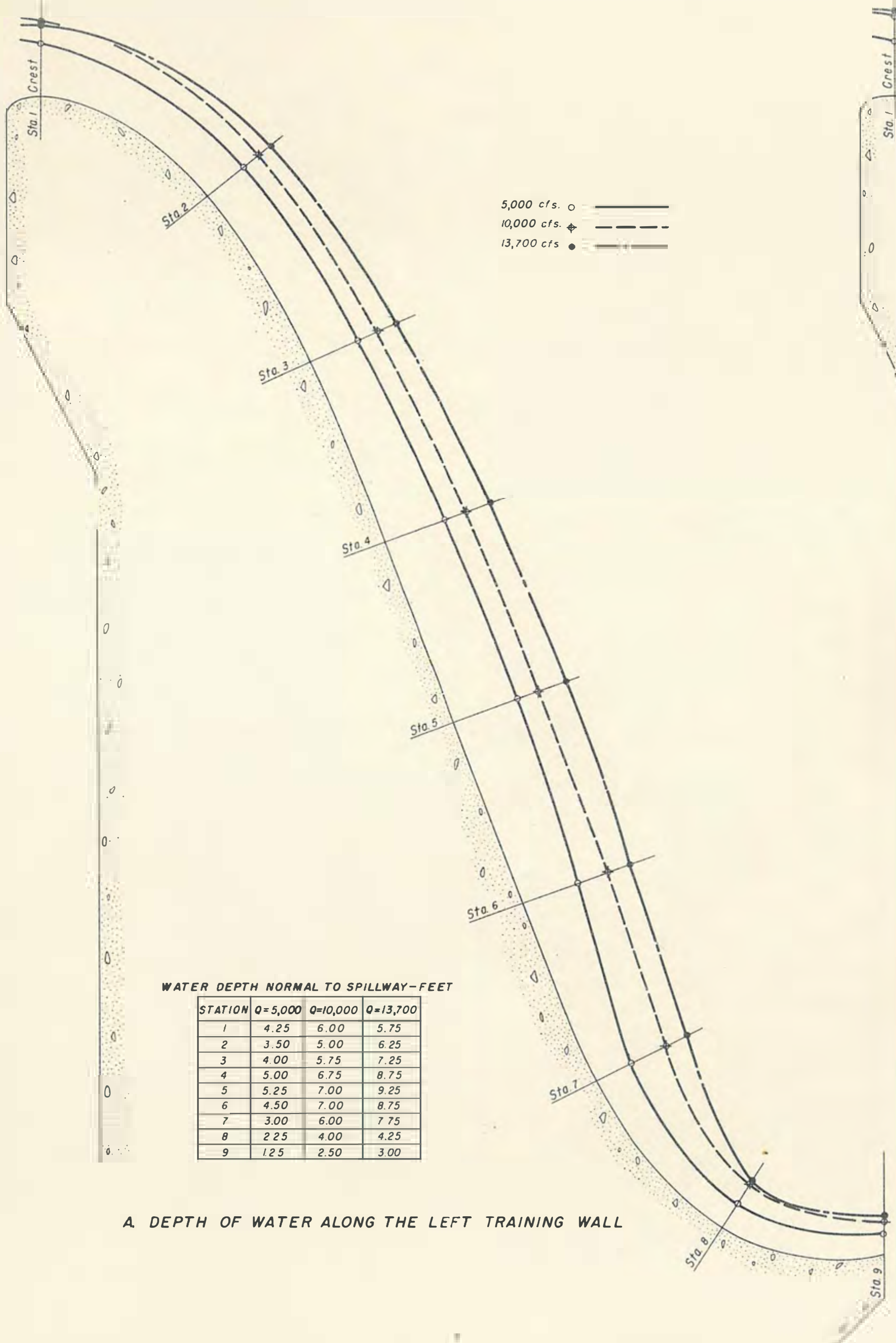


C. Recommended Design - 10, 000 cfs.

D. Recommended design - 13, 500 cfs.

ANCHOR DAM

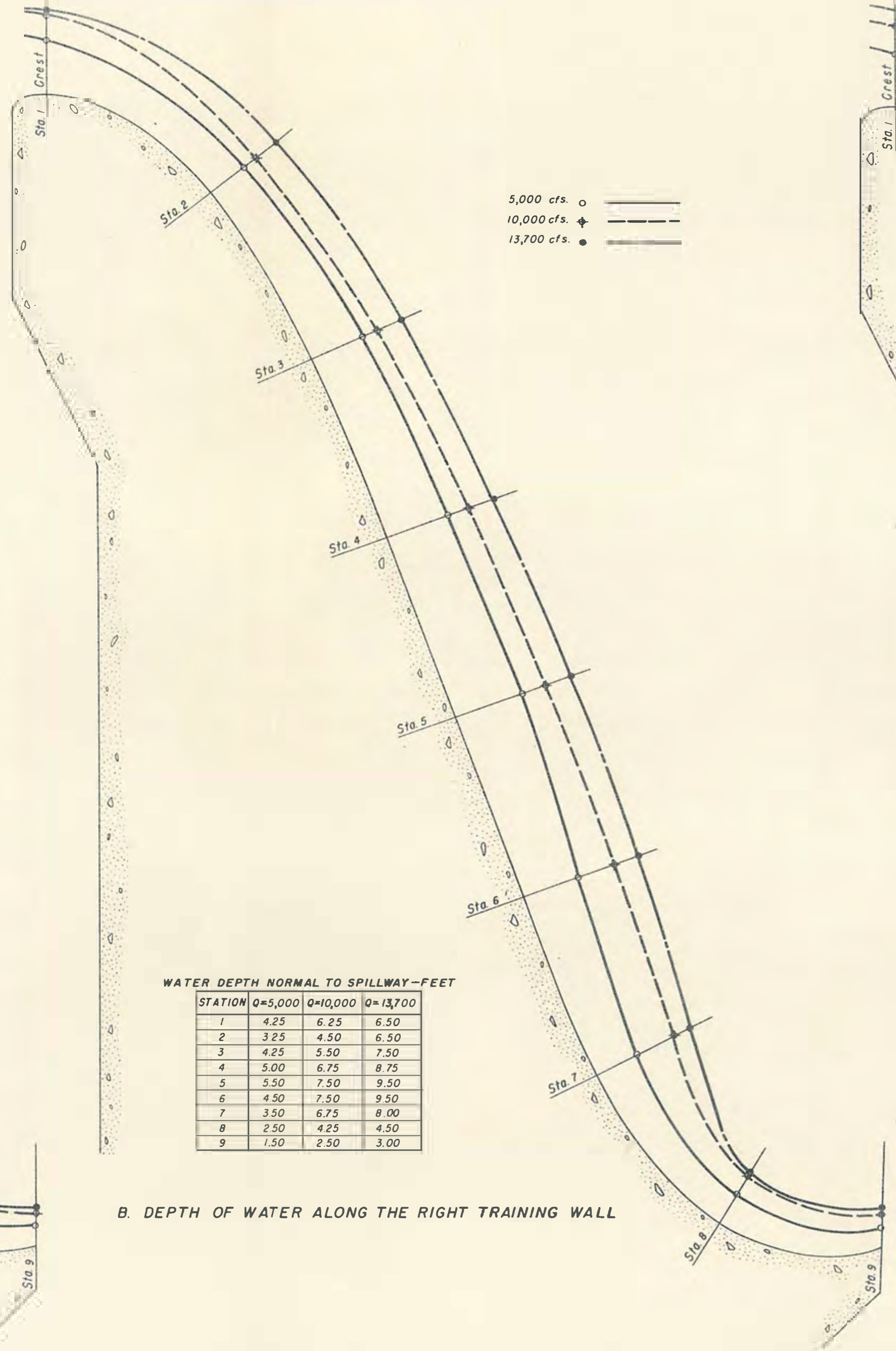
Flow Around the Pier Noses
1:30 Scale Model



WATER DEPTH NORMAL TO SPILLWAY–FEET

STATION	Q=5,000	Q=10,000	Q=13,700
1	4.25	6.00	5.75
2	3.50	5.00	6.25
3	4.00	5.75	7.25
4	5.00	6.75	8.75
5	5.25	7.00	9.25
6	4.50	7.00	8.75
7	3.00	6.00	7.75
8	2.25	4.00	4.25
9	1.25	2.50	3.00

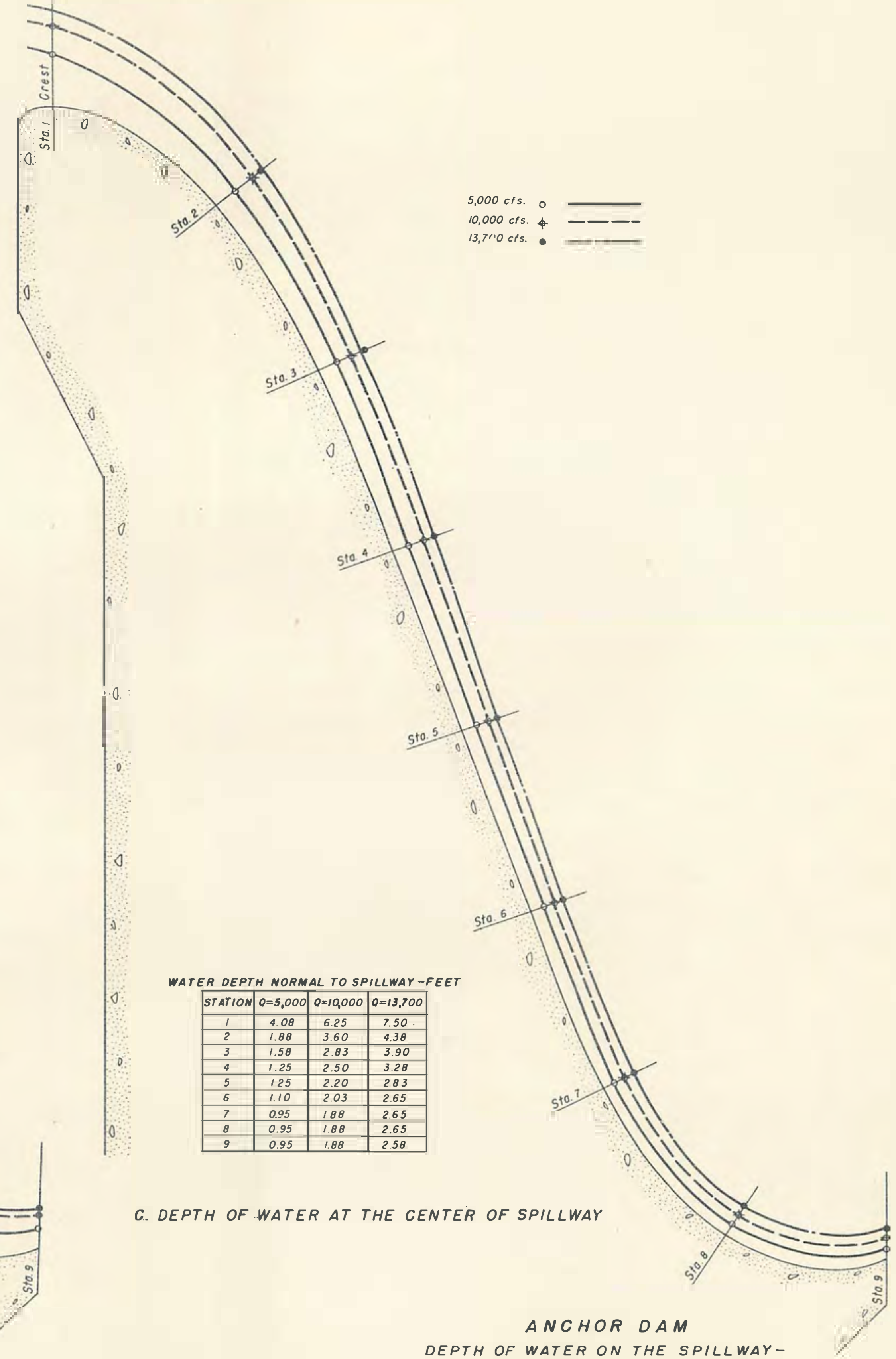
A. DEPTH OF WATER ALONG THE LEFT TRAINING WALL



WATER DEPTH NORMAL TO SPILLWAY–FEET

STATION	Q=5,000	Q=10,000	Q=13,700
1	4.25	6.25	6.50
2	3.25	4.50	6.50
3	4.25	5.50	7.50
4	5.00	6.75	8.75
5	5.50	7.50	9.50
6	4.50	7.50	9.50
7	3.50	6.75	8.00
8	2.50	4.25	4.50
9	1.50	2.50	3.00

B. DEPTH OF WATER ALONG THE RIGHT TRAINING WALL



WATER DEPTH NORMAL TO SPILLWAY–FEET

STATION	Q=5,000	Q=10,000	Q=13,700
1	4.08	6.25	7.50
2	1.88	3.60	4.38
3	1.58	2.83	3.90
4	1.25	2.50	3.28
5	1.25	2.20	2.83
6	1.10	2.03	2.65
7	0.95	1.88	2.65
8	0.95	1.88	2.65
9	0.95	1.88	2.58

C. DEPTH OF WATER AT THE CENTER OF SPILLWAY

ANCHOR DAM

DEPTH OF WATER ON THE SPILLWAY–
RECOMMENDED DESIGN

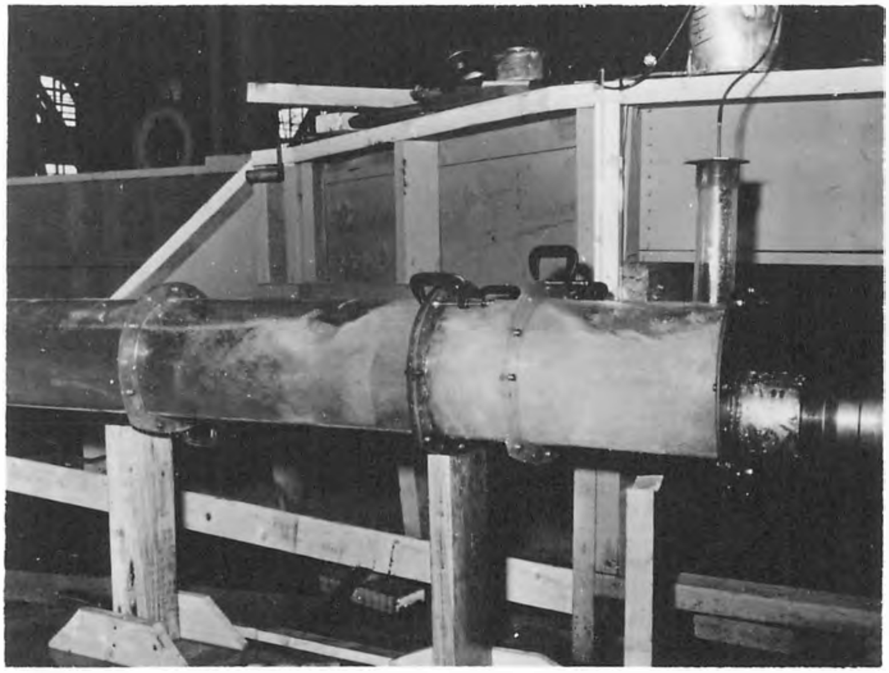
DATA FROM 1:30 SCALE MODEL

A. 5,000 cfs.

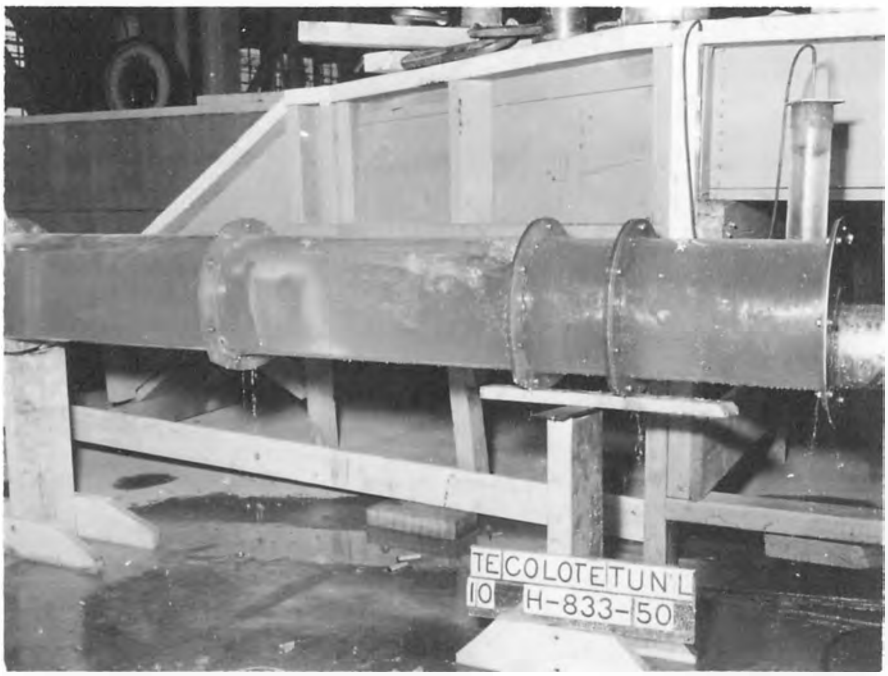
B. 13,500 cfs.

ANCHOR DAM

Flow on the Spillway and in the River Channel--Recommended Design
1:30 Scale Model



A. One tunnel baffle 15 feet from valve



B. Two tunnel baffles, 10 and 15 feet from valve. Note high water level in standpipe and the turbulence below the second baffle.

TECOLOTE TUNNEL
Operation of 1:10 Scale Model of Tunnel Baffle Designs with Flow of 100 cfs and 104 foot head on Valve.

A. Sand and gravel overburden placed on bedrock to elevation 6365.00.

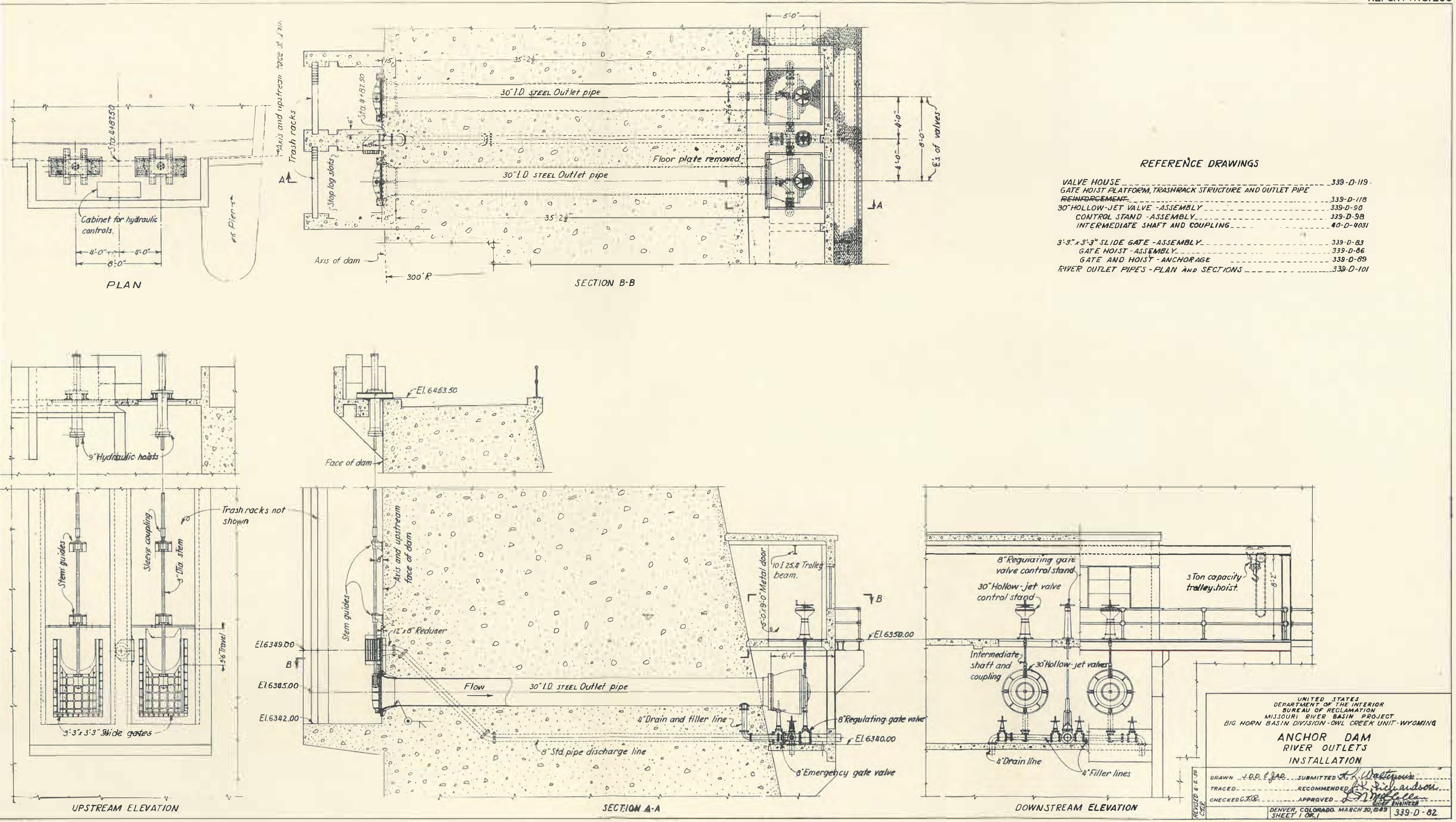
B. Scour after 30 minutes' operation at 5, 000 cfs.

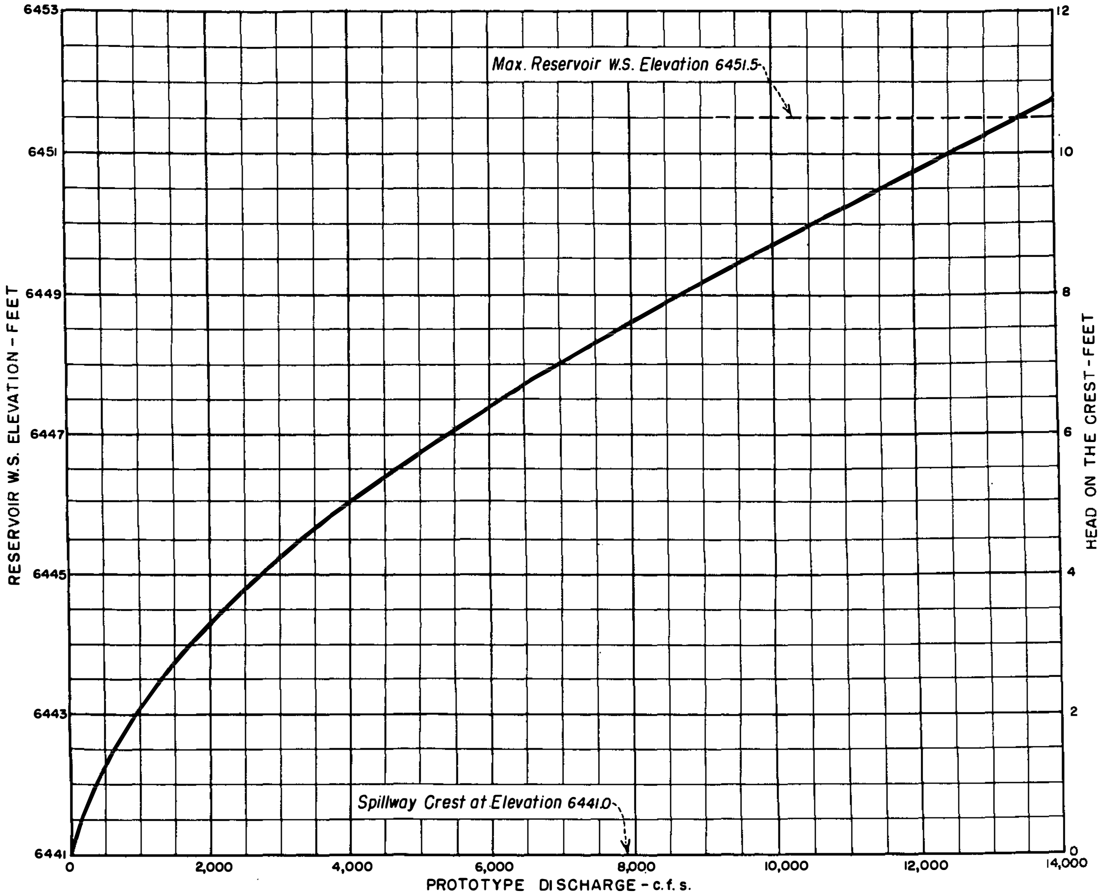
C. Scour after 30 minutes' operation at 13, 500 cfs.

ANCHOR DAM

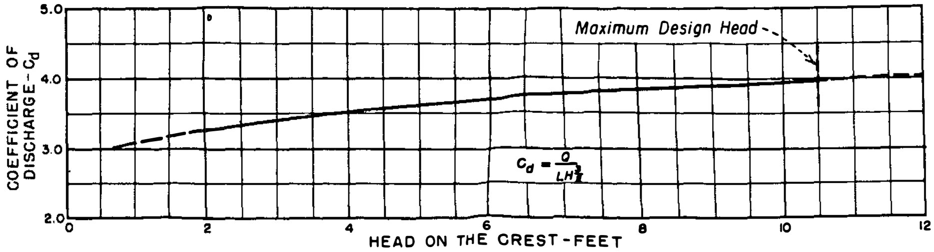
Scour in the River Channel
1:30 Scale Model

FIGURE 13
REPORT HYD. 289





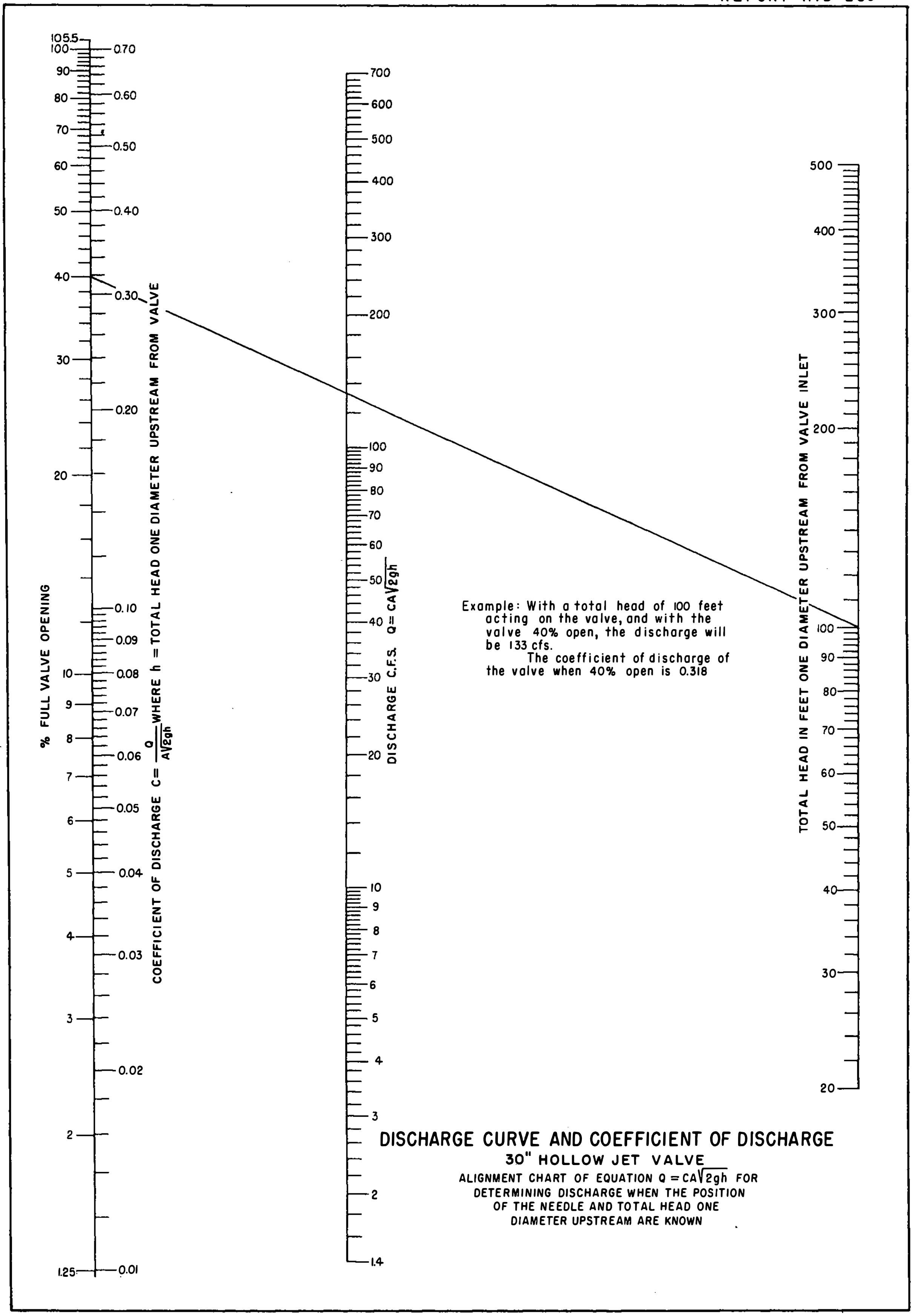
A. SPILLWAY DISCHARGE VERSUS RESERVOIR ELEVATION AND HEAD ON THE SPILLWAY CREST



B. COEFFICIENT OF DISCHARGE VERSUS HEAD ON THE CREST

ANCHOR DAM

DISCHARGE CURVE AND COEFFICIENT OF DISCHARGE FOR THE SPILLWAY
RECOMMENDED DESIGN
1:30 SCALE MODEL



DISCHARGE CURVE AND COEFFICIENT OF DISCHARGE

30" HOLLOW JET VALVE

ALIGNMENT CHART OF EQUATION $Q = CA\sqrt{2gh}$ FOR DETERMINING DISCHARGE WHEN THE POSITION OF THE NEEDLE AND TOTAL HEAD ONE DIAMETER UPSTREAM ARE KNOWN