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# HYDRAULIC MODEL STUDIES OF FALCON DAM

Hydraulic Laboratory Report No. Hyd. 276

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RESEARCH AND GEOLOGY DIVISION



BRANCH OF DESIGN AND CONSTRUCTION  
DENVER, COLORADO

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JULY 31, 1950

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FOREWORD

This report contains the results of tests on three hydraulic models of features of Falcon Dam, constructed by the Hydraulic Laboratory of the office of the Chief Engineer, United States Department of the Interior, Bureau of Reclamation, Denver, Colorado, during the period from August 1949 to April 1950. These studies were a part of the design procedure of the Bureau of Reclamation, which was entrusted with the design of Falcon Dam and Power Plant by agreement of the two Commissioners of the International Boundary and Water Commission and as approved by the appropriate agencies of both Governments. The model studies were made to observe the operation of preliminary designs, evaluate proposed changes in the structures, and obtain performance data.

The first part of the report contains a summary of the testing program, the modifications suggested for the preliminary design to provide improved performance, and the results obtained; while the remainder of the report contains detailed information regarding the tests.

The testing program was authorized by L. N. McClellan, Chief Engineer of the Bureau of Reclamation. H. G. Arthur, as design sponsor for the project, coordinated the testing program with the various sections of the Branch of Design and Construction and with the International Boundary Commission. The model studies were conducted by the Hydraulic Laboratory staff, with the cooperation of the staff of Spillway and Outlet Works Section 1.

The Hydraulic Laboratory was headed by J. E. Warnock until January 1950, and at present is under the direction of H. M. Martin. J. N. Bradley, A. J. Peterka, and E. J. Rusho supervised the tests, which were conducted by Arthur S. Reinhart. Messrs. D. C. McConaughy, C. J. Hoffman, and G. H. Austin of Spillways and Outlets Section 1, advised and assisted the Laboratory during the tests. In addition, other personnel of the Bureau, not named individually, participated in the testing program, and assisted in preparation of the report.

There is available, as a supplement to this report, a short motion picture showing the arrangement and operation of the various models. This motion picture, produced by the photographic staff of the Hydraulic Laboratory, is available for loan by writing to Mr. L. M. Lawson, Commissioner, United States Section, International Boundary and Water Commission, First National Bank Building, El Paso, Texas.

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Laboratory Report No. 276  
Hydraulic Laboratory  
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Subject: Hydraulic model studies of Falcon Dam

SUMMARY

Hydraulic characteristics of the various features of the Falcon Dam Project were investigated by means of three models constructed in the Hydraulic Laboratory at Denver, Colorado. A multiple-purpose model, constructed to a scale of 1:130, was made primarily to study general flow conditions resulting from the interrelation of various hydraulic features, as well as to study the performance of the spillway in detail. This project model included the spillway, powerhouses, intake structures, approximately 2 miles of river channel and sufficient adjoining topography to determine the effect of flood discharges. Two larger scale models were used to investigate the performance of the outlet works. Each of these models included 2 hollow-jet valves for regulating the discharge, a stilling basin, and sufficient downstream channel to give a true representation of operating conditions.

Tests were run on each model for a range of discharges up to and including the maximum predicted flows. Observations, notes, photographs and graphical records were made to evaluate both the preliminary designs and desirable modifications suggested as the tests progressed.

Observations of the 1:130 scale model in operation provided a better understanding of the complicated flow patterns which existed downstream from the dam for various combinations of flow. Tests showed that the problems resulting from the intersecting flows were less severe than might be anticipated. A portion of the area downstream from the spillway was modeled because it was possible that the riverbanks opposite the confluence of the excavated channels and river might erode sufficiently to make protective measures necessary in the prototype. Tests showed, however, that no appreciable erosion of the banks occurred for any flow condition. It was found that with high discharges, when tail water completely inundated the area below the spillway, flow from the spillway channel crossed the intervening area adjacent to the United States powerhouse and flowed into the powerhouse tailrace channel and the river proper. This cross flow eroded the bank of the spillway channel on the river side and appeared to be sufficient to interfere with

power plant operation. To prevent this undesirable flow, a dike was built along the left side of the spillway channel, Figure 9.

Some improvement in flow was also obtained by eliminating the large water area existing to the right of the spillway channel. A destructive eddy induced by spillway flows formed in this area and another dike, similar to that on the left, was built on the right of the channel (Figure 9). These dikes confined the flow to the excavated channel, resulting in greatly improved operation.

In contrast with the mild erosion tendencies in the channels and at the confluence of the excavated channels and the river, the areas immediately downstream from the corners of the spillway apron eroded severely. To reduce this erosion, 45° spur-type wing walls were added at both sides of the basin (Figure 11, Design S-4).

At the dividing pier between the spillway and United States power plant intake structure an undesirable depression in the water surface occurred on the spillway side, due to excessive contraction of the flow entering the end bay of the spillway. Studies of eight different pier designs were made to determine the most effective and satisfactory pier nose shapes. The pier recommended for field construction is shown in Figure 25.

Tests on the 1:30 scale model of the preliminary design of the Mexican outlet works showed a relatively rough water surface, both in and downstream from the basin, and excessive erosion in the channel downstream from the horizontal apron. An end sill added to the apron reduced the depth and extent of the erosion. Lengthening the basin provided a smoother water surface in the downstream channel. Adding tapered fillets to the sloping bottom of the basin and moving the converging walls upstream resulted in better energy dissipation in the stilling basin. The cellular dividing wall between the powerhouse draft tubes and the stilling basin, the wing wall at the right end of the basin, and the dividing wall in the basin were modified to improve flow conditions in and below the stilling basin. Each modification was tested and evaluated by means of erosion and sweep-out tests, described in the report, and by water surface profiles shown in Figures 57 and 58. The stilling basin structure recommended for field construction is shown in Figures 54 and 59, while its operation is shown in Figures 60, 61, and 62. Results of erosion tests at maximum discharge are shown in Figure 63.

Tests on the 1:24 scale model of the United States outlet works were similar to those made for the Mexican outlet works. Since the United States outlets discharged less water through smaller hollow-jet valves, it was possible to reduce the dimensions of certain components of the stilling basin. Tests to determine these dimensions were made. In addition, tapered fillets, an end sill, a longer horizontal apron, and a shortened center wall all provided improved performance, as had been found for the Mexican outlet works. The basin floor was lowered to increase the margin of safety against the jump sweeping off the apron for low tail

water. This gave the same margin of safety as that provided in the Mexican outlet works. Erosion tests and water surface profiles were used to evaluate the various modifications tested. The stilling basin recommended for field construction is shown in Figure 72. Figures 73, 74, and 75 show the structure in operation; and Figures 76 and 77 show the erosion to be expected for the maximum flow and the water surface profiles in the basin, respectively.

#### TERMINOLOGY

All measurements used in this report are prototype values unless otherwise noted. Horizontal dimensions are in prototype feet or stations and elevations are given in prototype feet above sea level.

The various arrangements tested are denoted by numbers, while the model itself is denoted by a letter preceding the number. For example all arrangements of the project model are preceded by the letter "S"; those of the Mexican outlet model are preceded by the letter "M"; and those of the United States outlet model by the letter "A." Thus, Test M-5 is the fifth arrangement or modification of the Mexican outlet works that was tested.

As a general rule, hydraulic dimensions are also given in prototype terms. The various model-prototype relations are determined according to the Froude law, and are given below. In this tabulation, the subscripts p and m denote prototype and model values respectively, and N denotes the scale ratio prototype-to-model ( $L_p/L_m$ ).

$$L_p = L_m \times N$$

$$Q_p = Q_m \times N^{5/2}$$

$$V_p = V_m \times N^{1/2}$$

$$H_p = H_m \times N$$

$$T_p = T_m \times N^{1/2}$$

where Q = discharge in cubic feet per second

V = velocity in feet per second

H = head in feet of water

T = time in hours, minutes, or seconds

L = any linear dimension

## INTRODUCTION

Under the Water Treaty of 1944, the United States and Mexico agreed to construct a series of dams on the Rio Grande between Fort Quitman, Texas, and the Gulf of Mexico. These dams, of which Falcon Dam is the furthest downstream, Figure 1, are to be constructed jointly by the two Governments, acting through their respective sections of the International Boundary and Water Commission. By agreement of the Commission, and with the concurrence of the respective Governments, the Bureau of Reclamation has been entrusted with the design of Falcon Dam, including the spillway, outlet works, and powerhouses.

Falcon Dam, located 72 miles southeast of Laredo, Texas, is an earth fill structure approximately 24,300 feet long, rising 150 feet above the present bed of the Rio Grande, Figure 2. Two power plants of similar design are located on either side of the river channel immediately downstream from the dam, and each house has an outlet works utilizing two hollow-jet valves, Figures 3 and 4. The valves in the Mexican outlet works are 90 inches in diameter; those in the United States outlet works are 72 inches in diameter. A chute-type spillway, Figure 3, located on the United States side of the river is equipped with six 50-foot by 50-foot rectangular leaf gates on the crest. At the downstream end of the spillway is a horizontal stilling basin 600 feet wide and 180 feet long with chute blocks at the upstream end and a dentated sill at the downstream end.

To study the operating characteristics of the structure, three hydraulic models were built in the Bureau of Reclamation Hydraulic Laboratory in Denver, Colorado. One model, constructed to a scale of 1:130, included the spillway, powerhouse, intake structures, one-third square mile of reservoir, and approximately 2 miles of downstream river channel. This model was constructed to study the performance of the spillway and powerhouse tailrace channels, and the effects of these three structures discharging into the Rio Grande, simultaneously or in any combination. The study was also made to furnish information relative to spillway capacity, flow patterns, and erosion both below the spillway and at the intersections of the various channels.

A second model, constructed to a scale of 1:30, included the outlet works and the power plant on the Mexican side of the river, and was constructed to study hydraulic performance of the outlet works stilling basin. One side of the model stilling basin was constructed of glass as an aid in observing the underwater performance in the stilling pool.

A similar model was built to study the operation of the outlet works on the United States side. To utilize the same size model valves, the model scale was increased to 1:24. For this model, no power plant discharge was provided, as tests of the 1:30 model indicated that discharges through the power plant had no appreciable effect on the action of the stilling basin below the valves. Detailed descriptions of each model will be found in succeeding sections of this report.

## PART I

### The Project Model

#### Description of the Model

To study the interrelation of spillway flow, power plant discharge, and flow in the river downstream from the dam, as well as details of spillway performance, a large over-all model was built in the Hydraulic Laboratory. Because of the location of the laboratory sump and other physical features, a "mirror image" model was constructed; the model being reversed from left to right, Figures 2 and 5. In the discussion of the various structures in this report the minor image arrangement is used. (Left and right hand are reversed.) Since in all other respects the model was similar to the prototype, this reversal did not affect the hydraulic performance.

The Falcon Dam spillway has a crest 350 feet long, with six 50- by 50-foot gates to control the flow. An inclined chute 1,168 feet long with diverging training walls leads to the horizontal stilling basin, which is 180 feet long and 600 feet wide, Figure 3. Chute blocks 8 feet high are at the upstream end of the apron, and a 12-foot high dentated sill is at the downstream end. The horizontal apron, at elevation 175, is 81.7 feet lower than the spillway crest. Maximum head on the spillway crest will be about 58 feet for the maximum predicted discharge of 456,000 cfs.

The model, Figures 5 and 6, covered approximately 3,400 square feet of floor space, and was constructed in two main sections: a head box 12 feet 2 inches by 43 feet constructed of wood lined with sheet metal, and a tail box 65 feet 10 inches by 43 feet constructed of cinder block and mortar. The cinder block walls were sealed to the concrete laboratory floor, eliminating the need for any other bottom.

Topography in the head box was made of concrete formed over metal lath, which in turn was supported by wooden forms. Most of the topography in the tail box was concrete formed over a gravel base, as shown in Figures 7A and B, but where erosion was to be studied, uniformly fine sand (average grain size 20 mm) was molded to the proper contour. The topography was placed in such a manner that any concrete area could be replaced by a sand area with little difficulty. While preliminary plans called for the placing of riprap downstream from the spillway and powerhouses, this was omitted in the model since the exact effect of riprap would be difficult to duplicate. As a result, erosion in the model was somewhat more severe than is anticipated in the prototype.

The spillway, Figure 8A, was formed of smooth concrete carefully formed to metal guides placed parallel to the flow. Transverse metal strips were placed between the templates to make possible alterations of spillway length with minimum time and effort. The training

walls, dentated sill, piers, and powerhouses were of wood, while the six spillway gates were made of sheet metal. These gates operated in metal-lined slots in the piers. The gates were fitted with racks which engaged pinions set on a single rod, so that the gates could be operated as a unit. If individual gate operation was desired, the pinions could be disengaged and any gate operated manually. To measure pressures on the spillway, piezometers were installed both in the crest and in the steep chute just upstream from the stilling basin.

Each power plant structure was supplied with water from two pipes connected to the head box. One pipe supplied water to the outlet works, while the other pipe supplied flow to the simulated turbines. No attempt was made to duplicate exactly the flow through the powerhouses as the small scale made this infeasible. Detailed studies of the outlet works and powerhouse discharges were made on other models.

Water was supplied to the model from the laboratory sump through a 6-inch pipe, in which was installed an orifice meter to measure the flow. To more closely duplicate the smooth flow characteristics of an actual reservoir, the water supply line was branched and water was introduced into the head box through two pipes placed at third points behind a 6-inch rock baffle constructed near the upstream end of the head box, Figure 8B. After flowing through the model, the water was returned directly to the laboratory sump.

Staff gages were installed in both the head box and tail box to indicate water levels. In addition, profiles of the water surface on the spillway were obtained by means of point gages.

#### Operation of the Model

It was necessary to increase the tail water elevation before discharging large amounts of water over the spillway to prevent premature erosion of the model bed. Accordingly, water was supplied to the downstream end of the model for this purpose. When the tail water had risen to a sufficient depth on the apron, the main supply pump was turned on and the reservoir area filled. A mercury manometer indicated the discharge as measured by the orifice meter, and the desired discharge was then set by means of a valve in the line. The maximum spillway discharge was considered to be 456,000 cfs without regard to the reservoir water surface. The elevation of the tail water, read from a staff gage, was controlled by a hinged gate at the downstream end of the model over which the water passed. Raising or lowering this gate raised or lowered the level of the tail water.

During initial tests of the model, flow performance was observed for the entire model, with particular attention to the following: Tendencies for erosion of the river banks at the intersection of the spillway channel with the river; erosion tendencies at the intersections of the powerhouse tailrace channels and the main river channel; tendency for flows to erode or leave established channels; erosion downstream from the

spillway apron; flow patterns in the spillway approach, at the spillway entrance at the powerhouse intakes; and hydraulic action of the spillway itself. The large project model made possible the study of all these problems and their relation to one other.

#### General Flow Problems

The problem of riverbank erosion opposite the entrance of the spillway channel into the Rio Grande was of particular interest because preliminary plans called for the location of a town site on the Mexican side of the river opposite the spillway channel, though the site location was later changed. Severe erosion of the riverbank would have endangered this town. However, model operations indicated no significant erosion in this area for any flow. At high spillway flows, including the maximum of 456,000 cfs, the large area inundated reduced the velocity sufficiently to prevent scour, while at lower spillway flows water in the river channel deflected the spillway flow and acted as a buffer to protect the riverbank. Even after several days of testing the model at various discharges, there was no appreciable erosion of the sand used to represent the channel material.

The channels below the powerhouses showed but little disturbance of the erodible bed. There was practically no movement of the sand under any conditions. Flow in the tailraces was very smooth and uniform for all flows, and it was apparent from observations of model operation that no problems were present in these areas. Model operation did show, however, that at discharges above 370,000 cfs there was considerable cross flow from the spillway to the United States power plant tailrace across the normally dry area separating the two channels. The cross flow caused some erosion of the channel banks and interfered with the flow from the power plant. In addition there was an undesirable eddy induced in the area to the right of the spillway channel. These conditions indicated that it would be advisable to confine spillway discharges to the spillway channel. Accordingly, earth dikes were placed along the channel for a distance of 1,800 feet downstream from the end of the basin, Figure 9. With this modification, the spillway flow remained in the spillway channel for a sufficient distance to prevent any interference with power plant operation. These dikes were retained for all future testing.

Observations of the flow in various locations in the model indicated that the general arrangement of the structures and excavated channels was satisfactory. With the dikes placed along the spillway channel there was no intermingling of spillway and powerhouse discharges before the flows joined in the intended channel. Flow throughout the model was smooth and uniform and followed the flow lines expected.

#### General Characteristics of the Spillway

In operation, the spillway performed generally as intended, Figure 10A. For the maximum discharge of 456,000 cfs, the jump was quite uniform and stayed well on the apron, Figure 10B, indicating a satisfactory

basin design with proper energy dissipation. Although a standing wave pattern, caused by the piers, was evident in the spillway chute, flow distribution was satisfactory throughout the entire length, being quite uniform where it entered the stilling basin, as evidenced by the uniformity of the jump. At lower flows the operation was also very satisfactory, with the flow continuing to be spread evenly across the spillway chute. Even when the gates were opened unsymmetrically the flow was spread satisfactorily. With one side gate open, a standing wave formed against the opposite wall of the chute. With any other gate open, a triangular wave pattern was formed in the chute. In either case, however, sufficient spreading of the flow occurred to produce a satisfactory jump in the stilling basin.

With the two outside gates open (Gates 1 and 6—gates are numbered from left to right), a triangular wave pattern formed on the spillway at all flows, with the base of the triangle at the downstream end of the chute. The triangle was not quite centered in the chute, as the differences in shape of the approaches caused a slightly greater concentration of flow in the left side of the chute. With Gates 2 and 5 open, a multiple diamond-shaped wave pattern formed in the chute at all flows. Again the distribution was slightly unsymmetrical, due to approach conditions. With the two center gates open, the wave pattern was a single diamond not quite closed at the downstream end. With all combinations of two gate openings, the flow spread and resulting jump in the basin was satisfactory.

#### Discussion of Tests

Although, as indicated above, the general operation of the spillway was satisfactory, there were four problems that warranted investigation and improvement. These were:

1. Erosion downstream from the stilling basin
2. Poor approach conditions, particularly along the left spillway entrance pier
3. Lack of free board on the spillway chute walls
4. Spillway capacity

To study these problems, various modifications were made and tested in the model. The modifications are numbered and the numbers are preceded by the letter "S"—thus Design S-2 is the second spillway change tested. When, later, it became necessary to change only the details of the left approach pier, each change was given a letter—Pier B, Pier C, etc. The change was then designated as a combination of the number representing the basic arrangement, and a letter representing the pier. Thus, S-7 with Pier B in place became S-7B.

Erosion Problems. Operation of the preliminary design showed that there was considerable erosion immediately downstream from the stilling basin, particularly at the right side. To measure and record the severity

of this erosion, as well as that obtained with proposed corrective changes, the following technique was used. The sand areas were formed by hand to represent the topography expected in the prototype. The maximum anticipated discharge of 456,000 cfs was then passed over the spillway for a 2-hour period. (Two hours in the model represented 22.8 hours in the prototype). During this time, photographs of the spillway in operation were made. At the end of the 2 hours the water was shut off, the model allowed to drain, and the resulting erosion pattern contoured and photographed. From observations and data, and assisted by photographs, a suitable modification was selected. The various modifications tested to reduce erosion and the results of each test are summarized below. Wing wall arrangements are shown in Figure 11.

The preliminary design of the spillway, S-1, provided for two wing walls placed at  $90^\circ$  to the line of flow. These wing walls extended from the stilling basin training walls to the sides of the trapezoidal channel below the basin. Operation of this design for 2 hours scoured a hole 50 feet deep (measured from the basin floor) at the right end of the apron, and a similar, lesser hole at the left end, Figure 12A. This erosion was due to eddies occurring immediately downstream from the end sill at each side of the basin (Figure 12B). The eddies occurring were caused by the abrupt increase in cross section immediately downstream from the wing wall. It was believed that the more extensive erosion on the right occurred because the flow leaving the end sill was intercepted almost immediately by the right bank causing a relatively more severe eddy at the right apron corner. At the left corner where the flow was, in effect, directed away from the channel bank the induced eddy and resulting erosion were less severe. Observation of the flow in the two areas indicated that the more violent eddy occurred on the right side for all ranges of flow. The severe resulting erosion necessitated revising the preliminary design.

In Design S-2, the wing walls were modified by extending them downstream 66.5 feet from the end of the apron at a  $45^\circ$  angle from which point they were continued normal to the line of flow until they intersected the sides of the trapezoidal channel, Figure 11. The walls thus eliminated the abrupt change in area and allowed the flow leaving the apron to expand more gradually, reducing the violence of the eddies causing the erosion. Tests of the modified Design S-2 were made for a discharge of 456,000 cfs, as previously described, with improved results over the first design. After 2 hours of operation it was found that a hole 10 feet deep had eroded at the base of the right wing wall, Figure 13A. In addition, the erosion exposed a portion of the cut-off wall below the end of the horizontal apron at the right side of the basin. While the result showed definite improvement, it was believed further improvement could be affected.

For Design S-3, "spur-type" wing walls, were added to the first design as shown in Figure 11. These walls were the same height as the stilling basin walls and extended downstream from the basin at  $45^\circ$  on the pool side, the back side of the wall being parallel to the spillway centerline. The resulting V-shaped wall extended 46 feet downstream from the apron. In operation the wall projected into the eddy path, intercepting

it, and reducing its erosive action. Tests with this wall showed a very satisfactory erosion pattern (Figure 13B). The scour along the wing walls was substantially reduced, and none of the cut-off wall was exposed. The only noticeable erosion was a hole approximately 200 feet downstream from the end sill, in the center of the spillway channel. Since this hole had occurred for all tests of all modifications, and since it was not a part of the wing wall problem, no consideration was given it at this time.

For Design S-4, the spur wall length was reduced to 29.5 feet as this was the width of the footings of the training walls. Tests on this shorter wall resulted in less satisfactory erosion patterns than were found for the longer wall of Design S-3. The shorter wall evidently provided less braking action on the induced eddies at the apron corners and resulted in slightly more erosion along the walls and end of the apron. After a 2-hour test at maximum discharge, the cut-off wall below the end sill was exposed approximately 2 feet, Figure 14A. However, this increase in erosion was considered relatively minor, and it was believed that, in the prototype, this wall would provide sufficient protection from undermining. Since riprap will be placed in the prototype areas subject to erosion (the same areas where fine sand was used in the model) the depth of erosion should be less severe in the prototype.

Design S-5 used the same length and type of spur wall as Design S-4, but the wall was only one-half as high, the top being placed at elevation 210. At maximum discharge and tail water elevation the top was 25 feet below the water surface. A 2-hour scour test with the maximum discharge of 456,000 cfs produced a deep hole immediately downstream from the right end of the basin, along the right spur wall. This hole was 35 feet deep, as shown in Figure 14B. The erosion was very similar to that which occurred with 90° wing walls. To determine the reasons for this erosion, dye streams were used to trace the flow lines. It was found that because the wall was below the water surface an eddy current circulated over the top of the wall, Figure 15. A vertical current also traveled down the inside face of the wall at high velocity causing the erosion noted above. Since the wall was found to be of no value in providing protection from excessive erosion, it was given no further consideration.

To verify the results of the erosion tests, independent corroborative tests were run using Designs S-4 and S-5. Results of these check tests compared closely with those obtained for the original tests. A photograph of the confirming test for Design S-5 is included in this report, as Figure 16. The similarity of these erosion patterns, obtained for two independent tests using identical procedures, operating conditions, and walls may be seen by comparing Figure 14B with the check test, Figure 16.

On the basis of these wing wall tests it is evident that wing walls of the spur type, with top elevation 245.0, aid in providing protection from erosion at the apron corners. Long walls provided more protection than short walls, and consequently the longest wall consistent with best structural arrangement and justifiable cost should be added at the end of the stilling basin.

Spillway Approach Pier Tests. Operation of the model with a free crest and flows up to about 200,000 cfs indicated the spillway approach conditions were satisfactory. Also, when the gates were partially open and the reservoir was sufficiently high to produce orifice discharge, no undesirable approach conditions existed. However, at flows exceeding 200,000 cfs, with free crest operation, excessive flow contraction occurred at each end of the spillway, resulting in depressed water surfaces adjacent to the abutments of the spillway crest, Figure 17A. The depression at the left approach pier was particularly noticeable, producing an undesirable water surface, creating an unbalanced pressure on the pier, and reducing the discharge through the end gates.

Photographs of the depression in the water surface showed that the flow accelerated as it traveled around the pier, Figure 17B. The flow lines tended to leave the pier face along the spillway side, resulting in an unbalanced pressure on the pier, together with a "piling-up" of the water against the first intermediate spillway pier.

To reduce the severity of the flow contractions three general procedures were possible:

1. To increase the radius of the pier nose. This solution was not feasible in this case because of the limiting fixed distance between the spillway crest and the power plant intake.
2. To increase the length of the pier, thereby moving the depressed water surface sufficiently far upstream so that recovery of head and a level water surface could be effected before the crest was reached.
3. To decrease the length of the pier, thereby moving the depressed water surface downstream from the crest.

Since the first method was not practical, in this case, the pier length was increased from 63.8 feet to 100 feet as shown in Design S-6, Figures 18A and B, retaining the original pier nose shape. For a flow of 456,000 cfs, full recovery of the depressed water surface occurred approximately 30 feet upstream from the crest as shown in Figure 18A. However, the hydraulic advantages offered by this scheme did not warrant the added cost involved in extending the pier.

To study the effect of reducing the pier length, seven different piers were installed and tested. With each pier in place, the water surface in the vicinity of the pier was measured and photographed for a discharge of 456,000 cfs. Relative discharge coefficients were also obtained for each design, using the equation

$$Q = CLH^{3/2}$$

where    Q = total discharge in cubic feet per second  
          C = coefficient of discharge  
          L = length of spillway crest in feet  
          H = head on crest in feet of water

To minimize error and make the coefficients directly comparable, the piers were tested without changing the discharge valve setting and without interrupting the water supply. A discharge of 456,000 cfs was passed through the model with all gates fully opened, and the piers were inserted and removed without otherwise altering the flow conditions. A specially designed, float-type, water level gage having a mechanically magnified dial was used to measure the change in water surface in the head box as the piers were changed. The headwater elevation and the metered discharge were then used to compute the discharge coefficients for each pier. Details of each pier and its performance are discussed in the following paragraphs. Different pier shapes are designated by letters of the alphabet, and all are considered, for purposes of discussion, to be variations of Design S-7. All values given in the pier discussions are for a discharge of 456,000 cfs with all gates open.

Pier A, Design S-7A, is the pier shown in the preliminary design. This pier extended 63.8 feet upstream from the crest, with an elliptically shaped nose. The maximum draw-down with this pier was 22.7 feet which occurred approximately 32.5 feet upstream from the crest. There was a recovery, or surface rise, of 7.7 feet before the crest was reached, making draw-down at the crest of 16.0 feet below reservoir water level. The discharge coefficient was found to be 3.36. The pier shape, water surface profile, and photograph of the draw-down are shown in Figure 19.

Pier B, Design S-7B, (Figure 20) was a short pier, extending 38.5 feet upstream from the crest, with a nose radius of 25 feet. With this pier in place, the point of maximum draw-down occurred approximately at the spillway crest. The elevation of the water surface at this point was approximately 28.2 feet below the reservoir water surface. Thirty feet upstream from the crest the water surface was reduced only 14 feet, as contrasted with 22 feet in the same location for the preliminary design. A discharge coefficient of 3.34 was obtained for a discharge of 456,000 cfs. Considering the draw-down and the discharge coefficient the performance of the pier was considered acceptable but an undesirable fin formed against the chute side wall and the flow was concentrated along the first spillway pier. Since this arrangement was not entirely satisfactory, the tests were continued.

Pier C, Design S-7C, shown in Figure 21, was 57 feet long. On the spillway side, a 50-foot radius was used to form the curve, and a 10-foot radius was used to form the nose of the pier. This pier performed much like the original pier; there was an abrupt draw-down at the nose, with some recovery before the crest was reached. The maximum draw-down was 22.5 feet for maximum discharge, and the position of the low point was approximately 30 feet upstream from the crest. A discharge coefficient of 3.34 was obtained for a discharge of 456,000 cfs.

Pier D, Design S-7D, (Figure 22) was 47.5 feet long, 16.3 feet shorter than the original pier. The curved face next to the spillway had a radius of 180 feet and the nose radius was 3 feet. This pier performed better than either the original or Pier C. A draw-down of 15.0 feet at maximum discharge, occurred approximately 41 feet upstream from the crest. Also, there was a recovery of 5 feet in surface elevation before the crest was reached. A discharge coefficient of 3.33 was obtained for a discharge of 456,000 cfs. However, with partial gate openings, there was a tendency for subsurface water to eddy about the nose of the pier, creating an undesirable condition in the power plant intake area.

Pier E, Design S-7E, was shaped as shown in Figure 23, in an attempt to reduce the eddies at the nose of the pier and to move the position of maximum draw-down further downstream. Although this was partly accomplished, the offset in the pier caused an undesirable eddy and vortex in the area to the right of the pier nose. A discharge coefficient of 3.33 was obtained for a discharge of 456,000 cfs. Because of the eddy and vortex the testing was continued.

Pier F, Design S-7F, (Figure 24) was a very short pier with a radius of 15 feet on the face next to the spillway. This pier extended only 22.17 feet upstream from the crest, and had a sharp angle where the radius intersected the upstream face. A severe disturbance occurred at the upstream end of the pier face next to the spillway which was caused by the sharp angle. Occasionally the flow broke completely free of the pier. Also, there was a "piling up" of the water against the first spillway pier, due to the tangential acceleration of the water as it passed around the approach pier and over the spillway crest. Maximum draw-down for this pier was 42.3 feet and occurred approximately 10 feet upstream from the crest. A discharge coefficient of 3.31 was obtained for 456,000 cfs. Because of the disturbance induced by the pier, instability of flow, and low discharge coefficient, this pier was considered unsatisfactory.

Pier G, Design S-7G, (Figure 25) was a refinement of Pier F. It was the same length, but the sharp angle was rounded with a 10-foot radius in an attempt to prevent the water from breaking free of the pier face as had occurred with Pier F. An improvement in the appearance of the flow was obtained as the flow followed the curve. A depressed water surface was evident along the left training wall, but this occurred well downstream from the crest. The maximum draw-down was 28.4 feet; and the discharge coefficient for 456,00 cfs was 3.31 which was the same value as previously obtained for Pier F. Because of the relatively low coefficient testing was continued.

Pier H, Design S-7H, (Figure 26) was 43 feet long, with a 180-foot radius on the face next to the spillway. The face next to the intake and the pier nose were formed to a continuous elliptical curve. The ellipse was introduced for the purpose of reducing the subsurface eddies that occurred with partial gate openings. The draw-down with this pier, 25 feet upstream from the crest, was 12.2 feet, which was considerable improvement over Pier A which was 20.8 feet longer. A coefficient of 3.33 was obtained for 456,000 cfs.

Considering the draw-down, discharge coefficient, over-all performance, and relative costs of all piers tested, Pier H, was the most satisfactory in alleviating the adverse flow conditions at the left pier. No modifications were made at the right abutment, since the draw-down there was not as severe as at the left abutment.

Pressures on the Spillway Face. Tests were run to determine the pressures on the spillway crest and on the vertical curve upstream from the stilling basin for various discharges and gate openings. A total of 14 piezometers had been installed to measure these pressures. Pressures were determined for discharges over the spillway ranging from 70,000 to 456,000 cfs. For each discharge except the maximum the flow was passed with all gates opened equally and the reservoir at maximum elevation. The lowest pressure which occurred just downstream from the crest for a discharge of 70,000 cfs, with gates opened 6 feet, was atmospheric. In all other cases the pressures were above atmospheric. Piezometer locations and pressures for discharges of 456,000 cfs and 70,000 cfs are shown in Figure 27. For 70,000 cfs the sheet of water on the vertical curve of the model was so thin that reliable measurements could not be made. It is certain, however, that these pressures, not shown in Figure 27, were above atmospheric.

Training Wall Studies. With a discharge of 456,000 cfs, the freeboard on the left training wall at Station 25/40 was only 2 feet, which was not believed sufficient to take care of spray, splash, and the bulking effect caused by greater insufflation of air in the prototype. By changing the slope of the top of the training walls, as shown in Figure 28, the minimum freeboard may be increased to 7 feet which would provide an adequate margin of safety against overtopping the chute walls.

Water Surface Profiles. Water surface profiles were measured along the spillway chute walls for discharges of 456,000, 300,000, 200,000, 100,000 and 60,000 cfs. Profiles for a discharge of 456,000 cfs were recorded with all gates open, while profiles for the lower discharges were obtained for flow through partial gate openings with the reservoir water surface at elevation 314.2. Longitudinal profiles are shown in Figures 28 and 29. These profiles along the wall were higher than the average profile through the spillway chute because the standing wave pattern, which originated at the spillway piers, was reflected from the chute walls. Transverse water surface profiles were measured near the crest and at Station 28/43 (immediately upstream from the upper limit of the hydraulic jump) as shown in Figure 30. These profiles indicate that the flow was well distributed across the chute just prior to entering the hydraulic jump, thus the divergence of the chute walls and the general arrangement of the upstream portion of the spillway were considered satisfactory.

Spillway Capacity. The spillway was calibrated with Pier H in place and the discharge for reservoir water surface 314.2 was found to be 436,000 cfs. The corresponding discharge coefficient  $C$  in the equation:  $Q = CILH^{3/2}$ , was computed to be 3.33. Thus, the capacity of the spillway,

based on the 1:130 scale model, was about 4.3 percent less than anticipated from design calculations. Experience with small models has shown, however, that because of differences in the thickness of the boundary layer, model to prototype, and because of surface tension effects, the prototype will discharge slightly more water than the model. Discharge curves obtained from the model are shown in Figures 31 and 32, and the relation between discharge coefficient and head is also shown in Figure 31.

#### The Recommended Design

The design recommended for construction in the field is basically the same as that proposed in the preliminary design. Certain modifications were made to improve the hydraulic performance of the structure as follows:

1. Two dikes, one along each side of the spillway channel and extending 1,800 feet downstream from the end of the stilling basin, Figure 9, were added to confine spillway discharges to the spillway channel.
2. Two spur-type wing walls 29.7 feet long downstream from the basin, Figure 11, were added to the existing wing walls to reduce the erosion at the corners.
3. Pier H (Figure 26) replaced the left abutment pier to improve the flow condition in the power plant intake and in the spillway entrance.
4. Higher training walls (Figure 28) were recommended for use in confining the flow in the spillway chute.

Operation of the recommended structure was satisfactory in all respects. Flow from the spillway channel did not interfere with flow from the power plant tailrace. Nor was there a tendency toward erosion below the power plants or at any of the channels intersecting the Rio Grande. Erosion at the corners of the spillway apron after maximum discharge for 2 hours was only 10 feet below the normal elevation of the channel (Figure 14) as contrasted with 35 feet for the preliminary arrangement. The depression in the water surface due to contraction about the left approach pier was reduced, the discharge coefficient of the spillway was increased slightly and the operating appearance of the structure was improved. Pier H was recommended as it gave satisfactory results and was economical.

Flow in the spillway chute was satisfactory at all discharges, as shown in Figures 33, 34, 35, and 36. At high discharges, the flow spread evenly in the chute and showed no tendency to overtop the training walls. At lower discharges the flow was also spread evenly over the spillway, if the gates were opened equally, Figures 33A and B. If the gates were not opened equally, standing waves formed in the chute, but they were not sufficiently high to be of concern and they had only a minor effect on the performance of the stilling basin, Figures 35 and 36. The stilling basin also performed well at all discharges; even with unsymmetrical gate openings, the jump remained well up on the slope of the chute. Figure 37 shows the entire model operating with a spillway discharge of 456,000 cfs.

## PART II

### The Mexican Outlet Works Model

#### Description and Operation of the Model

The hydraulic performance of the Mexican outlet works was studied on a 1:30 scale model which included the outlet control valves, the stilling basin, a simulated powerhouse, and approximately 450 feet of downstream channel. Figures 38 and 39 show the recommended design. The 1:30 scale was selected to utilize 3-inch hollow-jet valves, which were available in the laboratory, to represent the 90-inch hollow-jet valves of the prototype.

In the preliminary design, Figure 40, the Mexican outlet works contained two 90-inch hollow-jet valves which discharged over a 30° inclined floor onto a horizontal apron 80 feet long and 25.2 feet below the tail water elevation, for a 9,400 cfs flow in the downstream river channel. A center dividing wall extended the entire length of the basin, which resulted, in effect, in a separate stilling basin for each valve. Converging walls immediately below the valves were included to compress the jet of water from the sides before it plunged beneath the tail water and into the stilling basin. Forty-five degree wing walls at the ends of both stilling basin training walls permitted the flow to diverge into the combined outlet and power plant tailrace channel, 182.5 feet wide. From the end of the basin, the channel bottom sloped upward at 4:1 to river bed elevation. Figure 60, although it illustrates the recommended design developed from the tests, shows the general arrangement of the structure.

Maximum total discharge through the two valves was 4,570 cfs for a total head of 81.9 feet at the valves, while the maximum discharge for one valve was 2,400 cfs at a total head of 90.3 feet. These discharges and heads were used to determine the necessary dimensions of the structure since they resulted in the most severe operating conditions. It was found that a structure which performed satisfactorily for these maximum conditions would also be satisfactory for any other head and discharge.

The head box and tail box in which the model was contained were constructed of wood lined with sheet metal. The left side of the stilling basin contained a large glass panel through which subsurface hydraulic action in the stilling basin could be studied. Flow in the power plant tailrace was simulated by discharging the proper quantity of water through the model powerhouse which contained rock baffles in place of the turbines in the prototype structure. Thus the model powerhouse introduced the proper quantity of flow into the tailrace, at the correct velocity and in the proper direction. Separate but interconnected head boxes were used to supply the outlet valves and powerhouse. Water from the laboratory sump was pumped through an 8-inch pipe in which was placed an orifice meter to measure the flow. The 8-inch pipe discharged directly

into the head box used to supply the outlet valves, while the powerhouse head box was connected to the outlet valve head box with another 8-inch pipe containing a control valve.

Since only a short portion of the outlet conduit was represented in the model, reservoir elevations could not be set in the head box using a simple scale relationship. Instead, the losses in the prototype conduit were calculated down to the outlet valves and the resulting pressure head determined at this point. Piezometers were installed one pipe diameter upstream from the valve, and the model value of the calculated piezometric pressure was set by opening or closing the valve. The proper model discharge, as measured by a Venturi meter, was supplied to the head box and the model valves opened or closed until the desired head of the valve was obtained as indicated by the piezometer.

The topography in the downstream tail box was molded in sand having the following analysis:

<u>Size sieve</u>	<u>Percent passed</u>
4	100
8	91
16	63
30	27
50	3
100	0

Prior to each test, this sand was formed to represent the channel below the outlets, and after each test the resulting erosion was contoured and photographed.

#### Discussion of Tests

Development tests were made to improve the performance of the preliminary design by installing and testing modifications in various parts of the structure. The effectiveness of each modification was evaluated by operating the model for 1 hour with maximum discharge for two valves. Photographs were taken during the run to show action in and downstream from the basin. At the end of the run, the erosion was contoured and the results photographed. The same test procedure was then repeated, with only one valve discharging 2,400 cfs. Results of each test were then compared with results of previous tests, using the photographs and other data as an aid. As a rule, each modification that resulted in improved operation was left in place for all subsequent testing.

The various modifications tested are numbered in order, each number being preceded by the letter M which indicates tests on the Mexican outlet works. Thus Design M-3 was tested immediately following Design M-2. Each change in the wing walls was noted by a change in letter following the number. Thus, Design M-6A was Design M-6 with Wing Wall A in place; Design M-6B was Design M-6 with Wing Wall B in place. A tabulation of the model designs tested will be found at the end of this section.

Preliminary Design. Operation of the preliminary Design M-1, Figure 40, was reasonably satisfactory, but it appeared that some modifications would be desirable to improve the performance.

The first tests disclosed the following objectionable features: (1) excessive erosion of the channel bed immediately downstream from the stilling basin, and (2) turbulent water surface downstream from the stilling basin. A reduction in the length of the center wall also appeared to be possible.

With both valves operating at maximum discharge, the jets from the valves plunged beneath the surface of the water and dissipated most of their energy before the flow left the basin. The water surface was rough, both in the basin and downstream, Figures 41A and B. Also objectionable waves formed in the tailrace opposite the power plant as a result of the unstable action. With one valve operating the water leaving the basin expanded rapidly causing flow around the end of the dividing wall. The difference in elevation of the water surface between the two sides of the basin was approximately 2 feet.

Erosion downstream from the horizontal apron is shown in Figure 42. The channel was eroded to elevation 152 or 4 feet below the level of the apron after maximum flow for 1 hour. Since the original bed had sloped upward at 4:1 from the end of the apron, and since the eroded area occurred approximately 8 feet downstream from the end of the basin, the actual depth of the erosion was 6 feet. With one valve operating more scour occurred, although no measurements were made, than with both valves operating. In subsequent erosion tests the single-valve operation proved the most critical and this was considered the criterion in judging results.

Tests with and without the power plant operating showed very little difference in performance, Figures 41 and 43. With the power plant discharging the wave action below the stilling basin was slightly reduced, Figure 43, while the erosion was slightly greater at the left side of the stilling basin and slightly less at the right side. Since there was no appreciable difference with or without the power plant discharge, and since the tests could be made more rapidly without it, most of the subsequent testing was done without the power plant discharging.

Basin Design Tests. To reduce the depth of erosion downstream from the apron, various sill designs were tested with the center wall shortened 46 feet. The revised wall now extended to the footings of the powerhouse, Figure 40. Structural requirements of the powerhouse required that the remaining portion of the center wall be retained. The only effect noted from eliminating the center wall was the return of sand to the side of the basin below the nonoperating valve, when one valve was operating.

Design M-2A was Design M-2, with End Sill A added. This sill was dentated, 4 feet 6 inches high, and was installed at the end of the stilling basin, Figure 44. Tests with one 90-inch valve discharging a maximum of 2,400 cfs showed that the greatest erosion was to elevation 157, or 1 foot higher than the basin floor, Figure 45B. This erosion occurred

immediately downstream from the discharging valve. While this sill effectively reduced the erosion downstream from the stilling basin, the roughness of the water surface was increased. A high boil occurred directly over the sill, which resulted in an increase in wave height downstream, Figure 45A.

To reduce the boil over the end sill, the lower, solid block sill B, 1 foot 6 inches high, Design M-2B, Figure 44, was tested. This sill height was believed to be sufficient to deflect the water upward and prevent appreciable erosion at the toe of the basin without causing an excessive boil to form on the surface above the sill.

A test with one valve operating showed a considerably reduced boil at the end of the basin, but a hole to elevation 156 was eroded approximately 5 feet downstream from the end of the basin. This erosion was to the same elevation as the floor of the apron and was 1 foot deeper than with the dentated sill.

The tests on the sills of Designs M-2A and M-2B indicated that a sill of some type aided in reducing the erosion downstream from the basin. It appeared, however, that better stilling action in the basin could be obtained by modifications elsewhere; and since the erosion would be affected by such changes, it was considered desirable to proceed with that testing first. Development of the final sill was therefore deferred until other basin details had been decided.

In developing the stilling basin for the Boysen Dam Outlet Works (Hydraulic Laboratory Report 283) converging walls were shown to be valuable in aiding energy dissipation in the basin. It was also found that a similar convergence on the bottom of the jet was helpful. Consequently, in Design M-3, tapered fillets were introduced along the inclined floor beneath the valves, Figure 46. These fillets tapered from one-half the width between the converging walls at the bottom of the incline to zero width at the top of the incline, and the top surface of the fillets formed angles of  $45^{\circ}$  with the floor and converging walls. In operation, the wave action both in and downstream from the basin, resulting from this modification, was considerably lessened (Figure 47A) with no increase in the erosion over previous tests. The maximum depth of erosion was to elevation 156, or the elevation of the basin floor, Figure 47B. As with Design M-2, some sand was returned to the basin during single valve operation. Since the fillets were found to improve the basin performance, they were retained and used in the basin for all later tests.

Design M-4 was installed and tested in an effort to further reduce the wave action downstream from the stilling basin. For this design the angle of the inclined floor beneath the valves was reduced from  $30^{\circ}$  to  $24^{\circ}$  with the horizontal, Figure 46. The sloping surface was not extended to the floor, but stopped 4 feet 4.2 inches above the apron then dropped vertically to the floor, providing a cushion of water. Performance tests run at maximum discharge showed the surface violence in and downstream from the basin was reduced, but the water surface slope in the

stilling basin increased from that indicated in previous tests, Figure 48A. This effect was thought to be the result of less energy dissipation in the upper end of the basin with resulting higher velocities. In other respects, the basin performed as well as previous designs. Because this change offered no advantages over previous designs, it was not given further consideration.

For Design M-5 an attempt was made to obtain a level water surface in the basin; the slope of the inclined approach floor was returned to  $30^\circ$ , and the converging walls were moved 8 feet upstream as shown in Figure 46. This change in position of the converging walls resulted in a level water surface, Figure 48B, and improved the operation sufficiently to warrant its retention for all subsequent tests. In other respects, the performance was similar to that obtained with previous modifications.

In a further attempt to improve flow conditions downstream from the stilling basin, modifications of the wing walls at the end of the stilling basin were tested. Alterations in wing wall shapes are discussed as variations of Design M-6. Design M-6A, Figure 49, included outlet wing walls which were similar to the spur walls used on the spillway, Figure 11. Although the spur walls reduced erosion on the spillway, where an induced eddy was present, they did not appreciably reduce the erosion at the end of the outlet works basin where no serious eddies were present, Figure 50A. The maximum erosion with one valve discharging 2,400 cfs was 1 foot below the elevation of the basin floor or to elevation 155, Figure 50B. This occurred immediately downstream from the end sill, exposing the cut-off wall under the horizontal apron. Since the spur walls offered no appreciable improvement, other walls were tested.

The basin training walls were then extended 14 feet downstream without lengthening the basin floor, Figure 49. The wing walls were cut-back at  $45^\circ$ , intersecting the previous wing walls 14 feet from the inside face of the basin training walls. This modification showed improved performance over Design M-6A by reducing the severity of wave action downstream from the stilling basin, Figures 51A and B.

Although the quantity of sand carried back into the basin during one valve operation was not of major concern, modifications were tested to determine whether the quantity could be reduced. Previous designs, that made use of a short dividing wall (Designs M-2, M-3, M-4, M-5, and M-6), showed approximately the same amount of sand returned, while the preliminary design, in which the dividing wall extended the complete length of the basin, resulted in very little sand being deposited on the apron below the closed valve. For design M-7 Figure 49, a dividing wall 5 feet high extending from the end of the powerhouse structure to the end of the basin was used. With one valve discharging 2,400 cfs more sand was returned to the basin than was returned with the short dividing wall, Figure 52. Most of the sand was deposited against the dividing wall, with the remainder scattered over the basin floor. In addition, the erosion was slightly greater than for previous tests using the short wall. Maximum depth of erosion after a 1-hour discharge of 2,400 cfs through one valve was to

elevation 154 or 2 feet below the floor of the horizontal apron. This erosion occurred immediately downstream from the end of the basin and exposed 2 feet of the cut-off wall. The excessive amount of sand returned and the greater erosion made this design unsatisfactory.

To reduce the sand deposit on the apron and to further reduce the surface waves in the vicinity of the powerhouse, the basin floor was extended 14 feet to match the side walls which had previously been lengthened the same amount. Ninety degree wing walls were installed together with a dentated sill at the end of the apron, Figure 49. The short dividing wall was also used. Tests for both valves discharging (Figure 53A) indicated that the longer basin reduced the wave heights in the lower channel and resulted in less severe erosion of the river bed. Maximum depth of erosion was to elevation 156 or the elevation of the apron floor, Figure 53B. Considering the erosion, wave heights, and general performance, this arrangement provided the best operation of all designs tested.

Design M-8C, Figure 54, used a 3-foot high block sill in place of the 4-foot 6-inch dentated sill of Modification M-8. Because this sill was lower than the dentated sill, the boil at the end of the stilling basin was reduced, with consequent lessened wave action downstream. Also the erosion pattern was improved slightly--the maximum depth of erosion was to elevation 157 or 1 foot above the apron. This was 1 foot less erosion than was experienced with the dentated sill. Figure 55 shows a comparison of the erosion obtained with different end sill designs. Since End Sill C was superior in performance to the other sills tested it was recommended for use in the prototype structure.

Sweep-out Tests. Tests were run to obtain the minimum permissible tail water elevation for a given discharge and head. For these tests the valves were operated at maximum capacity and the tail water was gradually lowered, by means of a control gate at the downstream end of the model, until the jump "swept out" of the basin, Figure 56. The results indicated that with the total maximum discharge of 4,570 cfs through both valves, the tail water could be lowered 6 feet below elevation 181.2 before the jump swept out. Tail water elevation 181.2 is the normal tail water for a discharge of 4,570 cfs through the valves plus 5,400 cfs through the turbines. The sweep-out tests were repeated for lower basin discharges, and a greater margin of safety was found for these conditions. Thus, the lowest hydraulic safety factor occurs for the maximum basin discharge.

Pressures on the Stilling Basin Floor. Nine piezometers were installed in the stilling basin to measure the impact of the jet of water upon the floor. These piezometers were installed on the horizontal floor beneath the right valve, immediately downstream from the end of the inclined floor, Figure 57. They were spaced across the right half of the basin so that a transverse pressure distribution could be obtained, and were distributed longitudinally to measure the decrease in pressure as the jet spread downstream. These pressures were used by the designers as an aid in determining the thickness of the basin floor. In the direction of flow, the pressure increased to a peak value of 34 feet of water,

approximately 8 feet downstream from the toe of the incline, and then decreased to 24 feet of water 12.5 feet further downstream.

For purposes of comparison, pressures were also measured with the 24° inclined approach and step of Design M-4. Because of the step between the incline and the basin floor, which formed a water cushion, the pressures were lower than with the recommended design, especially near the toe of the incline. The pressures gradually increased in the direction of flow with a maximum pressure of 29 feet of water, occurring approximately 20 feet downstream from the toe of the incline. Pressures for both designs, together with the location of the piezometers are shown in Figure 57.

Water-surface Profiles. Water-surface profiles were taken through the recommended basin for discharges of 4,570 cfs with both valves operating and 2,400 cfs with one valve operating. These profiles, Figures 58 and 59, were taken transversely at the end of the center wall, at the end of the stilling basin and longitudinally along the left basin wall and along the centerline. The profiles show the relatively level water surface in the basin, even for one valve operation, and the lack of water surface disturbance over and downstream from the end sill. The transverse water-surface profiles, with one valve discharging 2,400 cfs, showed a 2-foot difference in water surface across the basin, Figure 59.

#### The Recommended Design

The recommended design, Design M-8C, Figures 54 and 60, was the most satisfactory of all the designs tested. Sufficient developmental testing had been done to be certain that the component parts of the structure were of a minimum size and that they were properly arranged to provide the best operation possible. The appearance of the structure in operation provided emphasis for this fact. Practically the entire volume of the basin was utilized in dissipating energy, yet the water surface in and downstream from the basin was level with a minimum of surface disturbance, Figures 61A and B. Downstream from the basin the flow entered the tailrace channel satisfactorily, producing negligible waves, Figures 62A and B. With one valve operating, Figures 63A and B, there was some unsymmetrical flow within the stilling basin, but at the end of the basin, the flow was well distributed across the entire width. Greater surface disturbances were evident in the downstream channel, but considering that one valve operation is an emergency condition, the performance was considered satisfactory. Improved performance was obtained with smoother water surfaces for discharges less than the maximum.

There was no severe erosion, downstream from the basin, Figures 64A and B, for any condition tested. After two valves had discharged a total of 4,570 cfs for 1 hour, the maximum erosion was still 1 foot above the floor of the basin. After a discharge of 2,400 cfs through one valve for 1 hour, the erosion depth was found to be the same as for two valves operating. Although a small amount of sand was carried

back into the basin with one valve operating, the condition was considered unimportant since the sand could be swept off the apron by opening the other valve. For normal operation both valves will be operated simultaneously, and sand should not be carried back onto the apron.

# FALCON DAM--MEXICAN OUTLET WORKS

## STILLING BASIN DESIGNS

1:30 Hydraulic Model Studies

DESIGN NUMBER	NEW FEATURES	PURPOSE	ELEVATION OF MAXIMUM EROSION		W. S. CHARACTERISTICS IN BASIN	W. S. CHARACTERISTICS D. S. FROM BASIN	REMARKS
			1 VALVE	2 VALVES			
M-1	-	-	Not tested	152	Turbulent slight upward slope	Moderately turbulent	Preliminary design
M-2A	Center wall shortened 4"-6" dentated end sill added	Reduce cost. Reduce erosion	157	157	Turbulent, high boil directly over sill	Turbulent	Short center wall retained High end sill rejected
M-2B	1'-6" block end sill	Reduce surface boil	156	156	Turbulent	Smoother than 2A	Sill not acceptable—too much erosion
M-3	Tapered fillets on inclined floor	Reduce violence of basin w.s.	Not tested		Mildly turbulent	Smoother than 2B	Fillets retained for future tests
M-4	Slope of inclined floor decreased 60 to 24°	Reduce surface violence	156	156	Sloping upward to end of basin	Moderate turbulence	Design rejected
M-5	Converging walls moved upstream; slope returned to 30°	Reduce surface violence	Not tested		Level & mildly turbulent	Moderate turbulence	Position of converging walls retained for future tests
M-6A	"Spur" type wing walls	Reduce erosion	155	156	Mild turbulence	Moderate turbulence	No improvement—design rejected
M-6B	90° wing wall. 14 feet long	Reduce surface violence and erosion	154	154	Same as 6A	Mildly turbulent	Walling the retained for subsequent tests
M-7	Low center wall extending to end of basin	Reduced sand returned to basin	154	Not tested	Similar to previous design	Similar to previous designs	More sand carried into basin than with no wall— not acceptable
M-8	Basin 14 feet longer	Reduce surface violence	157	156	Same as 6A	Less turbulent than previous designs	Recommended basin length
M-8C	3' block end sill	Reduce boil	157	157	Good	Better than previous designs	Recommended design

## PART III

### The United States Outlet Works Model

#### Description and Operation of the Model

The preliminary design of the United States outlet works, Figure 3, was similar to that used on the Mexican side, but with some dimensions reduced to compensate for the 50 percent less discharge. Two 72-inch hollow-jet valves were used in the United States outlets. These valves discharged up to a total of 2,920 cfs into the stilling basin, over an inclined floor and onto a horizontal apron 60 feet long. Downstream from the basin were two 40° wing walls, one separating the outlet discharge from the tailrace, and the other intersecting the side of the downstream trapezoidal channel. The horizontal apron of the stilling basin was at elevation 160, or 4 feet higher than the Mexican basin. The maximum total head at the valves was 81.5 feet with both valves fully open.

To test and develop the outlet works design, a 1:24 model was built in the Denver laboratory. The model scale was selected to utilize available 3-inch hollow-jet valves to represent the 72-inch valves of the prototype. A "mirror image" model was built, one that was reversed from left to right, to utilize equipment existing in the laboratory.

Included in the model (Figure 65) were two hollow-jet valves with approach conduits, a head box, a stilling basin, approximately 250 feet of downstream channel and a tail box equipped with an adjustable gate. No provisions were made to duplicate the powerhouse tailrace, as tests on the Mexican outlet works had shown that hydraulic conditions were no worse with the powerhouse discharging.

The head box and tail box were constructed of wood lined with sheet metal and one side of the stilling basin contained a glass panel to study the underwater action. Water was pumped from the laboratory sump to the head box used to represent the prototype reservoir. Discharges were measured by a Venturi meter placed in the supply line. The tail box contained graded sand, similar to that used and described for the Mexican outlet works, Figure 66. Erosion of this sand during operation of the model provided one criterion for judging the effectiveness of various proposed designs.

#### Discussion of Tests

The purpose of these tests was to make the United States outlet works as nearly like the Mexican outlet works as possible, both as to hydraulic performance and as to the general arrangement of the outlet works structure. Since the maximum discharge through the United States outlets was only two-thirds that of the Mexican outlets (2,920 cfs as compared to 4,570 cfs) and since the width of both stilling basins was

the same--because they were both contained in powerhouse structures of identical design--the major modifications in basin dimensions were made in the depth and length of the structures. The United States outlet works was, therefore, tested to determine the amount each dimension could be reduced to provide hydraulic performance similar to that obtained for the Mexican basin.

Stilling basin apron elevation tests. Tests of the preliminary design (Figure 67) of the Mexican outlet works indicated that the water surface in the basin was rough, there was considerable wave action downstream from the basin, Figure 68A, and erosion occurred at the end of the stilling basin apron, Figure 68B. The stilling action was less effective in the United States basin than in the Mexican basin, Figure 69A. To investigate this condition, sweep-out tests were made and the results compared with similar test results obtained and described for the Mexican basin. When the sweep-out margin for the United States outlets was compared with that of the Mexican outlets, it was found that there was approximately 1.5 feet less margin for the United States basin. This was responsible for the more violent action in the United States basin. The elevation of the basin floor, consequently, was lowered 1.5 feet and further sweep-out tests were made. The basin action, Figure 69B, was then found to be comparable to the action in the Mexican basin and the sweep-out margin was found to be 5 feet. This lower apron elevation was used for all subsequent tests.

Stilling basin tests. In determining the modifications to be tested in the model, information obtained from the previously tested Mexican outlet works was used. Design A-2 included tapered bottom fillets, dentated end sill, and converging walls. Because of the arrangement of the structure, however, a 40° wing wall was located to the left of the basin, Figure 70. The performance of this design was satisfactory in most respects; the water surface, both in and below the basin, was relatively smooth, Figure 71A, and the erosion pattern in general was satisfactory. With one valve discharging 1,600 cfs, the maximum scour was to elevation 159, or 0.5 foot above the basin floor, Figure 71B. However, considerable sand was deposited on the floor of the basin when one valve was operating. It was believed that this deposition was due to an eddy below the nonoperating valve which was intensified by the 40° wing wall to the left of the basin.

In Design A-3 the 40° wing wall was changed to a 90° wing wall, 14 feet 8 inches long at the end of the left side of the basin, as shown in Figure 70. Tests on this wall arrangement showed that the eddy previously noted was not directed back into the basin, Figure 72A, but tended to stay downstream. As a result the amount of sand returned to the basin was reduced considerably, as shown in Figure 72B.

#### The Recommended Design

The recommended design was similar to Design A-3, but an end sill 3 feet high was installed on the end of the apron. This sill was

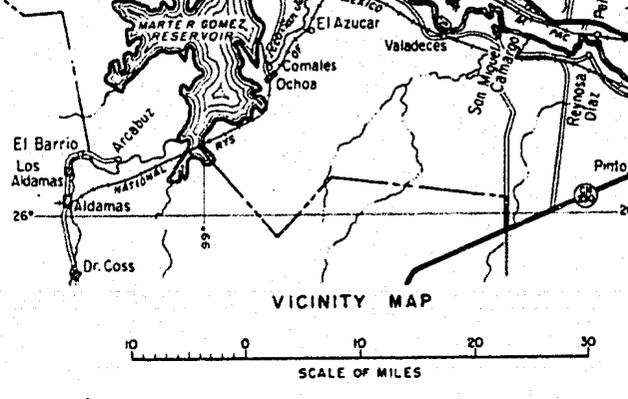
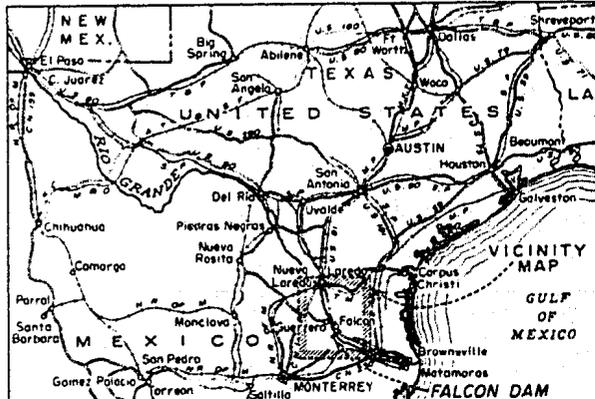
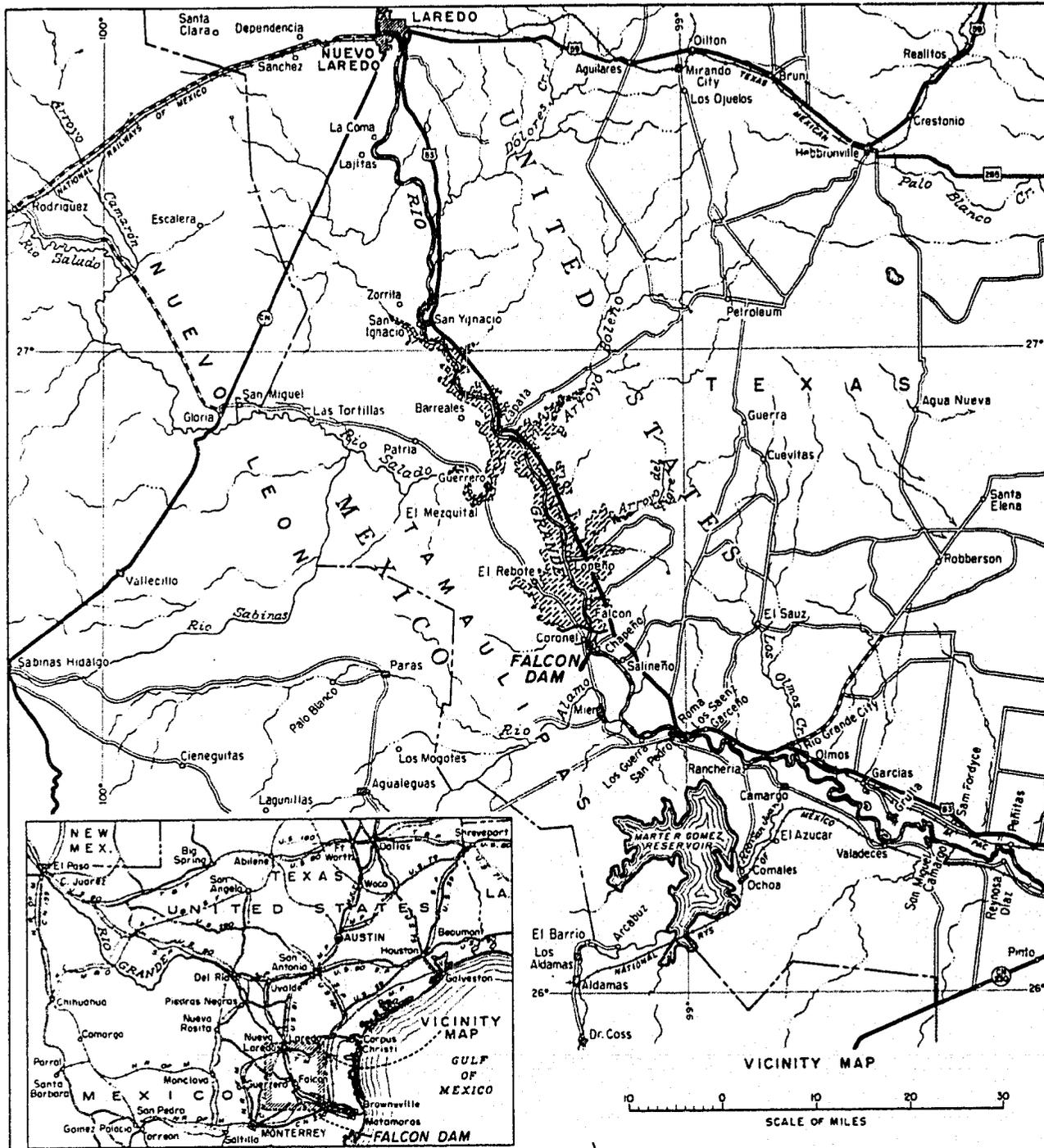
previously found to be satisfactory for the Mexican outlet works and also improved Design A-3. The recommended design is shown in Figure 73. Tests on this design showed satisfactory operation both with one valve, Figures 74A and B and 75A, and with two valves, Figures 75B and 76A and B. With both valves operating the water surface downstream from the basin was relatively quiet. No severe erosion occurred downstream in the basin. After 1 hour the maximum scour was to elevation 158, or 0.5 foot below the basin floor which occurred 5 feet downstream from the end of the apron, Figure 77A.

With one valve discharging 1,600 cfs, operation also appeared satisfactory. Some cross flow occurred in the stilling basin and the waves below the basin, Figure 74A, were somewhat more pronounced than with both valves operating, but these were not considered objectionable. Erosion after a discharge of 1,600 cfs for 1 hour (Figure 77B) resulted in a hole located 5 feet downstream from the end of the apron, on the operating side, to the same depth as the basin floor, 158.5. The amount of sand returned to the basin was not excessive; in fact it was similar in amount to that found in the Mexican outlet works for similar operating conditions.

Wave action in the downstream channel was not considered of sufficient magnitude to cause concern. The direction of wave propagation was primarily downstream for all discharges, with only slight action reflected into the tailrace area downstream from the power plant. The most pronounced wave action occurred with one valve discharging 1,600 cfs, but disturbances did not cross into the tailrace to any great degree, Figure 75A.

Water surface profiles were taken, both transversely and longitudinally for maximum discharge with one and both valves operating, Figures 78 and 79. These profiles show the relatively level water surface in the basin for the most critical operating conditions.

FIGURE 1



**INDEX MAP**

**EXPLANATION**  
(VICINITY MAP ONLY)

- OILED OR PAVED ROADS
- GRAVELED ROADS
- IMPROVED ROADS

MEXICO  
MINISTRY OF HYDRAULIC RESOURCES  
IRRIGATION & RIVER CONTROL

CHECKED BY: [Signature]  
DATE: [Date]

APPROVED: SECRETARY

MEXICO D.F.

UNITED STATES  
DEPARTMENT OF THE INTERIOR  
BUREAU OF RECLAMATION

DRAWN BY: [Signature]  
TRACED, C.W.A. RECOMMENDED BY: [Signature]  
CHECKED BY: [Signature] APPROVED BY: [Signature]

DA-4-1  
DENVER, COLO. - AUG. 2, 1942

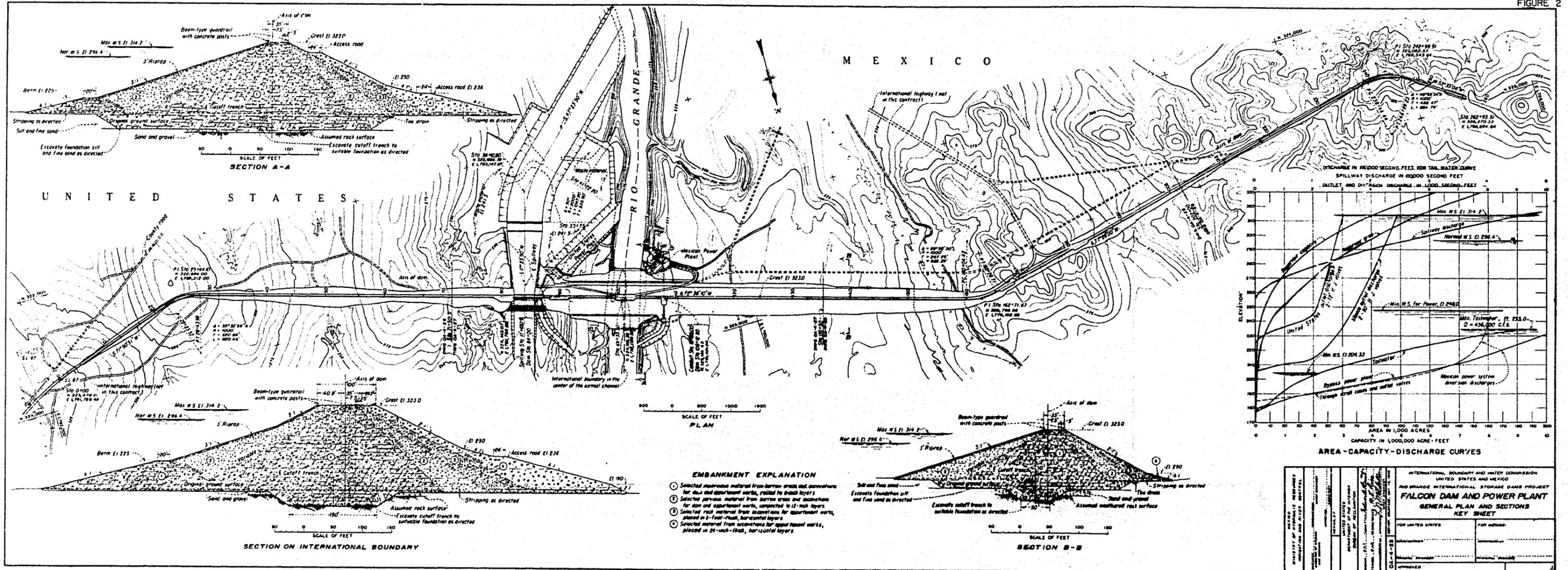
INTERNATIONAL BOUNDARY AND WATER COMMISSION  
UNITED STATES AND MEXICO

**RIO GRANDE INTERNATIONAL STORAGE DAMS PROJECT**

**FALCON DAM AND POWER PLANT**

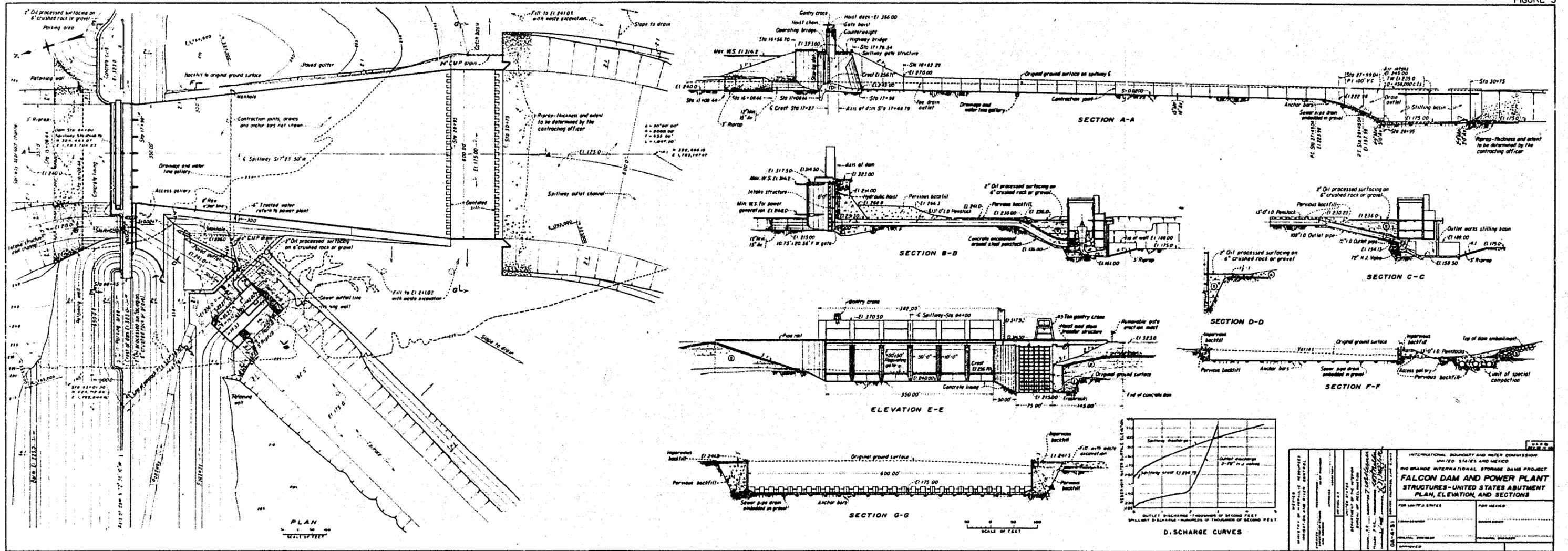
**LOCATION MAP**

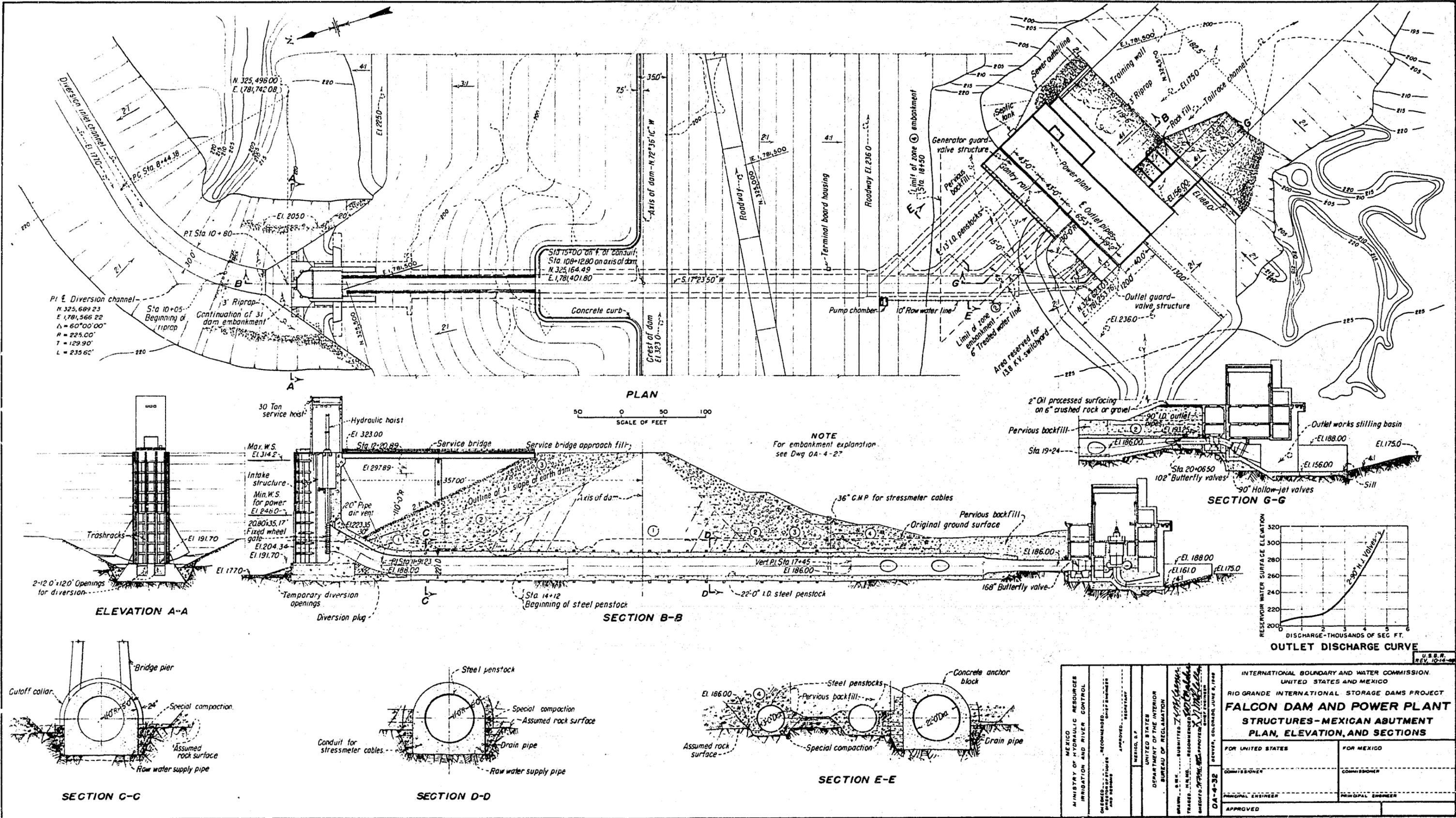
FOR UNITED STATES	FOR MEXICO:
COMMISSIONER	COMMISSIONER
PRINCIPAL ENGINEER	PRINCIPAL ENGINEER
APPROVED	



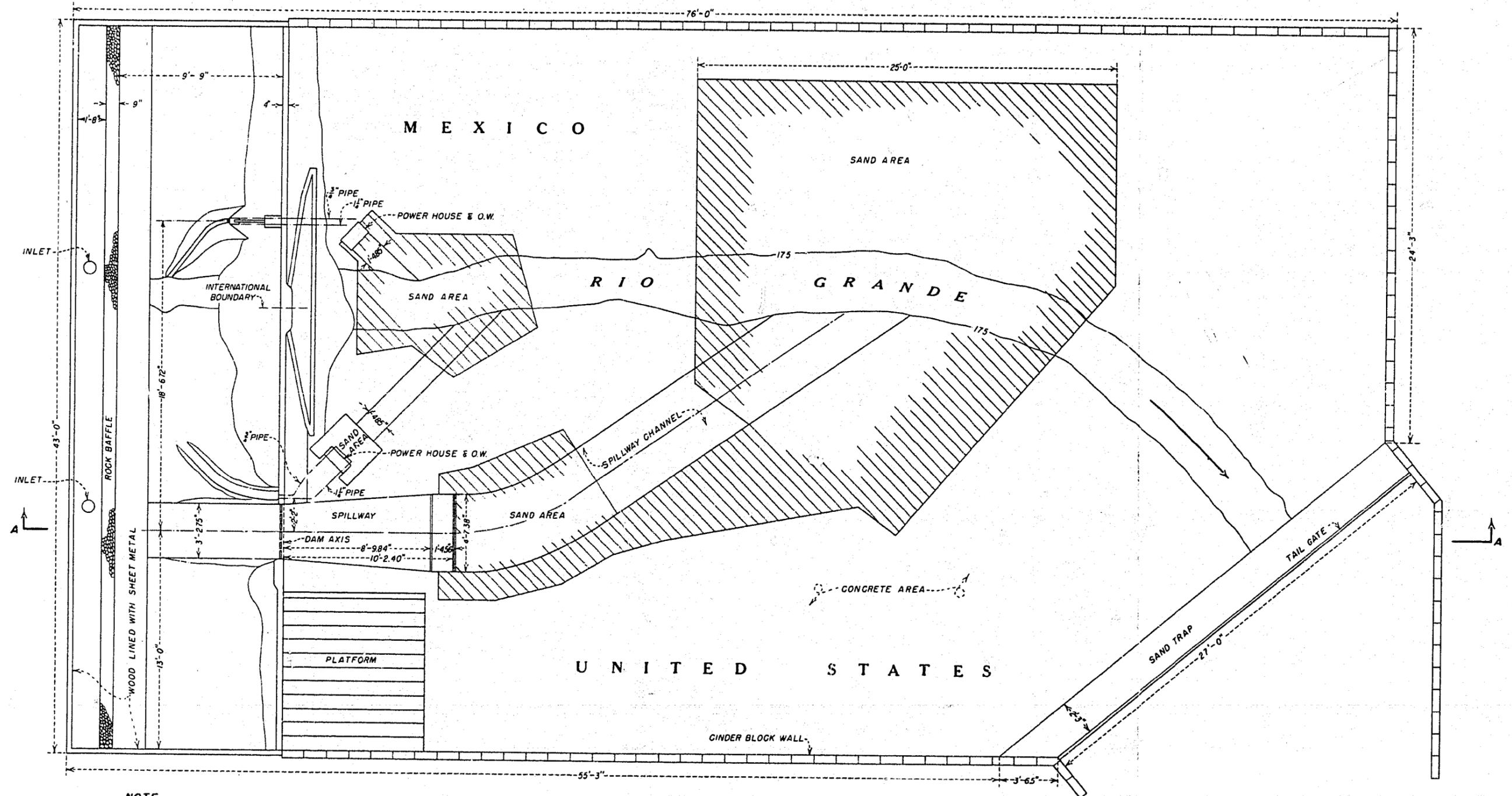
INTERNATIONAL BOUNDARY AND WATER COMMISSION UNITED STATES AND MEXICO	
RIO GRANDE INTERNATIONAL STORAGE DAMS PROJECT	
FALCON DAM AND POWER PLANT	
GENERAL PLAN AND SECTIONS	
KEY SHEET	
FOR UNITED STATES	FOR MEXICO
DESIGNED BY: [Blank]	DESIGNED BY: [Blank]
CHECKED BY: [Blank]	CHECKED BY: [Blank]
APPROVED BY: [Blank]	APPROVED BY: [Blank]

FIGURE 3



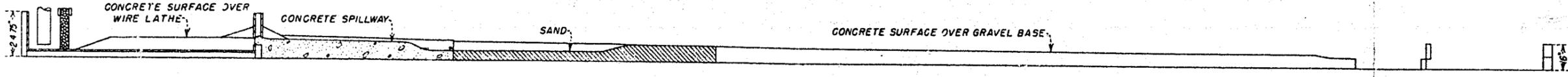


MEXICO MINISTRY OF HYDRAULIC RESOURCES IRRIGATION AND RIVER CONTROL DIRECTOR GENERAL RECOMMENDED BY: [Signature] APPROVED: [Signature]		UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION SUPERVISOR RECOMMENDED BY: [Signature] APPROVED: [Signature]	
INTERNATIONAL BOUNDARY AND WATER COMMISSION UNITED STATES AND MEXICO RIO GRANDE INTERNATIONAL STORAGE DAMS PROJECT <b>FALCON DAM AND POWER PLANT</b> STRUCTURES - MEXICAN ABUTMENT PLAN, ELEVATION, AND SECTIONS		FOR UNITED STATES COMMISSIONER [Signature]	FOR MEXICO COMMISSIONER [Signature]
DENVER, COLORADO, JUNE 9, 1949 D.A.-4-32		PRINCIPAL ENGINEER [Signature]	PRINCIPAL ENGINEER [Signature]
APPROVED			



NOTE  
GATES AND PIERS ON SPILLWAY  
CREST NOT SHOWN

P L A N

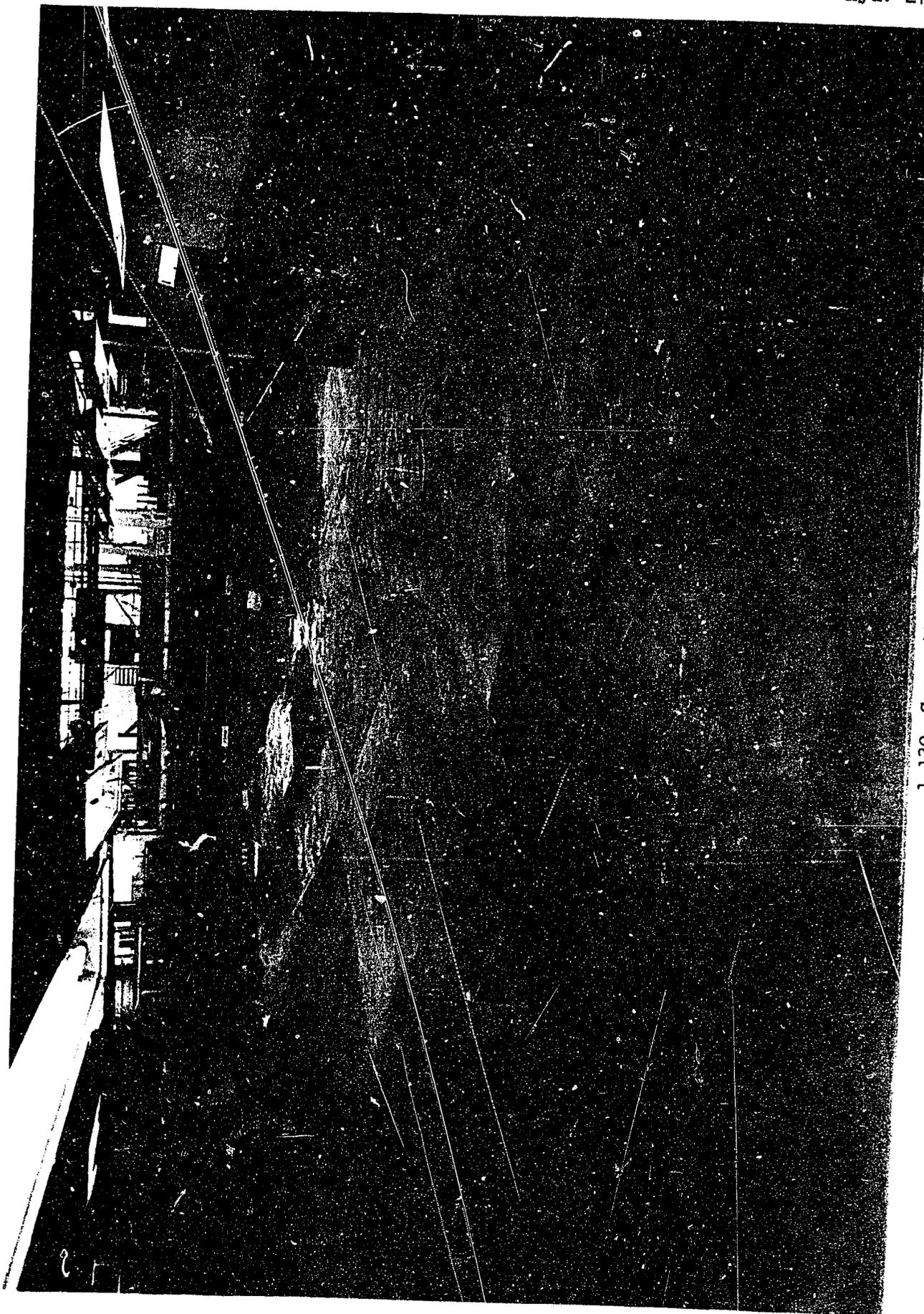


SECTION A-A

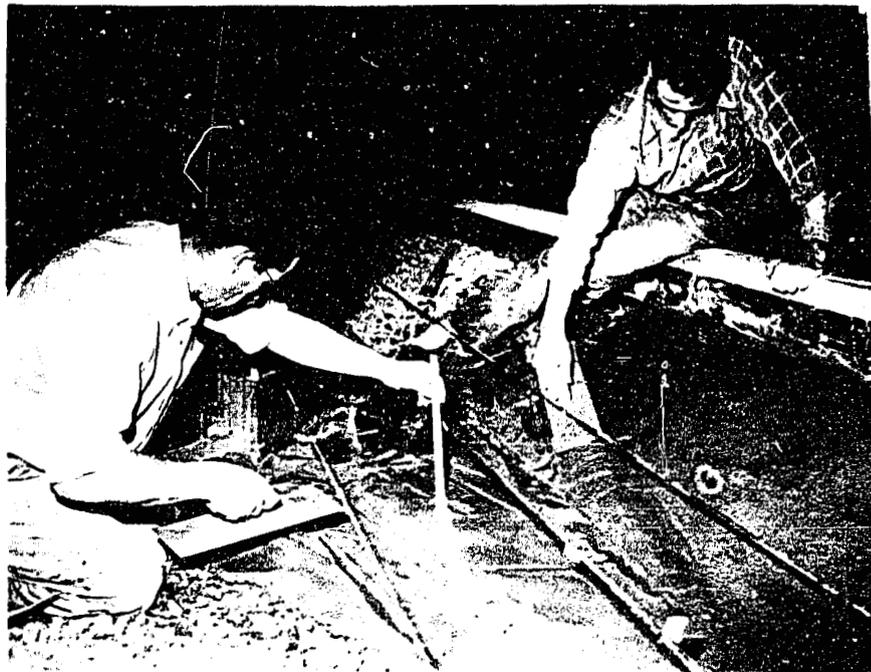
**FALCON DAM**  
HYDRAULIC MODEL STUDIES  
THE PROJECT MODEL  
MODEL ARRANGEMENT  
MODEL SCALE 1:130

369  
1-50

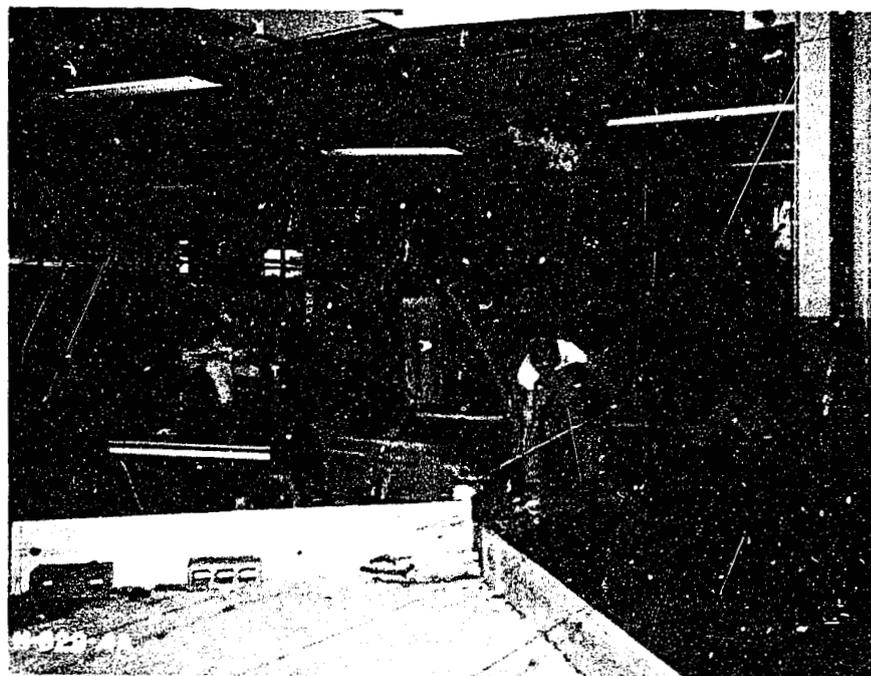
Figure 6  
Hyd. 276



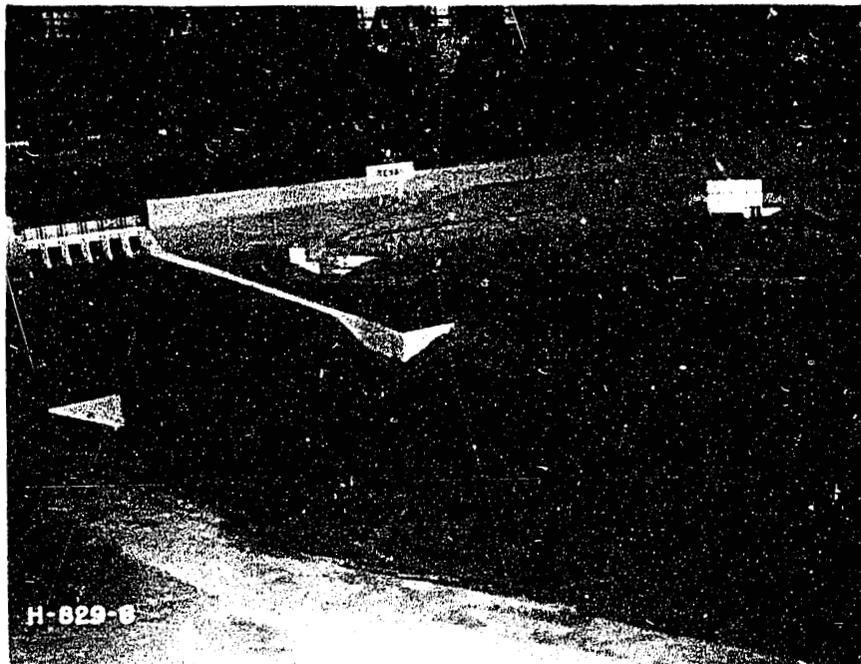
1-130 Composite Model  
FALCON DAM



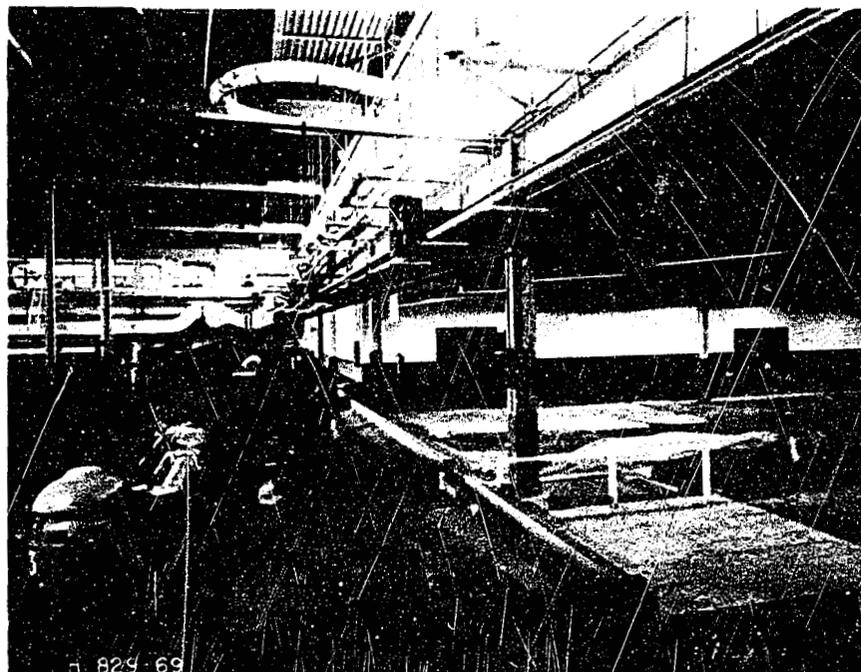
A. Placing topography in concrete. The cords are at fixed elevations and are used as references in forming the topography.



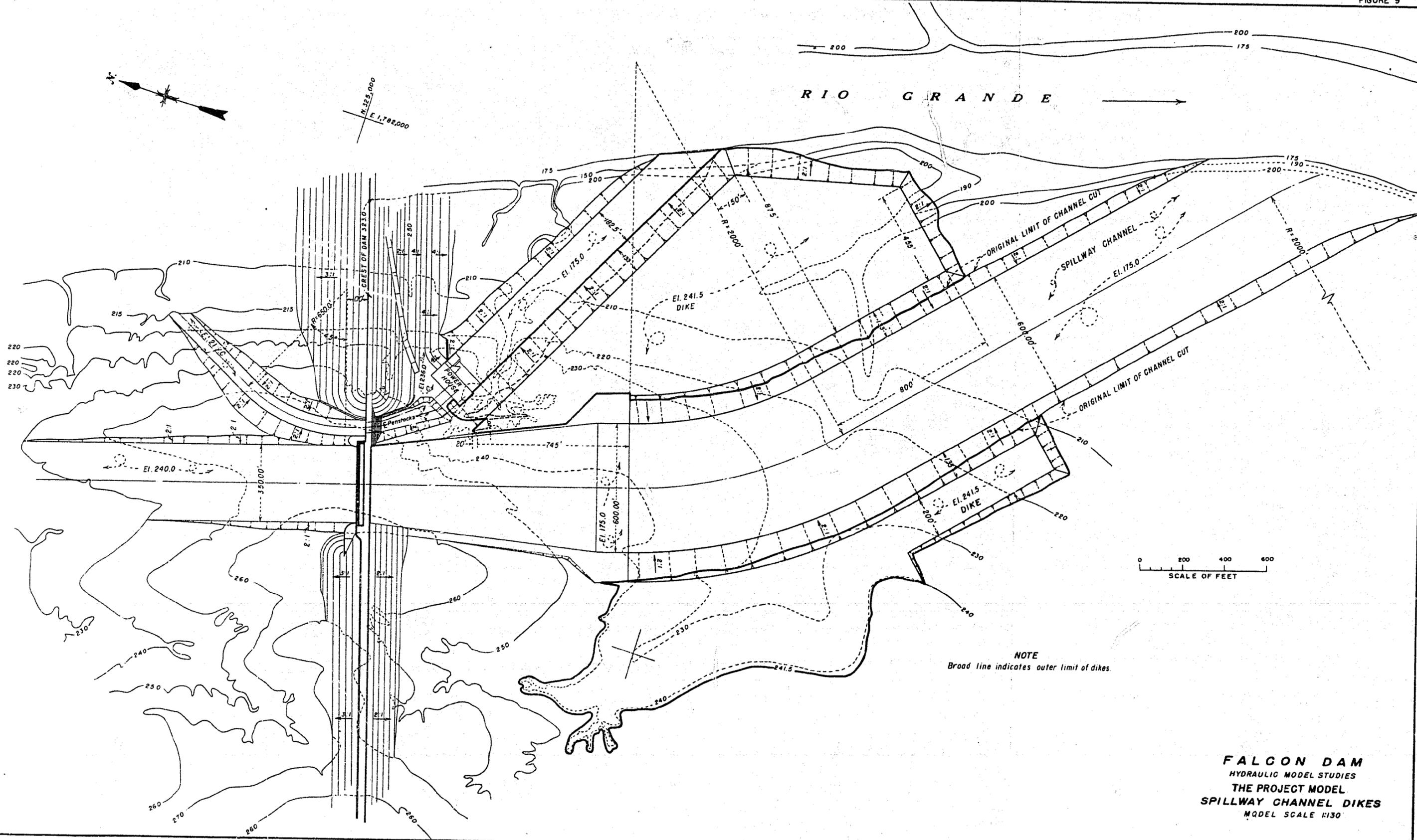
B. Placing topography in concrete. The area in the foreground will be filled with fine sand.



A. The spillway as originally constructed, prior to operation. Note the demarcation between the sand (lighter area) and the concrete.

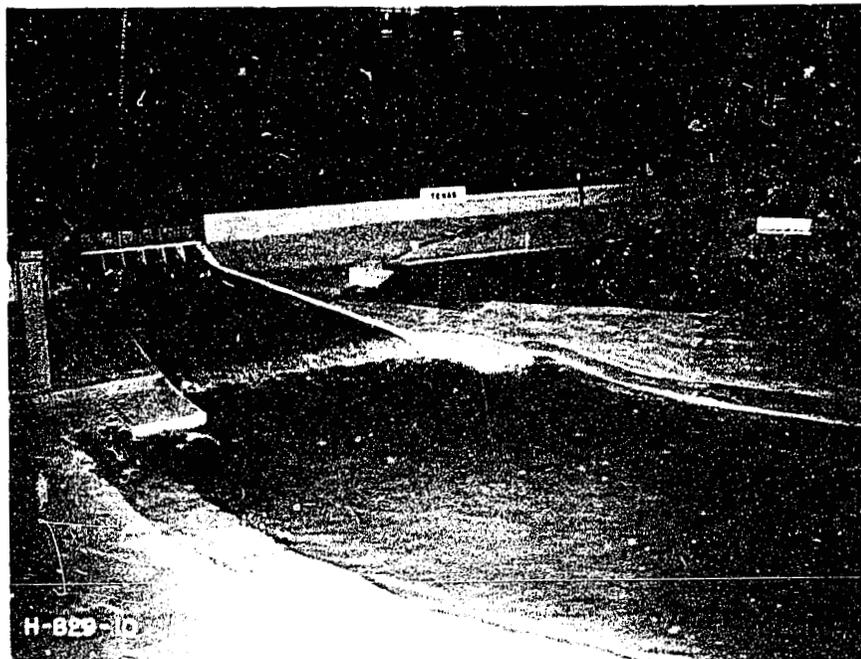


B. The reservoir area. Water is supplied by the pump in the foreground. Note rock baffle at the left, and the gages in the foreground.

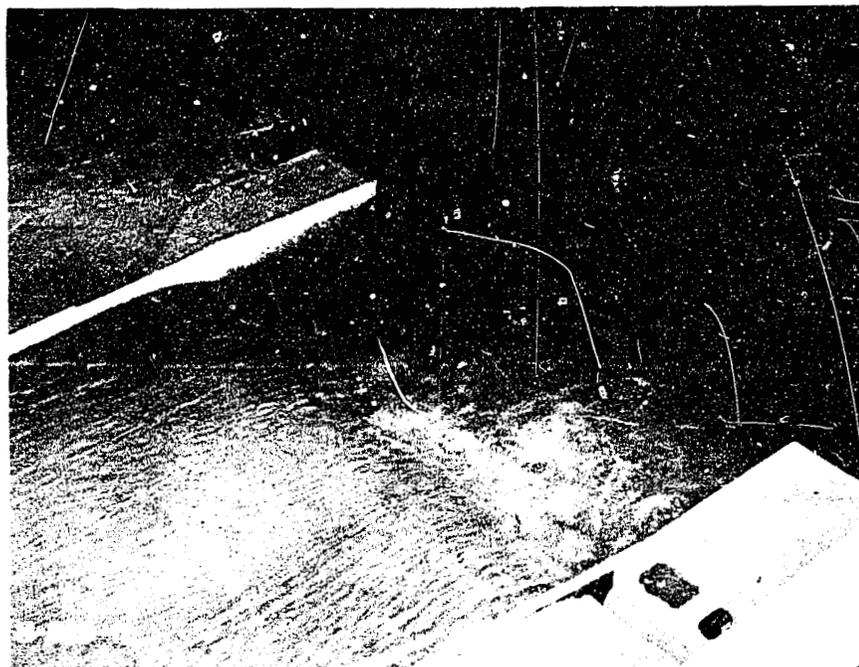


NOTE  
Broad line indicates outer limit of dikes.

**FALCON DAM**  
HYDRAULIC MODEL STUDIES  
THE PROJECT MODEL  
SPILLWAY CHANNEL DIKES  
MODEL SCALE 1/30

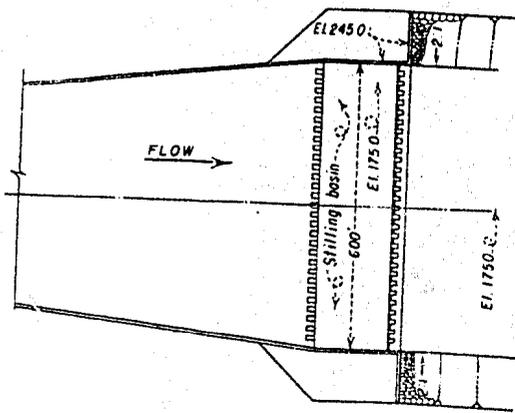


A. The spillway and spillway channel.

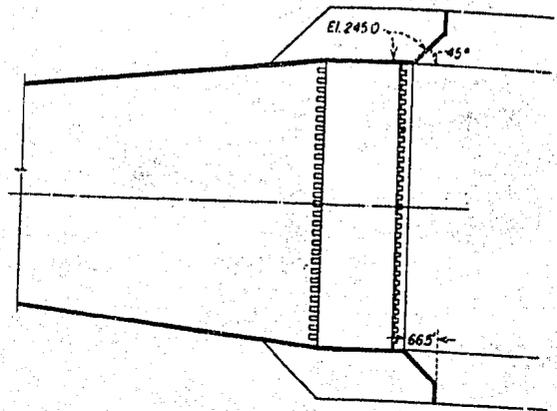


B. The stilling basin.

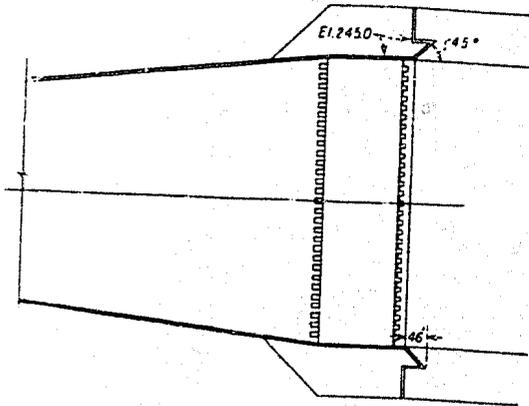
THE SPILLWAY---PRELIMINARY DESIGN. DIS-  
CHARGE OF 456,000 CFS WITH ALL GATES  
FULLY OPEN.



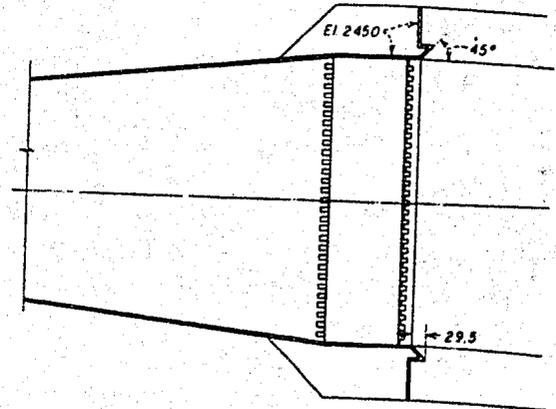
PRELIMINARY DESIGN



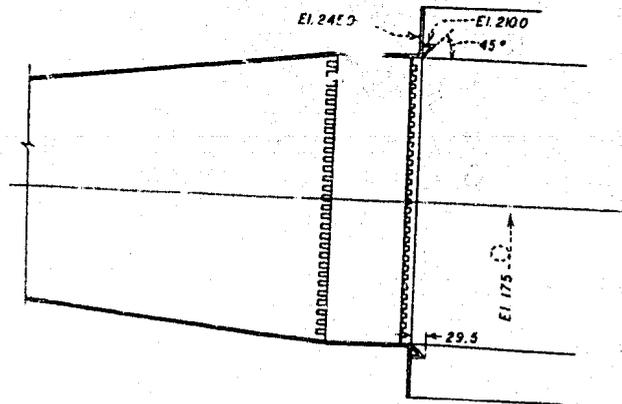
DESIGN S-2



DESIGN S-3

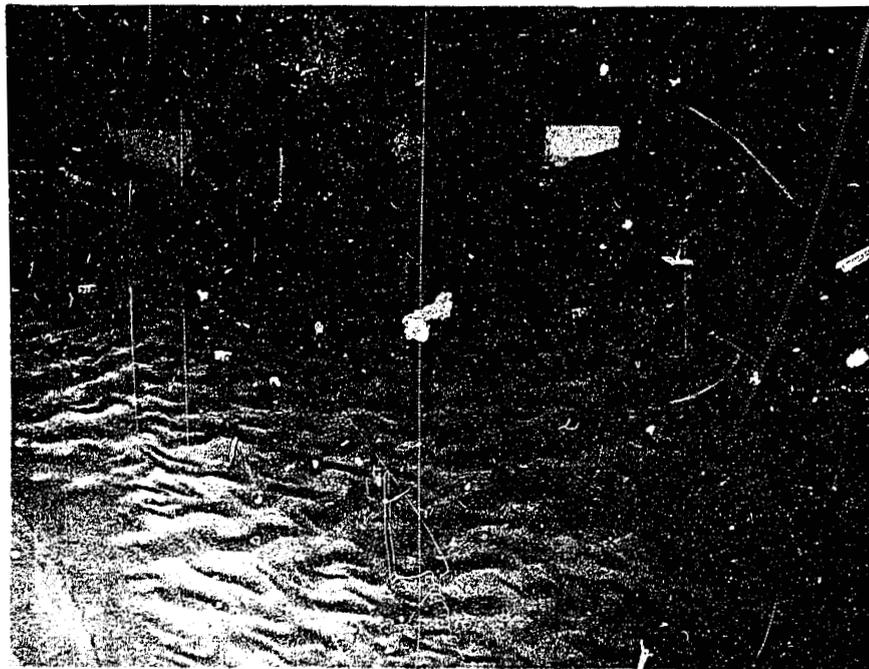


DESIGN S-4

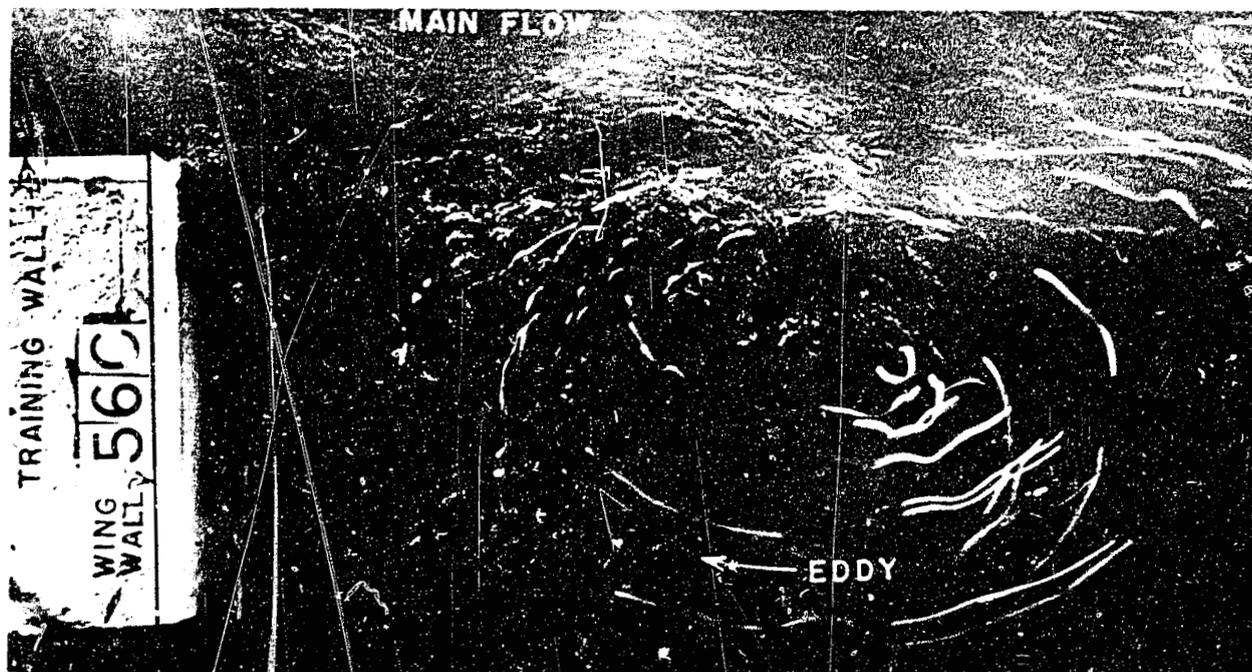


DESIGN S-5

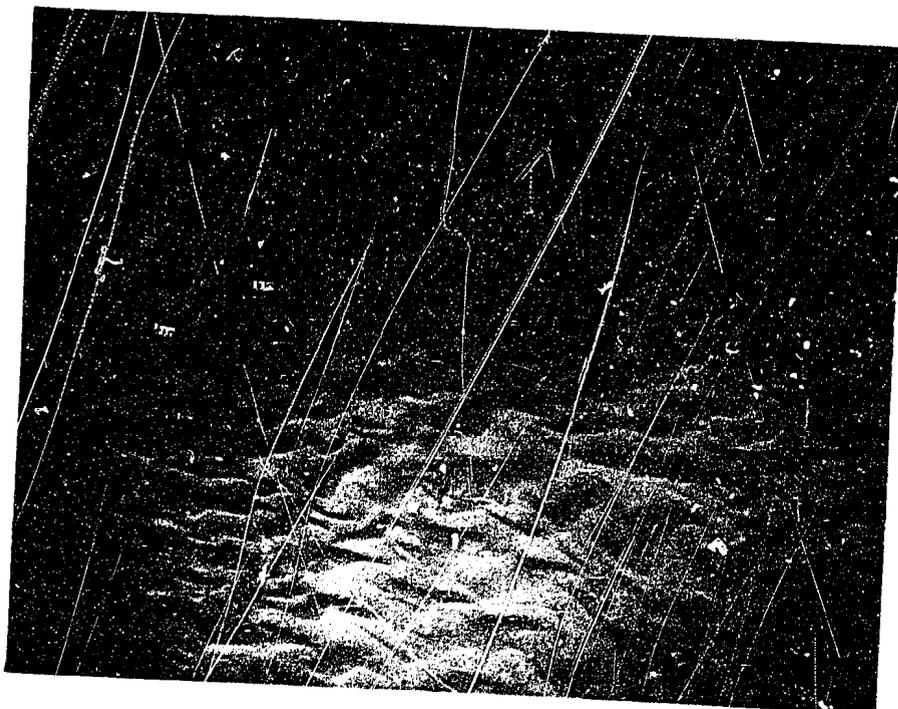
**FALCON DAM**  
 HYDRAULIC MODEL STUDIES  
 SPILLWAY  
 WING WALL DESIGNS  
 MODEL SCALE 1:130



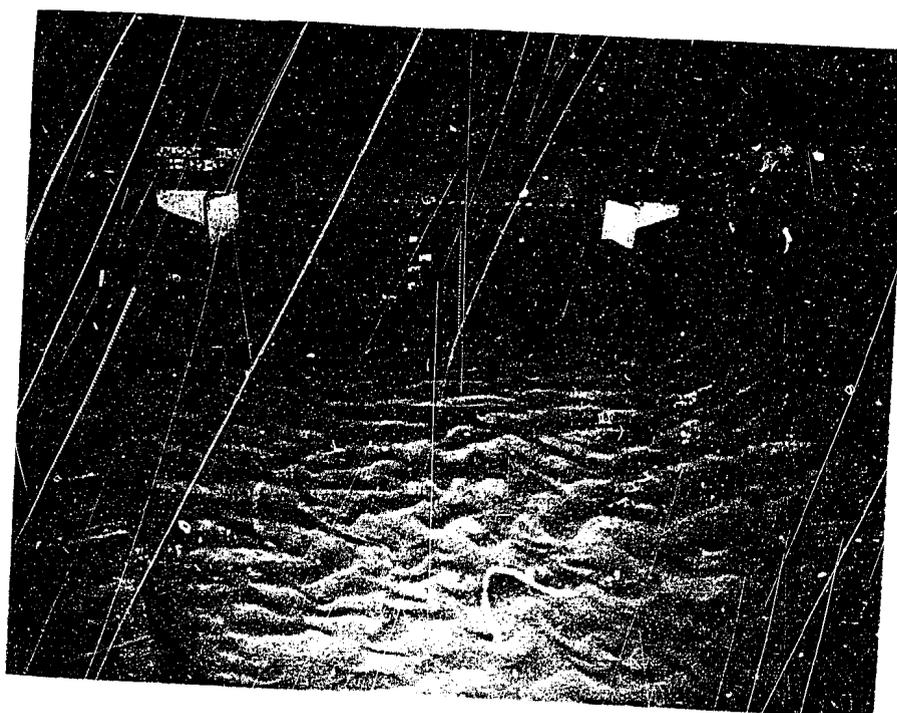
A. Erosion below the stilling basin after a discharge of 456,000 cfs for 2 hours. All gates open. Note the deep hole at the end of the stilling basin.



B. Eddy in area immediately downstream from right end of stilling basin.  $Q = 456,000$  cfs. It is believed that this eddy was the cause of the severe erosion shown above.

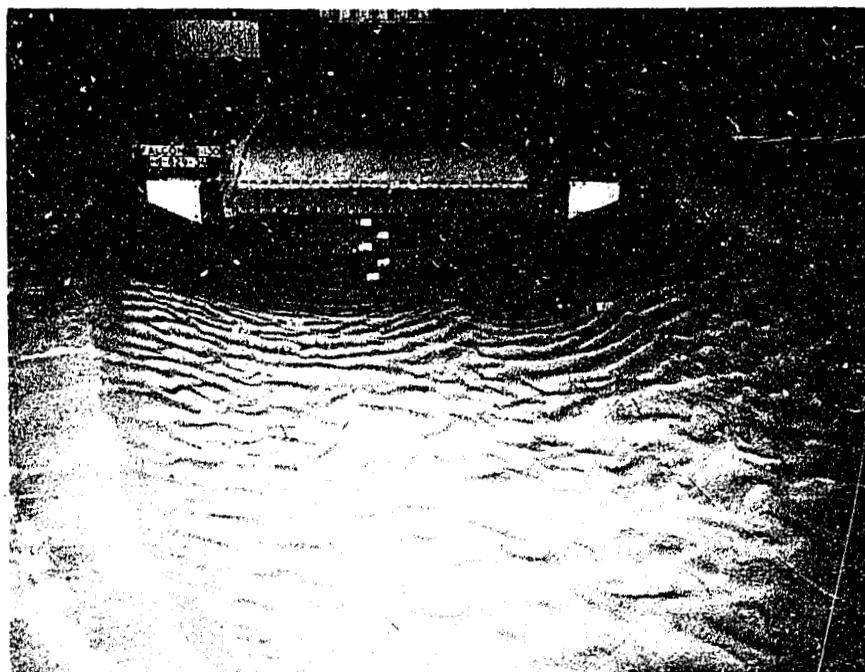


A. Design S-2. Erosion after a discharge of 456,000 cfs for 2 hours. All gates fully opened.

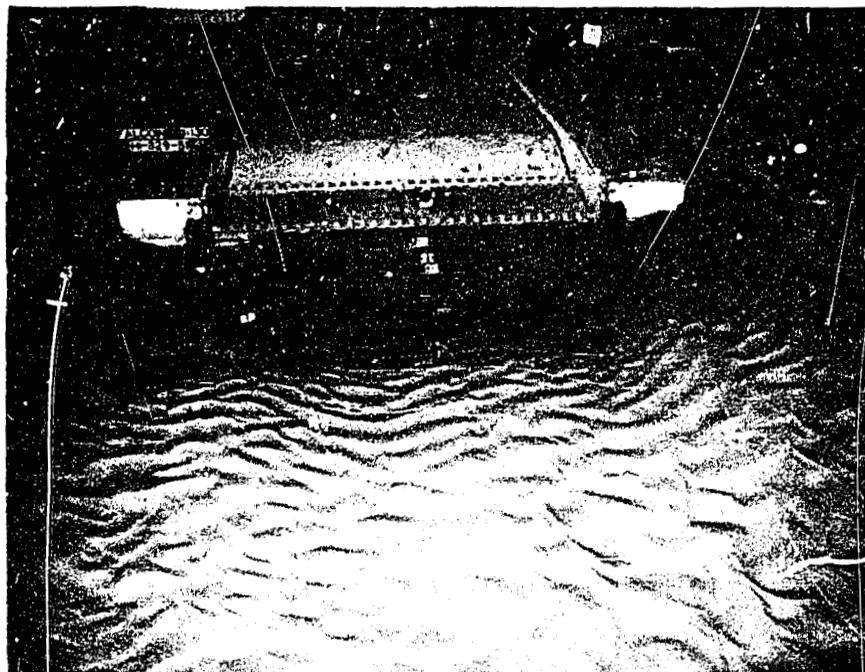


B. Design S-3. Erosion after a discharge of 456,000 cfs for 2 hours. All gates fully opened. Note the reduction in erosion from previous designs.

THE SPILLWAY. EROSION BELOW THE STILLING BASIN.

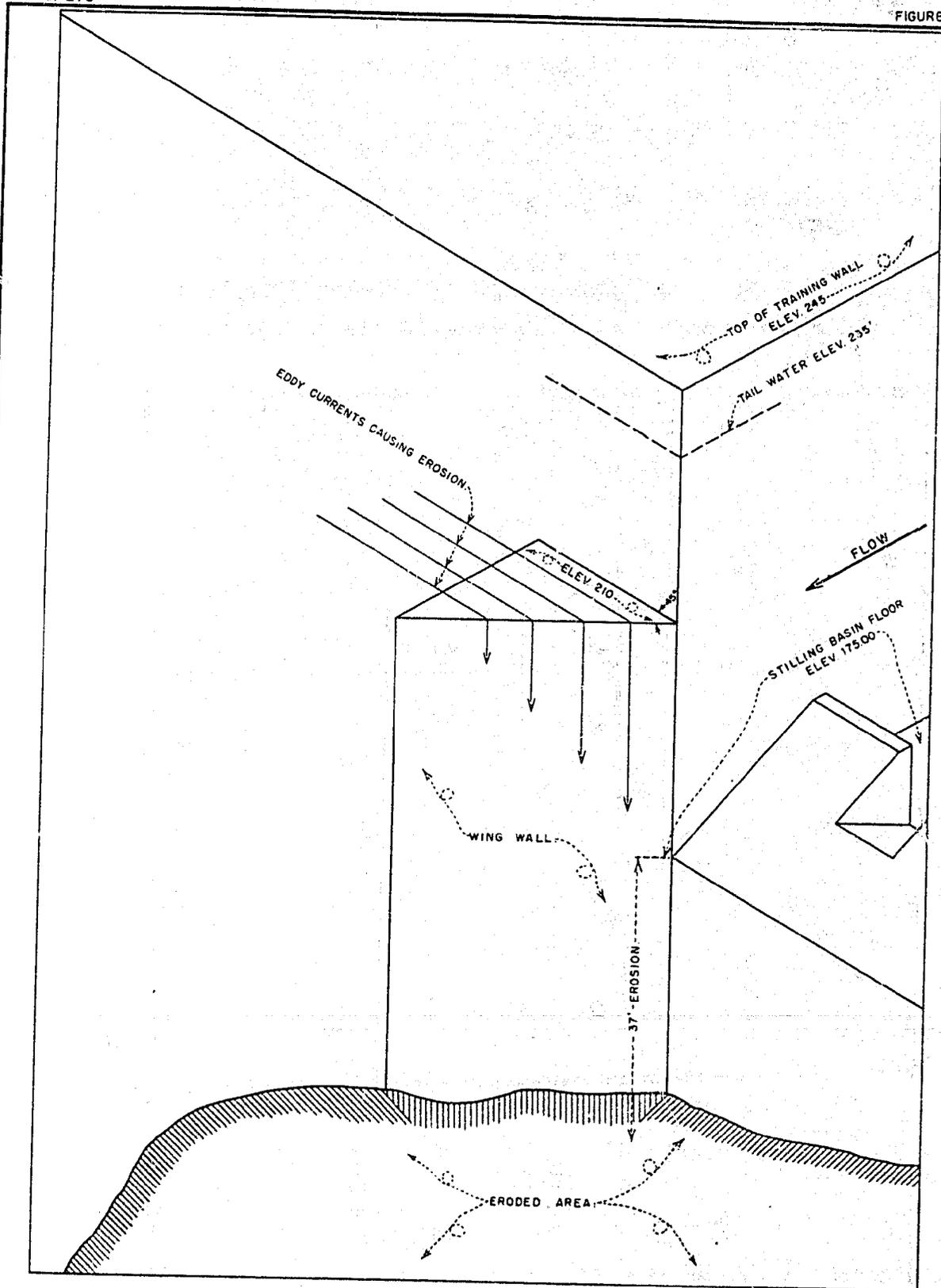


A. Design S-4. Erosion after a discharge of 456,000 cfs for 2 hours. All gates fully opened. Erosion at apron corners is negligible.



B. Design S-5. Erosion after a discharge of 456,000 cfs for 2 hours. All gates fully opened. Note the similarity of erosion with this design to that obtained with the preliminary design.

THE SPILLWAY. EROSION BELOW THE STILLING BASIN.



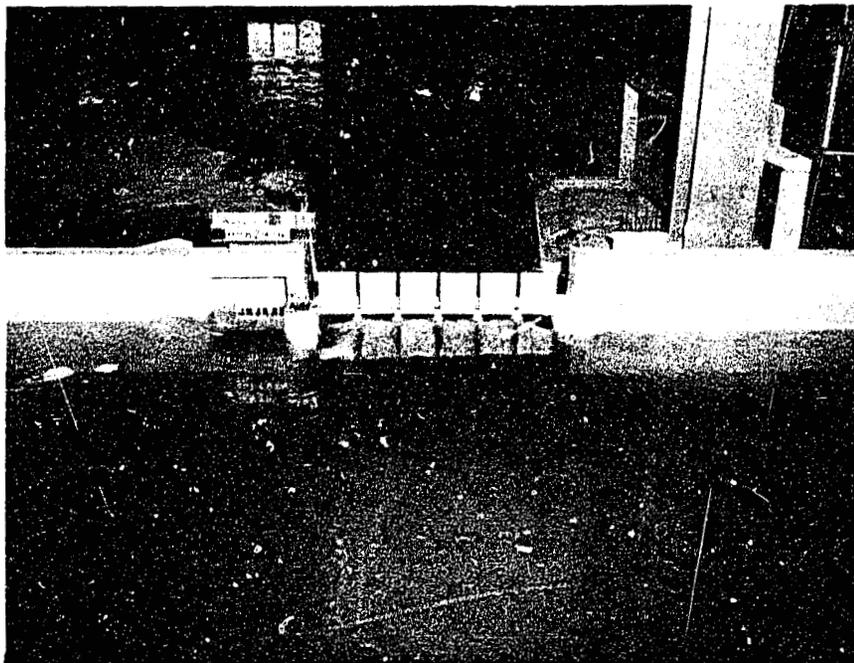
FALCON DAM  
HYDRAULIC MODEL STUDIES  
SPILLWAY  
EROSION CURRENTS WITH  
LOW WING WALL  
MODEL SCALE 1/30

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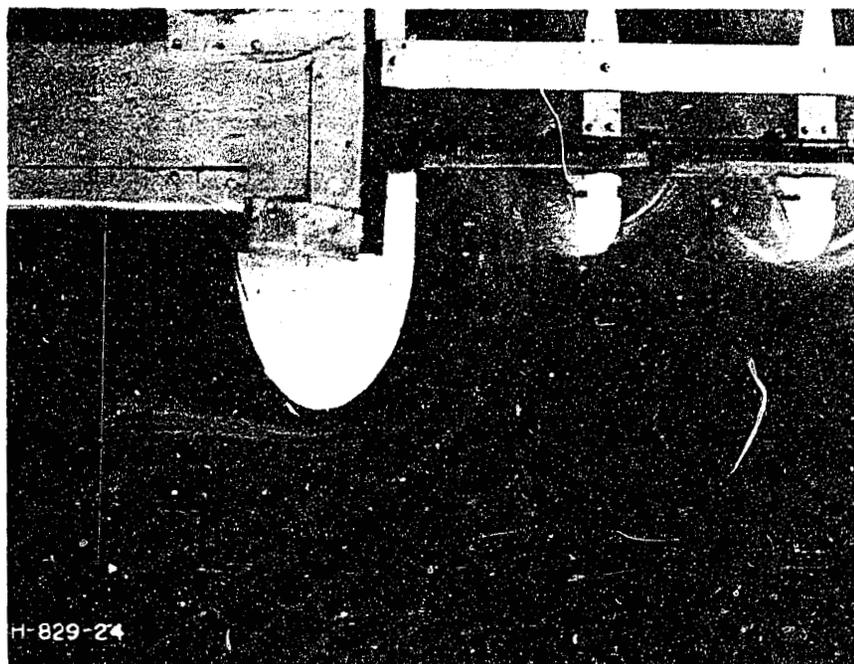


Results of a second erosion test with Design S-5. Erosion after a discharge of 456,000 cfs for two hours. Compare with Figure 14B.

THE SPILLWAY. DESIGN S-5. EROSION BELOW THE SPILLWAY.

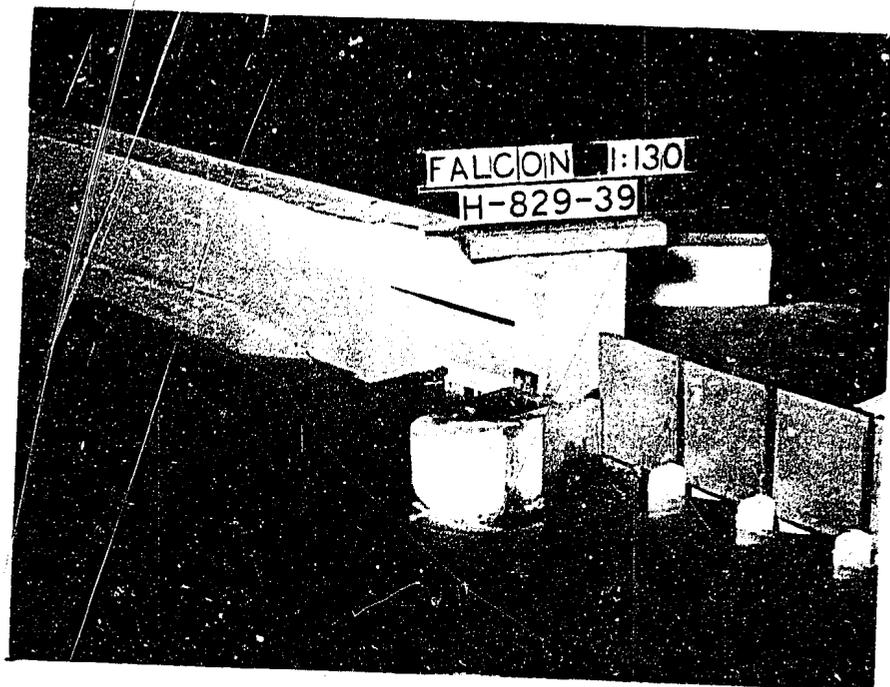


A. Drawdown at both piers.

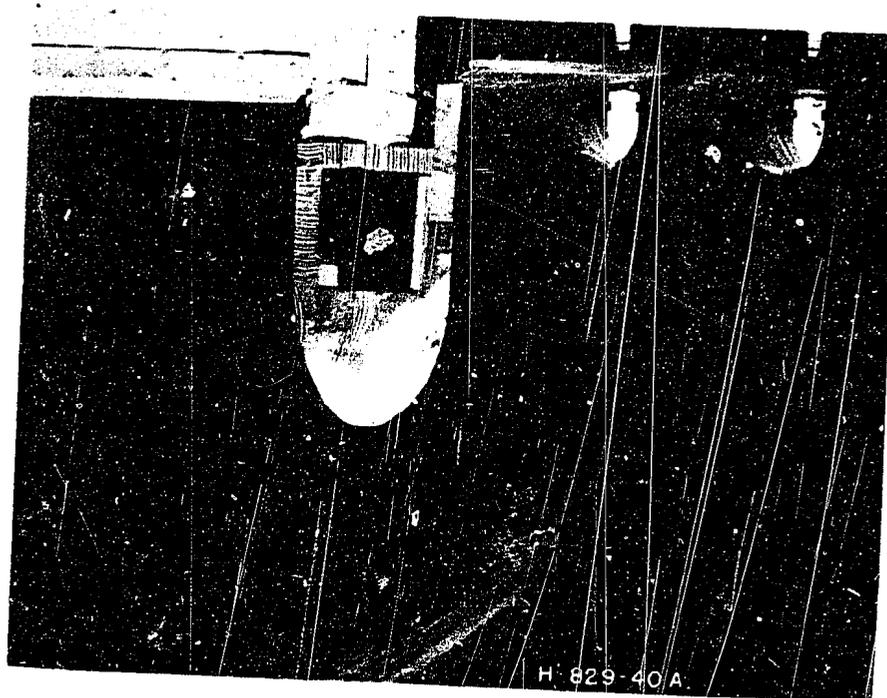


B. Currents about the left approach pier. Preliminary design. Note how the flow tends to leave the face of the pier next to the spillway.

THE SPILLWAY. CONTRACTION AT THE APPROACH.  
456,000 CFS BEING DISCHARGED.

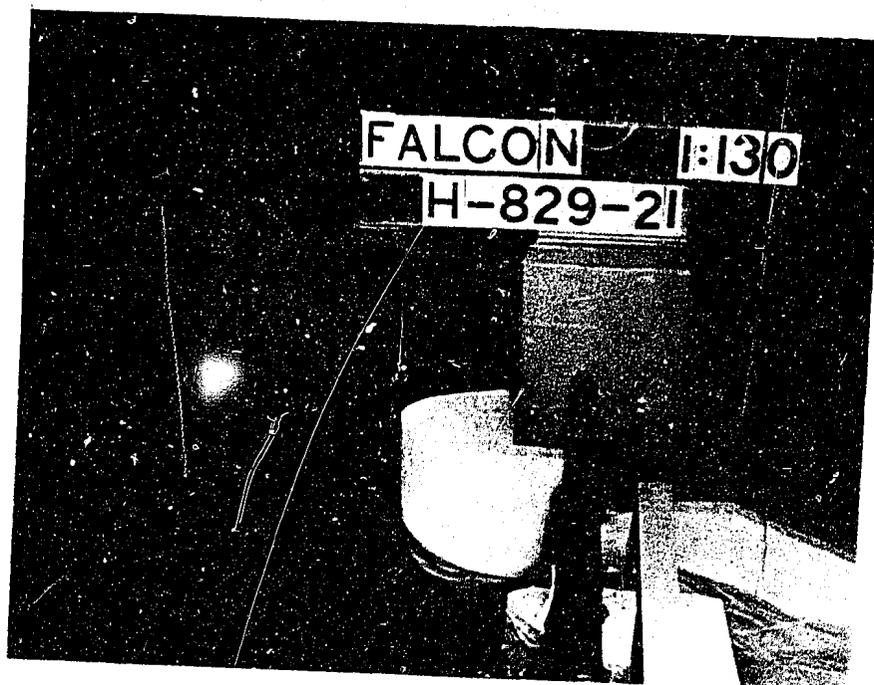


A. Excessive contraction causes a depressed water surface or drawdown along the pier. Note that the elevation of the water surface dips, then rises again before the crest is reached.

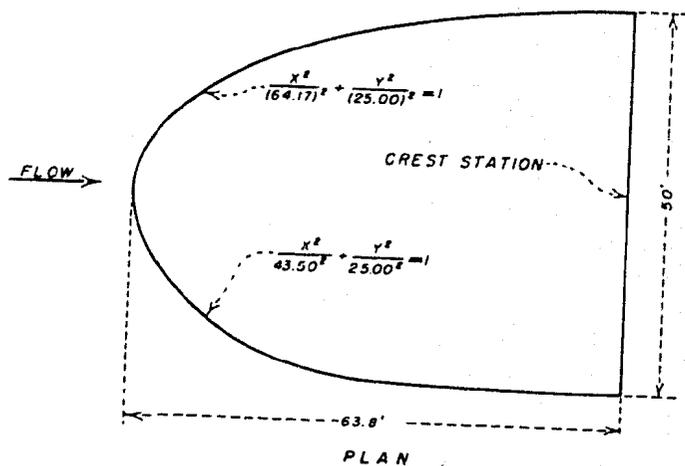


B. Currents about the pier. Note the eddy on the left, in front of the powerhouse intake. The powerhouse is not discharging.

THE SPILLWAY. DESIGN 8-6--LEFT APPROACH PIER 100 FEET LONG.  
456,000 CFS BEING DISCHARGED.

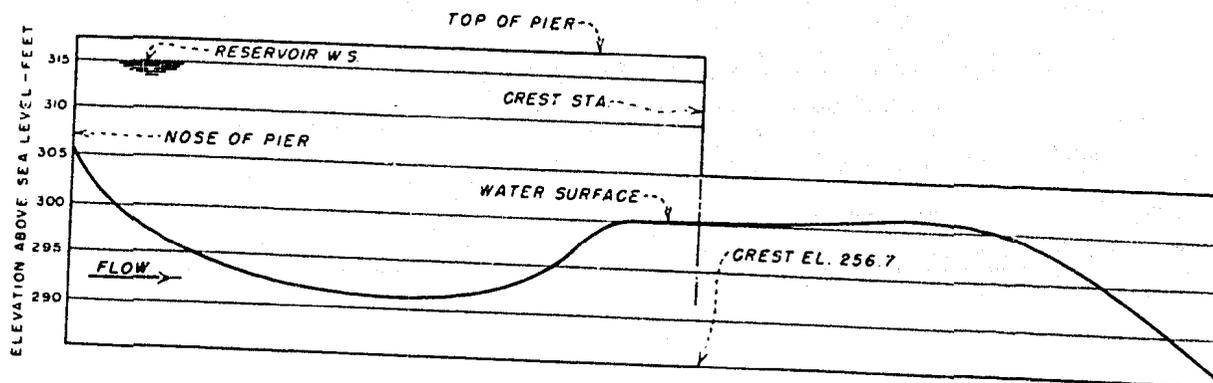


DRAWDOWN DURING OPERATION - Q = 456,000 C.F.S.

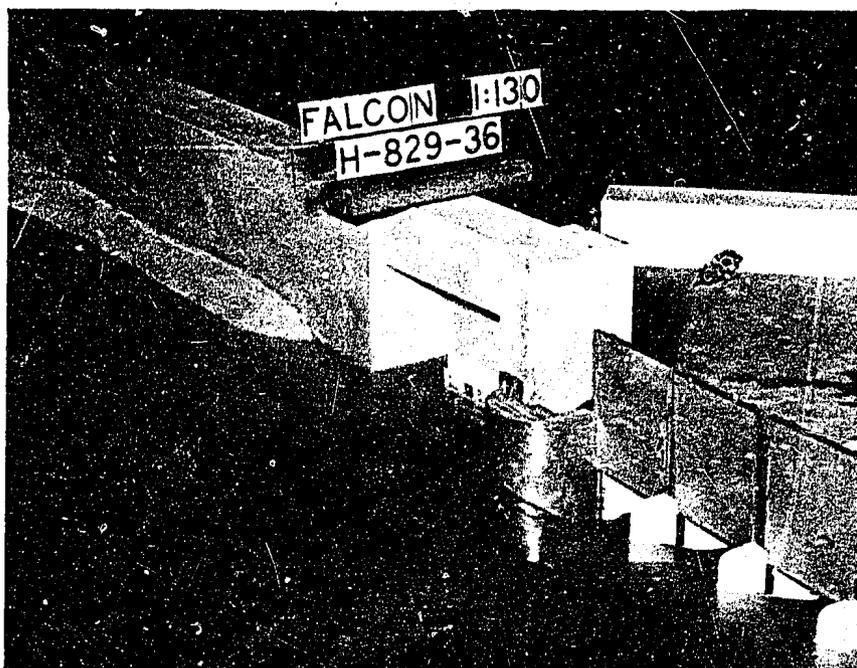


COEFFICIENT OF DISCHARGE = 336

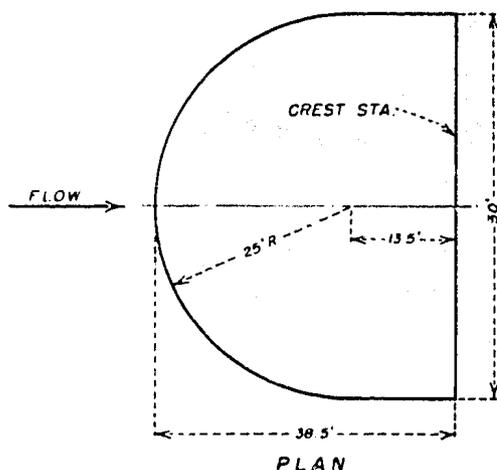
PLAN AND PERFORMANCE DATA



PROFILE OF DRAWDOWN-SPILLWAY SIDE - Q = 456,000 C.F.S.  
 LEFT APPROACH PIER-PIER DESIGN "A"

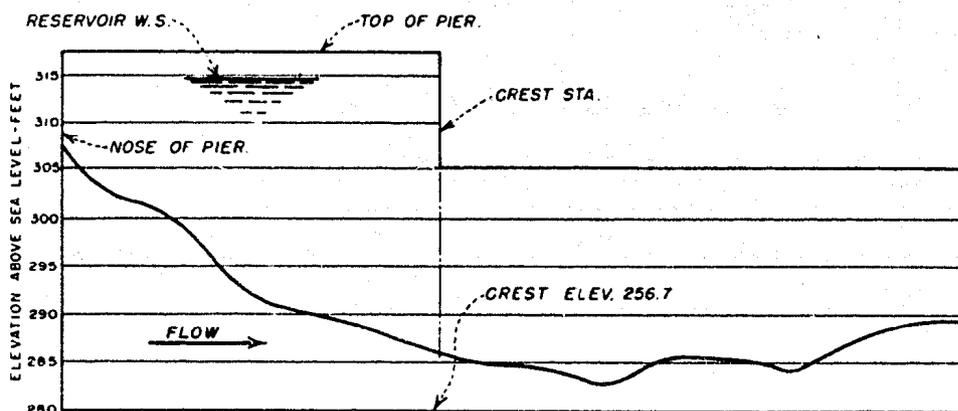


DRAWDOWN DURING OPERATION -  $Q = 456,000$  C.F.S.



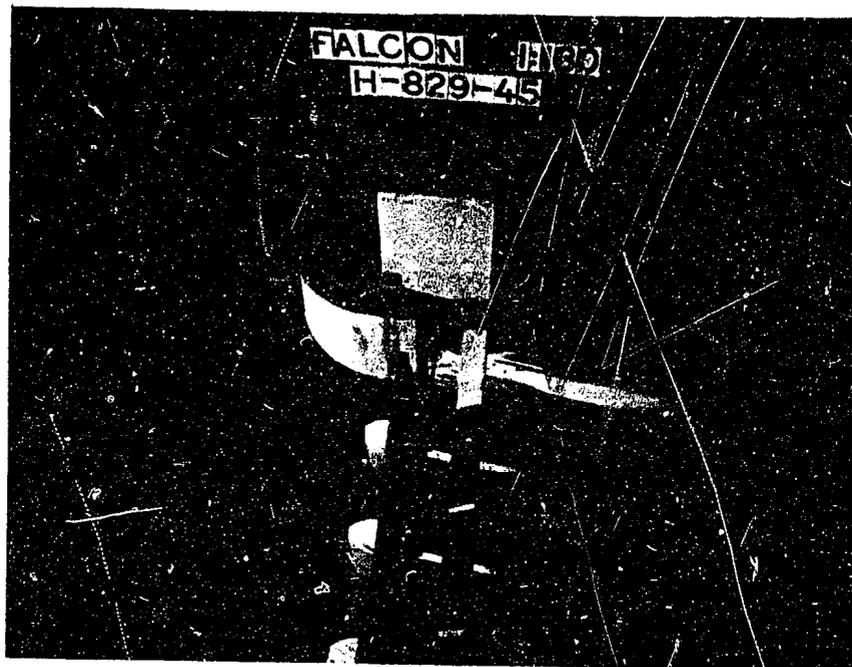
COEFFICIENT OF DISCHARGE = 3.34

PLAN AND PERFORMANCE DATA

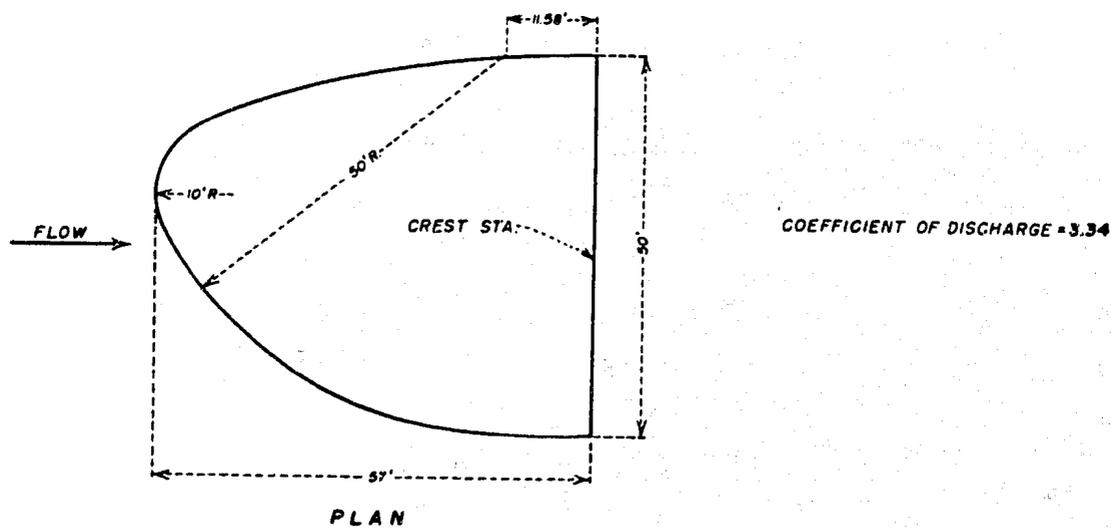


LEFT APPROACH PIER - PIER DESIGN "B"

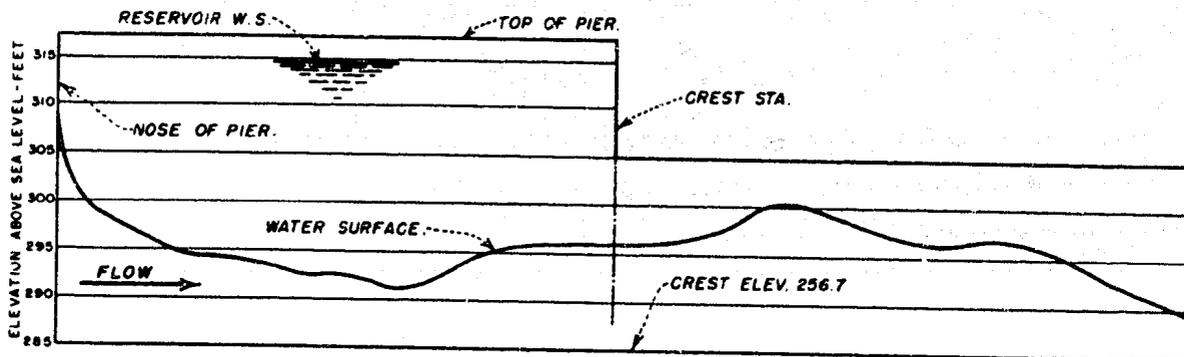
369



DRAWDOWN DURING OPERATION -  $Q = 456,000$  C.F.S.



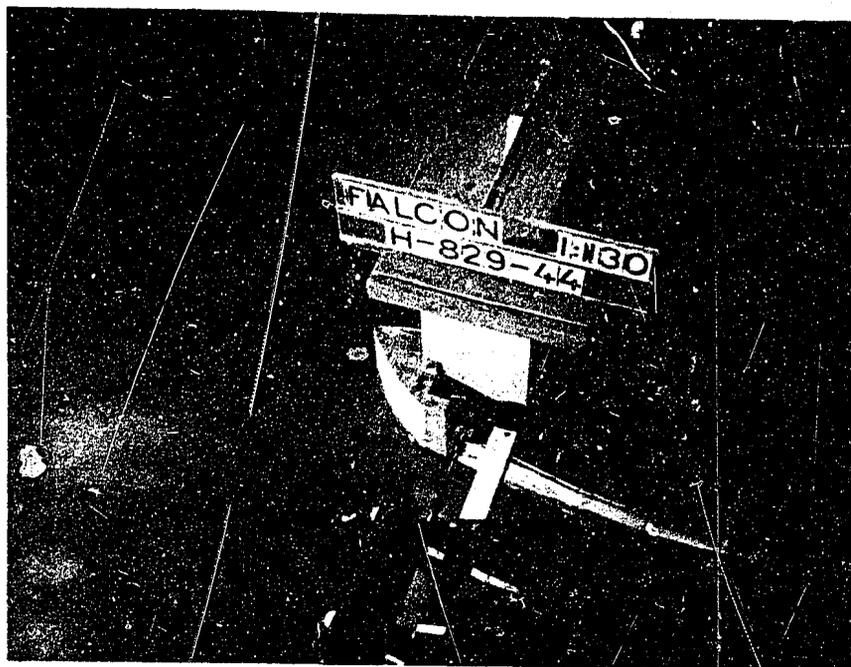
PLAN AND PERFORMANCE DATA



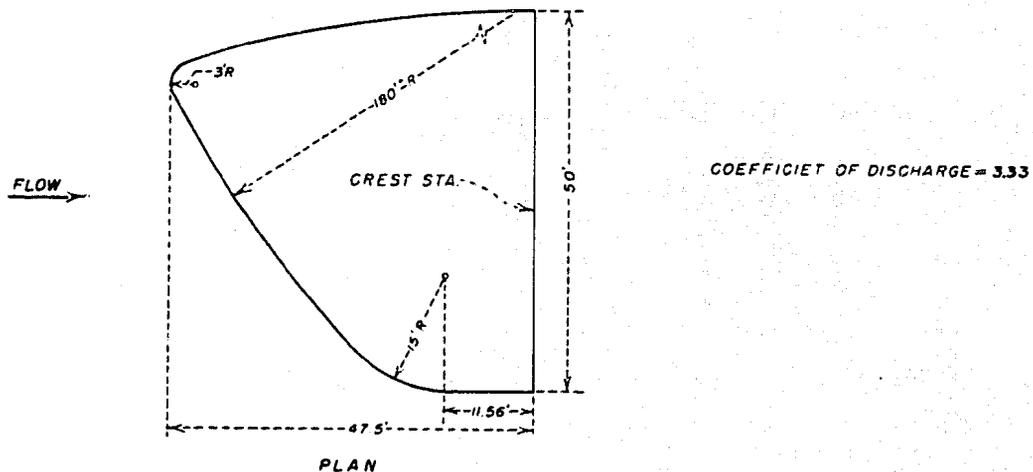
PROFILE OF DRAWDOWN - SPILLWAY SIDE  $Q = 456,000$  C.F.S.

LEFT APPROACH PIER - PIER DESIGN "C"

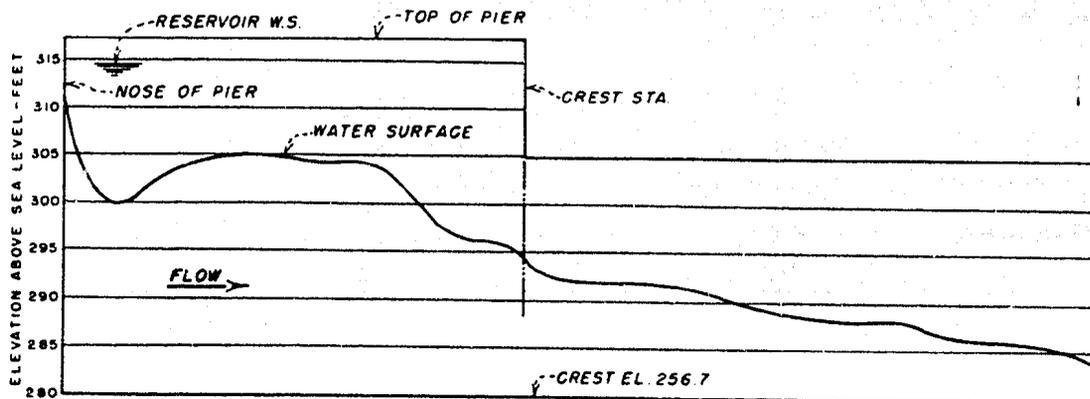
369



DRAWDOWN DURING OPERATION -  $Q = 456,000$  C.F.S.



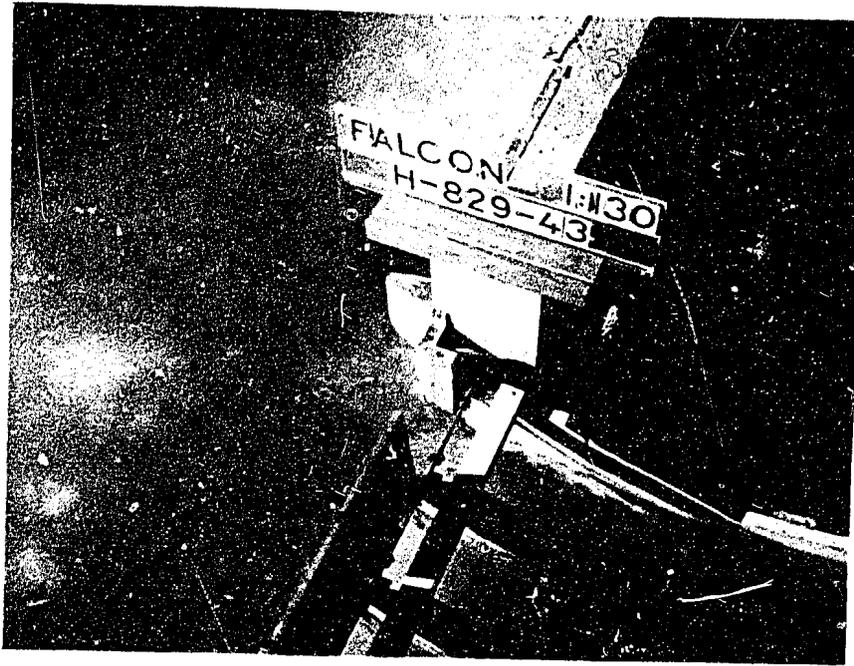
PLAN AND PERFORMANCE DATA



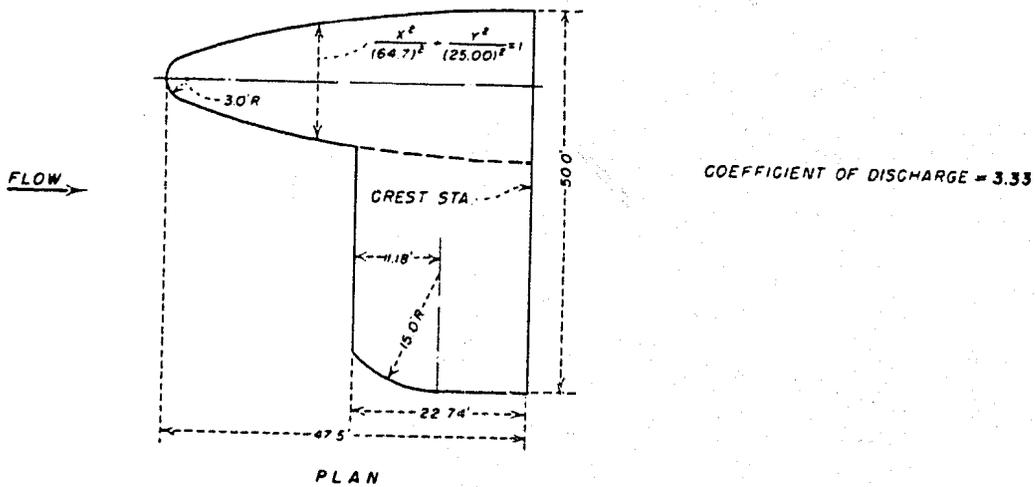
PROFILE OF DRAWDOWN - SPILLWAY SIDE  $Q = 456,000$  C.F.S.

LEFT APPROACH PIER - PIER DESIGN "D"

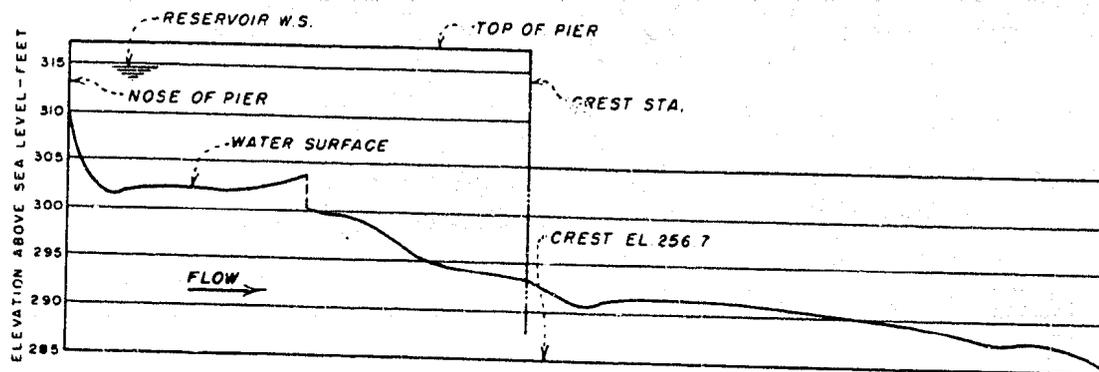
2069



DRAWDOWN DURING OPERATION—Q=456,000 G.F.S.

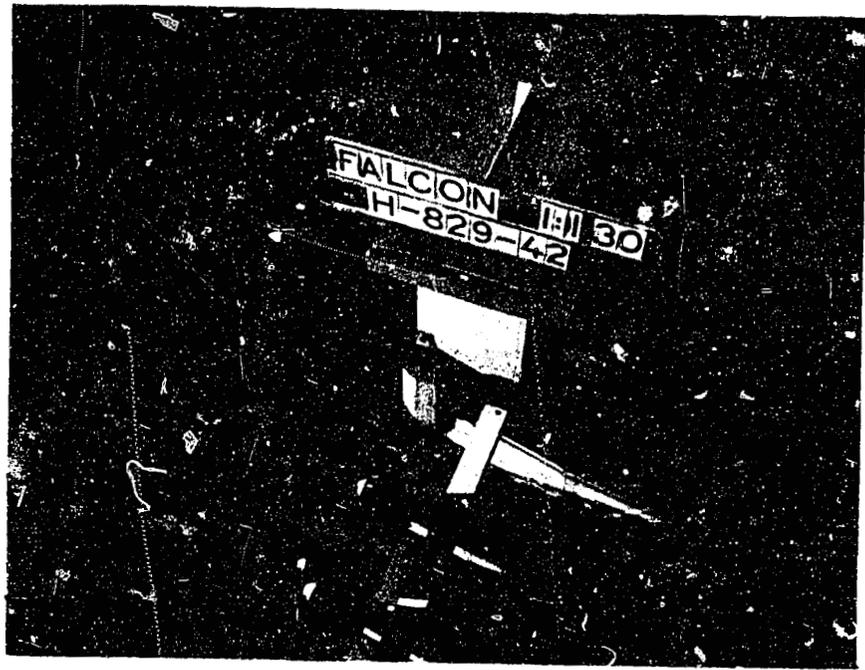


PLAN AND PERFORMANCE DATA

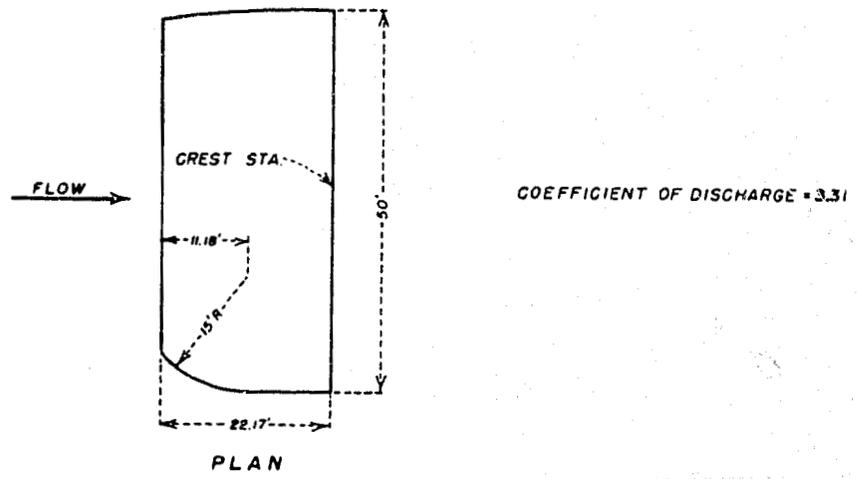


PROFILE OF DRAWDOWN—SPILLWAY SIDE Q=456,000 G.F.S.  
LEFT APPROACH PIER—PIER DESIGN "E"

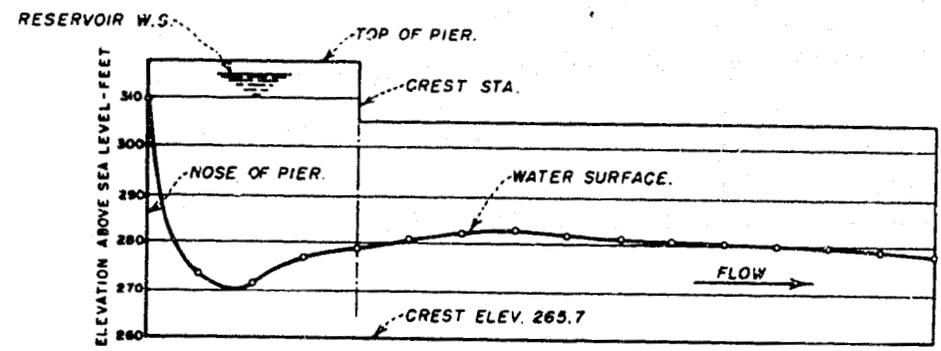
369



DRAWDOWN DURING OPERATION -  $Q = 456,000$  C.F.S.



PLAN AND PERFORMANCE DATA



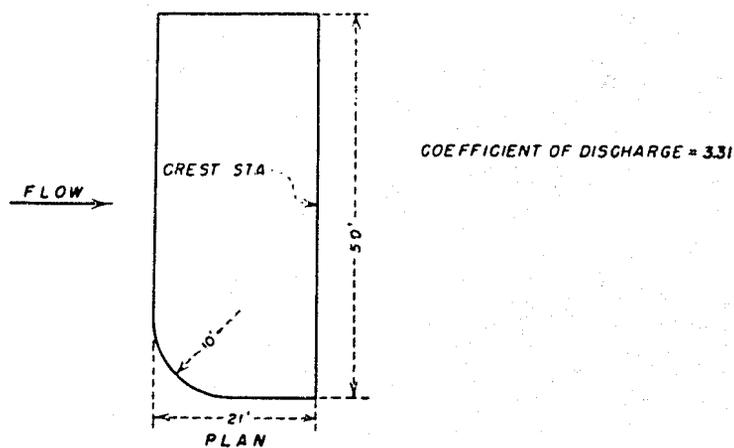
PROFILE OF DRAWDOWN - SPILLWAY SIDE  $Q = 456,000$  C.F.S.

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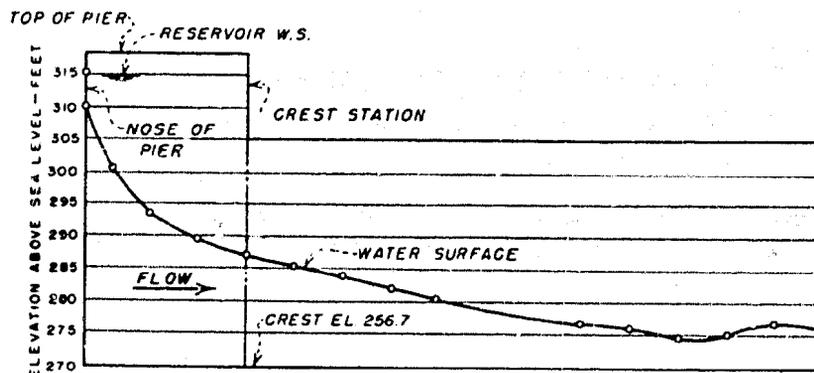
LEFT APPROACH PIER - PIER DESIGN "F"



DRAWDOWN DURING OPERATION -  $Q = 456,000$  C.F.S.



PLAN AND PERFORMANCE DATA



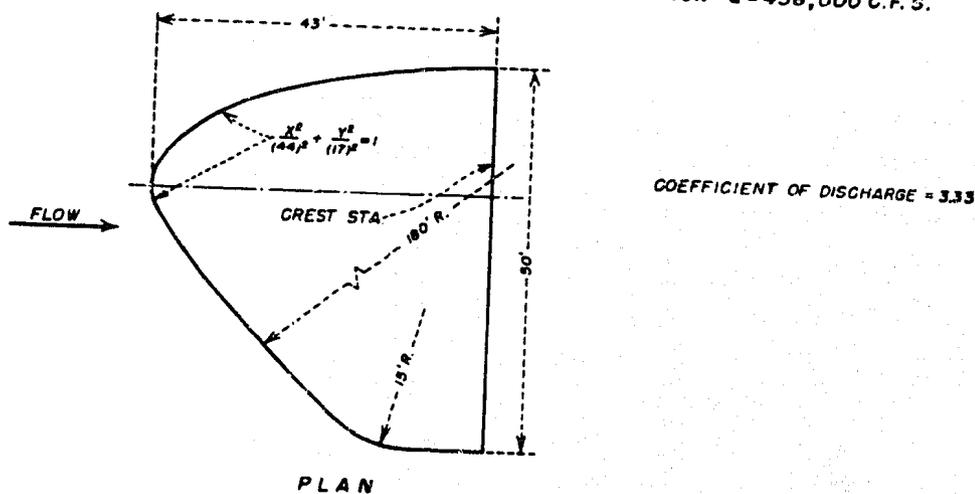
PROFILE OF DRAWDOWN - SPILLWAY SIDE  $Q = 456,000$  C.F.S.

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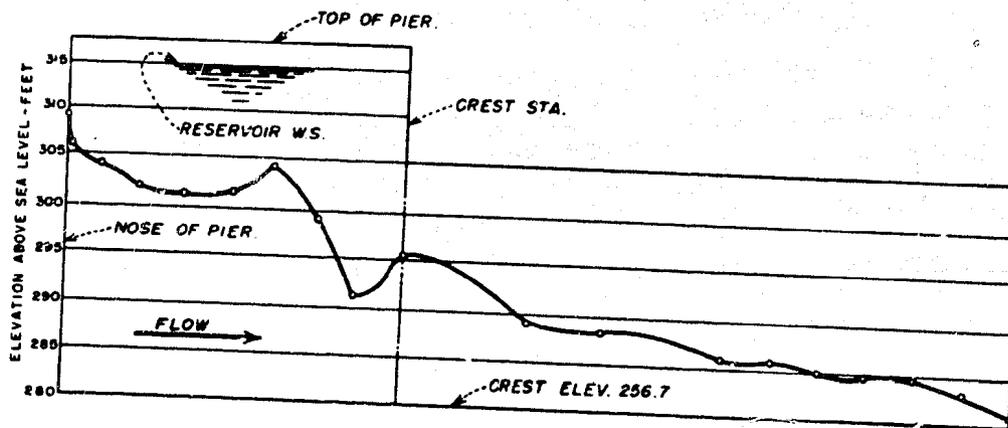
LEFT APPROACH PIER - PIER DESIGN "G"



DRAWDOWN DURING OPERATION - Q = 456,000 C.F.S.



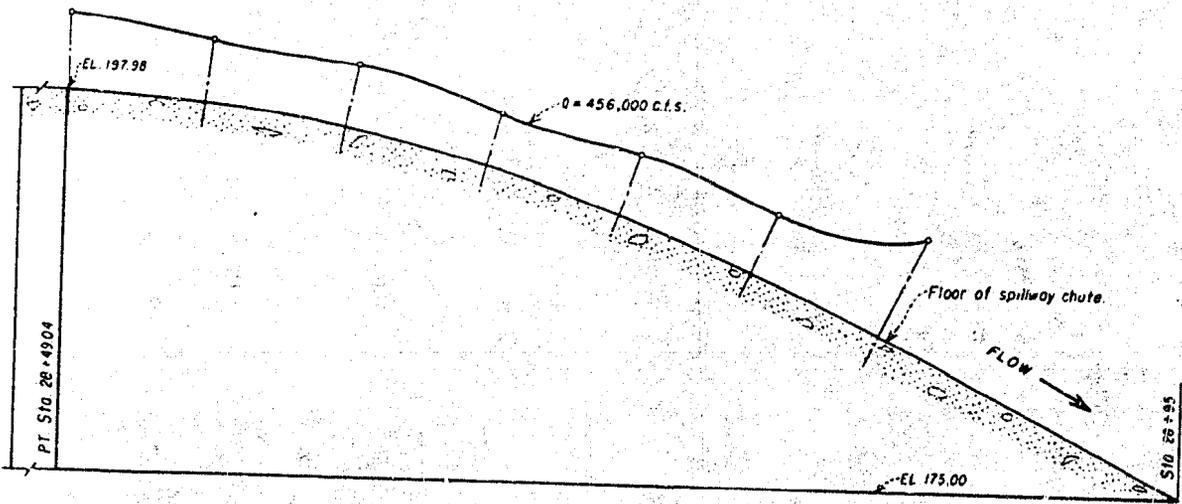
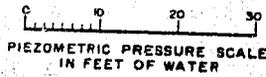
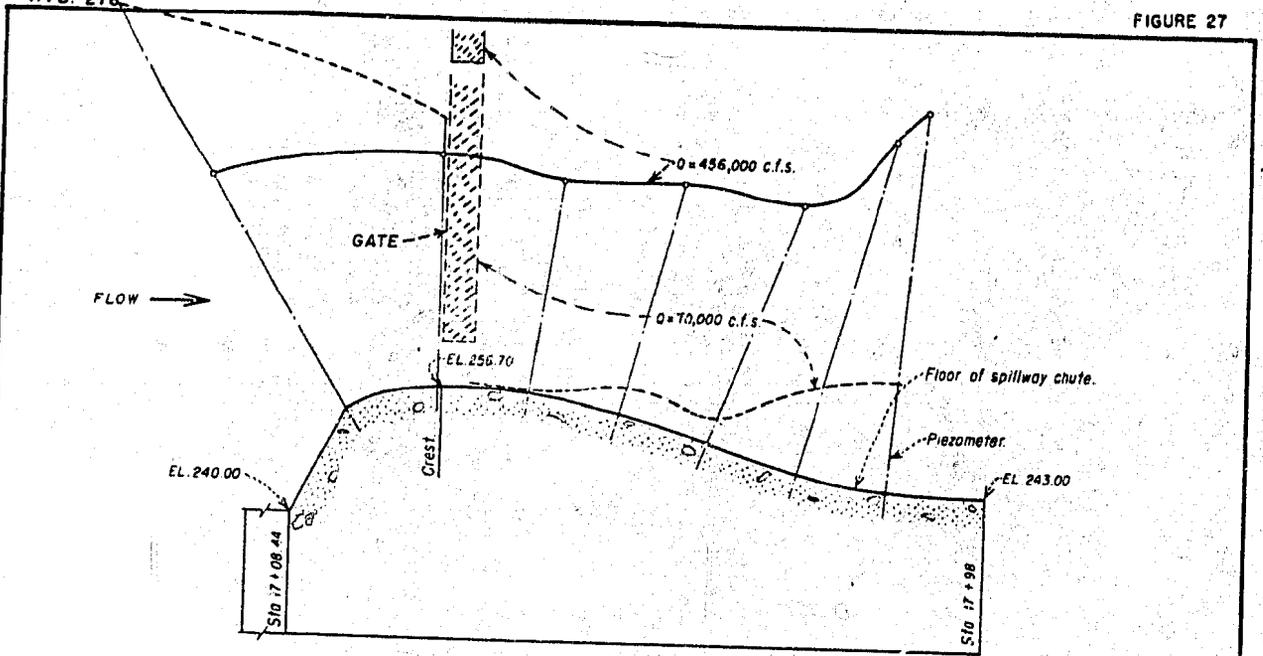
PLAN AND PERFORMANCE DATA



PROFILE OF DRAWDOWN-SPILLWAY SIDE - Q = 456,000 C.F.S.

LEFT APPROACH PIER - PIER DESIGN "H"

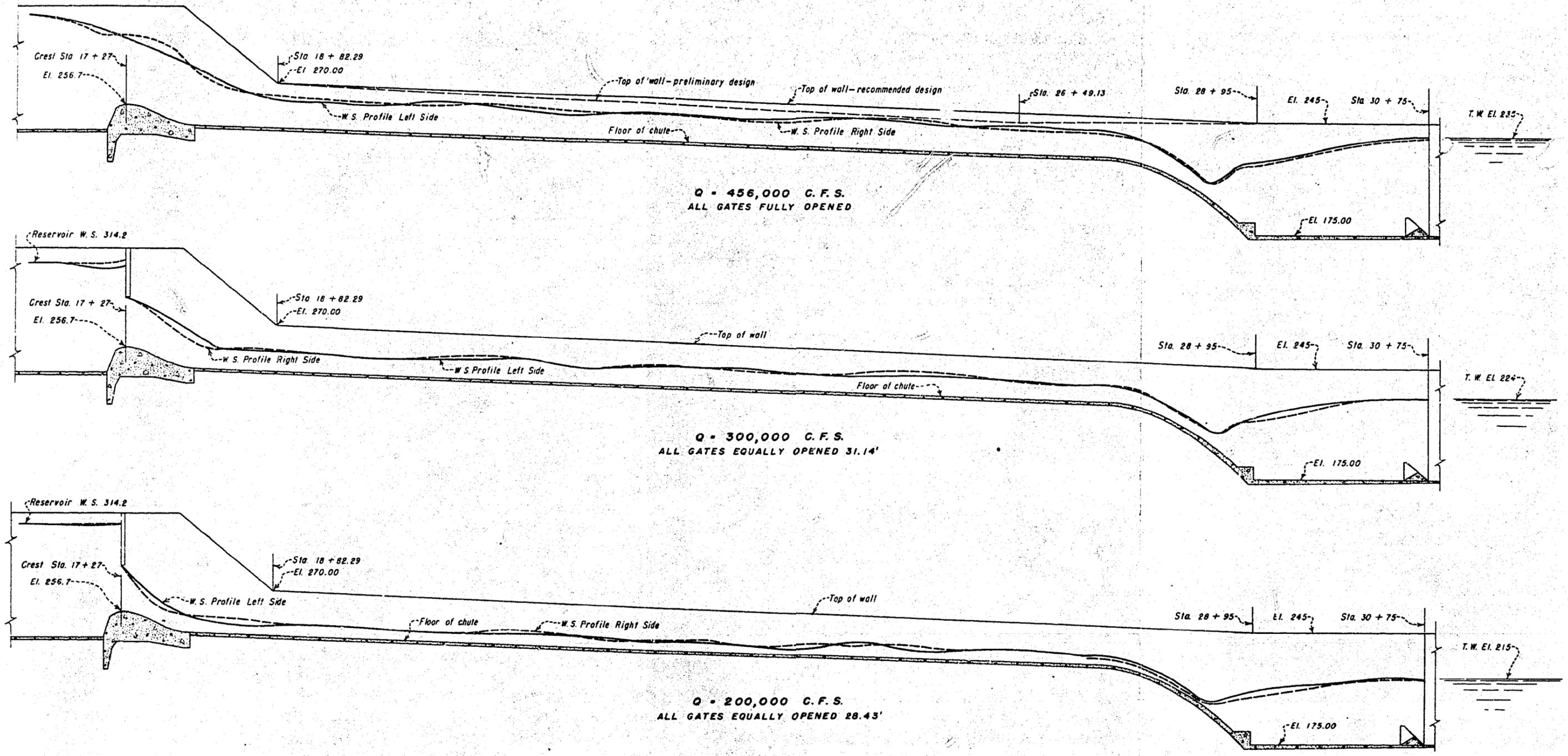
389



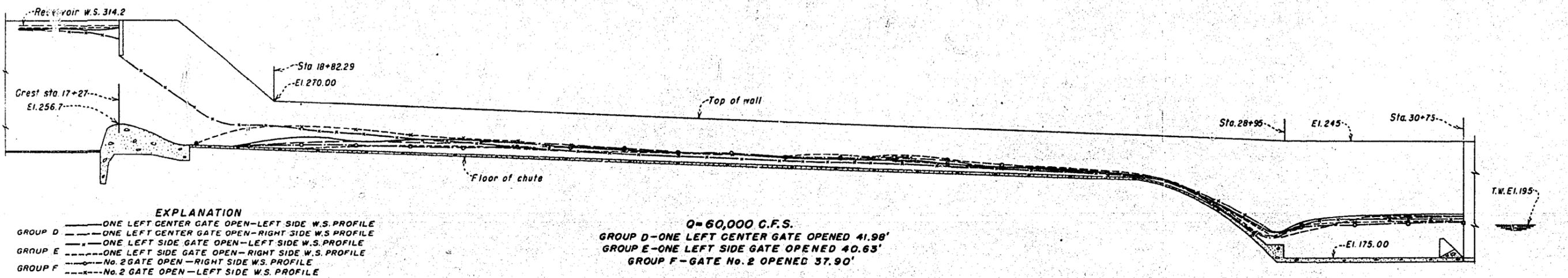
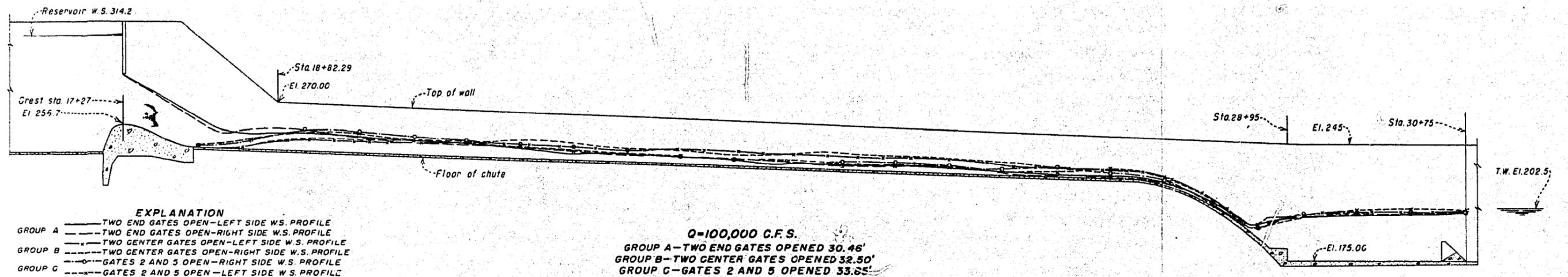
**NOTE**

Pressures are in feet of water, with the chute floor being the zero line. Pressures above the line are above atmospheric pressure. All pressures remained above atmospheric for the complete range of discharges.

**FALCON DAM**  
HYDRAULIC MODEL STUDIES  
SPILLWAY  
PIEZOMETRIC PRESSURES  
MODEL SCALE 1:130

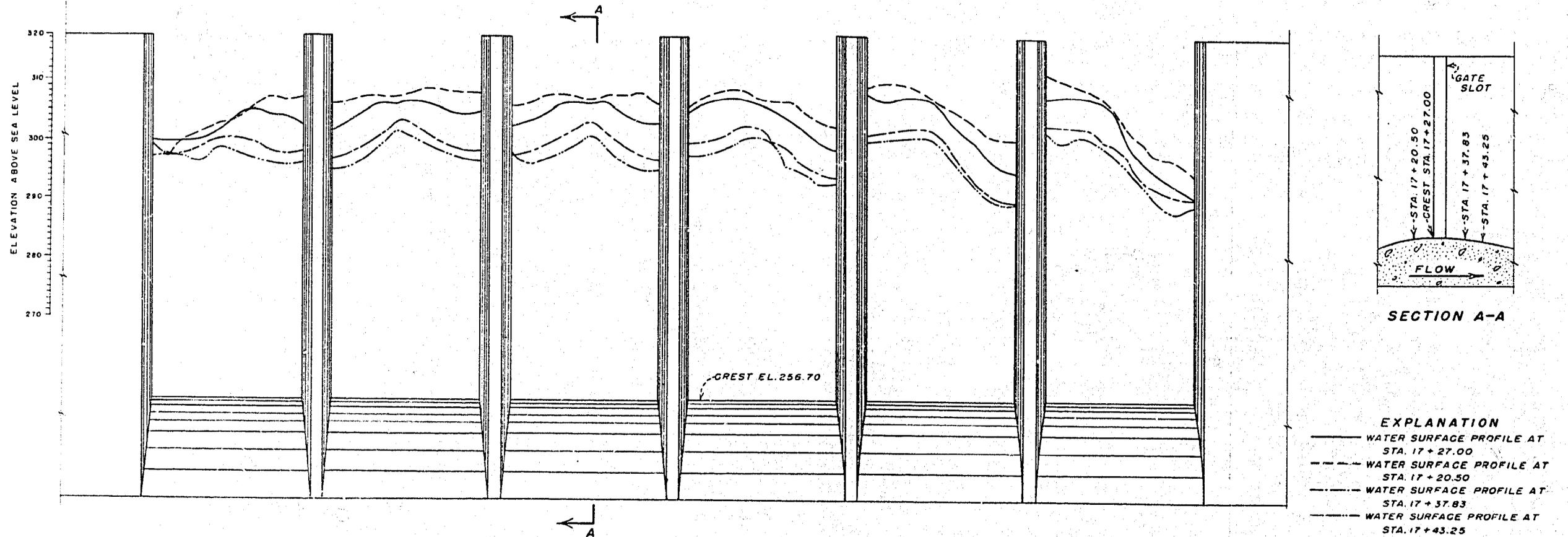


**FALCON DAM**  
HYDRAULIC MODEL STUDIES  
SPILLWAY RECOMMENDED DESIGN  
WATER SURFACE PROFILES  
MODEL SCALE 1:30



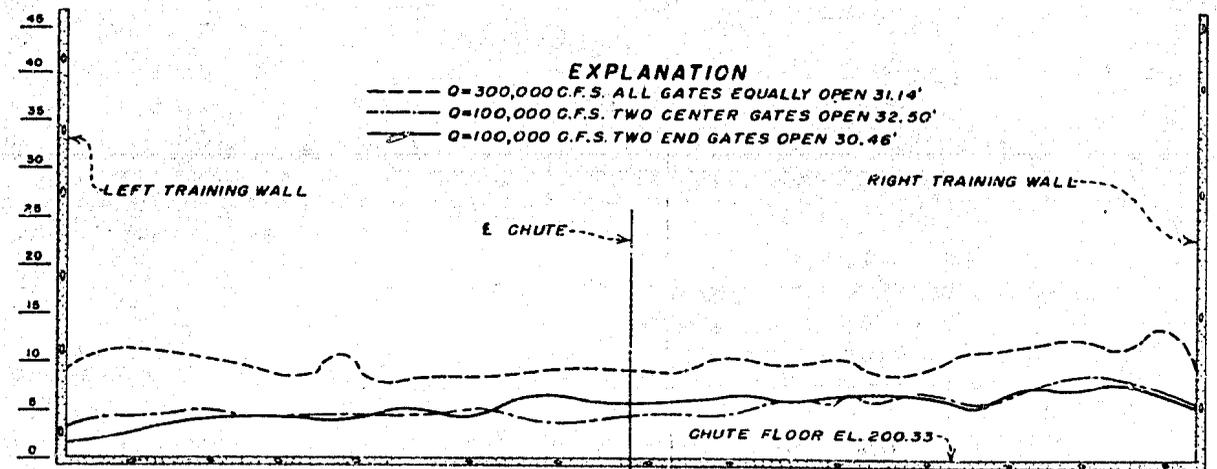
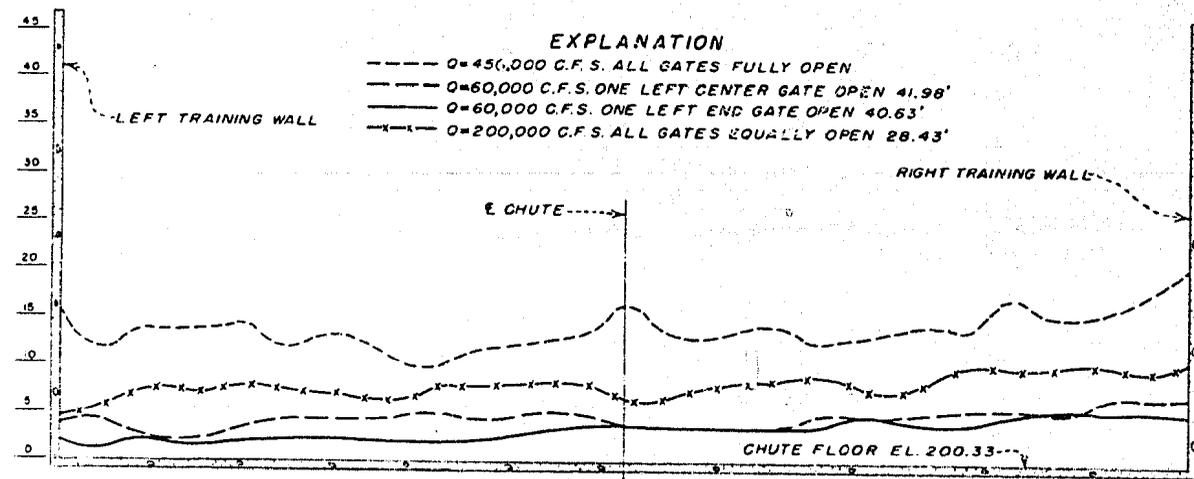
**FALCON DAM**  
 HYDRAULIC MODEL STUDIES  
 SPILLWAY—RECOMMENDED DESIGN  
 WATER SURFACE PROFILES  
 MODEL SCALE 1:130

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**NOTES**  
 1. VERTICAL SCALE IS TWO TIMES HORIZONTAL SCALE  
 2. FOR ALL PROFILES Q = 456,000 C.F.S.

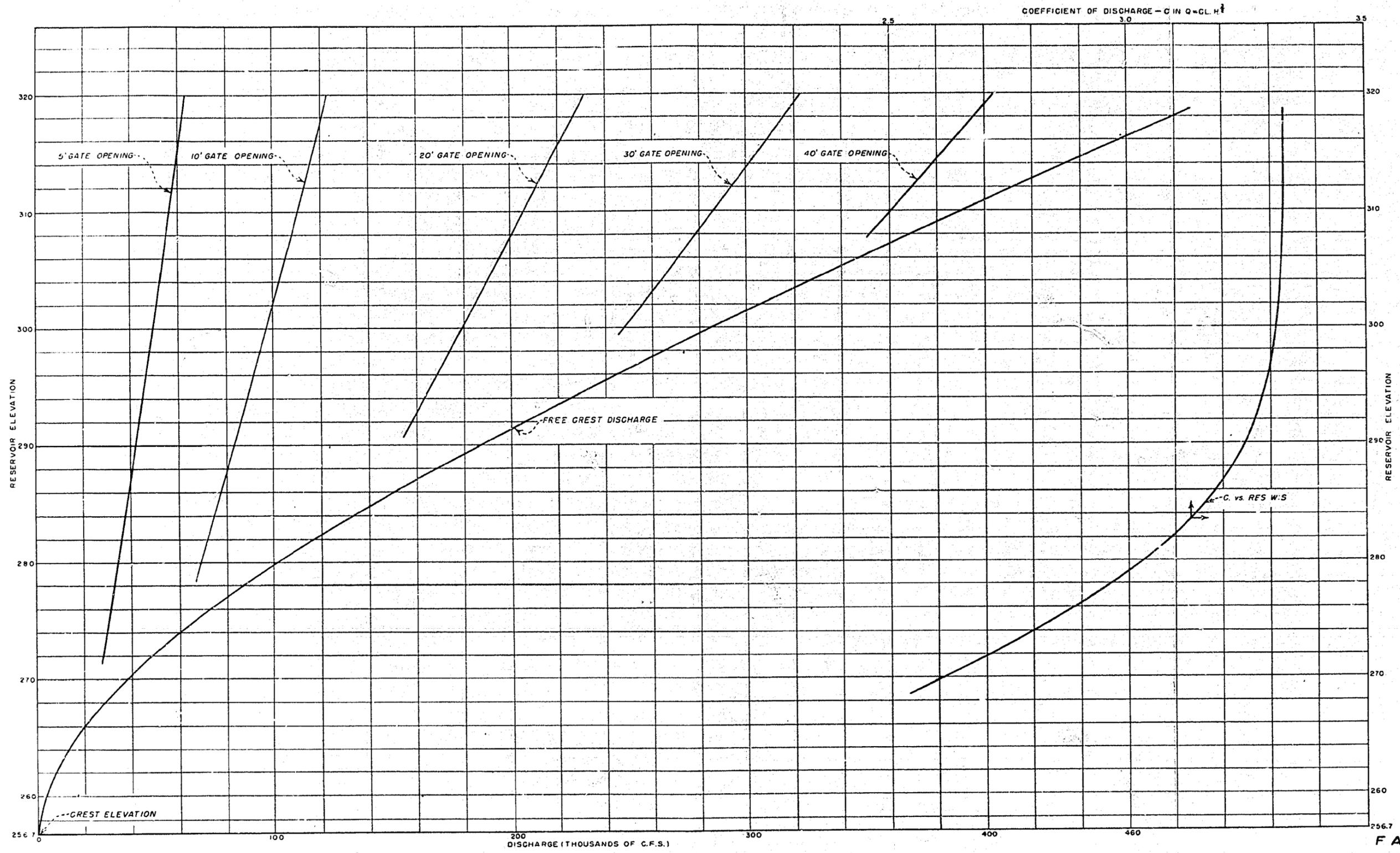
UPSTREAM VIEW OF SPILLWAY CREST



PROFILES AT STATION 28 + 43

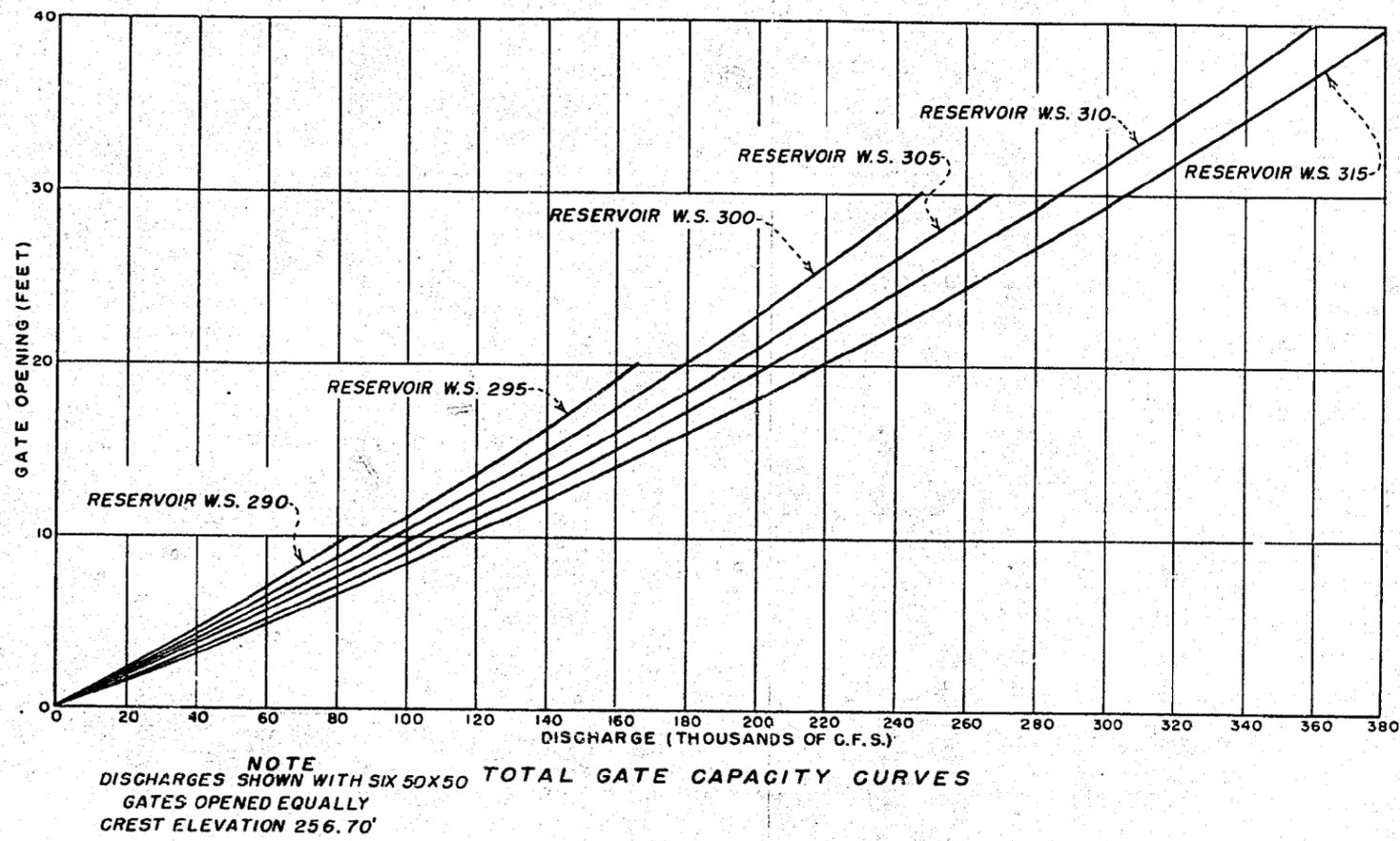
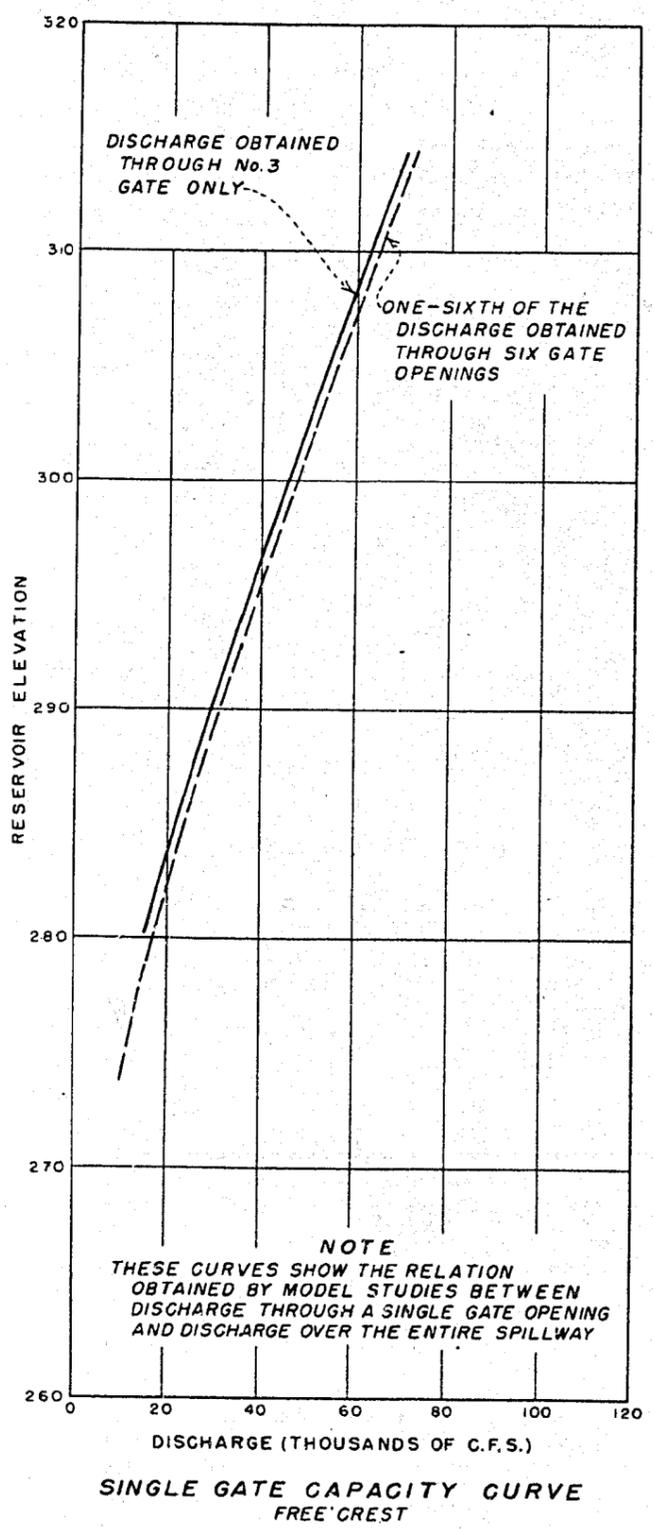
**NOTE**  
 STA. 28+43, WHERE THESE PROFILES WERE MEASURED IS JUST UPSTREAM FROM THE UPPER LIMIT OF THE HYDRAULIC JUMP AT MAXIMUM DISCHARGE

**FALCON DAM**  
 HYDRAULIC MODEL STUDIES  
 SPILLWAY—RECOMMENDED DESIGN  
 WATER SURFACE PROFILES  
 MODEL SCALE 1:130



NOTE  
 ALL GATES OPENED EQUALLY  
 SIX 50' X 50' LEAF GATES

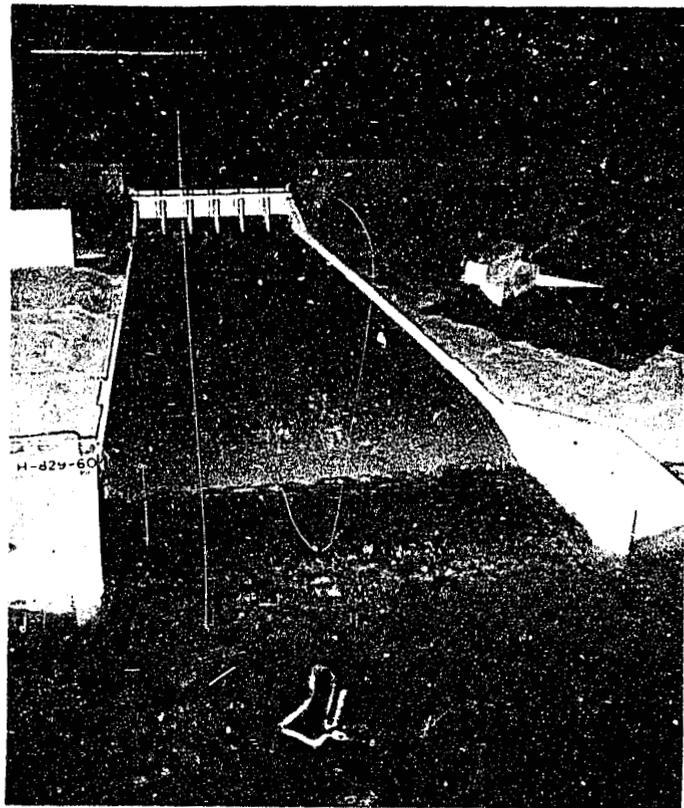
FALCON DAM  
 HYDRAULIC MODEL STUDIES  
 SPILLWAY—RECOMMENDED DESIGN  
 DISCHARGE CURVES  
 MODEL SCALE 1:130



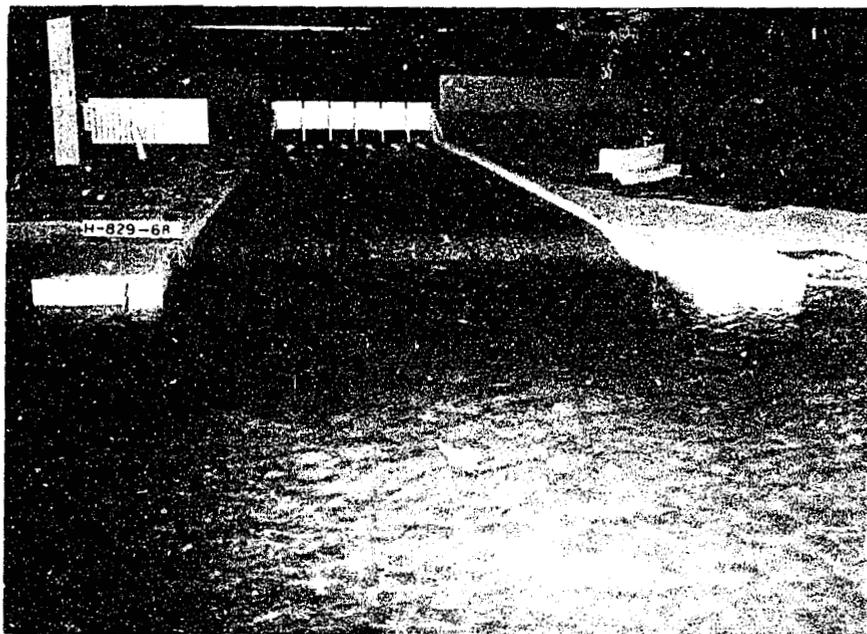
FALCON DAM  
HYDRAULIC MODEL STUDIES  
SPILLWAY-RECOMMENDED DESIGN  
DISCHARGE CURVES  
MODEL SCALE 1:130

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-4-50

Figure 33  
Hyd. 276



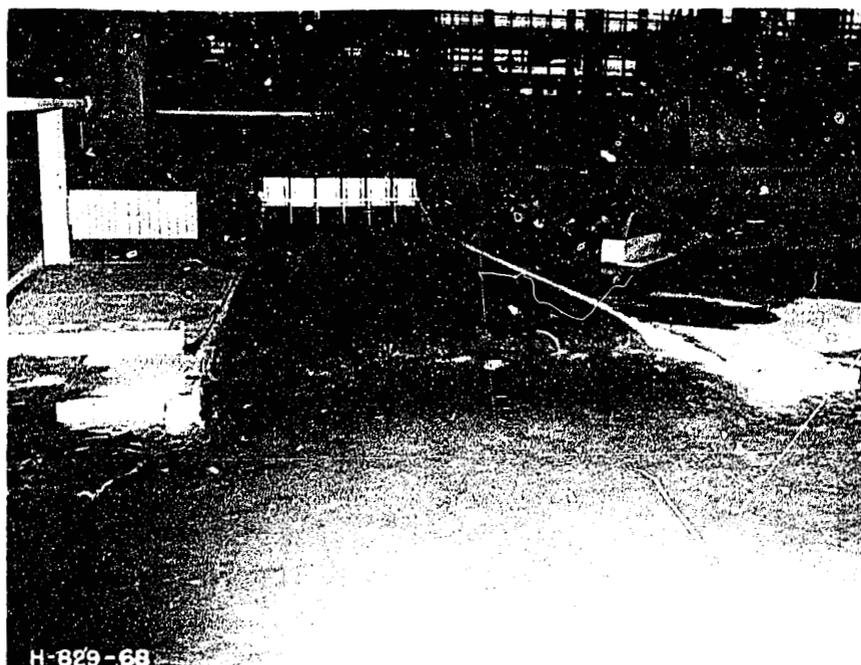
100,000 cfs being discharged with all gates open 8 feet and the reservoir water surface at elevation 314.2.



200,000 cfs being discharged with all gates open 19 feet and the reservoir water surface at elevation 314.2.

THE PROJECT MODEL. DISCHARGE OVER THE SPILLWAY WITH PARTIAL GATE OPENINGS.

Figure 34  
Hyd. 276



A. General operation of the spillway.

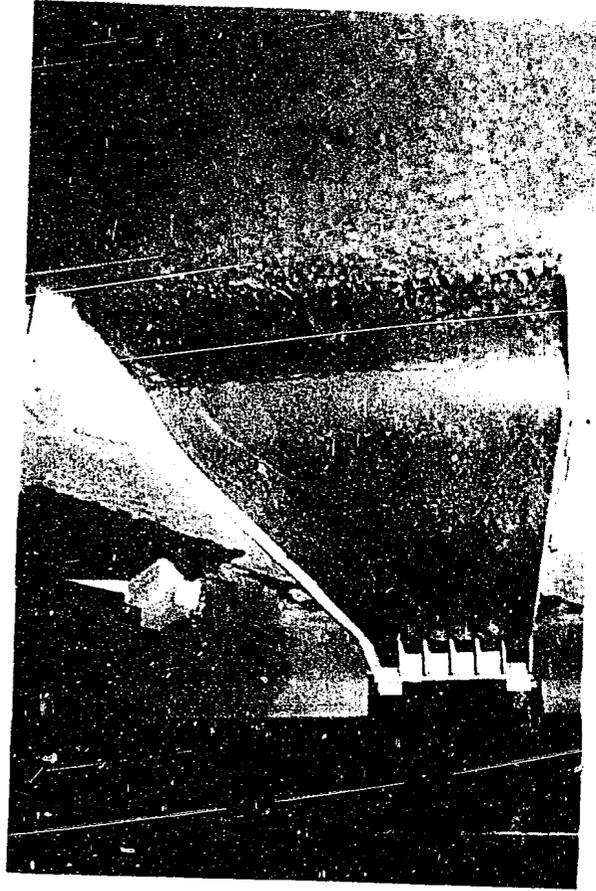


B. Operation of the stilling basin.

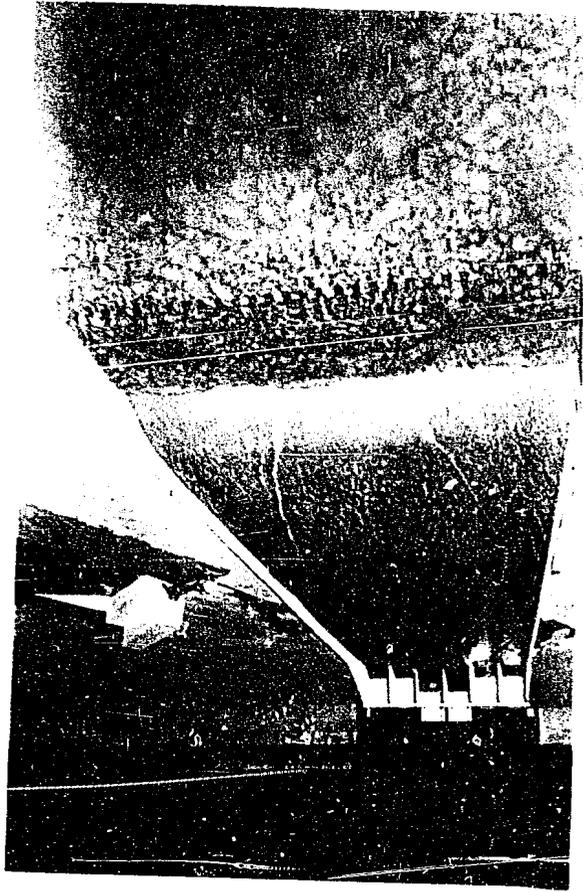
THE SPILLWAY---RECOMMENDED DESIGN. 456,000 CFS  
DISCHARGING WITH ALL GATES FULLY OPEN.

THE SPILLWAY---RECOMMENDED DESIGN. 100,000 CFS  
BEING DISCHARGED THROUGH TWO GATE OPENINGS WITH  
THE RESERVOIR WATER SURFACE AT ITS MAXIMUM. NOTE  
THE STANDING WAVE PATTERN.

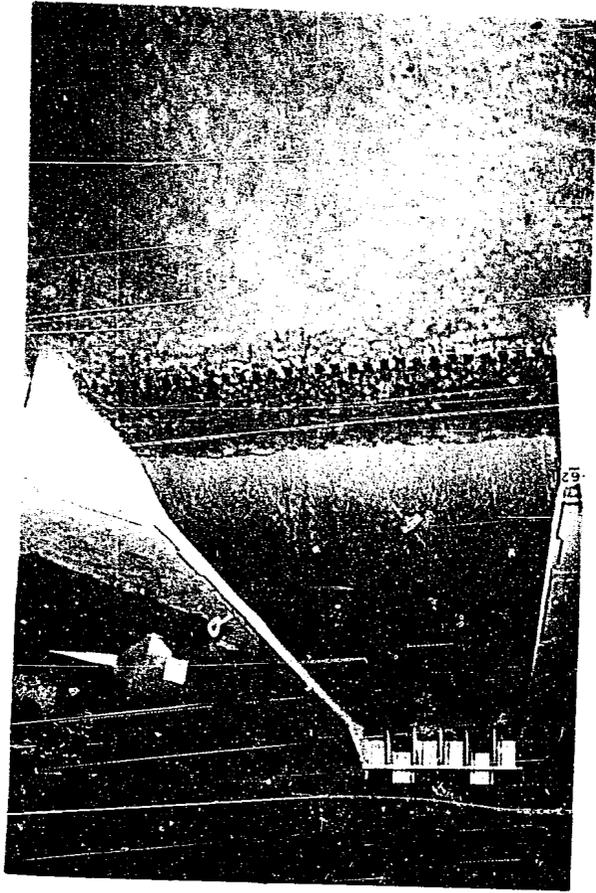
A. Gates Nos. 1 and 6 open.

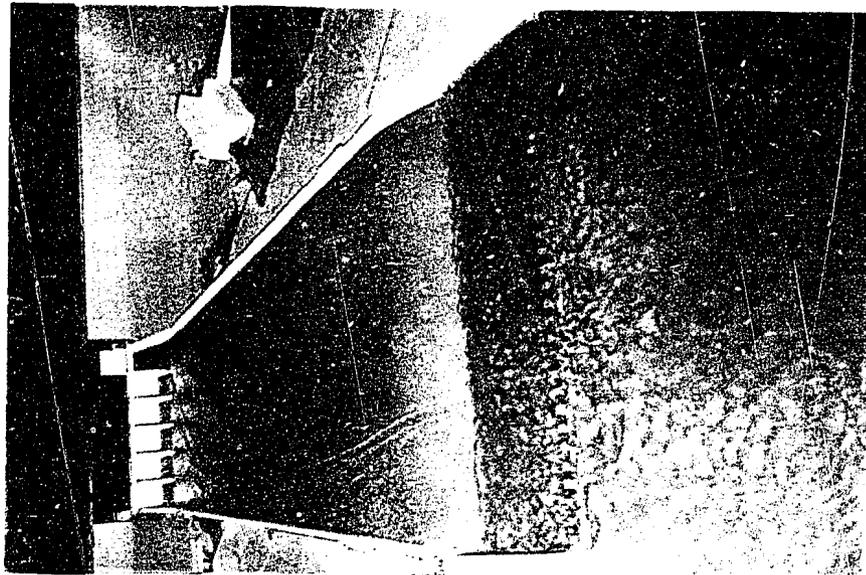


B. Gates Nos. 3 and 4 open.

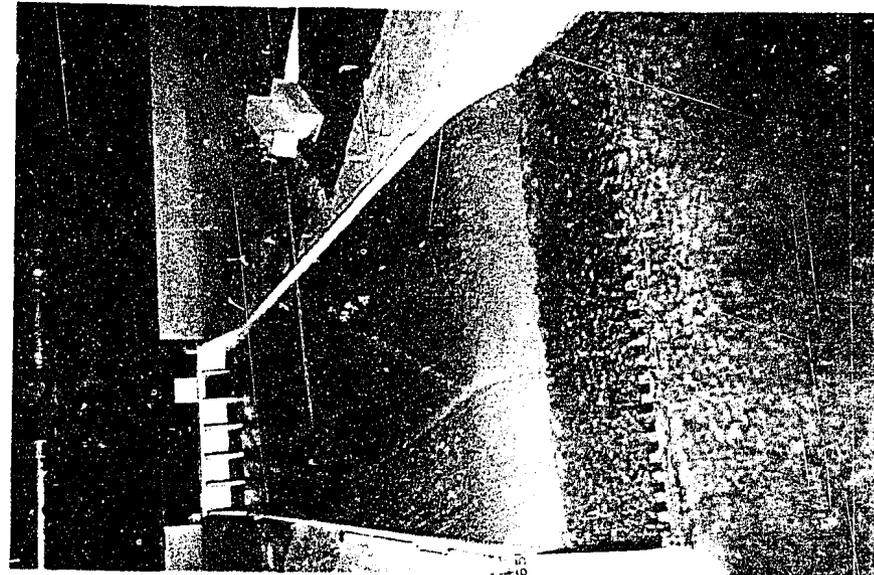


C. Gates Nos. 2 and 5 open.

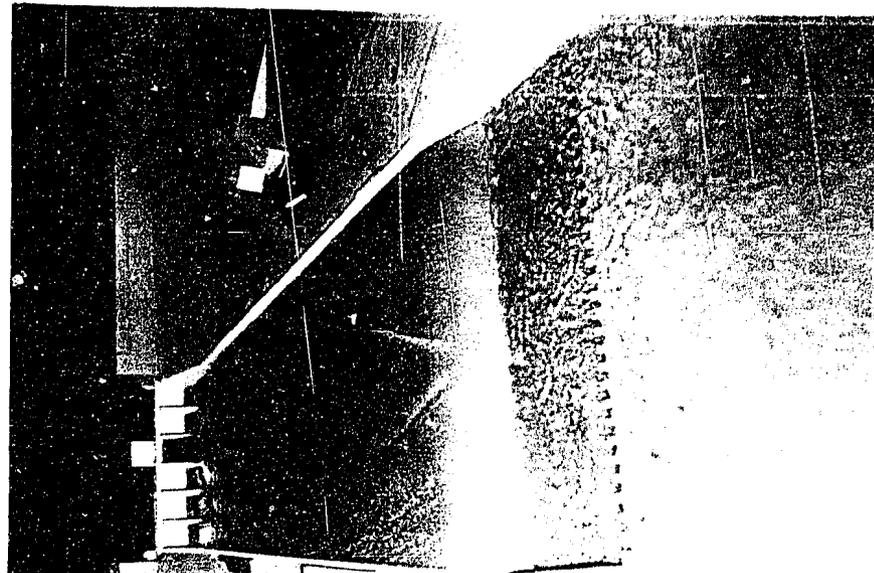




A. No. 1 gate open.

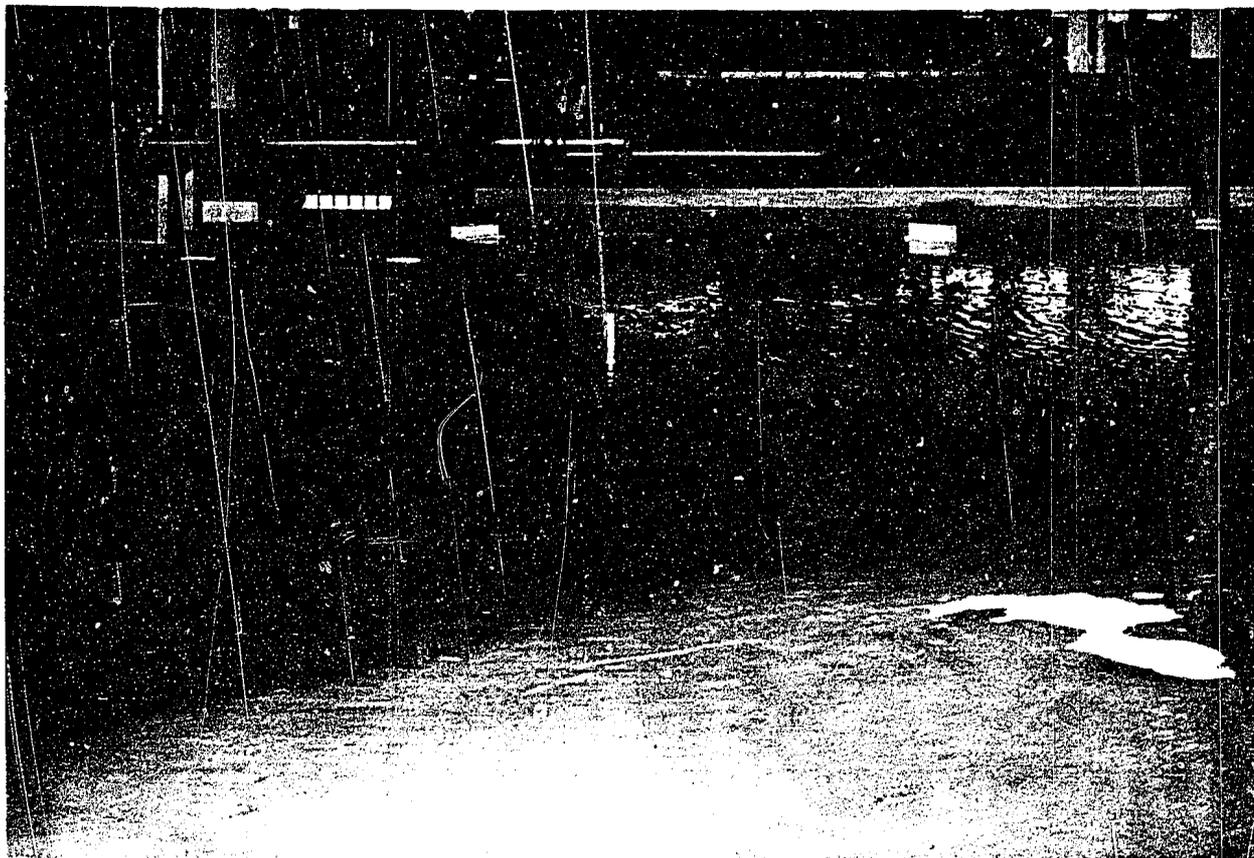


B. No. 2 gate open.

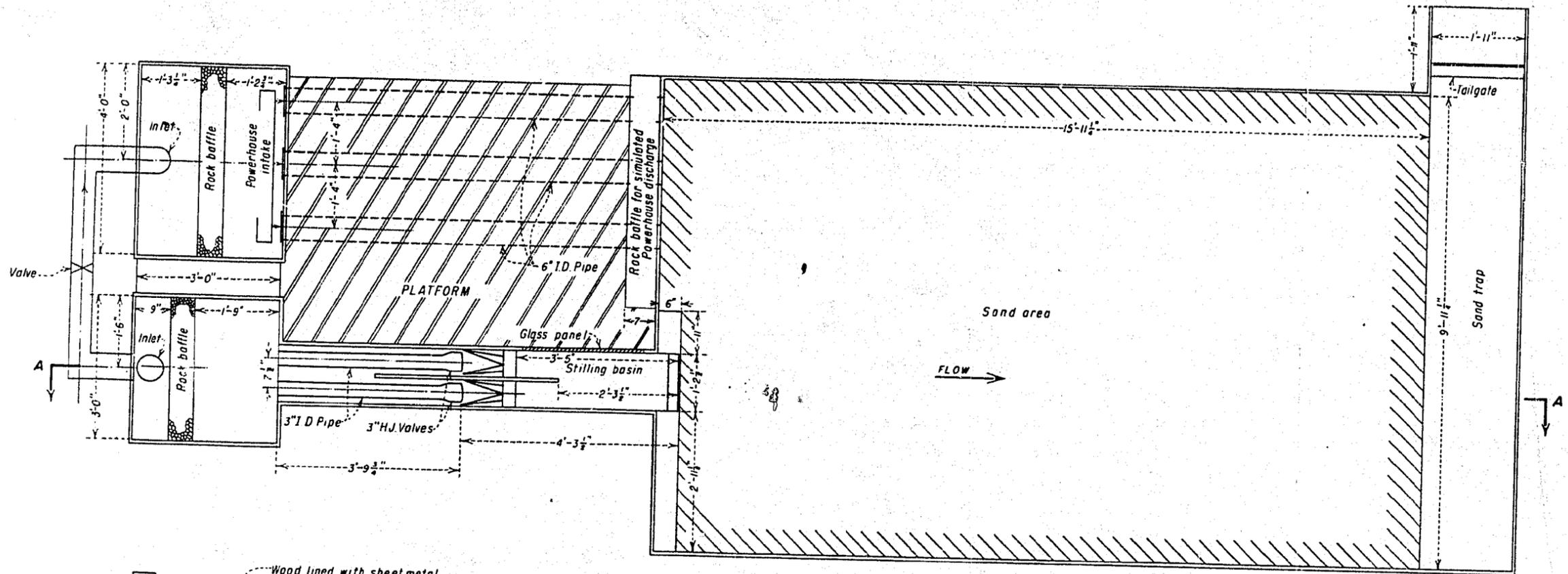


C. No. 3 gate open.

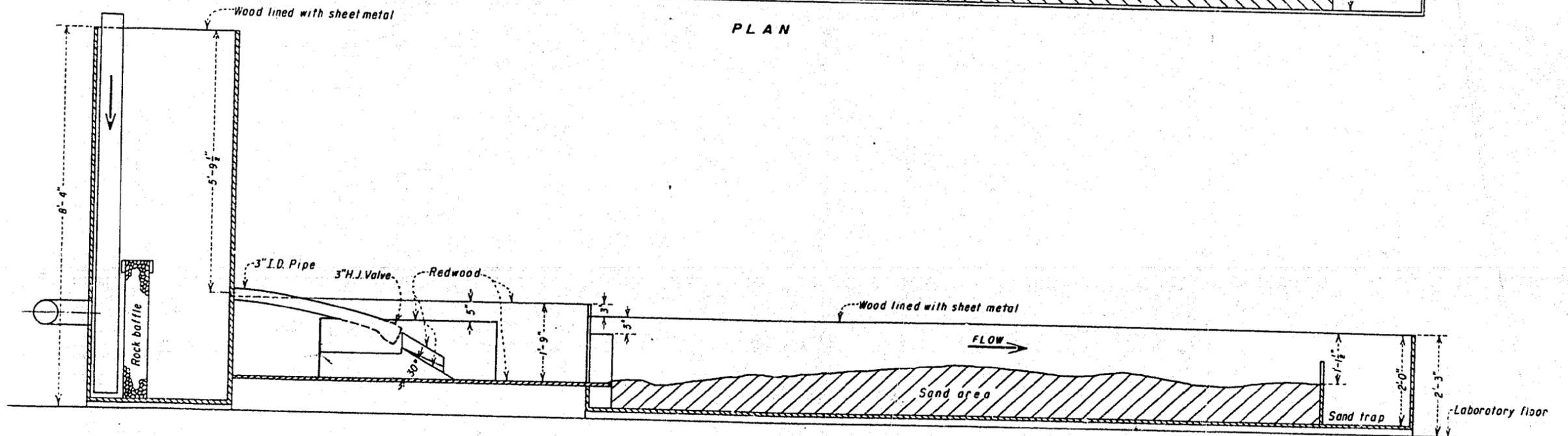
THE SPILLWAY---RECOMMENDED DESIGN. 60,000 CFS  
BEING DISCHARGED THROUGH ONE GATE OPENING. NOTE  
THE STANDING WAVES IN THE CHUTE.



THE PROJECT MODEL---RECOMMENDED DESIGN WITH A DIS-  
CHARGE OF 456,000 CFS. ALL GATES FULLY OPEN. TAIL-  
WATER ELEVATION 235.



PLAN

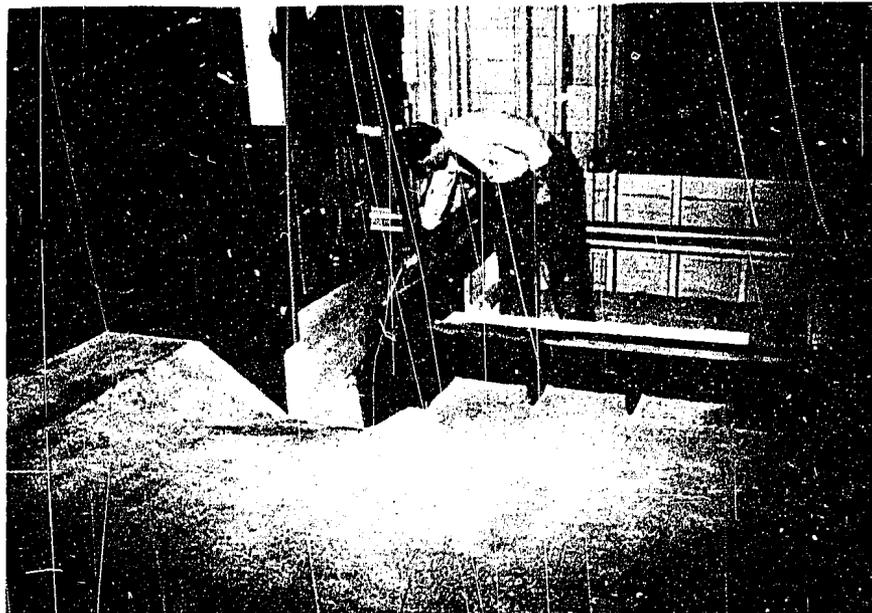


SECTION A-A

**FALCON DAM**  
 HYDRAULIC MODEL STUDIES  
 MEXICAN OUTLET WORKS  
 MODEL ARRANGEMENT  
 MODEL SCALE 1:30

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 3-50

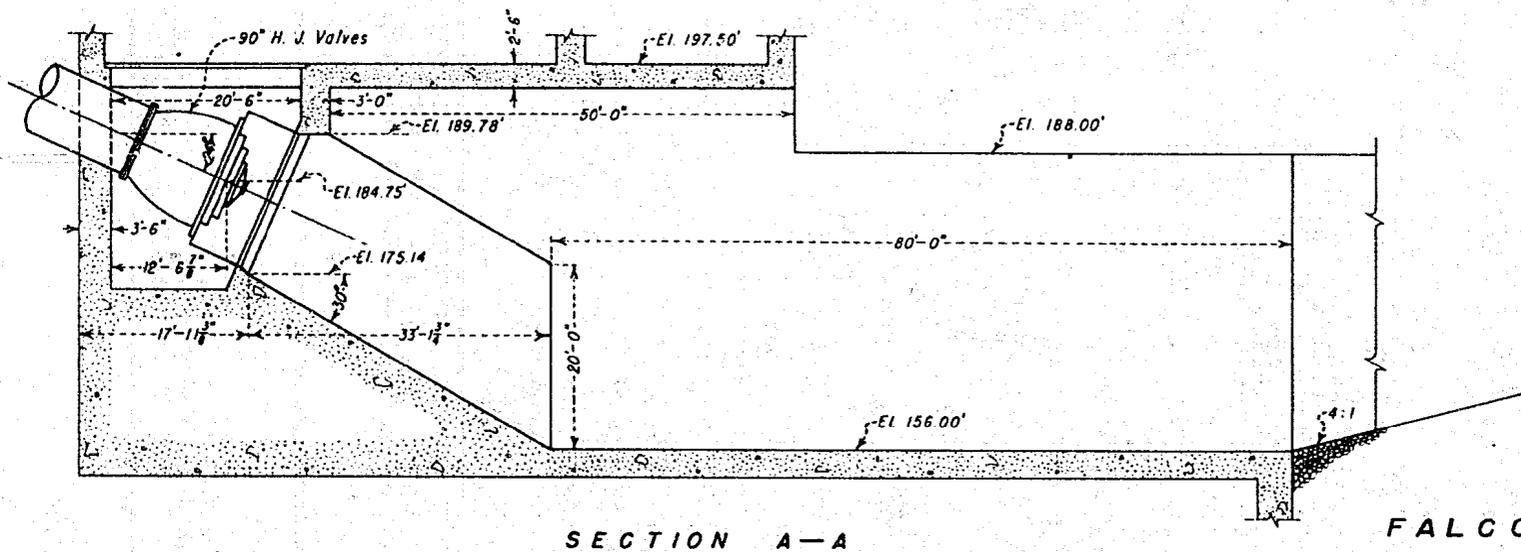
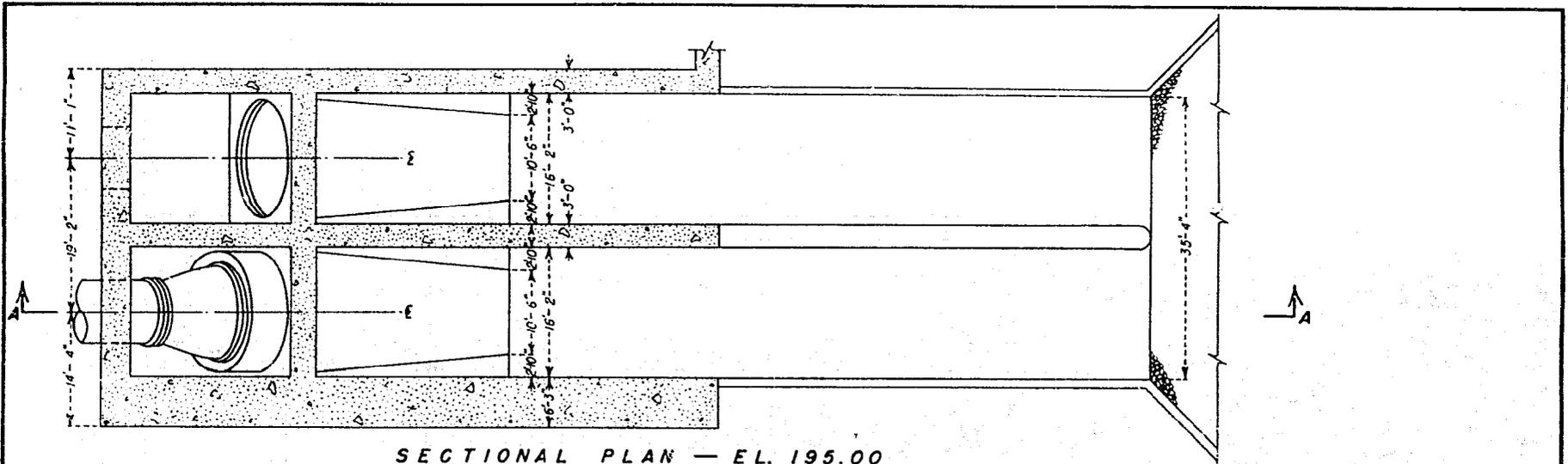
Figure 39  
Hyd. 276



A. Model arrangement. Note the conduit leading from the head box to the valve, the stilling basin, and the downstream topography. At the right are the three simulated power plant openings.



B. The stilling basin. Note the valves, the inclined floor, the converging walls, the fillets, the horizontal floor, and the end sill. This is the recommended design.



**FALCON DAM**  
 HYDRAULIC MODEL STUDIES  
 MEXICAN OUTLET WORKS  
 PRELIMINARY DESIGN  
 MODEL SCALE 1:30

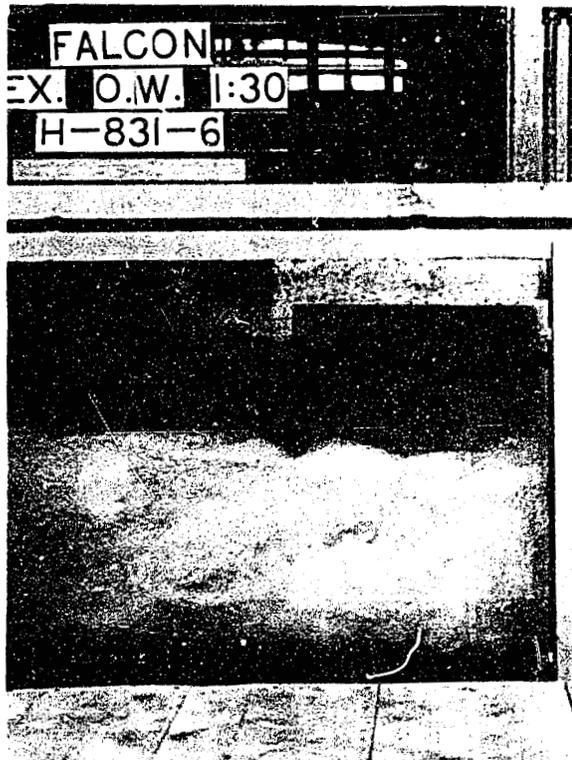
369

1-50

Figure 41  
Hyd. 276



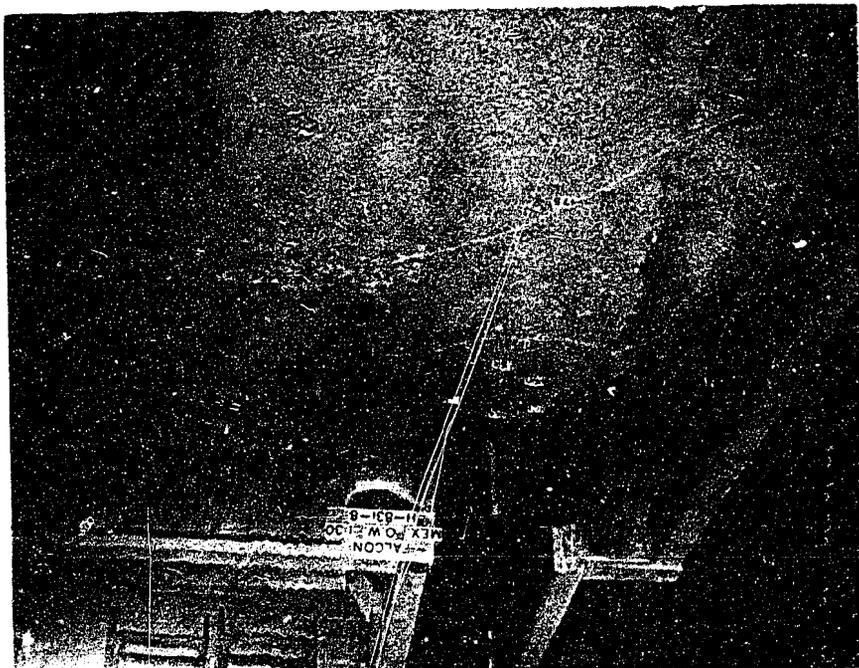
A. Operation of the stilling basin. Both valves discharging-- $Q = 4,570$  cfs.



B. Action in the stilling basin. Photograph taken through glass panel in left side of model. Both valves discharging-- $Q = 4,570$  cfs.

THE MEXICAN OUTLET WORKS---PRELIMINARY DESIGN  
OPERATION WITHOUT POWER PLANT DISCHARGE.

Position after a discharge of 4,570 cfs for 1 hour.  
Both valves operated during the entire test. No discharge  
through power plant.



Position after a discharge of 4,570 cfs through  
the valves and 5,400 cfs through the powerhouse for 11 hours.

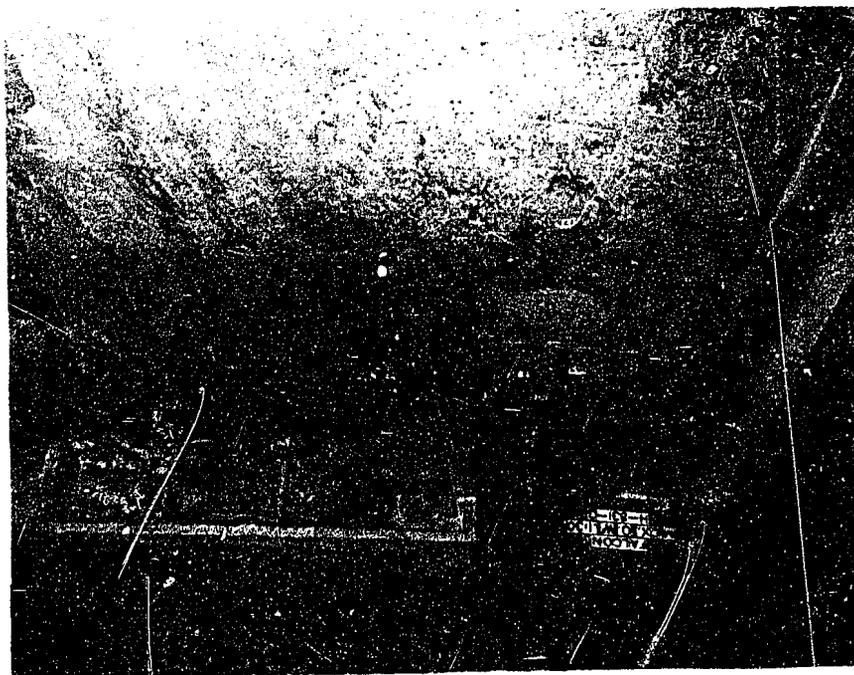


Figure 43  
Hyd. 276

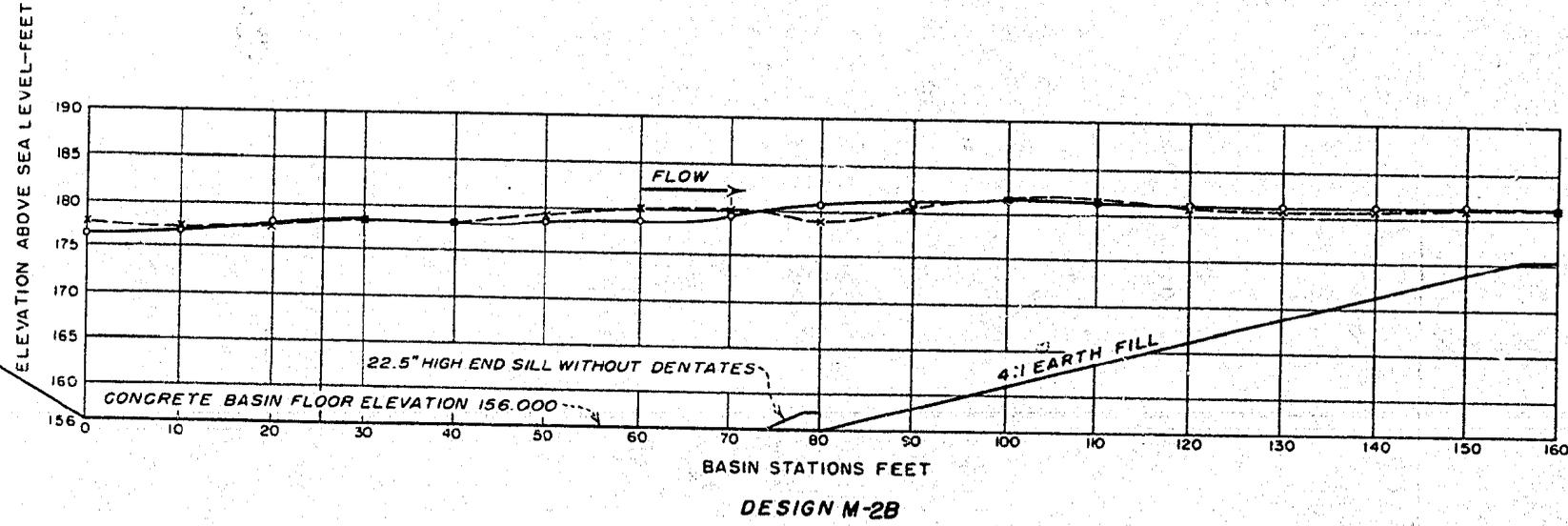
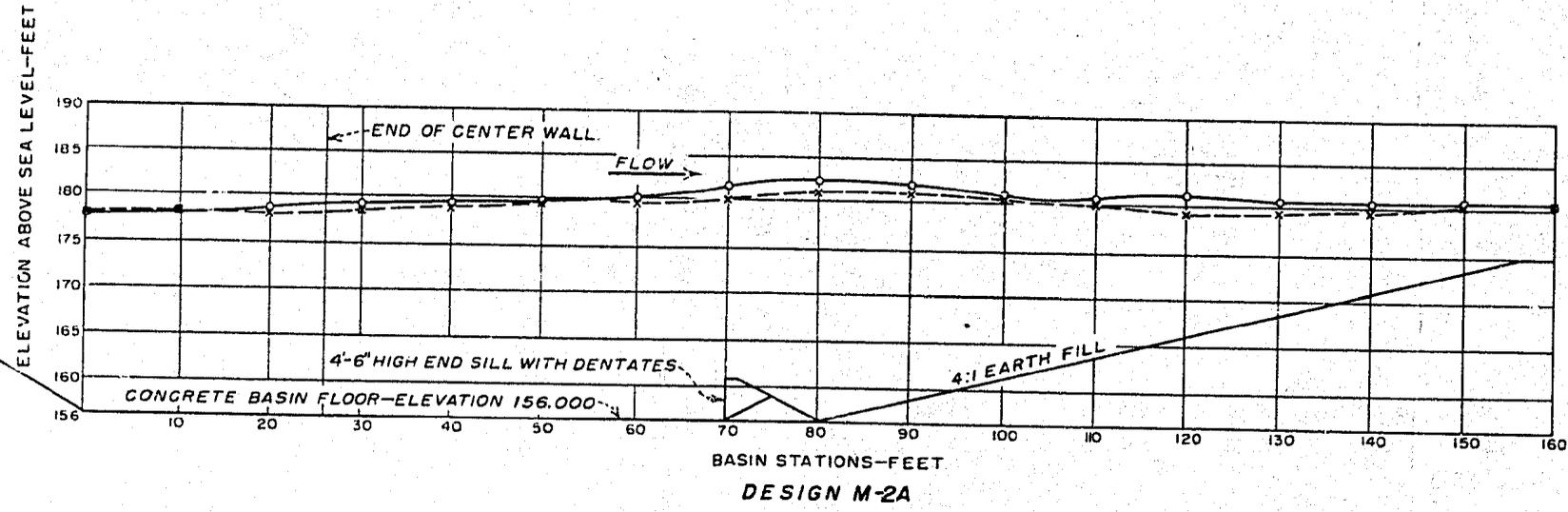


A. The stilling basin. 4,570 cfs discharging through both valves.



B. Action in the basin. 4,570 cfs discharging through both valves.

THE MEXICAN OUTLET WORKS. PRELIMINARY DESIGN. OPERATION WITH THE POWER PLANT DISCHARGING 5,400 CUBIC FEET PER SECOND.

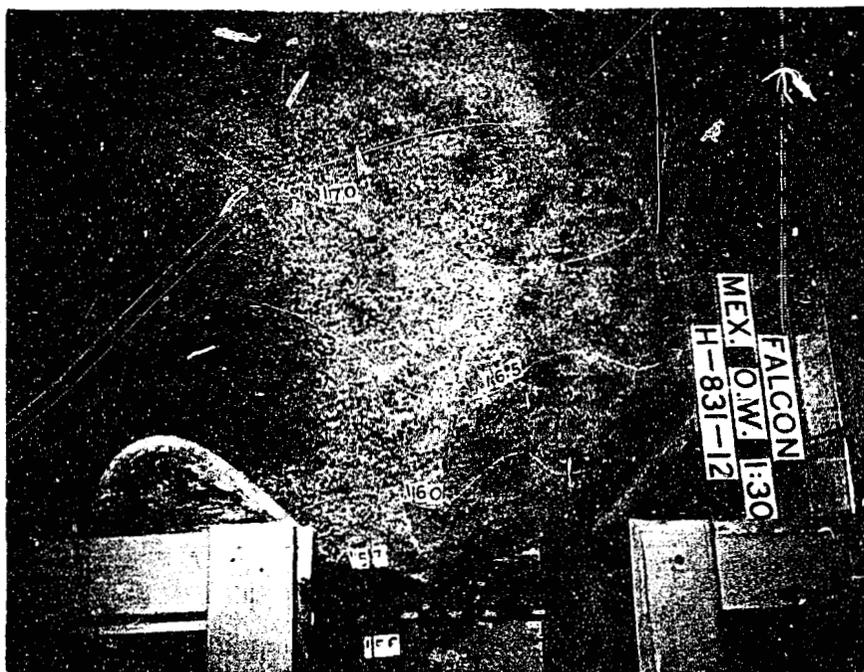


————— Q=4570 C.F.S. BOTH VALVES OPEN—PROFILE ALONG LEFT WALL.  
 - - - - - Q=2400 C.F.S. LEFT VALVE OPEN—PROFILE ALONG LEFT WALL

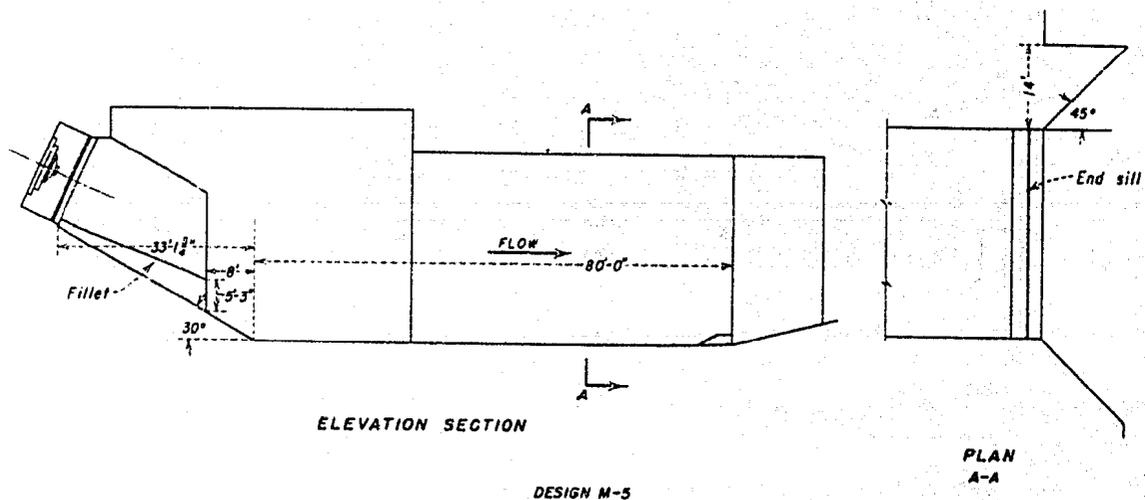
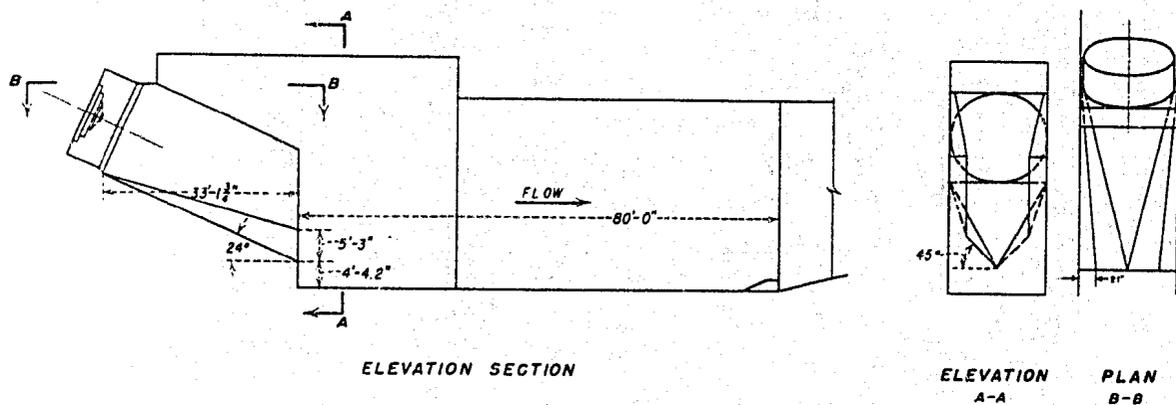
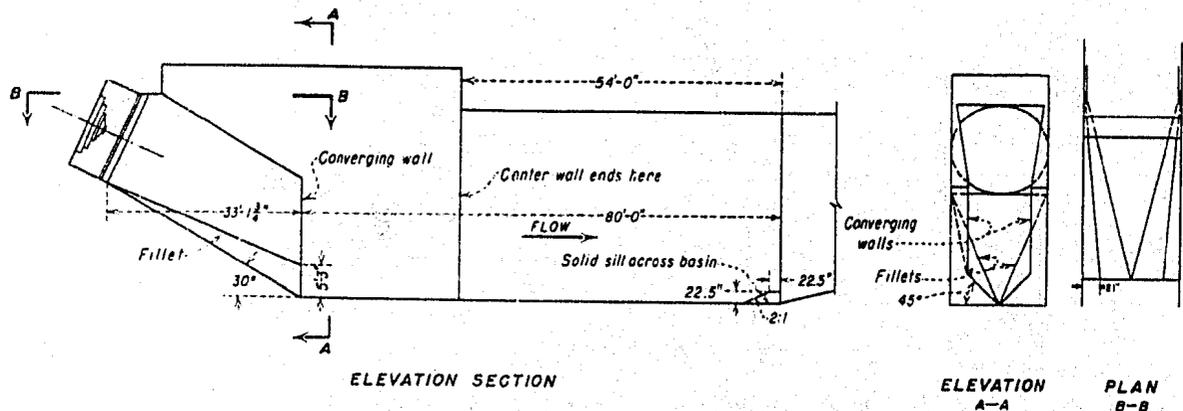
**FALCON DAM**  
 HYDRAULIC MODEL STUDIES  
 MEXICAN OUTLET WORKS  
 STILLING BASIN DESIGNS  
 MODEL SCALE 1:30



A. One valve discharging-- $Q = 2,400$  cfs.

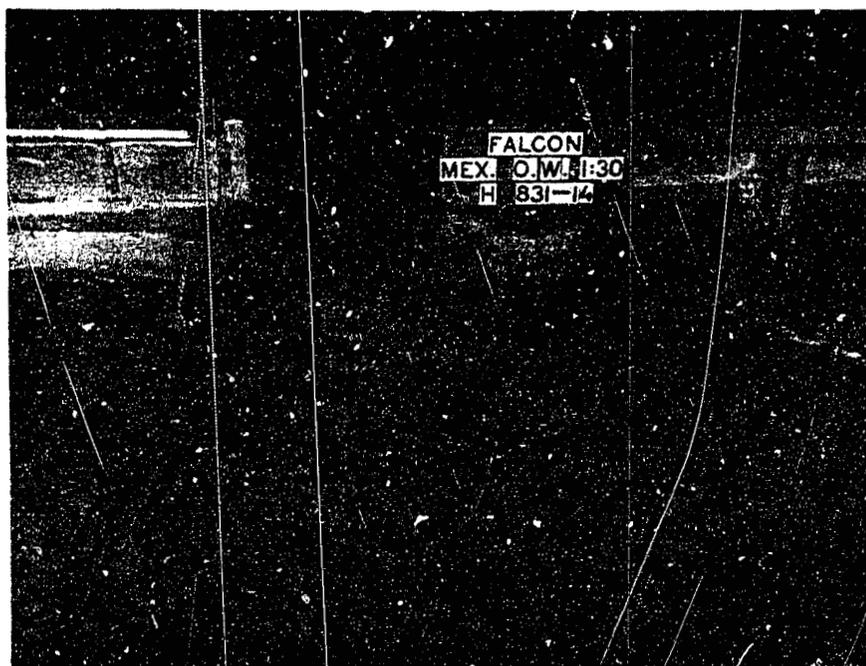


B. Erosion after a discharge of 2,400 cfs for 1 hour through left valve.



FALCON DAM  
 HYDRAULIC MODEL STUDIES  
 MEXICAN OUTLET WORKS  
 STILLING BASIN DESIGNS  
 MODEL SCALE 1:30

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A. Left valve discharging 2,400 cfs.



B. Erosion after a discharge of 2,400 cfs for 1 hour through left valve. Note sand deposited in basin.

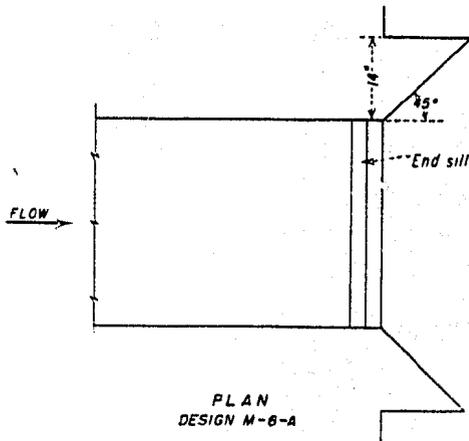


A. Design 4. Slope of inclined approach to basin  $24^{\circ}$ . Note slope of water surface from left to right.  $Q = 4,570$  cfs, both valves discharging. Flow from right to left.

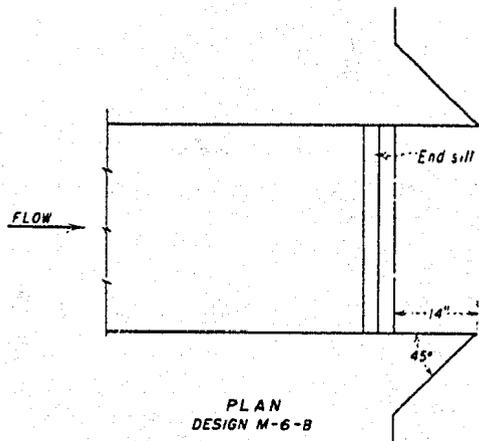


B. Design 5. Slope of inclined approach to basin  $30^{\circ}$ . Surface of water has approached the horizontal.  $Q = 4,570$  cfs, both valves discharging.

THE MEXICAN OUTLET WORKS---DESIGNS M-4 AND M-5.  
WATER SURFACES IN THE STILLING BASIN.

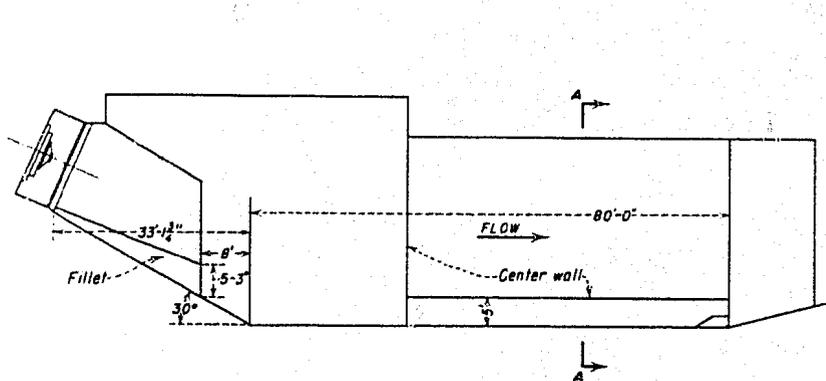


PLAN  
DESIGN M-6-A



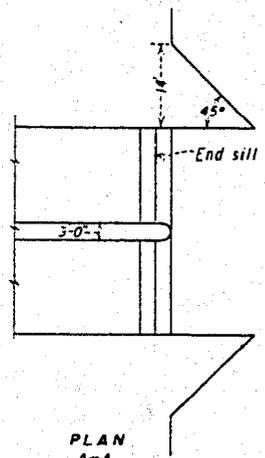
PLAN  
DESIGN M-6-B

DESIGN M-6  
UPSTREAM SECTION OF  
BASIN IS THE SAME AS  
DESIGN M-5

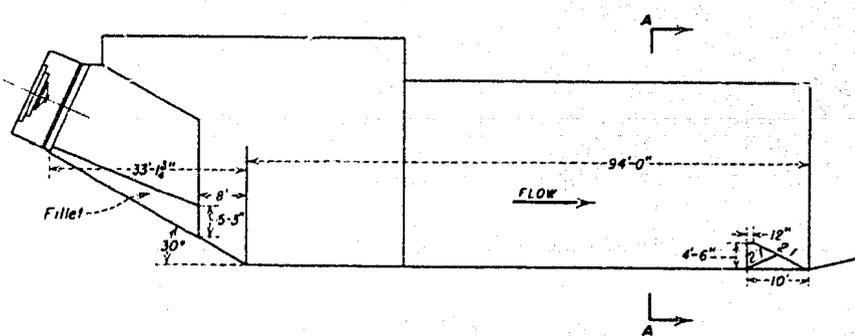


ELEVATION SECTION

DESIGN M-7

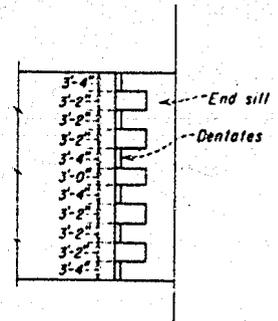


PLAN  
A-A



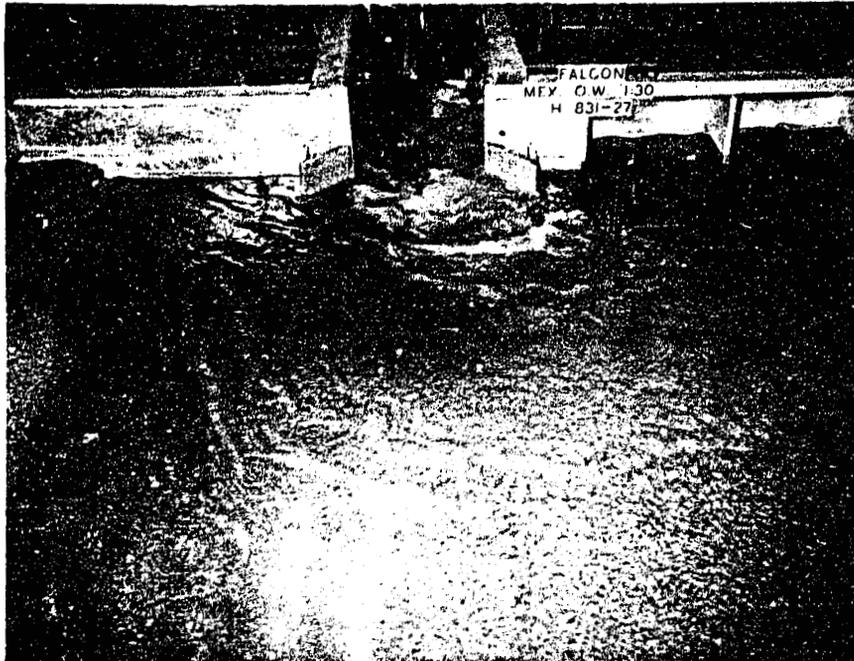
ELEVATION SECTION

DESIGN M-8



PLAN  
A-A

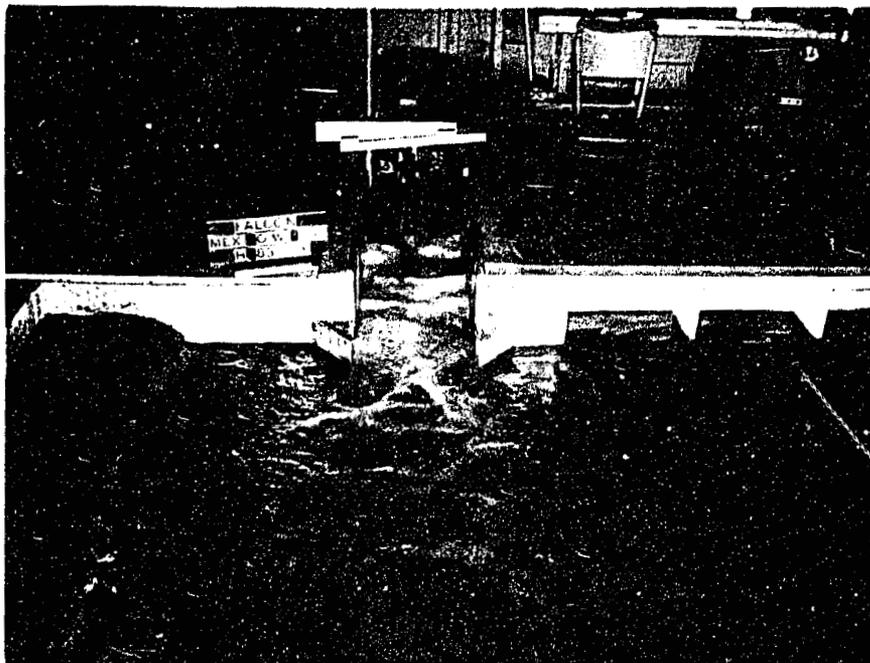
**FALCON DAM**  
HYDRAULIC MODEL STUDIES  
MEXICAN OUTLET WORKS  
STILLING BASIN DESIGNS  
MODEL SCALE 1/30



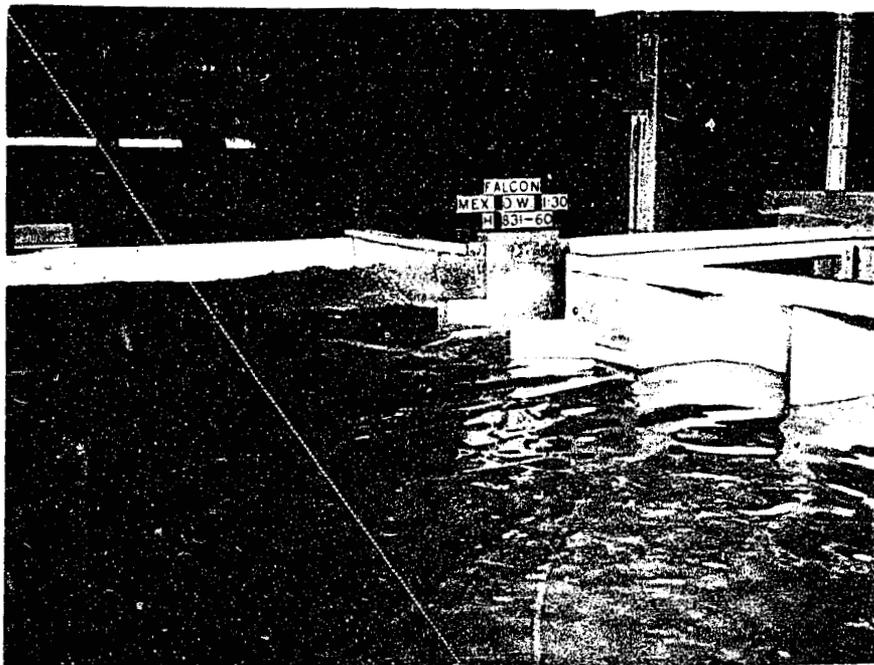
A. One valve discharging 2,400 cfs



B. Erosion after a discharge of 2,400 cfs for 1 hour through left valve. Note that the erosion is below the elevation of the basin floor.



A. Action of the stilling basin. Both valves discharging--  
 $Q = 4,570$  cfs.



B. Water surface downstream from the basin. Both valves  
discharging-- $Q = 4,570$  cfs.

Erosion below the stilling basin after a discharge of 2,400 cfs for 1 hour through left valve. Note the sand deposited to the left of the low dividing wall.

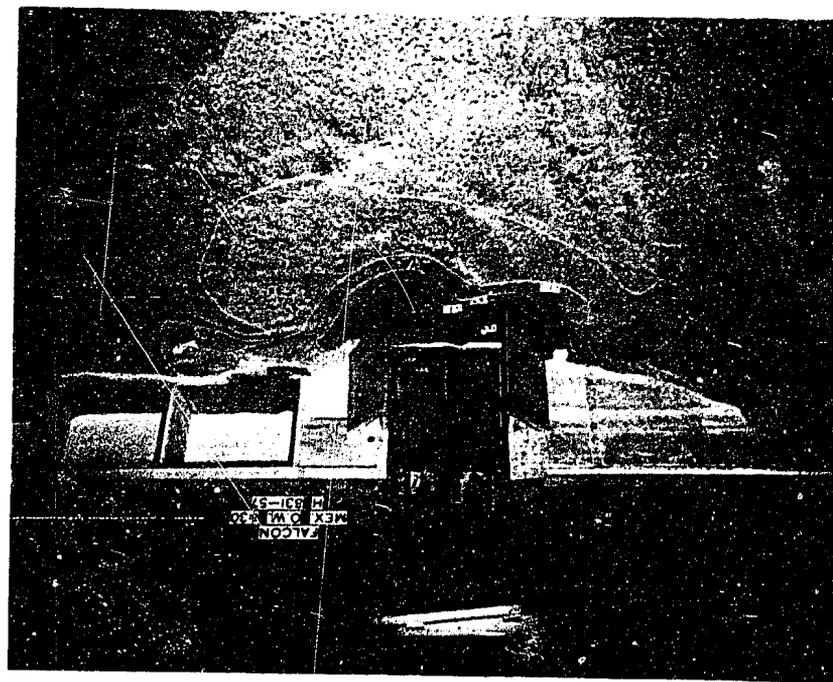
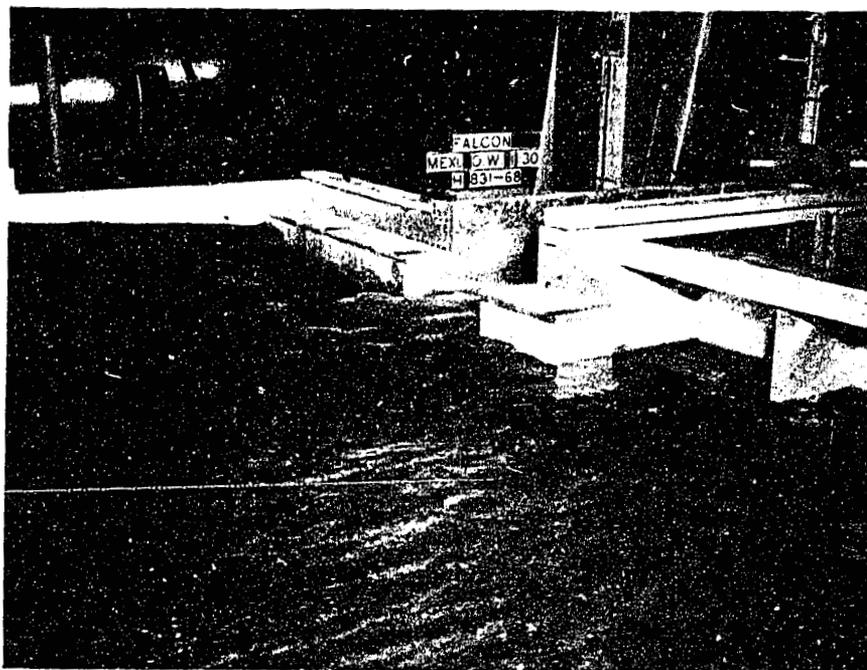


Figure 53  
Hyd. 276

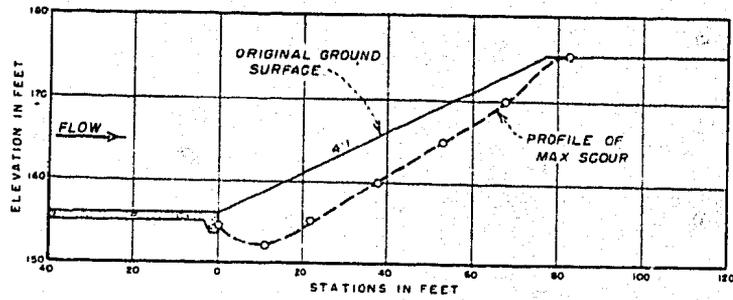


A. Water surface downstream from the stilling basin. Both valves discharging a total of 4,570 cfs.

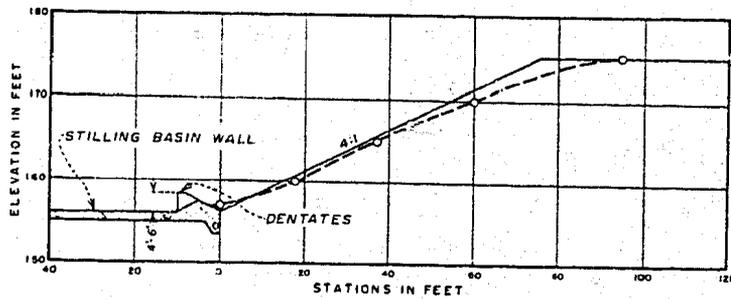


B. Erosion after a discharge of 4,570 for 1 hour through both valves.

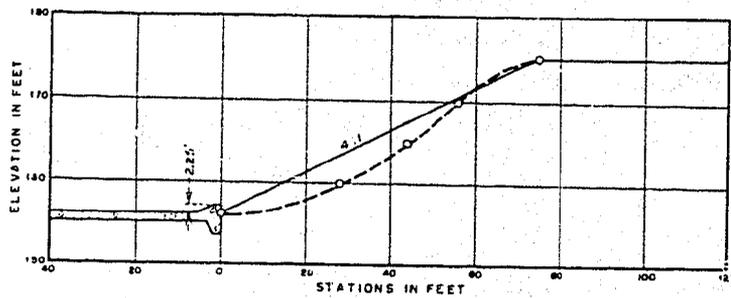




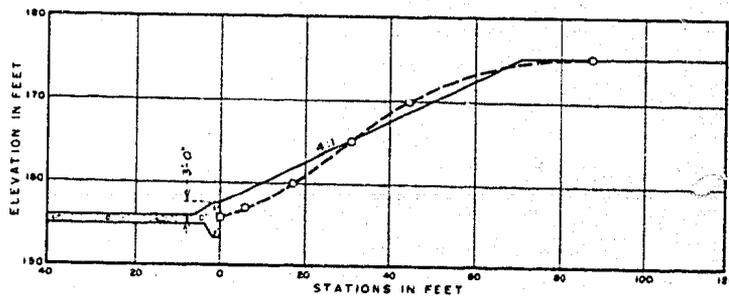
PRELIMINARY DESIGN-NO END SILL



END SILL "A"



END SILL "B"



END SILL "C"

NOTE

PROFILES TAKEN THROUGH POINTS OF  
MAXIMUM EROSION FOR EACH DESIGN.

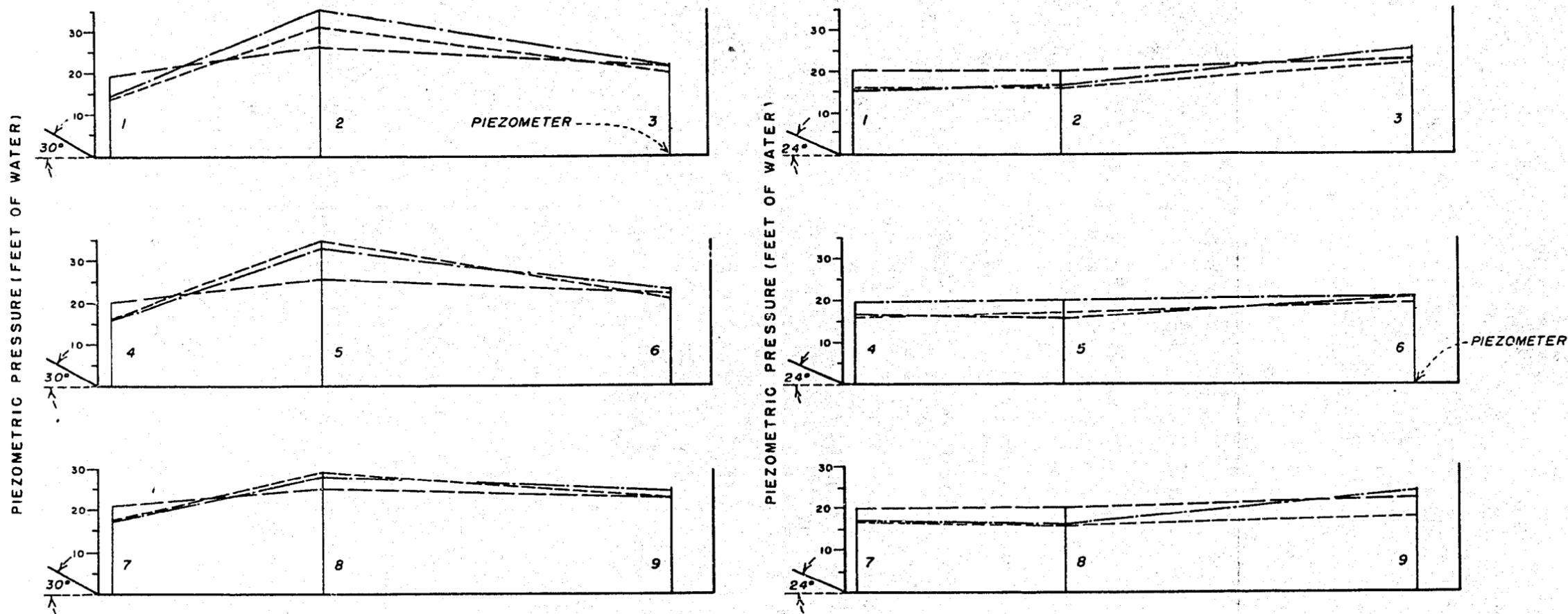
FALCON DAM  
HYDRAULIC MODEL STUDIES  
MEXICAN OUTLET WORKS  
EROSION WITH DIFFERENT  
END SILL DESIGNS  
MODEL SCALE 1:30

369

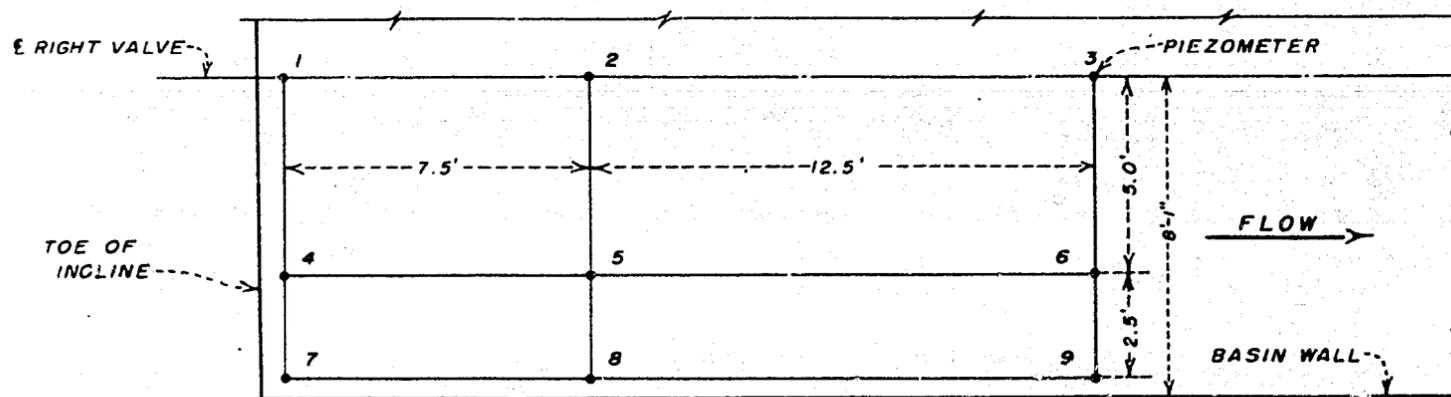
3-30



4,570 cfs discharging through two valves, tailwater elevation 175.0.  
This condition, which will not occur in the prototype, was used in  
determining the hydraulic margin of safety of the stilling basin.



PIEZOMETRIC PROFILES

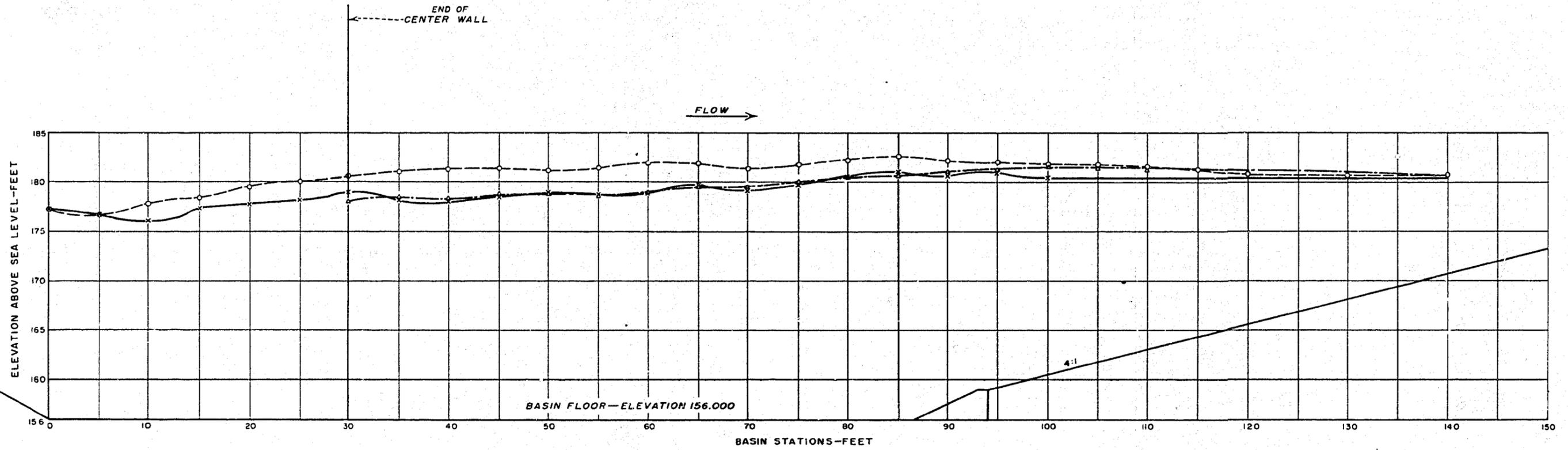


PLAN  
PIEZOMETER LOCATION

EXPLANATION

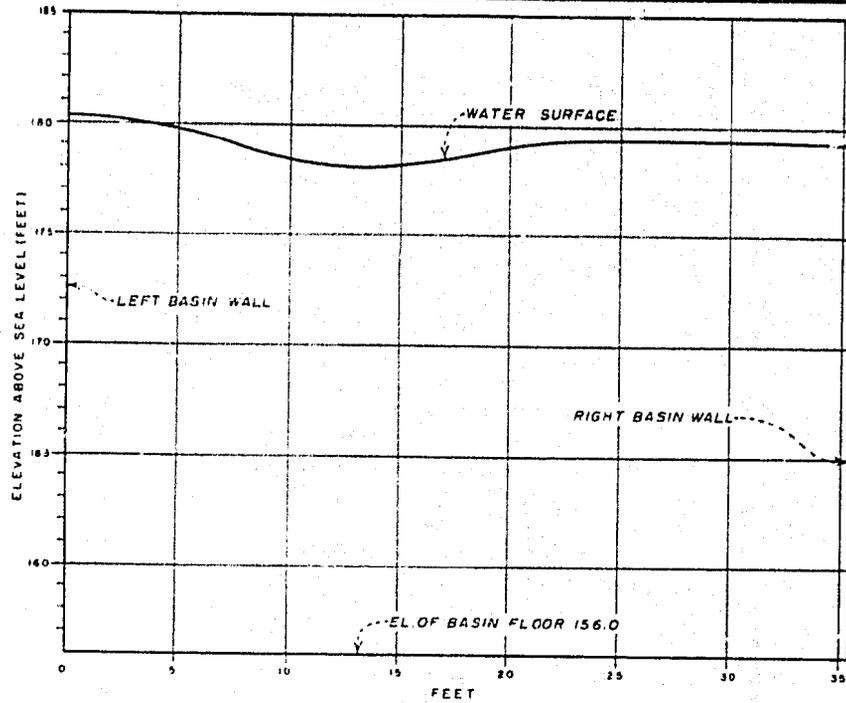
- Q = 4.570 C.F.S. - BOTH VALVES DISCHARGING
- Q = 3.210 C.F.S. - BOTH VALVES DISCHARGING
- Q = 2.400 C.F.S. - RIGHT VALVE ONLY DISCHARGING

**FALCON DAM**  
 HYDRAULIC MODEL STUDIES  
 MEXICAN OUTLET WORKS  
 PIEZOMETRIC PRESSURES  
 ON STILLING BASIN FLOOR  
 MODEL SCALE: 1:30

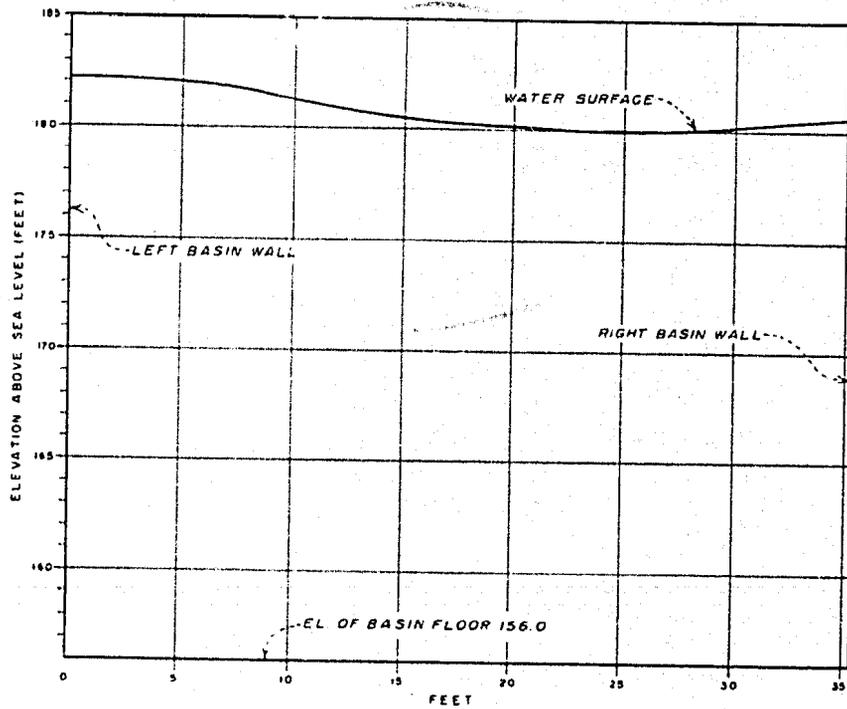


——— Q=4570 C.F.S. BOTH VALVES OPEN—PROFILE ALONG LEFT WALL  
 - - - Q=2400 C.F.S. LEFT VALVE OPEN—PROFILE ALONG LEFT WALL  
 - · - Q=4570 C.F.S. BOTH VALVES OPEN—PROFILE ALONG CENTERLINE OF BASIN.

**FALCON DAM**  
 HYDRAULIC MODEL STUDIES  
 MEXICAN OUTLET WORKS  
 WATER SURFACE PROFILE—RECOMMENDED DESIGN  
 MODEL SCALE 1:30



PROFILE AT END OF CENTER WALL

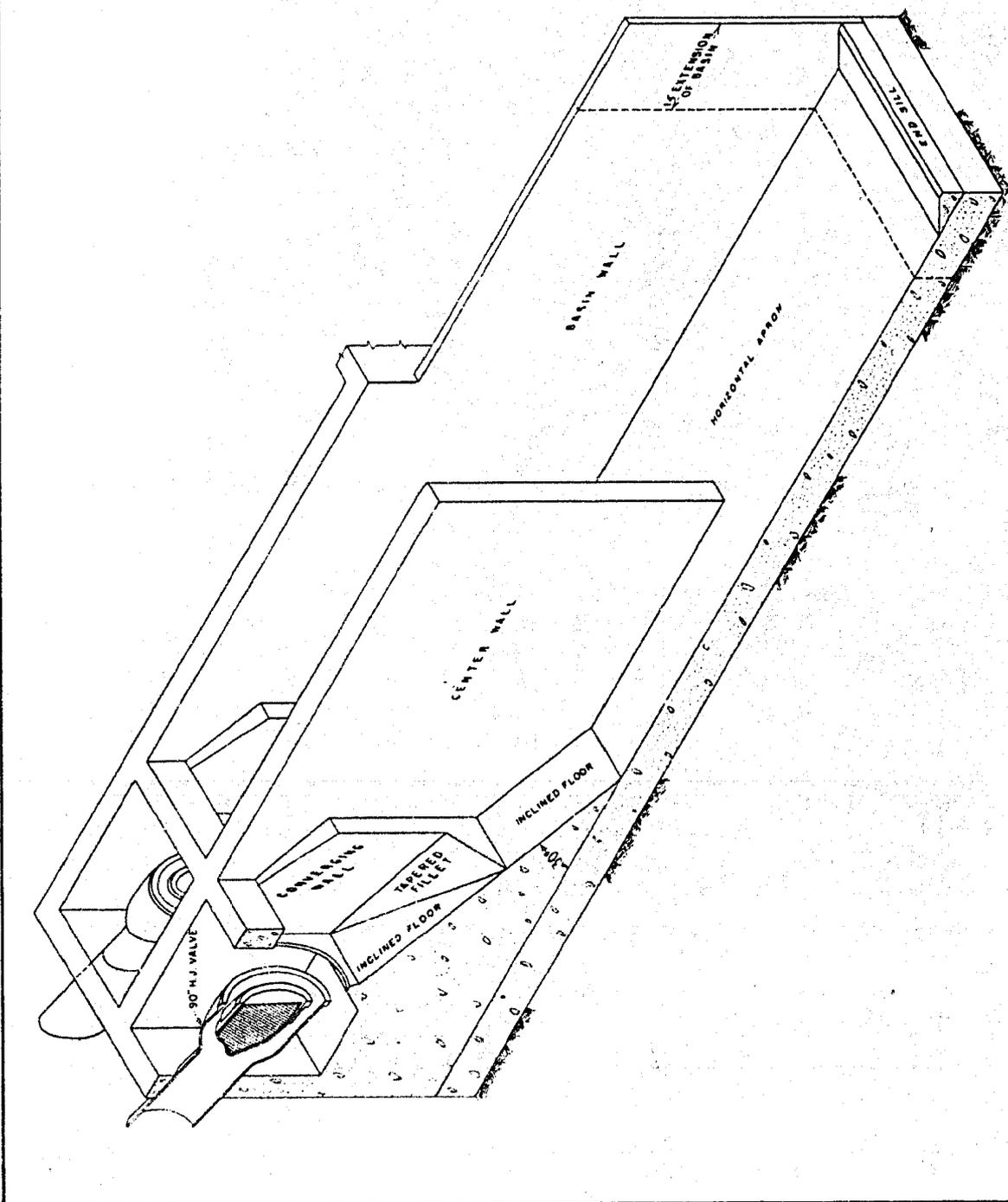


PROFILE AT END OF STILLING BASIN

NOTE  
 Q = 2,400 C.F.S.  
 LEFT VALVE ONLY DISCHARGING

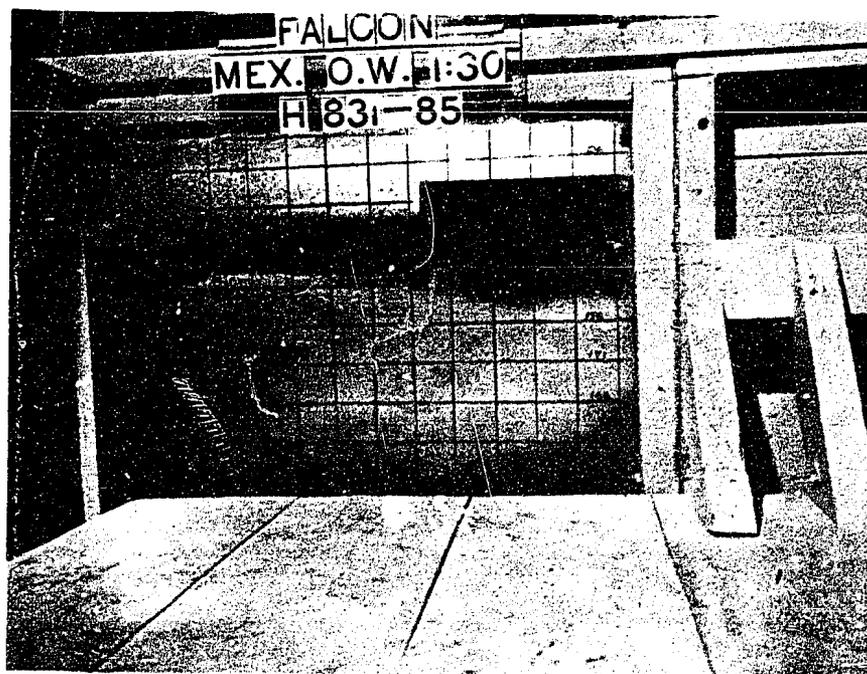
FALCON DAM  
 HYDRAULIC MODEL STUDIES  
 MEXICAN OUTLET WORKS  
 TRANSVERSE WATER SURFACE  
 PROFILES IN STILLING BASIN  
 MODEL SCALE 1:30

FALCON DAM  
HYDRAULIC MODEL STUDIES  
MEXICAN OUTLET WORKS  
STILLING BASIN FEATURES  
MODEL SCALE 1:30





A. The left valve discharging 2,400 cfs.

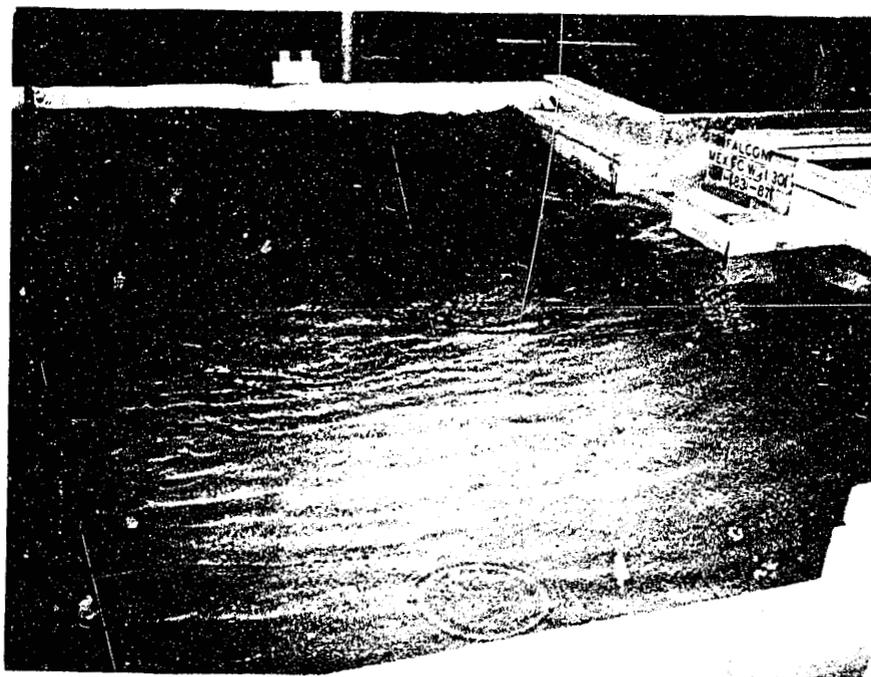


B. Both valves discharging 4,570 cfs.

THE MEXICAN OUTLET WORKS--RECOMMENDED DESIGN.  
WATER SURFACES IN THE STILLING BASIN



A. Action in the basin..

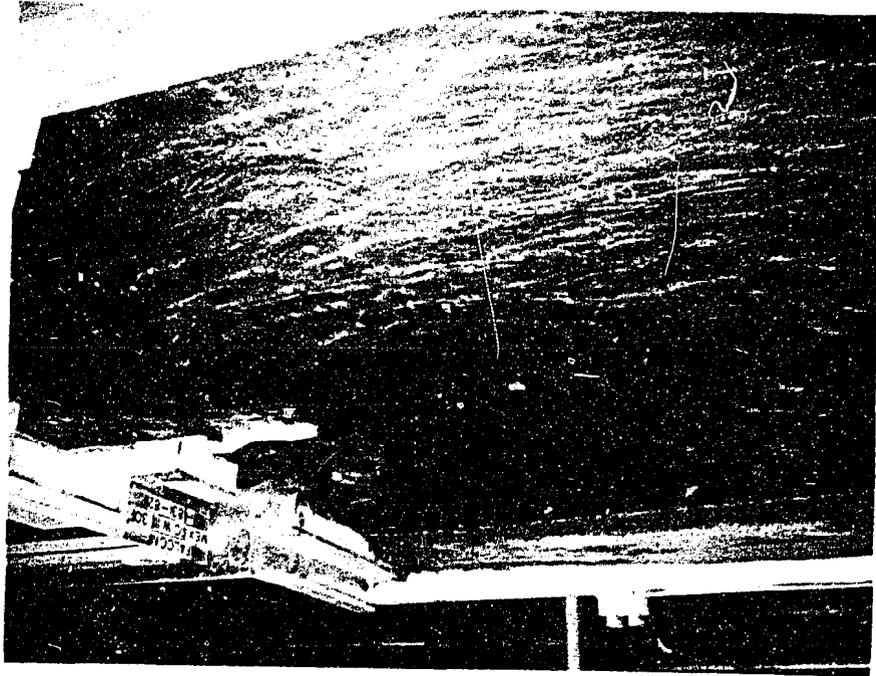


B. Flow in the channel.

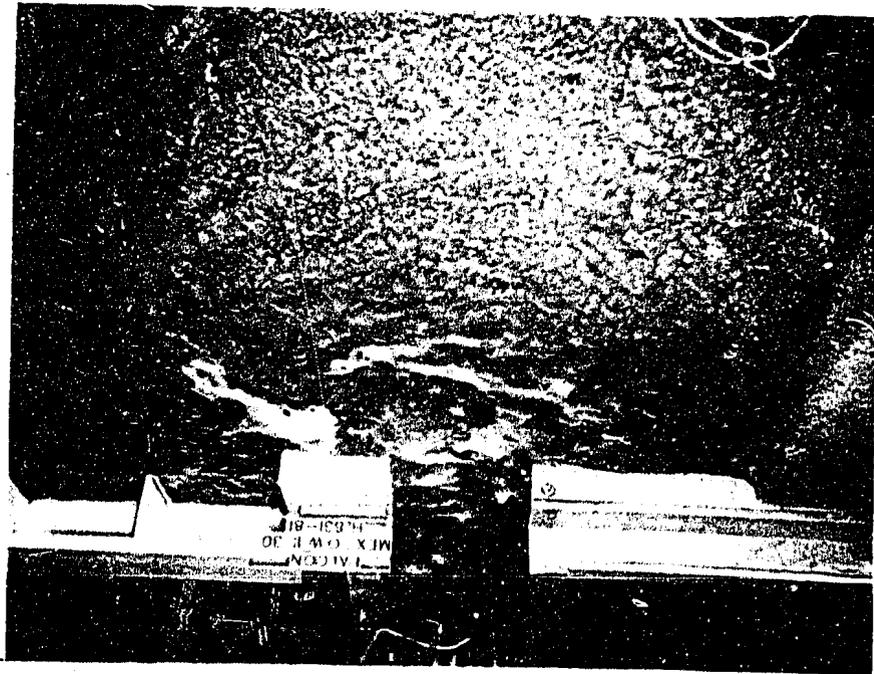
MEXICAN OULET WORKS.--RECOMMENDED DESIGN.  
4,570 CFS, BOTH VALVES DISCHARGING.

MELICAN OVERTAKE WORKS.--RECOMMENDED DESIGN.  
2,400 CFS, DISCHARGING THROUGH LEFT VALVE ONLY.

B. Flow in the channel.

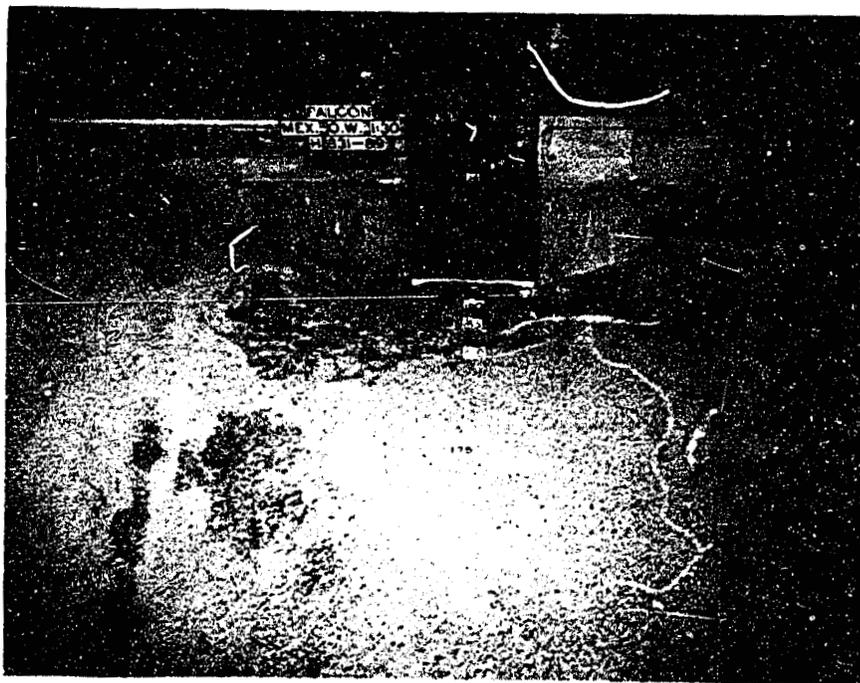


A. Action in the basin.



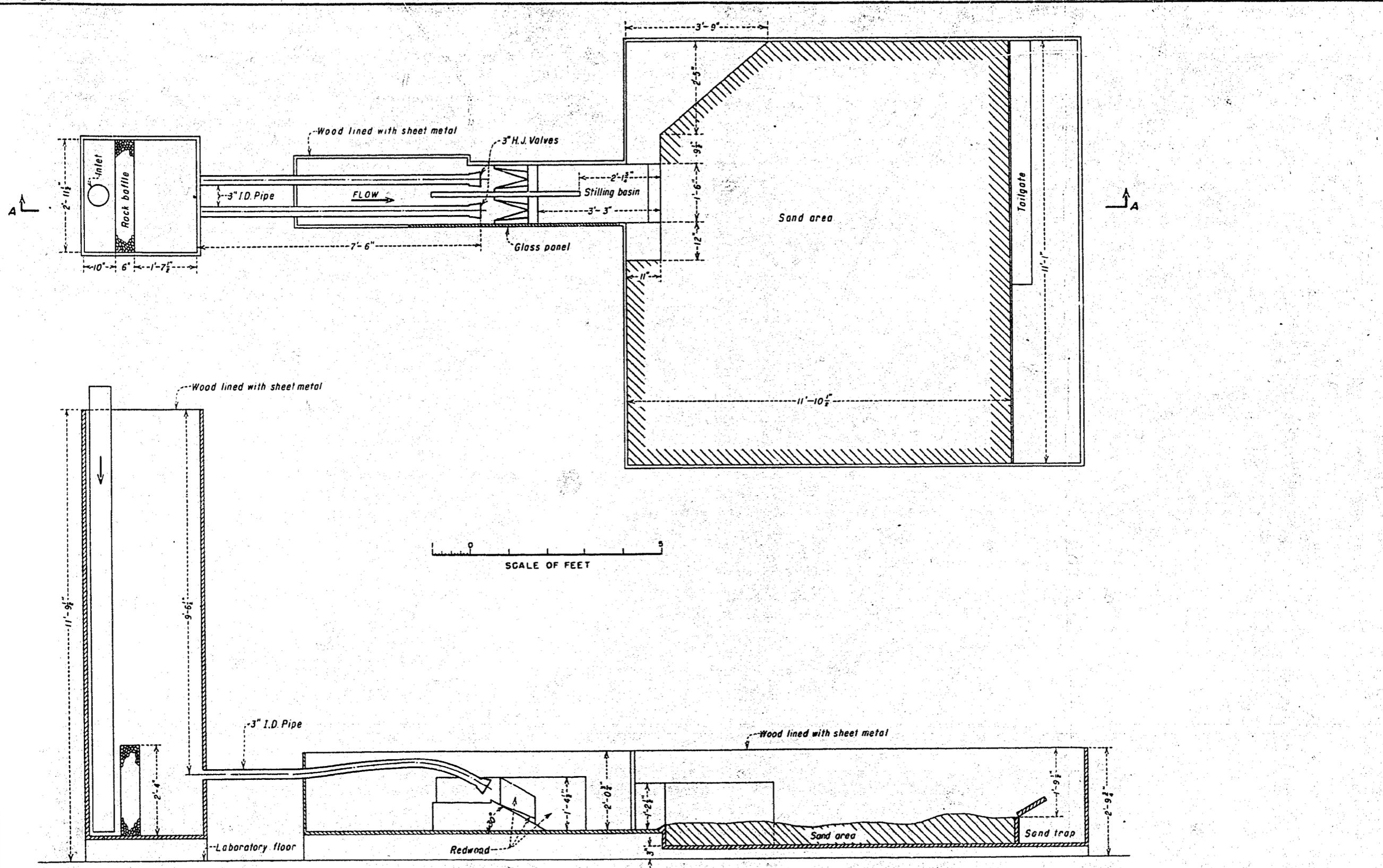


A. Erosion after a discharge of 2,400 cfs for 1 hour through left valve.



B. Erosion after a discharge of 4,570 cfs for 1 hour through both valves.

THE MEXICAN OULET WORKS.--RECOMMENDED DESIGN.  
EROSION DOWNSTREAM FROM STILLING BASIN.



SECTION A-A

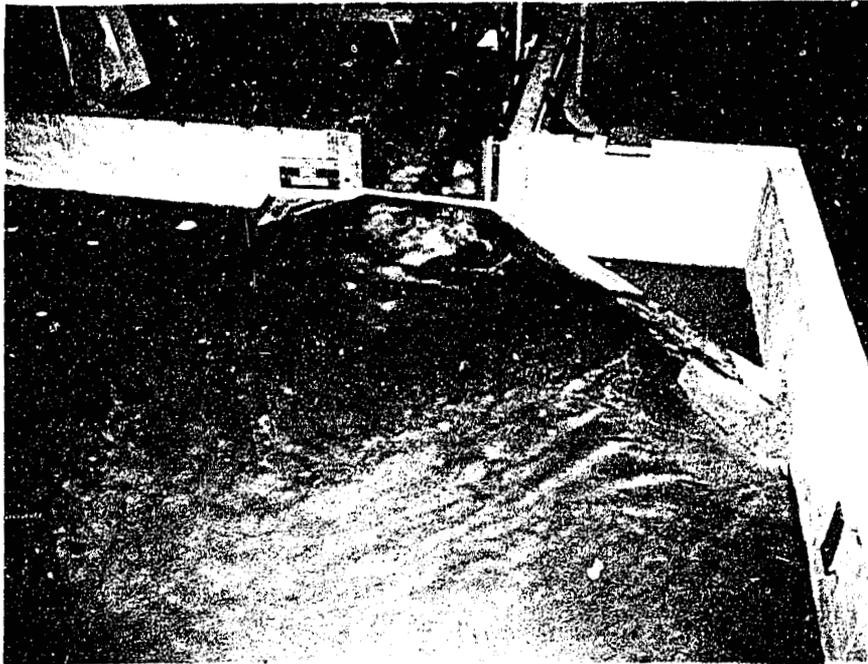
FALCON DAM  
 HYDRAULIC MODEL STUDIES  
 UNITED STATES OUTLET WORKS  
 MODEL ARRANGEMENT  
 MODEL SCALE 1:24

369  
 1-50



Note the two valves, the horizontal floor, the wing wall arrangement, and the sand in tailrace channel.





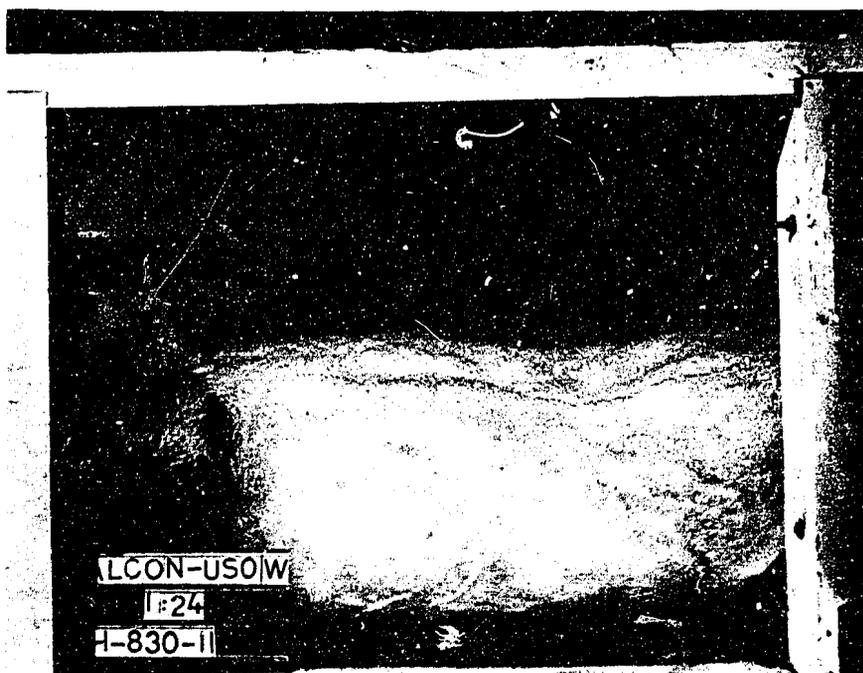
A. Operation of stilling basin. Both valves discharging a total of 2,920 cfs. Note violent water surface.



B. Erosion after a discharge of 2,920 cfs through two valves for 1 hour.

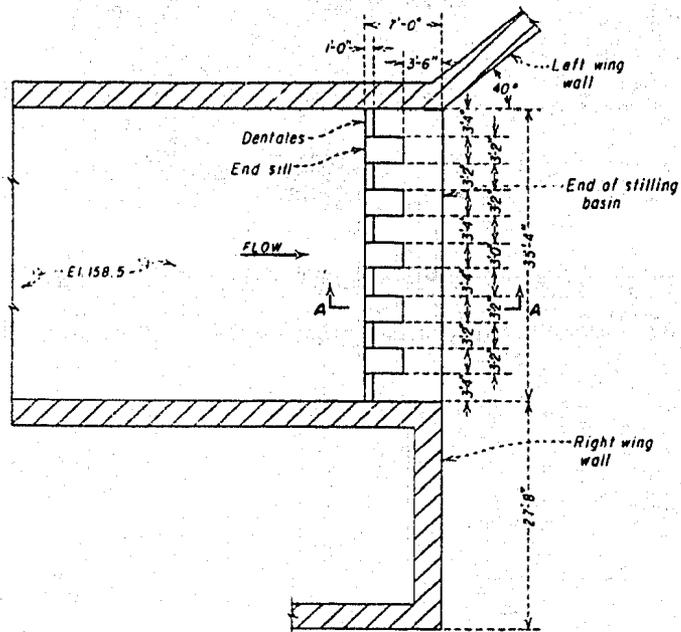


A. Basin floor elevation 160. Note the rough water surface in the basin.

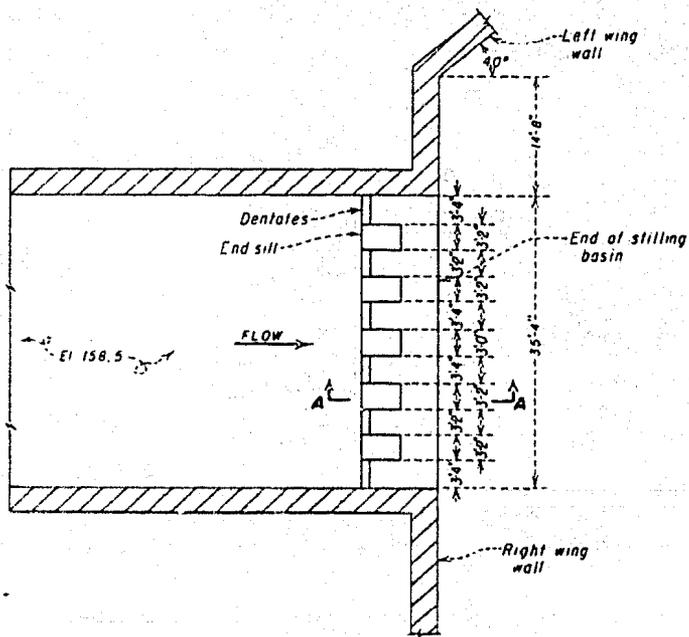


B. Basin floor lowered to elevation 158.5. Note reduction in violence of surface waves.

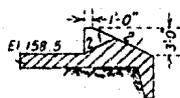
THE UNITED STATES OUTLET WORKS. BASIN PERFORMANCE.  
2,920 CFS THROUGH BOTH VALVES.



SECTIONAL PLAN  
DESIGN A-2



SECTIONAL PLAN  
DESIGN A-3



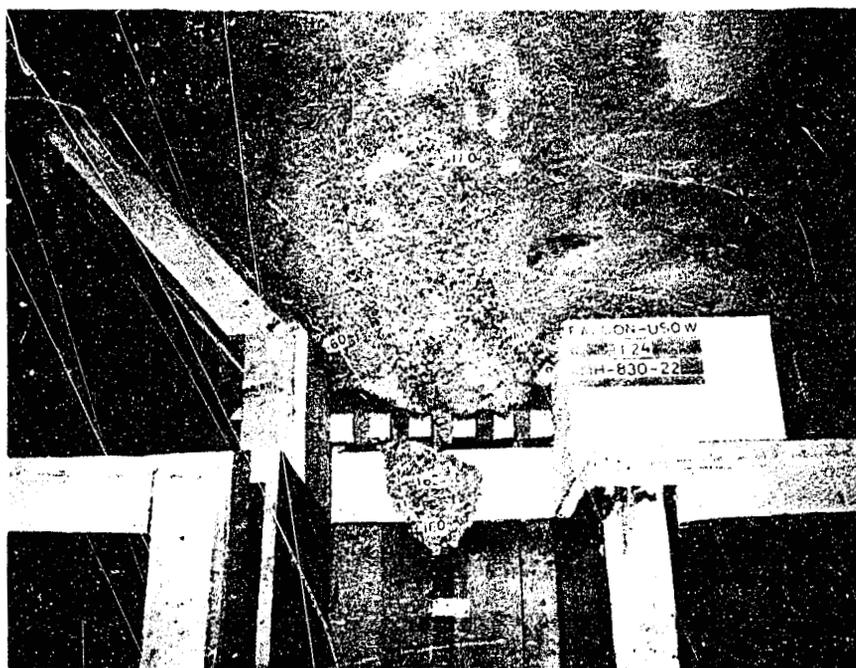
SECTION A-A

Note Upstream end of basin and basin length same as recommended design

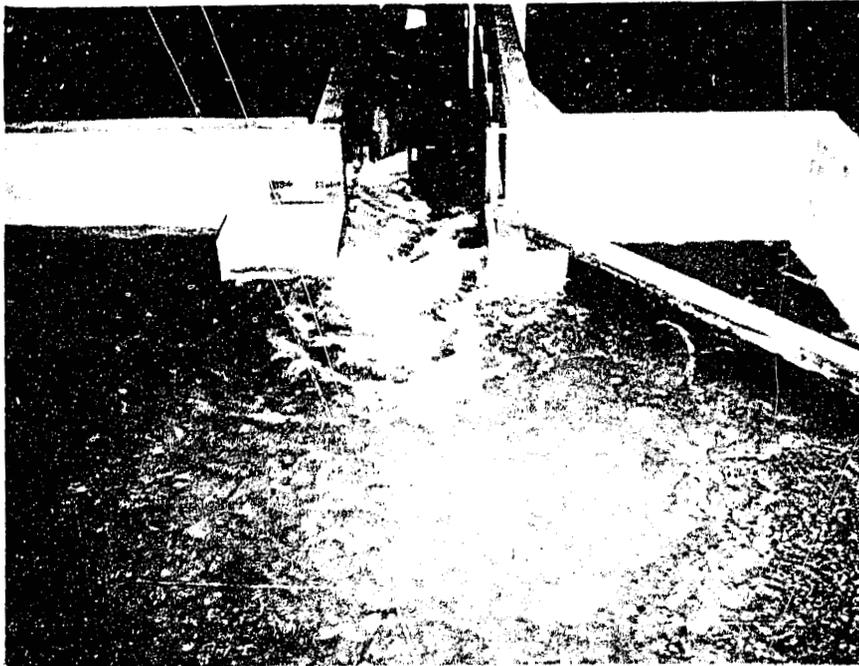
**FALCON DAM**  
HYDRAULIC MODEL STUDIES  
UNITED STATES OUTLET WORKS  
STILLING BASIN DESIGNS  
MODEL SCALE 1:24



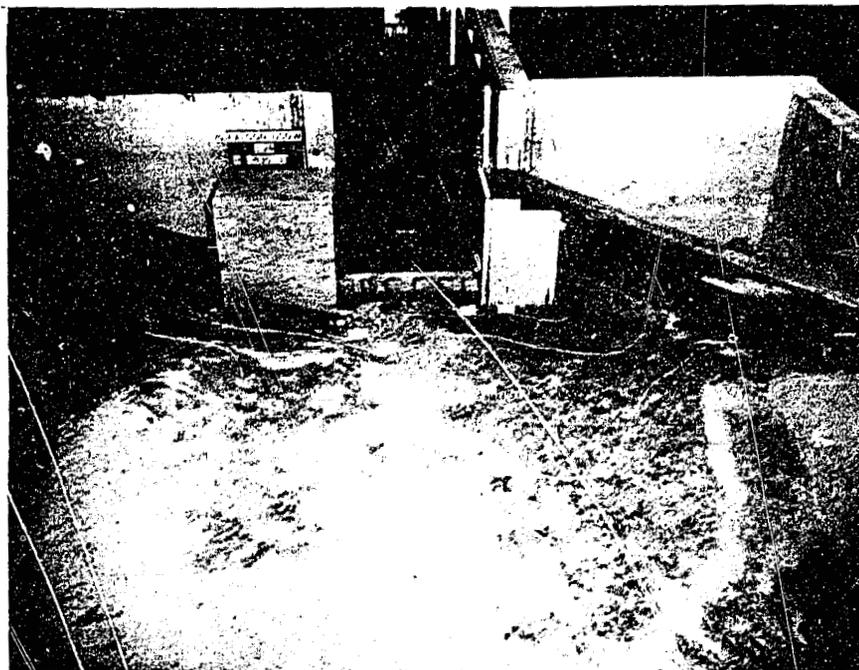
A. Operation of stilling basin. Right valve discharging 1,600 cfs.



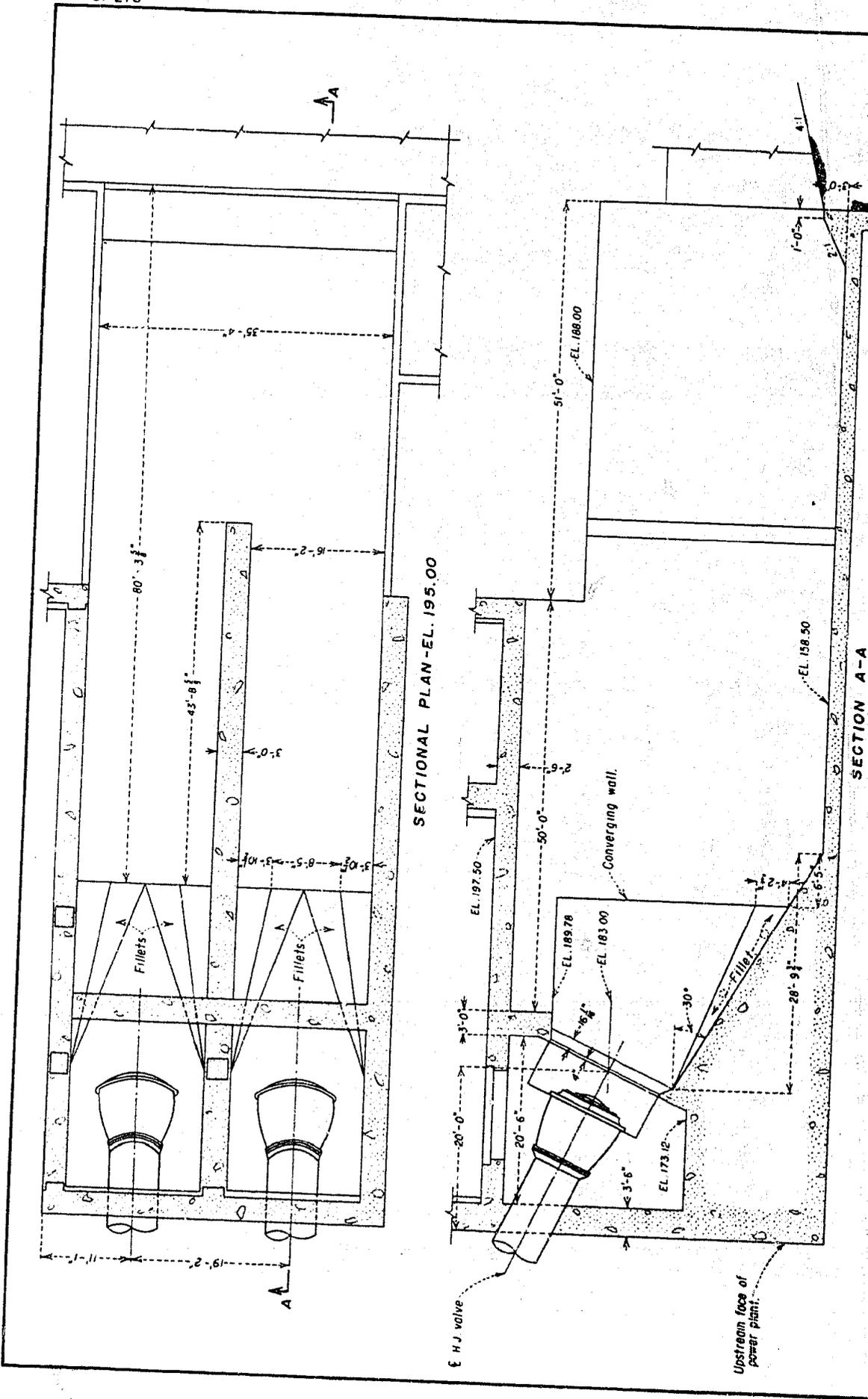
B. Erosion after discharge of 1,600 cfs through right valve for 1 hour. Note sand deposited in basin.



A. Operation of stilling basin. Right valve discharging 1,600 cfs.

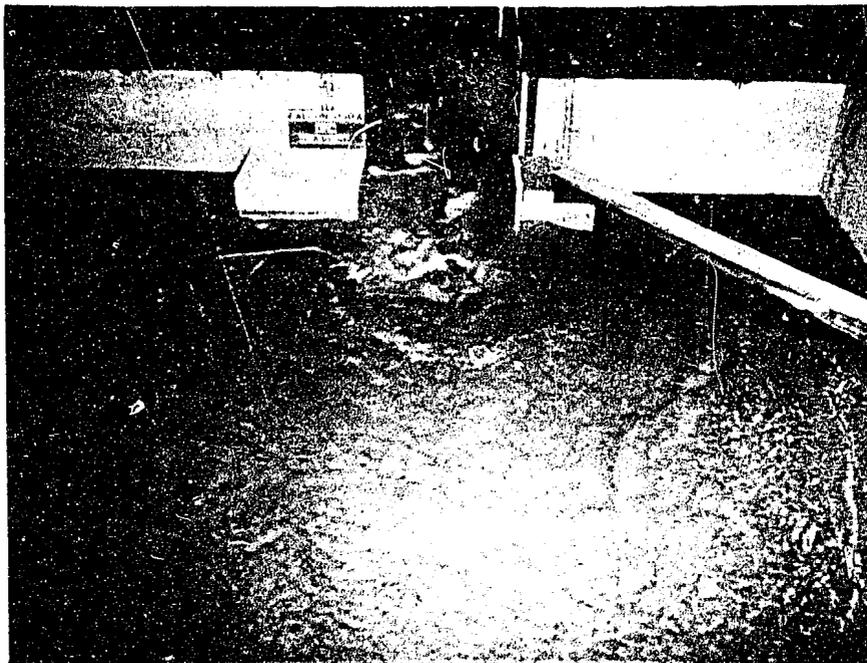


B. Erosion after a discharge of 1,600 cfs for 1 hour through right valve. Note reduction in sand deposited in basin.

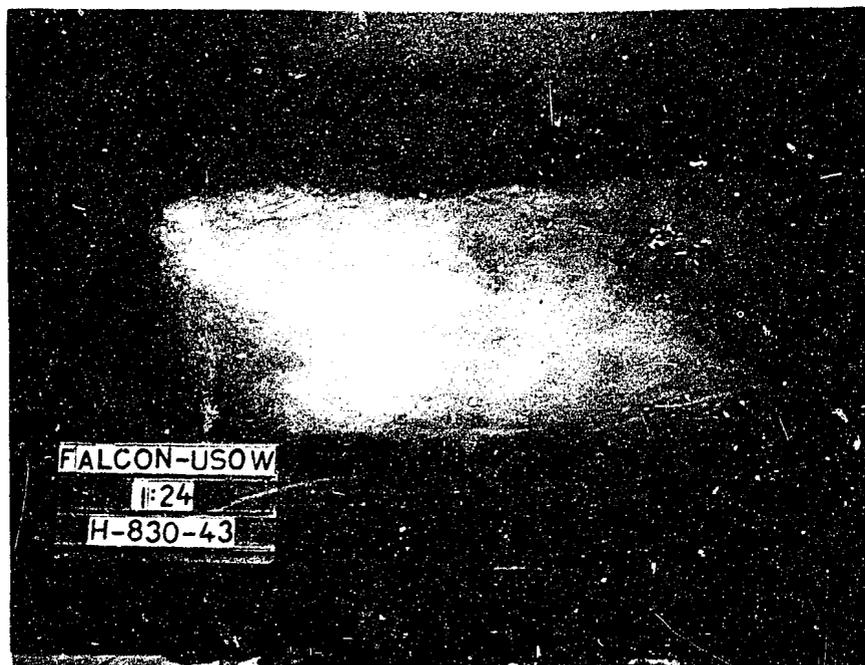


**FALCON DAM**  
 HYDRAULIC MODEL STUDIES  
 UNITED STATES OUTLET WORKS  
 RECOMMENDED DESIGN  
 MODEL SCALE 1:24

369  
 P-50



A. Right valve discharging 1,600 cfs. Note that turbulence is confined to one side of basin.



B. Right side view of basin--right valve discharging 1,600 cfs. Note vertical roll of water below converging wall.

THE UNITED STATES OUTLET WORKS. RECOMMENDED DESIGN.  
OPERATION OF STILLING BASIN.

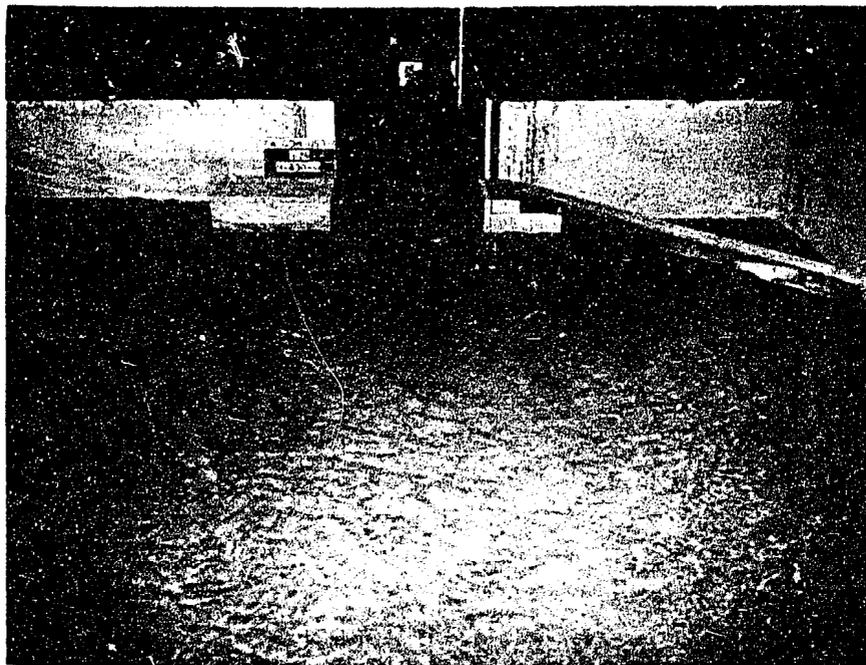


A. Water surface below basin. Right valve discharging 1,600 cfs. Tailwater elevation 180.5.

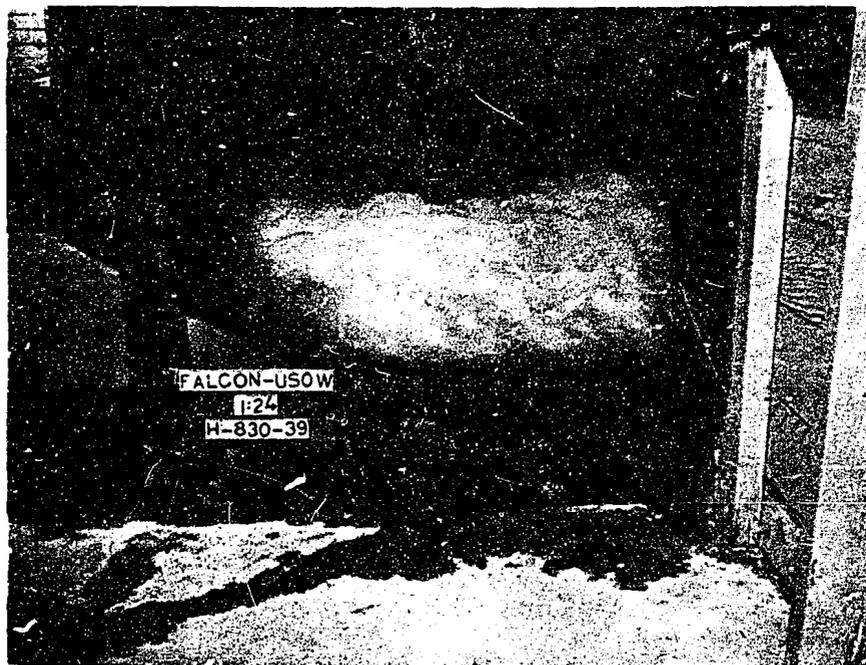


B. Water surface below basin. Both valves discharging a total of 2,920 cfs. Tailwater elevation 180.8.

THE UNITED STATES OUTLET WORKS. RECOMMENDED DESIGN.  
OPERATION OF STILLING BASIN.



A. Both valves discharging a total of 2,920 cfs.  
Tailwater elevation 180.8.

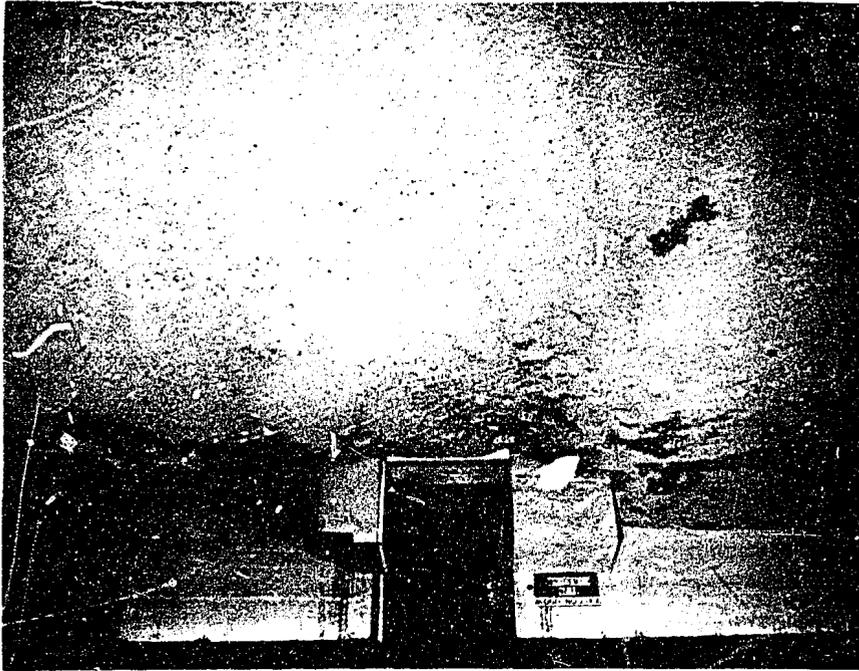


B. Both valves discharging total of 2,920 cfs.  
Note how the flow travels along the floor.

THE UNITED STATES OUTLET WORKS. RECOMMENDED DESIGN.  
OPERATION OF STILLING BASIN.

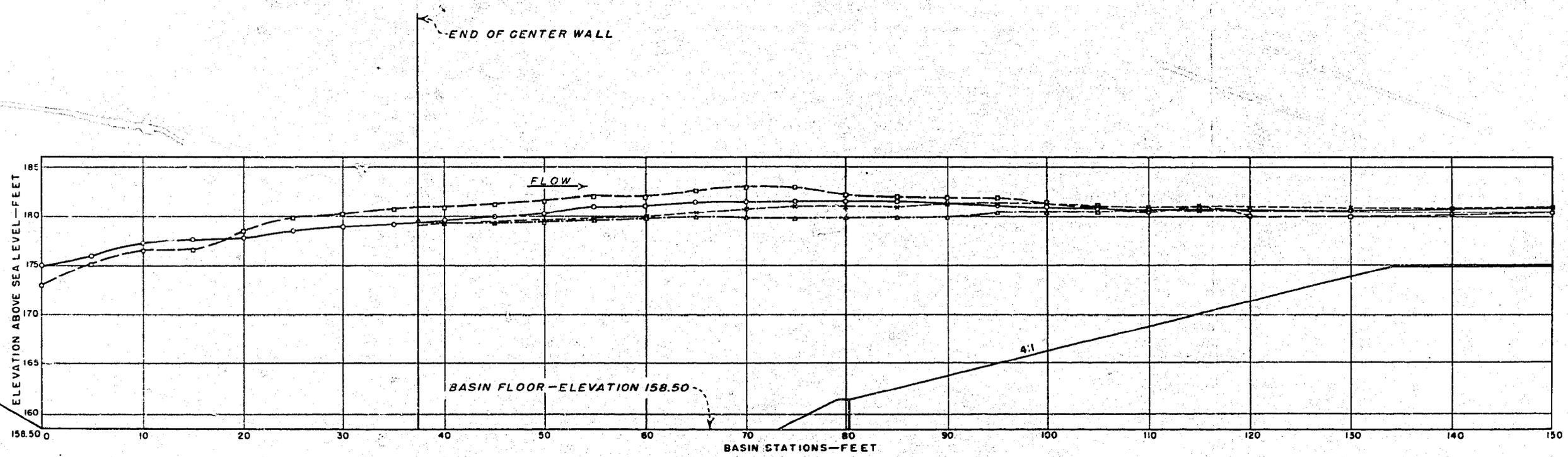
THE UNITED STATES OUTLET WORKS. RECOMMENDED DESIGN.  
EROSION BELOW STILLING BASIN.

B. Erosion after a total discharge of 2,920 cfs  
for 1 hour through both valves.



A. Erosion after a discharge of 1,600 cfs for  
1 hour through right valve.

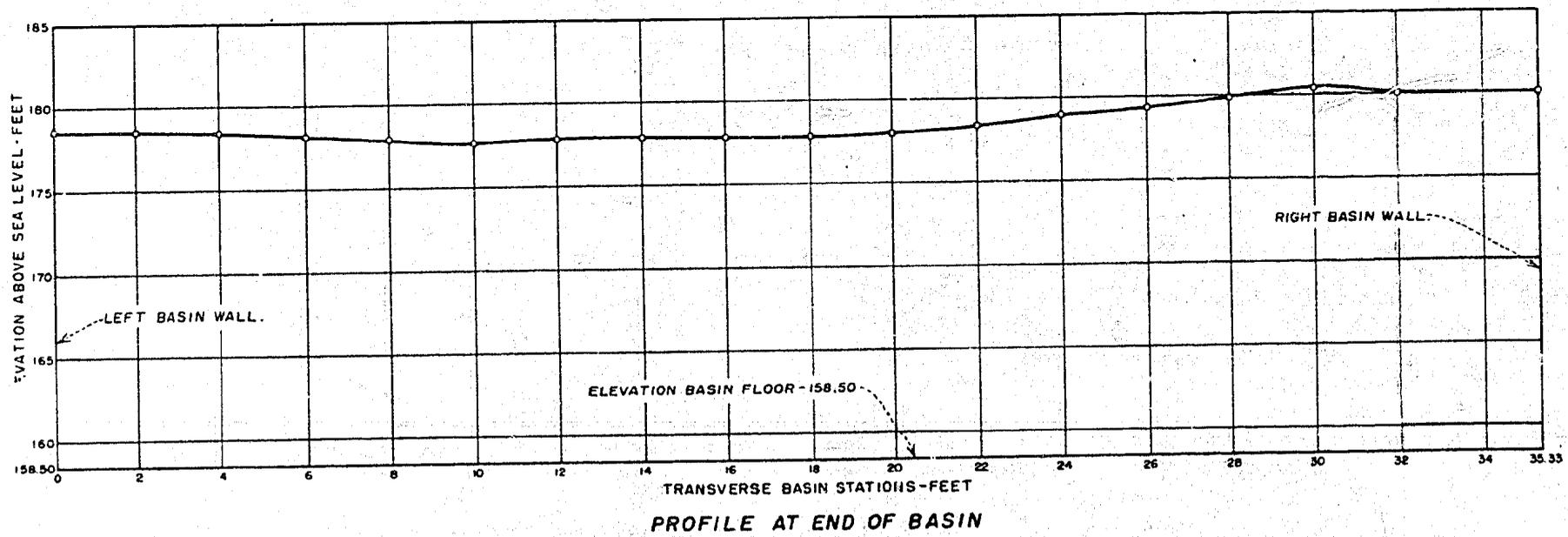
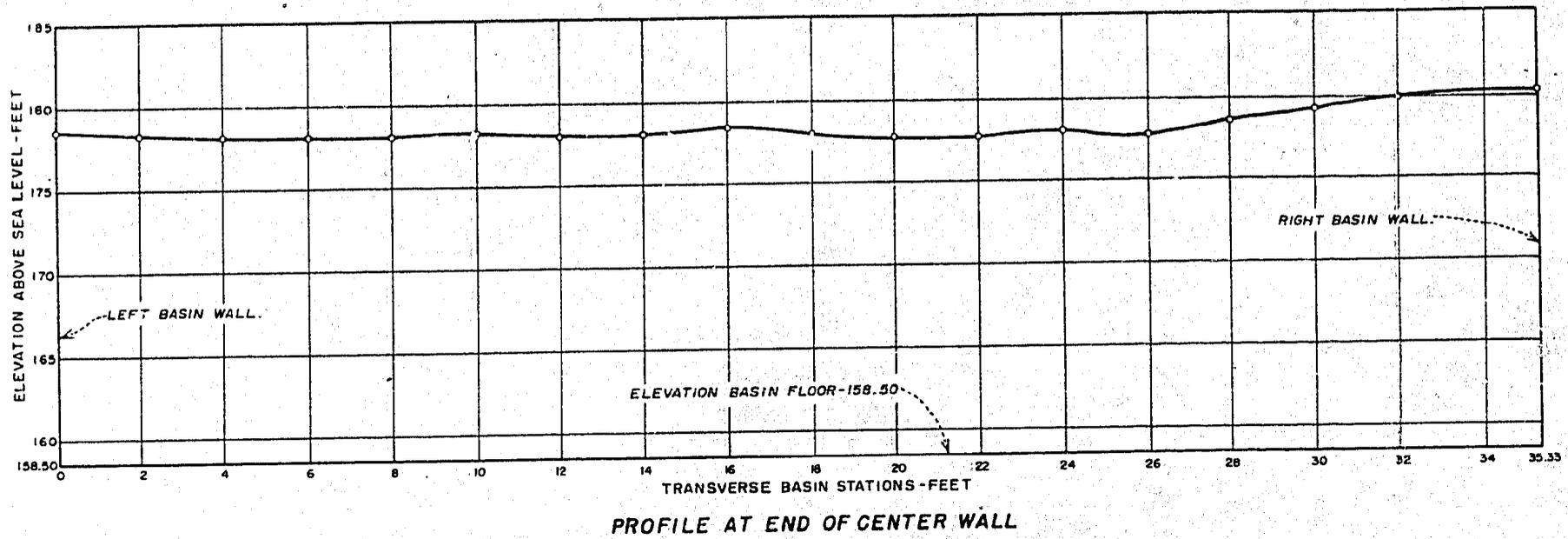




EXPLANATION

- Q=2920 C.F.S. BOTH VALVES OPEN—PROFILE ALONG RIGHT WALL
- -○- - Q=2920 C.F.S. BOTH VALVES OPEN—PROFILE ALONG CENTERLINE OF BASIN
- Q=1600 C.F.S. RIGHT VALVE OPEN—PROFILE ALONG CENTERLINE OF BASIN
- -□- - Q=1600 C.F.S. RIGHT VALVE OPEN—PROFILE ALONG RIGHT WALL

FALCON DAM  
 HYDRAULIC MODEL STUDIES  
 UNITED STATES OUTLET WORKS  
 WATER SURFACE PROFILES  
 RECOMMENDED DESIGN  
 MODEL SCALE 1:24



**NOTE**  
 Q = 1600 CFS - RIGHT VALVE ONLY DISCHARGING.

**FALCON DAM**  
 HYDRAULIC MODEL STUDIES  
 UNITED STATES OUTLET WORKS  
 TRANSVERSE WATER SURFACE  
 PROFILES IN STILLING BASIN  
 MODEL SCALE 1:24