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UNITED STATES
DEPARTMENT OF THE INTERIOR
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HYDRAULIC MODEL STUDIES OF GORGE HIGH DAM SPILLWAY AND OUTLET WORKS

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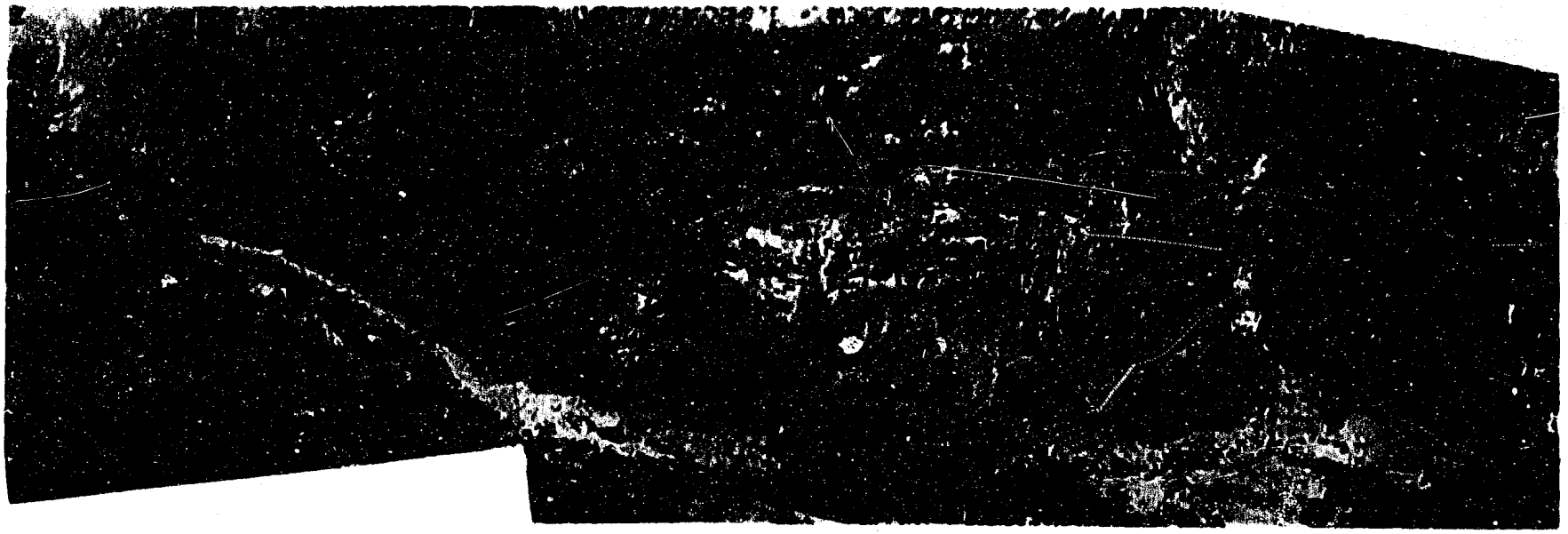

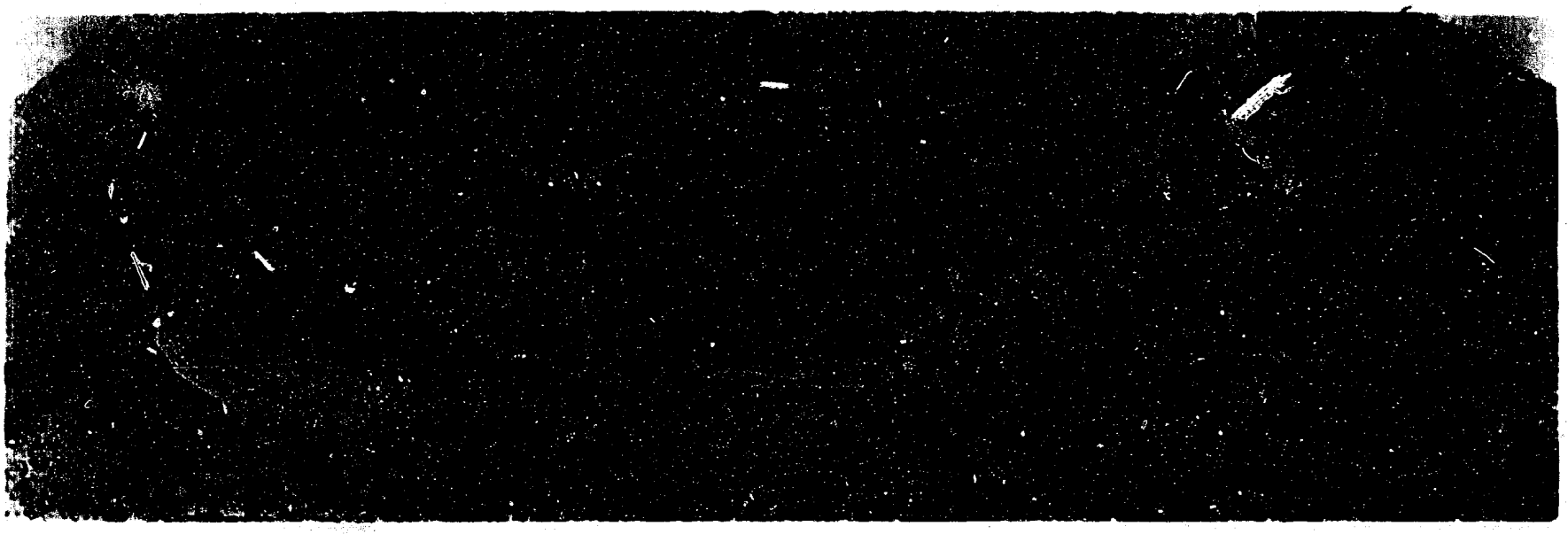

SITE OF GORGE HIGH DAM

PREFACE

Hydraulic model studies of the spillway and outlet works for Gorge High Dam, Skagit Project, Washington, were conducted in the Hydraulic Laboratory of the Bureau of Reclamation at Denver, Colorado, under Contract No. 14-06-D-787 between the City of Seattle, Washington, and the Bureau of Reclamation.

The final plans, evolved from this study, were developed through the cooperation of the staffs of the City of Seattle Lighting Department, Consulting Engineer John L. Savage, and the Hydraulic Laboratory.

During the course of the studies, John L. Savage and his staff frequently visited the laboratory to observe the model studies and discuss test results. Dr. Paul J. Raver, Superintendent of Lighting; Mr. E. R. Hoffman, Consultant; and Messrs. C. E. Shevling, J. M. Nelson, E. R. Schindler, and C. R. Hoidal, Lighting Department, City of Seattle, visited the laboratory on two occasions to observe and discuss the model tests.

The studies were conducted by W. E. Wagner under the supervision of Messrs. J. N. Bradley and A. J. Peterka of the Hydraulic Laboratory staff.

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INTRODUCTION

Gorge High Dam is located on the Skagit River about 2-1/2 miles east of Newhalem and approximately 24 miles northeast of Rockport, Washington, Figure 1. Constructed solely for power purposes Gorge High Dam replaces the existing Gorge Diversion Dam and will provide approximately 110 feet of additional head for power generation. Water from the reservoir impounded by Gorge High Dam will pass through an existing tunnel, 17 feet 9 inches in diameter, to Gorge powerhouse located some 5 miles downstream from the point of diversion.

Gorge High Dam is a combination gravity and arch-type structure approximately 670 feet in length at the top and rising 140 feet above the present riverbed, Figure 2. The principal hydraulic features of the dam are the spillway, power intake, two outlets, and a log sluice.

The overflow spillway, located in the gravity section of the dam, has a crest length of 104 feet and is designed for a maximum discharge of 145,000 second-feet at a head of 55 feet. Flow over the spillway is controlled by two 47- x 50.5-foot fixed-wheel gates. Due to the remote location of the dam, up to 7,000 second-feet may be released in emergencies over the top of the spillway gates when the gates are closed.

The two identical outlets are located to the right of the spillway and are 8 feet 9 inches wide and 8 feet 9 inches high with an arched top. Designed for a maximum discharge of 10,000 second-feet at a head of 110 feet, the outlets discharge directly into the river.

Another feature of the structure is the log sluice, which is located to the right of the spillway and outlets, near the top of the dam. Its purpose is to release logs and surface debris from the reservoir when the reservoir pool is near the top of the spillway gates. After passing through the sluice, the logs drop down the downstream slope of the gravity section of the dam.

The power intake is located approximately 190 feet upstream from the left dam abutment. Flow into the tunnel is controlled by a 20- x 25-foot fixed-wheel gate. The power intake is connected to the existing tunnel to Gorge Powerplant by constructing a new adit about 140 feet in length.

Hydraulic model studies were conducted to develop the hydraulic design of the spillway, to determine the flow characteristics of the outlets and log sluice, to study the approach conditions at the power intake when the spillway is operating, and to improve the flow characteristics of each of the features, wherever possible. Two hydraulic models were used in the studies: An over-all model including all the hydraulic features of the dam and a larger scale model of one of the outlets. The over-all model was built primarily to study the flow characteristics of the spillway. However, a general study of the outlets and log sluice, such as the properties of the flow leaving the outlets and log sluice, was made using the over-all model. To permit a more detailed study of the outlets, a separate model of one outlet was constructed to a larger scale. To avoid confusion in presenting the results, each of the hydraulic models is discussed separately in this report.

PART I--SPILLWAY STUDIES

THE 1:48 SCALE MODEL

An over-all model, including all the principal hydraulic features of Gorge High Dam, was built to a geometrical scale of 1:48, Figures 3 and 4. The model consisted of a head box, containing topography for a distance of 380 feet upstream from the dam, the gravity section of the dam and most of the arch dam, and a tail box containing topography below elevation 800 feet for a distance of 1,050 feet downstream from the dam. The spillway, trashrack structure of the power intake, outlets and log sluice were accurately represented in the 1:48 model, including all the hydraulic appurtenances of these features except the outlet gates and the log sluice gate. Accurate reproduction of the sluice and outlet gates was deemed unnecessary since the outlets will operate only when the gates are fully open and no hydraulic problems are anticipated in the log sluice which operates with a free water surface at a maximum head of only 7 feet.

The spillway was constructed by screeding neat cement to metal templates which were accurately cut to the ogee shape, Figure 5. A row of piezometers, spaced 8 to 12 feet apart, was placed along the center line of the right spillway bay. The training walls and piers were cut and formed from plywood and sugar pine, while the spillway gates, outlets and log sluice were fabricated from sheet metal. The prototype topography was reproduced in the head and tail box by pouring a thin layer of roughened concrete over metal lath tacked to wooden contours, Figure 5B. Depending on the irregularity of the topography, the contours were placed at 10- to 30-foot intervals.

Initially the downstream river channel was constructed with concrete similar to the other topography. Later in the investigation when studies of the erosive effects of the spillway flow were desired, the stable concrete channel was replaced with a movable bed consisting of 3/8- to 1-1/2-inch gravel.

Water was supplied to the model by a 12-inch horizontal centrifugal pump and measured through a bank of 4 to 12-inch venturi meters.

THE INVESTIGATION

General

The spillway development studies were concerned primarily with the entrance conditions to the spillway, the distribution of flow down the spillway, and the action of the spillway flow as it entered the downstream river channel. The entrance conditions in the preliminary design were such that a large contraction formed at the right pier causing a reduction in the discharge coefficient and an uneven flow distribution in both the right and left spillway bays. Flow leaving the spillway was rough and impinged on the left river bank before entering the main river channel.

Some improvement of the approach conditions was accomplished by constructing an approach channel upstream from the spillway and backfilling the excavation holes at the dam. Further improvement of the flow conditions in the spillway was achieved by changing the shape of the right pier. Eleven different pier designs were tested before satisfactorily reducing the contraction and improving the flow distribution along the right training wall, Figure 6. The adequacy of each pier design was determined by comparing the measured discharge for each pier with the theoretical discharge, by comparing the depth of contraction at each pier, and by observing the flow distribution in the right spillway bay.

After the spillway approach had been improved and a satisfactory right pier had been developed, detailed studies of the spillway flow were undertaken. Eleven spillway designs were tested, Figures 7 through 9. The various designs differed in the alignment and width of the spillway and in the type of energy dissipating device, which included two jump-type basins and several superelevated flip buckets. Each of the designs was evaluated by the action of the flow as it entered the downstream channel, including the flow distribution in the stilling basin or flip bucket, the effect on the left river bank, side eddies, and the manner in which the flow entered the river channel.

General studies were also made on the outlet works and log sluice with the 1:48 model. The outlet works studies were limited to observation of the flow leaving the outlets, since a detailed study of

the flow through the outlets was conducted on a larger scale model, page 21. Flow through the log sluice was studied by investigating the approach conditions to the sluice and the flow down the face of the dam.

The Spillway Approach

In the preliminary design, which is shown in Figures 2 and 10A, no provision was made to channel the flow from the reservoir to the spillway. The main river channel or deepest part of the reservoir is located approximately 150 feet to the right of the spillway; thus, most of the flow approached the spillway from the right creating an undesirable contraction at the right pier and causing surface boils along the shoreline between the spillway and power intake. The contraction at the right pier is discussed in detail under Pier Studies, page 5. Water entering the left spillway bay was very rough, Figure 10A. Water "piled-up" on the center pier and the flow distribution through the left bay was very uneven. The surface boils along the waterline and the poor flow distribution were attributed to the general lateral approach of the flow from the right and the deep excavation hole immediately upstream from the spillway. The excavation hole caused the flow to accelerate nonuniformly from the reservoir to the spillway.

In Figure 10B, a large cut was made in the canyon wall upstream from the power intake to permit more flow to approach the spillway in a direct line. Although not shown in the photograph, the excavation hole upstream from the spillway was filled to elevation 750. These changes greatly improved the flow through the left spillway bay, Figure 10B. The "pile up" of water on the center pier was reduced and the surface of the water entering the spillway was comparatively smooth.

As a result of the first stilling basin studies, discussed on page 9, the spillway was moved 20 feet toward the river channel to improve the flow entering the downstream river channel. Also an approach channel which is required during diversion was placed in the model, Figure 29. These modifications helped to improve the spillway approach conditions.

Following these design changes, the model was operated with the excavation holes upstream from the dam unfilled, Figure 11A. The flow entered the spillway in a more direct line; however, the water surface along the upstream face of the dam and between the spillway and power intake was comparatively rough and indented with small boils of water, as indicated by the absence of confetti near the dam, Figure 11A.

In Figure 11B, the excavation holes were backfilled to elevation 750. The reservoir surface along the upstream face of the dam is comparatively smooth and the water approaching the spillway is

more uniform as evidenced by the improved flow condition at the center pier.

Therefore, it is recommended that the excavation holes immediately upstream from the dam be backfilled to approximately 750 feet.

Although the tunnel to the powerplant was not constructed in the model, the appearance of the flow in the vicinity of the power intake when the spillway is operating indicated that the spillway flow will cause no adverse entrance conditions at the power intake.

Right Spillway Pier Studies

General. Filling the excavation holes, moving the spillway, and constructing the approach channel improved the flow conditions at the right spillway pier by reducing the turbulence in the approaching flow. However, due to the general lateral approach of the spillway flow, the large contraction at the right pier still formed. The contraction caused a reduction in the capacity of the spillway and an irregular and fluctuating water surface down the spillway face. To overcome these undesirable flow conditions, 11 pier designs were tested, Figure 6.

Preliminary pier. The maximum depression in the water surface caused by the contraction, measured from the top of the pier, was 38 feet, Figure 12. After recovering from the contraction, the flow along the right training wall continued to rise forming a fin of water which rose above the training wall in the lower reaches of the spillway. Numerous "ropes" of water formed on the surface of the water flowing down the spillway. The decrease in maximum discharge using the theoretical discharge as a reference was 5.9 percent.

Pier No. 2. The preliminary pier was cut diagonally in elevation and rounded from the lower upstream end to a point on top of the pier 2 feet downstream from the axis of the crest, Figure 6. This change moved the contraction downstream making the minimum cross-sectional flow area form downstream from the crest of the spillway. Near the right training wall, the contraction was so prominent that the surface of the spillway at the gate seat could be seen. Flow down the spillway was rough, Figure 12B, but no fin of water formed along the right training wall as in the preliminary design. For Pier No. 2, the maximum depth of contraction measured from the top of the pier was 57 feet and the percent decrease in discharge was 8.5.

Pier No. 3. With the same elliptical curve on the upstream face as the preliminary pier, Pier No. 3 had a 16-foot overhang between elevations 862.5 and 882.5 feet, Figures 6 and 13. This pier design caused an unstable contraction at maximum discharge. At one moment the contraction would have a depth of 23 feet, Figure 13A, then an unstable flow condition would develop and the water would fall away

from the pier making a maximum contraction of 44 feet, Figure 13B. When the contraction was approximately 23 feet deep, the flow down the spillway was fairly uniform. However, when the contraction increased to 44 feet, the water was extremely rough and overtopped the right training wall, Figure 13B. The decrease in discharge was 2.8 percent. Although no pressure tests were conducted, the unstable flow condition at the pier was probably due to the formation of low-pressure regions along the face of the pier. When the pressures become sufficiently low, the flow separated from the pier providing aeration to the low-pressure region. With the increase in pressure from aeration, the water surface rose on the pier until the low-pressure region again formed, thus causing the alternately high and low water surface along the pier.

Pier Nos. 4, 5, 6, and 7. In an effort to eliminate the unstable contraction, several variations of Pier No. 3 were constructed and tested as Pier Nos. 4 through 7, Figure 6. The performance of Pier Nos. 4 and 5 were very similar in operation to Pier No. 3. The unstable contraction still formed with approximately the same extreme depths and decrease in discharge. Figure 14A shows the operation of Pier No. 4 at maximum discharge when the contraction was a minimum.

Although the contraction was stable when Pier No. 6 was installed, the depth of contraction was 40 feet with a high fin of water forming along the right training wall and a decrease in discharge of 5.3 percent.

With Pier No. 7 installed, the maximum contraction of 35 feet was well upstream from the axis of the crest and was reduced to 21 feet as it passed over the spillway crest. Flow down the spillway was satisfactory with no appreciable fin along the training wall. However, the decrease in discharge was 3.8 percent.

Pier No. 8. Since the previous pier designs accomplished little in improving the flow conditions, Pier No. 3 was thickened from 9 to 14 feet with a more gradual curvature on the upstream face of the pier, Figure 6. The performance of Pier No. 8 was much improved over the previous designs. The contraction was stable at 19 feet below the top of the pier and, except for a small fin along the training wall, the flow down the spillway was comparatively smooth and uniform, Figure 14B. The discharge over the spillway was only 1.3 percent less than the theoretical discharge.

Pier No. 9. To determine whether the shape of the pier as well as the thickness was a contributing factor in reducing the contraction, and to simplify the pier construction if possible, Pier No. 9 (Figure 6) was tested in the model. The shape of Pier No. 9 was such that the overhang increased linearly from zero at elevation 818.08 to 17 feet at the top of the pier. Although the flow conditions using Pier No. 9 were stable and fairly uniform at all flows, the change in pier shape increased the depth of contraction from 19 to 24 feet. No data were

taken to determine the effect on discharge but all the previous pier studies have indicated that the decrease in discharge is proportional to the depth of contraction. Therefore, it can be assumed that the decrease in discharge was in excess of 1.5 percent.

Pier No. 10. Since Pier No. 8 gave a smaller contraction than Pier No. 9, it was decided to accept the general shape of Pier No. 8 and to determine the amount of overhang necessary to give satisfactory flow conditions with a minimum decrease in discharge.

In Pier No. 10, the amount of overhang was increased from 16 to 30.65 feet maintaining the same slope above elevation 845 feet as in Pier Nos. 3 and 8, Figure 6. Increasing the overhang reduced the depth of contraction from 19 to 14 feet, and the fin of water which was observed along the right training wall with Pier No. 8, Figure 14B, was completely eliminated with Pier No. 10. The decrease in discharge for both designs was approximately the same, 1.3 and 1.5 percent.

Pier No. 11. The studies using Pier Nos. 8 and 10 indicated that the optimum overhang should be somewhere between 16 and 30 feet. It was desired that the fin along the training wall be eliminated and yet keep the amount of overhang at a minimum. Discussions with the structural designers indicated that a 24-foot overhang was satisfactory from the structural and construction standpoints. Therefore, Pier No. 11 with a 24-foot overhang was constructed and tested. Figure 15 shows the flow conditions for discharges of 50,000, 100,000 and 145,000 second-feet. The depth of contraction for maximum flow was 17 feet and the spillway discharge at maximum reservoir elevation was approximately the same as the computed discharge. Flow down the spillway was very smooth and uniform at all flows with no appreciable fin of water along the right training wall, Figure 15A. Therefore, Pier No. 11 is recommended for use in the prototype.

The spillway was also operated at various discharges with partial gate openings to assure that no adverse flow conditions developed when the flow was controlled by the gates. Except for small vortexes forming upstream from the gate, the flow in the vicinity of the pier was smooth and satisfactory for all discharges. Figure 15C shows the flow conditions at the right pier for a discharge of 100,000 second-feet when the gates are partially closed.

The tests on Pier Nos. 3 and 4 indicated that the pressures on the pier face were subatmospheric since the flow at times separated from the pier. Ten piezometers were placed in Pier No. 11--two rows of five piezometers each at elevations 828 and 849 feet, Figure 16. Pressures were recorded for discharges of 100,000 and 145,000 second-feet. At the lower discharge, the pressures at all the piezometer openings were atmospheric or higher. At maximum flow of 145,000 second-feet, the pressures at Piezometer Nos. 2, 3, 4, 9, and 10 were below atmospheric with the lowest pressure of 8 feet below atmospheric recorded at Piezometer No. 4, Figure 16. The observed subatmospheric

pressures were not considered serious since they were well above the cavitation range and before reaching the cavitation range the flow would no doubt separate from the pier and provide aeration to the low-pressure region as explained in the tests on Pier Nos. 3 and 4.

Left Spillway Pier Studies

No comprehensive studies of the left pier were conducted since the contraction at the left pier presented no unusual problems due to the close proximity of the pier to the canyon wall and the more direct approach of the flow to the left spillway bay. The undesirable flow conditions observed early in the pier studies, Figure 17A, were caused primarily by the large contraction at the right pier. After the contraction was reduced by installing right Pier No. 11 (Recommended), the flow through both bays was improved considerably, Figure 17B and C. To make the two piers more nearly alike, the thickness of the left pier was increased to 14 feet--the same thickness as the right pier. Increasing the thickness of the left pier reduced the contraction from 23 to 21 feet and slightly improved the flow conditions in the left bay, especially along the left training wall.

It is recommended that the thickness of the left pier be increased to 14 feet with the same curvature on the pier nose as the right pier.

A recapitulation of the spillway pier studies is shown in Figure 18.

Stilling Basin Studies

General. Studies on the preliminary spillway design were conducted at the same time the pier and approach studies were being made. The recommended piers and approach channel were installed permanently in the model during the studies on Spillway Design No. 2. Thus, the approach to the spillway was the same for Design Nos. 3 through 11, Figures 7-9. Minor alterations were made to the downstream end of the center pier during the spillway studies. These modifications are discussed under the appropriate spillway design in which the changes were made.

The geology of the dam site and the river channel downstream from the dam, among other considerations, placed certain limitations on the spillway alignment and the means used to dissipate the energy of the spillway flow. Immediately downstream from the dam, the left canyon wall is a talus slope extending from the river to approximately 200 feet above the river channel. Since a future access road will cross near the top of the talus slope and movement of loose material into the river channel would endanger the road, it was desirable that the spillway flow either be directed away from the slope or be sufficiently quieted that the flow would not move the material. Another consideration was the depth to solid rock below the river channel which varies

from approximately 20 feet at the axis of the dam to 130 feet a distance of 600 feet downstream from the dam. The comparatively steep downward slope of the solid rock limited the length of stilling basin or bucket which could economically be used.

These two limitations greatly influenced the thinking in developing a satisfactory spillway and energy dissipator design.

Preliminary design. The preliminary spillway design, Figures 2 and 7A, was of the conventional ogee type with a short horizontal apron extending 18 feet downstream from the curved toe of the spillway profile. Baffle piers, 5 feet in height and curved on the upstream face, were placed on the apron to aid in breaking up the high-velocity flow. Although a hydraulic jump formed at the downstream end of the spillway for very small discharges, the spillway was designed to release the larger flows horizontally into the river with all energy dissipation taking place in the river channel.

To study the spillway flow without having to consider the changing flow pattern due to erosion, the downstream river channel, including the talus slope, was initially molded in concrete to form a stable channel. Figure 19 shows the spillway operation at the near-maximum discharge of 140,000 second-feet.

Due to the side displacement of the spillway with reference to the river channel, the spillway flow climbed the slope of the excavated channel, turned, and entered the river channel with no appreciable deceleration. Eddies formed on each side of the spillway causing flow to re-enter the spillway jet by overtopping the training walls and the spillway flow. The intermixing of the spillway flow with flow from the side eddies caused considerable splashing and extremely rough water downstream from the spillway.

Figure 19B shows the spillway discharging 100,000 second-feet. Although the side eddies were less pronounced, the general pattern of flow was the same as observed at maximum flow. For discharges less than 25,000 second-feet, a hydraulic jump formed on the horizontal apron and the flow entered the river channel satisfactorily.

From the studies on the preliminary design, it was apparent that, to save the talus slope downstream from the spillway, the spillway flow should be released more directly into the river. This could be accomplished in several ways: (1) Place the spillway in the center of the river over the arch section of the dam. (2) Rotate the axis of the spillway in its preliminary location to release the flow more directly into the river. (3) Move the entire spillway toward the river channel within the gravity portion of the dam. (4) Superelevate and curve the spillway to direct the flow into the river channel.

The first possibility was eliminated because of design problems and the extreme depth to bedrock in the river channel. The

second choice was rejected for economic reasons and the less favorable approach conditions. The third choice offered a reasonable solution. Since the right portion of the gravity section of the dam is a thrust block for the arch dam, the amount the spillway could be moved to the right was limited to 20 feet by the designers, however Therefore, it was decided to gain as much directional effect as possible by moving the entire spillway 20 feet toward the river channel and to further direct the flow into the river channel by superelevating the spillway (fourth choice).

Design No. 2. In Design No. 2 the spillway was moved 20 feet to the right and a superelevated bucket with baffle piers was placed in the spillway, Figures 7B and 20A. Rather than move the spillway in the model which would require the costly procedure of rebuilding the spillway crest and gravity section of the dam, the tail box containing the downstream topography was shifted 20 feet to the left and the topography upstream from the dam was removed and rebuilt, conforming to the 20-foot shift.

Shifting the spillway toward the river channel reduced the left side eddy, but the unretarded flow still impinged on the talus slope, Figure 20A. The superelevation in the bucket was insufficient to accomplish any noticeable turning of the flow toward the river channel, but the spillway flow was raised sufficiently to prevent eddy flow from passing over the top of the jet leaving the bucket.

After impinging on the talus slope, the flow was deflected to the opposite river bank. It appeared that less flow would be deflected to the opposite bank if the downstream end of the outlet channel was modified by removing the point indicated by the carpenter's rule in Figure 20A.

Figure 20B shows the flow entering the river channel after the point was removed. The spillway flow entered the river channel with less deflection to the opposite bank. Figure 21 indicates the amount the outlet channel was modified.

Design No. 3. Since the superelevation in Design No. 2 was insufficient to turn the spillway flow, a bucket with an extreme superelevation was constructed in Design No. 3, Figures 7C and 22A. Although Design No. 3 helped to turn the flow, Figure 22A, the bulk of the jet was still directed to the left bank of the river.

The above tests indicated that even with extreme superelevation of the bucket the jet was not turned the desired amount. It was evident that superelevation should start near the crest of the spillway and continue through the bucket. In addition, the spillway needed to be turned or curved more toward the river channel.

Design No. 4. In Design No. 4 the right training wall downstream from the axis of the dam was rotated 6-1/2° toward the river

channel and the bucket was extended downstream so that its end was normal to the right training wall, Figures 7D and 22B. The left training wall was curved to make the downstream end of the bucket 104 feet wide. The superelevation was less than that used in Design No. 3 but started higher up the spillway.

By comparing Figures 22A and 22B, it can be seen that turning the spillway helped to place the flow more directly into the river channel. The flow in the spillway was unevenly distributed, with some of the flow from the left bay crossing over the top of the flow in the right bay, creating a large fin of water which spread to the right of the spillway.

Using Design No. 4, tests were run with and without baffle piers on the superelevated bucket. From visual observations, it appeared that the baffle piers were of little value in breaking up the jet. Therefore, the baffle piers were removed from the model.

Design No. 5. To more evenly distribute the flow in the superelevated bucket and to flip the jet upward to a greater degree, the curvature of the bucket was increased in Design No. 5. In general, this change smoothed out the water surface in the center of the spillway but caused fins of white water to form along the edges of the jet, Figure 23A. The greater curvature of the bucket raised the trajectory of the jet permitting the flow to enter the pool at a steeper angle. The bulk of the flow was directed into the river channel but the left edge of the jet still impinged on the talus slope.

Design No. 6. The studies on Design No. 5 indicated that the left portion of the jet should be further turned toward the river channel. This was accomplished in Design No. 6, Figure 8B, by introducing more curvature in the left training wall, thus gradually reducing the width of the spillway to a minimum of 75 feet at the downstream end of the bucket.

In general, Design No. 6 gave the best results of any of the designs previously tested. The entire jet was directed away from the talus slope and into the river channel, Figure 23B. Although the jet was concentrated in a smaller area and a thin sheet of water folded over the right side of the jet, it was evident that the basic features of Design No. 6 could be refined into a satisfactory spillway design.

Figure 24 shows the operation of Design No. 6 for flows of 57,500 and 25,000 second-feet with both gates open and with the right gate closed. Except for the sheet of water spreading to the right of the main jet, the flow was satisfactory when both gates were equally open, Figure 24A and C. With the right gate closed and discharges above 35,000 second-feet, the flow through the left gate spread and created a fountain-like disturbance near the downstream end of the right training wall, Figure 24B. This disturbance could be eliminated by opening the right gate from 6 to 8 feet to permit flow through

the right spillway bay. For discharges less than 35,000 second-feet through the left gate, the jet spread across the bucket giving an excellent flow distribution, Figure 24D.

Design No. 7. To investigate an entirely different type of energy-dissipating device, the hydraulic jump basin was employed in Design No. 7, Figures 8C and 25A. The general plan of Design No. 6 was modified slightly and extended downstream to the stilling basin floor at elevation 680 feet. To aid in dissipating the high-velocity flow, baffle piers with a curved upstream face 7 feet wide and 28 feet high were placed in the downstream end of the stilling basin.

Figure 25B through D shows the operation of the stilling basin at various flows. For discharges less than 60,000 second-feet, a hydraulic jump formed but the flow was concentrated on the left side of the basin. Between 60,000 to 75,000 second-feet, the jump was on the verge of sweeping out, Figure 25C. The high-velocity flow struck the baffle piers and was deflected upward causing extreme surface turbulence at the downstream end of the basin. For discharges above 80,000 second-feet, the jump swept out and the high-velocity flow continued into the river channel, Figure 25D.

Although the flow conditions in Design No. 7 were generally unsatisfactory, the jump basin could be used by lengthening and lowering the stilling basin and by changing the superelevation to evenly distribute the flow entering the stilling basin. The cost of constructing Design No. 7 was considerably more than Design No. 6, and to enlarge the stilling basin sufficiently to perform satisfactorily at maximum flow would make the construction costs prohibitive. Therefore, it was decided to abandon the jump basin and concentrate on improving the superelevated bucket of Design No. 6.

Design No. 8. Design No. 6 was modified by raising the bucket and changing the superelevation to improve the flow distribution and raise the jet above the tail water, Design No. 8, Figure 9A. Also, the downstream end of center pier was turned 3° to conform to the curvature of the training walls. The operation of Design No. 8 at maximum flow, Figure 26A, was improved over Design No. 6, Figure 23B. The flow was more evenly distributed and the jet was more stable.

The flow disturbance at the downstream end of the right training wall was still prevalent when the left gate was discharging 57,500 second-feet. In an effort to eliminate the flow disturbance, the height of the right training wall was reduced by sloping the top of the wall downward to the lip of the bucket. However, this change offered no improvement over the flow conditions observed in Design No. 6, Figure 24B and 26B.

Design No. 9. In Design No. 9, a dividing wall, extending from the center pier to the end of the bucket, was used to separate the flow in the two bays, Figure 9B. At maximum flow, the flow was unevenly distributed and a large fin of water formed along the left side of the dividing wall, Figure 27A. Several different superelevated buckets in each bay were tested, but poor flow conditions still persisted. The dividing wall was then shortened by removing the downstream 50 feet of wall, Figure 27B. This change reduced the height of the fin of water along the dividing wall; however, there was considerable disturbance where the flow in each bay joined downstream from the dividing wall, Figure 27C and D.

Since the dividing wall increased the over-all cost of the spillway with no apparent improvement in the flow conditions, the design utilizing a dividing wall was abandoned.

Design No. 10. Design No. 10 was similar to Design No. 6 except the right training wall was curved and the width of the bucket lip was reduced from 75 to 72 feet, Figure 9C. Curving the right training wall helped to smooth out the flow down the spillway, Figure 28A. The flow was uniformly distributed and there was no tendency for the jet to climb either training wall. The trajectory of the jet after leaving the spillway was comparatively flat and lowered the tail water downstream from the spillway.

To refine the spillway design, several modifications to Design No. 10 were tested. These modifications included buckets at different elevations, varying amounts of superelevation, bucket lips which were straight or curved in plan, and varying lengths of buckets. Figure 28B shows the operation of one of these modifications at maximum discharge. In this case the lip of the bucket was raised to elevate the trajectory of the jet. Except for the steeper trajectory, the flow in the spillway was similar to Design No. 10.

Recommended spillway design. As a result of the extensive studies described above, Design No. 10 was modified slightly in the recommended spillway design, Figure 9D. The lip of the bucket was raised to permit the jet to enter the pool at a steeper angle and the right training wall was made straight for a distance of 50 feet downstream from the axis of the dam and then curved to the lip of the bucket. The general plan and elevation of the dam are shown in Figure 29 and details of the crest, piers, bucket and training walls for the recommended spillway are shown in Figure 30.

The recommended spillway was initially operated with a stable channel downstream from the spillway, Figure 31A. The channel was molded in concrete conforming to the topography shown in Figure 29 and included the modified spillway channel shown in Figure 21. When the model was operated at maximum discharge, Figure 31A, the flow struck the stable channel and spread laterally and downstream without forming any pool to dissipate the high-velocity

flow. Since the channel is composed of loose material to depths of 150 feet, the flow pattern shown in Figure 31A will exist only during the initial phase of spillway operation. Soon after the spillway first operates, erosion will occur, a pool will form, and the flow pattern will change. The depth and location of the erosion will depend on the amount of water being spilled and which gates are used to release the flow.

Considerable thought was given to the problem of best demonstrating the flow pattern at different discharges and various gate openings. Erosion studies (discussed on page 16) were made to determine the location and depth of erosion for various discharges. However, since the results of model erosion studies are only qualitative, exact reproduction of the prototype erosion in the model could not be expected. Also, the spillway will undoubtedly operate over a wide range of discharges for an unknown length of time. Therefore, it was decided to obtain a composite erosion pattern in the river channel which combined the erosion patterns for a discharge of 57,500 second-feet with the gates in three positions: (1) left gate fully open and right gate closed, (2) right gate fully open and left gate closed, and (3) both gates partially open an equal amount. The composite erosion pattern obtained from the above operation was then stabilized by placing a thin layer of concrete over the movable material, Figure 31B. The model was then operated for other discharges using the composite erosion pocket. At all flows a more representative flow pattern was obtained although it is realized that the erosion pocket gives true flow conditions only for a discharge of 57,500 second-feet. All photographs of the recommended design showing flow conditions in the downstream river channel, except Figure 43, were taken with the composite erosion pocket in the downstream river channel.

Figures 32 and 33 show the spillway operation with both gates equally open for discharges of 10,000, 25,000, 60,000, 100,000, and 145,000 second-feet. Throughout the range of discharges, the flow is directed into the river channel without impinging on the talus slope on the left bank of the river. An indication of the improvement obtained by turning the spillway flow can be seen by comparing the preliminary design, Figure 19A, with Figure 33B. In the improved design most of the turbulence from the jet occurs in the river and along the right bank, which is solid rock and capable of withstanding the impact of the spillway flow.

Discharges of 10,000, 25,000, and maximum flow through one gate at reservoir elevation 875 feet are shown in Figures 34 and 35. For discharges less than 40,000 second-feet, the flow was well distributed and the spreading of the jet was confined to the width of the bucket. Above 40,000 second-feet, the spreading flow reached the right training wall, causing a high fin of water along the training wall, Figure 34C. Flow through the right gate was similar in operation except that the spreading flow reached the left training wall at discharges above 19,000 second-feet, Figure 35.

Log Sluice and Outlet Studies

Detailed studies of the flow in the outlet works and log sluice could not be made on the 1:48 scale model because of the small scale of the model. However, general studies were made of the log sluice and the jets leaving the outlet works.

Figure 36B shows the outlets discharging the maximum flow of 10,000 second-feet with the reservoir at elevation 875 feet. Due to the alignment of the spillway and the outlets, part of the flow from the left outlet glances off the right training wall of the spillway creating considerable splash and disrupting the flow of the jet. Consideration was given to moving the outlets to the right or placing a deflector wall along the left side of the outlet. However, the outlets cannot be moved farther to the right without endangering the stability of the thrust block for the arch dam. The deflector wall was considered unnecessary since the outlets will probably operate only during diversion. Normally, the reservoir water will be released through the powerhouse and spillway, and the outlets will be used to release water only in emergencies after the dam is constructed. Therefore, it was decided to make no changes in the alignment or location of the outlets.

Figure 36C shows the log sluice operating at the same time as the outlets. Although this operating condition will probably never occur, the figure shows that water may be released from the log sluice without disrupting the flow from the outlets.

The approach conditions to the log sluice are demonstrated in Figure 36D. Confetti on the water surface indicated that the flow approaches the sluice uniformly. Figure 36E and F shows logs approaching the sluice and passing down the spillway face. The logs correspond to prototype logs, 1 to 2 feet in diameter and 8 to 20 feet in length. Tests indicated that the logs tend to approach the spillway with the long axis normal to the dam and only occasionally would a log become lodged across the sluice entrance.

It should be noted that the junction of the arch and thrust block in the model is Station 5+20 instead of Station 5+00 in the Recommended Design as shown in Figure 29. This discrepancy occurred when the positions of the spillway and outlets were changed in the model by moving the topography without rebuilding the dam and spillway (see Design No. 2, page 10). The line marked "500" near the base of the thrust block in Figure 36A and C is Station 5+00 and indicates the true width of the block. Therefore, part of the flow from the log sluice, Figure 36C and F, will spill over the edge of the thrust block.

THE RECOMMENDED SPILLWAY

General

Considerable data was obtained from the model for later correlation with data from the prototype, including water surface profiles, erosion studies, spillway pressures, and elevation of the tail water for different flow conditions. In addition, spillway rating curves for free flow, partial gate openings and flow over the gates were obtained. Studies were also made to determine the procedure for operating the gates to obtain optimum spillway performance.

Water Surface Profiles

Profiles of the water surface along each training wall and the center pier were obtained for discharges of 50,000, 100,000, and 145,000 second-feet with both gates fully open, Figure 37. Cross-sections of the flow were also obtained at the downstream end of the bucket and at a distance of 50 feet downstream from the axis of the dam. The profiles indicate that the training walls have sufficient free-board and that the depth of flow is comparatively uniform across the width of the spillway.

Section G-G, Figure 37, shows the trajectory of the left side of the jet as it leaves the bucket. The angle at which the flow enters the tail water varies from 50° for 50,000 second-feet to 30° for the maximum discharge of 145,000 second-feet.

Figure 38 gives the water surface profiles along the training walls and the center pier for discharges of 25,000 and 57,100 second-feet through the left gate when the right gate is closed. Except for the fin of water along the right training wall, the water is well distributed as it leaves the spillway bucket. Similar profile data with the right gate operating and left gate closed are shown in Figure 39.

Erosion Studies

For the erosion tests, the stable channel was removed for a distance of 575 feet downstream from the dam and replaced with 3/8- to 1-1/2-inch gravel to conform to the topography shown in Figure 29.

Prior to placing the gravel, concrete topography was installed in the model to represent the solid rock foundation as indicated by test holes drilled at the dam site. The depth and location of scour were determined for three operation conditions: (1) left gate discharging 10,000, 25,000, and 57,500 second-feet with the right gate closed and the reservoir at 875 feet, (2) right gate discharging 10,000, 25,000, and 59,100 second-feet with the left gate closed and the reservoir at 875 feet, and (3) both gates equally open and discharging 10,000, 25,000, 60,000, 100,000, and 145,000 second-feet at maximum reservoir elevation with no flow over the top of the gates. The model was

operated for 1 hour first at the lowest discharge and then progressively through the highest discharge without remolding the erosion pocket. However, before starting the erosion studies for the second and third operating conditions, the erosion pockets were filled and remolded to conform to the original topography.

Figure 40 shows the erosion pattern for the three discharges through the left gate. The erosion pocket occurs well to the right of the outlets and from 100 to 150 feet downstream from the spillway bucket. The elevation of the lowest point in the erosion pattern varies from 710 feet for 10,000 second-feet to 690 feet for 57,500 second-feet.

Erosion patterns with the right gate operating are shown in Figure 41. The erosion pockets occur directly downstream from the outlets and approximately 100 to 200 feet from the spillway bucket. The channel eroded to elevation 714 feet with a discharge of 10,000 second-feet and to elevation 695 with a discharge of 59,100 second-feet.

Figures 42 and 43 show the progressive scour patterns when both gates are discharging from 10,000 to the maximum of 145,000 second-feet. For discharges of from 10,000 to 25,000 second-feet, the erosion pocket occurs directly downstream from the outlets, Figure 42A and B. For discharges above 25,000 second-feet, the erosion pattern moves to the right of the outlets and approximately 200 feet downstream from the spillway bucket, Figures 42C and 43. For discharges above 100,000 second-feet a side eddy, which increases in intensity with discharge and the length of time the spillway operates, forms to the left of the spillway flow, Figure 43A and B. The concentration of flow at the higher discharges, coupled with the side eddy, caused the channel to erode to the floor of the tail box, or elevation 668 feet.

The erosion studies indicated that the depth of scour was moderate for discharges up to 50,000 second-feet through one or both gates. Above 50,000 second-feet, an erosion pocket from 50 to 100 feet deep may form in the river channel to the right of the talus slope. At all flows, the erodible material remained intact for a minimum distance of approximately 100 feet downstream from the dam. Since the erosion pocket will form well downstream from the dam and spillway bucket, the safety of the structure is assured against undercutting and erosion.

Spillway Calibration

Spillway rating curves, reservoir elevation versus discharge, for free-flow and partial gate openings in increments of 2 feet, Figure 44, were determined from the model and checked using independent banks of venturi meters. The curves for free-flow and gate openings in increments of 4 feet were determined by calibration tests, and the intervening 2-foot gate openings were determined by interpolation

and spot-checked by actual calibration tests. The gate opening is the vertical distance from the spillway crest to the bottom of the gate.

The spillway coefficient curve for free-flow is also shown in Figure 44 giving "C" in $Q = CLH^{3/2}$ in terms of reservoir elevation. In this equation Q is the discharge in second-feet, L is the length of crest between piers, and H is the difference in elevation between the spillway crest and the reservoir elevation. At elevation 875, top of gates, the discharge is 120,000 second-feet. The maximum design discharge of 145,000 second-feet with a coefficient, C, of 3.68 occurs at reservoir elevation 881.

A discharge curve for flow over the top of the gates in the seated position was also determined from the model. However, due to viscous effects and surface tension in the relatively low flows, a coefficient of discharge of 4.0 to 4.2 was obtained in the model. Since a discharge coefficient of such magnitude is seldom attained on rounded crests, a rating curve, Figure 45, was computed from data contained in Engineering Monograph No. 9.1/ The curve is based on a coefficient of 3.95 at design head and assumes an aerated nappe downstream from the gate.

No rating curve for the log sluice was computed or obtained from the model, since the sluice will operate only intermittently and for short periods of time. The rating curve for the outlets is discussed on page 26.

Pressure Measurements on Crest and Training Wall

Piezometric pressures on the spillway were obtained for free-flow discharges of 50,000, 100,000, and 145,000 second-feet and gate openings of 4 to 36 feet, in increments of 4 feet, at maximum reservoir elevation. The row of 10 piezometers was located along the center line of the right spillway bay. The longitudinal location of the piezometers and a tabulation of the observed pressures for the various flow conditions are shown in Figure 46.

The observed pressures for free-flow were all above atmospheric with the lowest pressure, 0.7 foot of water, observed at Piezometer No. 9 for a discharge of 50,000 second-feet. At partial gate openings, subatmospheric pressures were observed at Piezometer Nos. 6 through 9 for gate openings of 4 to 20 feet. The lowest observed pressure was 5.8 feet below atmospheric at Piezometer No. 9 for a gate opening of 20 feet. Since the subatmospheric pressures are well above the cavitation range, the spillway is adequately designed against cavitation pressures.

1/"Discharge Coefficients for Irregular Overfall Spillways," by J. N. Bradley, Bureau of Reclamation, March 1952.

To aid in the structural design of the training walls, pressures were observed at 20 piezometers spaced at different locations along the left training wall and at 4 piezometers near the downstream end of the right training wall, Figure 47. Pressure readings were recorded for free-flow discharges of 50,000, 100,000, and 145,000 second-feet and discharges of 25,000, and 60,000 second-feet through one gate with the other gate closed.

The maximum observed pressure on the left training wall was recorded near the bottom of the wall at Piezometer No. 13, where a pressure of 63.5 feet of water was observed at the maximum discharge of 145,000 second-feet. The pressure tests indicated that near the bottom of the right training wall in a region from 20 to 50 feet upstream from the end of the wall, the pressures were maximum and varied from 61 to 63.5 feet of water (see Piezometer Nos. 6, 7, 8, 12, 13, 14, and 50, Figure 47). Both upstream and downstream from this region, the pressures near the bottom of the wall lessened gradually to 4.8 and 12.0 feet of water, respectively.

With flow through one gate and the other gate closed, maximum pressures were also observed in the same region of the left wall, regardless of which gate was open or closed. The maximum pressures observed in the region varied between 36.1 and 41.8 feet of water at the maximum discharge of 60,000 second-feet through one gate. Although the water in spreading from the right spillway bay struck the left training wall, the pressure measurements indicated that the impact pressures on the left wall were no higher than the pressures observed when the maximum flow was released through the left spillway bay.

The pressures observed near the downstream end of the right wall were approximately atmospheric or lower when both gates were discharging, Piezometer Nos. 21 through 24, Figure 47. Similar pressures were obtained when the right gate was open and the left gate closed. However, when 60,000 second-feet was released through the left gate (right gate closed), the impact of the spreading flow gave a maximum pressure of 25.0 feet of water at Piezometer No. 23. The magnitude of the pressures observed on the right wall may be explained as follows. When water is released through the left gate with the right gate closed, the spreading flow strikes the right wall transforming part of the velocity head into pressure head giving a higher piezometric pressure along the wall.

Tail-water Elevations

Since no prototype tail-water data was available for the model studies, approximately 1,100 feet of river channel downstream from the dam was included in the model. The model channel was shaped in concrete to conform to the prototype topography obtained from field surveys. No attempt was made to manually control the tail water at the downstream end of the tail box, since the length of channel was considered sufficient

for the flow to reach equilibrium and establish its own tail-water control. During erosion studies, the concrete in a stretch of the river channel for a distance of 575 feet downstream from the dam was replaced with gravel.

Under the above test conditions, the maximum tail-water elevations immediately below the dam varied approximately 18 feet depending on whether the channel was stable or erodible. When the erosion tests were conducted with gravel in the river channel, the tail-water elevation for maximum discharge was 770 feet at the beginning of the test. After the model had operated for 1 hour and most of the gravel had washed over the downstream end of the tail box, Figure 43B, the tail-water elevation at the dam was 765 feet. With the stable channel and no erosion, Figure 31A, the tail water was 752 feet, while tail water 756 feet was observed with the stabilized erosion pocket, Figure 31B.

From the above tests, it can be seen that an accurate determination of the prototype tail water cannot be obtained from the model. Tail-water elevations on the prototype will depend on how much riverbed material is moved, how rapidly it is carried downstream out of range of the control section, and where deposits of eroded material will be made. However, it is believed that the two extreme tail waters are included in the above tests and that the tail water for maximum flow will be between 760 and 765 feet.

Gate Operating Procedure

The model tests showed that certain combinations of gate openings were better than others in releasing water through the spillway. In general, flow was smoother and better distributed in the spillway when the gates were opened an equal amount. However, the jet leaving the bucket was also more concentrated with both gates operating and caused slightly deeper erosion in the channel, especially for discharges above 30,000 second-feet.

Normally, the spillway will be operated with both gates opened an equal amount. With the gates equally open, the flow is turned away from the talus slope and enters the river channel in a direct manner. However, the position of the jet in the channel may be moved to the right or left by manipulating the gates, Figure 48. By releasing more flow through the left gate, the position of the jet is shifted to the right of the main river channel. Conversely, the jet position is shifted to the left when more flow is released through the right gate. Figure 48 illustrates the amount the jet may be shifted when one gate is closed for discharges of 25,000 and 57,500 second-feet.

The ability to shift the jet from one position to another position in the channel is a valuable feature of the spillway design. If excessive erosion or undesirable side eddies occur in the downstream river channel, the jet may be moved to another portion of the channel by manipulating the amount of flow through each gate.

When near-maximum discharges are released through one gate with the other gate closed, a fin of water forms along the training wall, Figures 34 and 35. This fin of water may be eliminated by opening the closed gate approximately 8 feet, Figure 49. By releasing a relatively small amount of water through the second spillway bay, the main flow is prevented from spreading to the opposite training wall and the flow is evenly distributed across the lip of the spillway bucket.

PART II--OUTLET WORKS STUDIES

INTRODUCTION

The construction schedule for Gorge High Dam calls for the construction of the gravity section of the dam first, followed by the arch section of the dam. The outlets are required to divert the excess river flow, above that released through the power intake, while the arch section of the dam is under construction. After the dam is completed, flow will normally be released through the power intake and over the spillway. However, the outlets are designed to release a maximum of 10,500 second-feet at reservoir elevation 875 feet.

The two identical outlets were originally located 16 feet 3-1/4 inches from the face of the right spillway training wall. When the spillway was shifted 20 feet toward the river as a result of the spillway studies, the outlets were moved a distance of 25 feet. Thus, in the recommended design, Figure 50, the center line of the left outlet is located 21 feet 3-1/4 inches to the right of the training wall. The outlets are 108 feet long, 8 feet 9 inches wide and 8 feet 9 inches high with an arched top and spaced 17 feet between center lines. The outlets are horizontal with the inverts at elevation 760 feet.

Flow through each of the outlets is controlled by an 8.75- by 8.75-foot fixed-wheel gate. The entrance to the outlet is elliptical with a ledge or corbel placed upstream from the tunnel invert which serves as a seat for stop logs. Air vents are provided to aerate the conduit downstream from the gates.

Model studies of the outlet works were conducted to study the flow characteristics in the bellmouth entrance, gate chamber, and the outlet proper.

THE 1:16.47 MODEL

A geometrical scale of 1:16.47 was chosen for the outlet model to make use of existing pipe from another model in the laboratory. The model consisted of a 4- by 5-foot head box, 11 feet high, and one of the outlets connected to one side of the head box, Figures

51 and 52. The head box was constructed of wood and lined with sheet metal. To permit observation of the flow in the outlet, the outlet and gate chamber were constructed of transparent plastic. All hydraulic features of the outlet were accurately dimensioned according to prototype drawings furnished by the City of Seattle. Since the outlets normally will be operated wide open, no gate was installed in the model and no attempt was made to obtain data for partial gate openings.

Fifty piezometers were placed in the bellmouth entrance and outlet to evaluate the pressure distribution within the outlet.

Water was supplied to the model by an 8-inch centrifugal pump and measured through venturi meters.

THE INVESTIGATION

General

The investigation of the outlet works was concerned with the characteristics of the flow in the outlets. Preliminary observations of the outlet operation, both in the 1:48 model and the larger 1:16.47 model, indicated that the flow downstream from the gates was satisfactory for all discharges. However, pressure measurements indicated that a region of low pressures existed in the bellmouth entrance to the outlet. Therefore, the studies were concerned primarily with developing an entrance which was free of cavitation and which could be placed in the limited space between the gates and the upstream face of the dam.

Bellmouth Entrance Studies

Investigation of the preliminary bellmouth, $\frac{X^2}{(4.81)^2} + \frac{Y^2}{(1.27)^2} = 1$, indicated a low pressure region along the sides of the entrance, Figure 53. Two rows of piezometers, Nos. 34 to 39 in the crown of the entrance and Nos. 21 to 26 located on the side of the entrance 4 feet above the invert, showed pressures varying from 22 feet of water below atmospheric near the face of the dam to 11 feet above atmospheric at the gate. In general, the row of piezometers in the crown of the bellmouth indicated pressures approximately 10 feet of water lower than those observed in the row of piezometers on the side of the bellmouth. A third row of piezometers, Nos. 8 to 13, showed positive pressures ranging from 14 to 25 feet of water in the corner near the invert of the entrance.

Since pressures dangerously near the cavitation range were observed in the preliminary bellmouth, it was necessary to develop a new entrance shape.

Method of Designing Entrances to Conduits

Numerous experiments have been made to develop methods of designing an entrance to a circular conduit. Results of tests on a jet issuing from a circular sharp-edged orifice, conducted by the Bureau of Reclamation, 2/ showed that the jet shape followed an elliptical curve rotated about the axis of the opening. The equation of the ellipse was expressed in terms of the conduit diameter, $\frac{X^2}{(0.50D)^2} + \frac{Y^2}{(0.15D)^2} = 1$. The shape of the preliminary bellmouth for Gorge High Dam outlets was designed using an ellipse, $\frac{X^2}{(0.57D)^2} + \frac{Y^2}{(0.15D)^2} = 1$, which is similar to the above general equation.

However, the cross section of the Gorge outlets is not circular, but a combination of a semicircle and a rectangle. Tests by H. A. Thomas3/ on jets issuing from rectangular orifices showed that the contraction for rectangular orifices was greater than the contraction for circular orifices, requiring an opening at the face of the dam equal to 1.6 times the diameter of the conduit as compared to 1.3 diameters for a circular conduit. Likewise, the length of bellmouth required for a rectangular conduit is approximately 3 times that required for a circular conduit.

Pressure tests on the preliminary bellmouth for the Gorge Dam outlets, Figure 53, indicated that a conduit with a cross section combining a semicircle and a rectangle would require a larger bellmouth opening than a circular conduit. Therefore, it was decided to develop a bellmouth having a contraction approaching that required for a rectangular conduit, limiting the bellmouth length to 6 feet which is the distance between the face of the dam and the gate slot.

Tests with Air

To study various bellmouth entrances in the 1:16.47 model would require costly and time-consuming changes. Since a test setup utilizing air as a flow medium was available in the laboratory, Figure 54A, the tests were conducted in the air model where various bellmouth shapes could be tested with a minimum of effort and materials.

Pressures in the preliminary bellmouth were obtained for the following test setups: (1) full preliminary bellmouth, Figure 54B;

2/Report No. Hyd-66, "Model Studies for the Design of Sluice Entrances for Grand Coulee Dam," Bureau of Reclamation, September 15, 1939.

3/"Design of Bellmouths for Entrances to Conduits of Circular, Square and Rectangular Cross Sections," H. A. Thomas, Carnegie Institute of Technology, July 1946.

(2) circular bellmouth with lower or rectangular portion of the preliminary bellmouth blocked off, Figure 54C; and (3) rectangular bellmouth with upper or circular portion blocked off, Figure 55A. The pressures, expressed in terms of velocity head, for the three test setups are shown in Figure 55A. For comparison, the pressures obtained in the water model are also shown.

The pressures observed in the air model with the full bellmouth are slightly lower than those observed in the water model. This variance was no doubt due to the difference in the test setups, since the air model included only the bellmouth entrance while the water model contained the entire outlet. The pressures obtained for the rectangular and circular bellmouths were almost identical to those measured in the preliminary bellmouth except near the face of the dam where the pressures measured at Piezometer No. 21 were considerably lower, especially for the rectangular bellmouth. Thus, the observed pressures shown in Figure 55A illustrated the need for a larger opening to accommodate the contraction in the conduit entrance.

Two additional bellmouths were tested to determine the shape necessary for an outlet barrel, rectangular in cross section, Figure 56A. Bellmouth No. 2, $\frac{X^2}{(6.00)^2} + \frac{Y^2}{(2.62)^2} = 1$, was based on the general curve for rectangular entrances except that the length was reduced from 1.5 to 0.7 times the conduit diameter. The equation for Bellmouth No. 3 was $\frac{X^2}{(6.00)^2} + \frac{Y^2}{(1.97)^2} = 1$, or a mean between Bellmouth Nos. 1 and 2.

Figure 55B shows the results of the pressure tests on the three entrance shapes. The pressure, expressed in terms of velocity head, for Bellmouth No. 2 was approximately zero in the upstream half of the entrance while slightly lower in the downstream half. The lower pressures observed in the downstream portion of the entrance probably were due to the shortened length of the bellmouth. The pressures observed in Bellmouth No. 3 fell between those observed in Bellmouth Nos. 1 and 2, with a pressure equal to 0.3 of the velocity head below atmospheric measured at Piezometer No. 21.

THE RECOMMENDED DESIGN

The air tests on the three entrance shapes indicated that Bellmouth No. 2 would give approximately atmospheric pressures when used with a rectangular conduit. Therefore, it was decided to construct the full bellmouth using an elliptical curve similar to Bellmouth No. 2, and test it in the 1:16.47 water model with the gate chamber and conduit attached. The equation of the new entrance, Bellmouth No. 4, was

$\frac{X^2}{(6.04)^2} + \frac{Y^2}{(2.54)^2} = 1$ which was slightly different than Bellmouth No. 2 but fitted better into the dimensioning and spacing of the outlets.

The pressures in feet of water observed in Bellmouth No. 4 and the outlet are shown in Figure 55C. The observed pressures in both the tunnel and bellmouth were above atmospheric except at Piezometer Nos. 38 and 39 in the crown of the bellmouth immediately upstream from the gate where the pressures were slightly below atmospheric. The lowest observed pressure was 4-1/2 feet below atmospheric, which is well above the cavitation range. No pressures were observed on the surface of the corbel projecting into the reservoir. In this region, velocities are low with corresponding higher pressures.

The above model setup represented flow entering the outlet with full contraction. However, since the outlets are comparatively close together and the left outlet adjoins the spillway corbel, the contraction on the left side of the left outlet would be suppressed when either the left or both outlets were operating. The approaching flow between the outlets would also be suppressed when both outlets are operating. Since piezometers had been installed on only one side of the outlet it was believed advisable to obtain data more representative of prototype approach conditions. Pressures were therefore observed with the following model arrangements:

Arrangement A. Same as shown in Figure 51. Flow approached outlet unsuppressed from all sides, representing approach conditions when only right outlet was operating.

Arrangement B. Wall of symmetry placed on left side of outlet. This arrangement represented the approach conditions in the right outlet when both outlets were operating.

Arrangement C. Spillway and right pier installed on right side of outlet, representing approach conditions when left outlet was operating.

Arrangement D. Spillway on one side and wall of symmetry placed on the other side of outlet to represent approach conditions in left outlet when both outlets were operating.

Pressures for each of the above arrangements with the reservoir at elevations 780, 820, and 880 feet are shown in Table 1. The observed pressures in the critical regions of the bellmouth did not vary appreciably with reservoir elevation for the four model arrangements. Slightly lower pressures were observed at Piezometer Nos. 38 and 39 for the operating condition when both gates were discharging, but the difference was negligible. Thus, the four model arrangements had no appreciable effect on the pressures in the conduit. The lowest pressure recorded was 4-1/2 feet below atmospheric at Piezometer No. 39 for Arrangement D with the reservoir at elevation 880 feet.

The appearance of the flow in the gate chamber and the general appearance of the jet leaving the outlet are shown in Figures 56B and 57 for discharges of 1,000, 2,000, 3,000, 4,000, and 5,250 second-feet.

DISCHARGE CURVE FOR OUTLET WORKS

The actual calibration points, reservoir elevation versus discharge, which were obtained for each of the model arrangements of the recommended bellmouth are shown in Figure 58. All the points fall on or near the same curve, indicating that the different arrangements had no measurable effect on the discharge. Therefore, the curve shown is equally applicable to either outlet. The maximum discharge through both outlets was 10,500 second-feet when the headwater was at the top of the gates, elevation 875.

Interior - Reclamation - Denver, Colo.

FIGURE 1

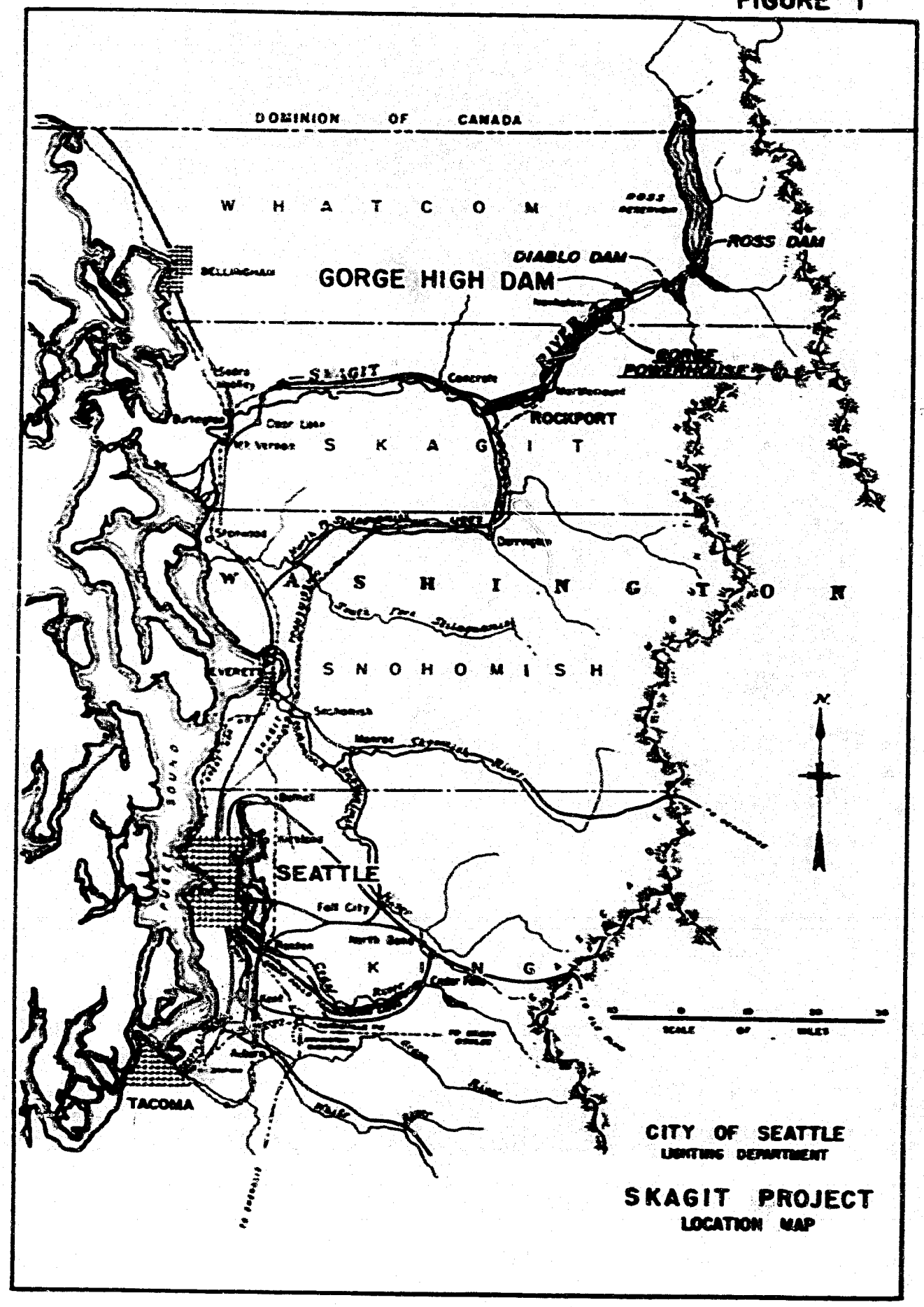
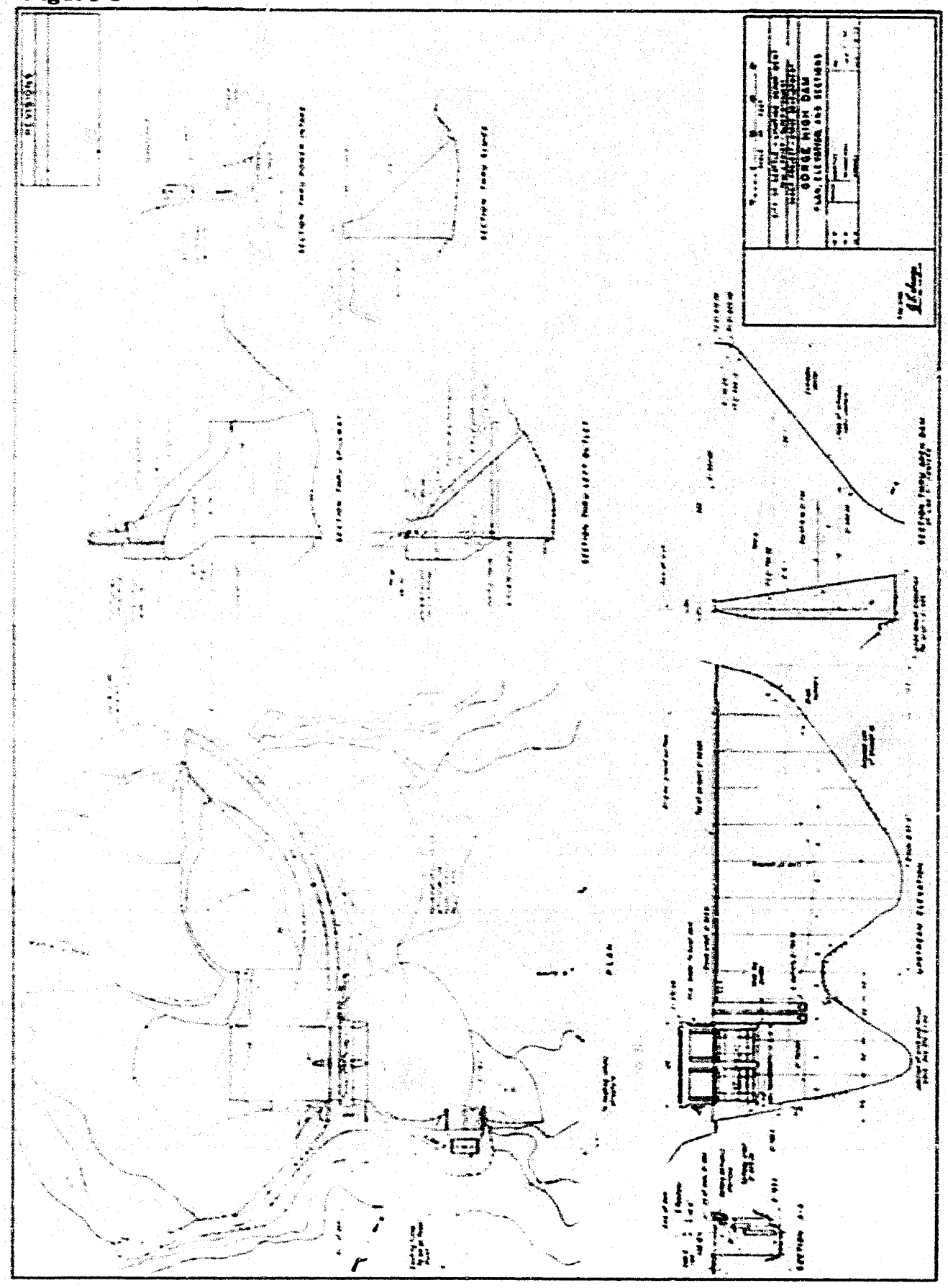
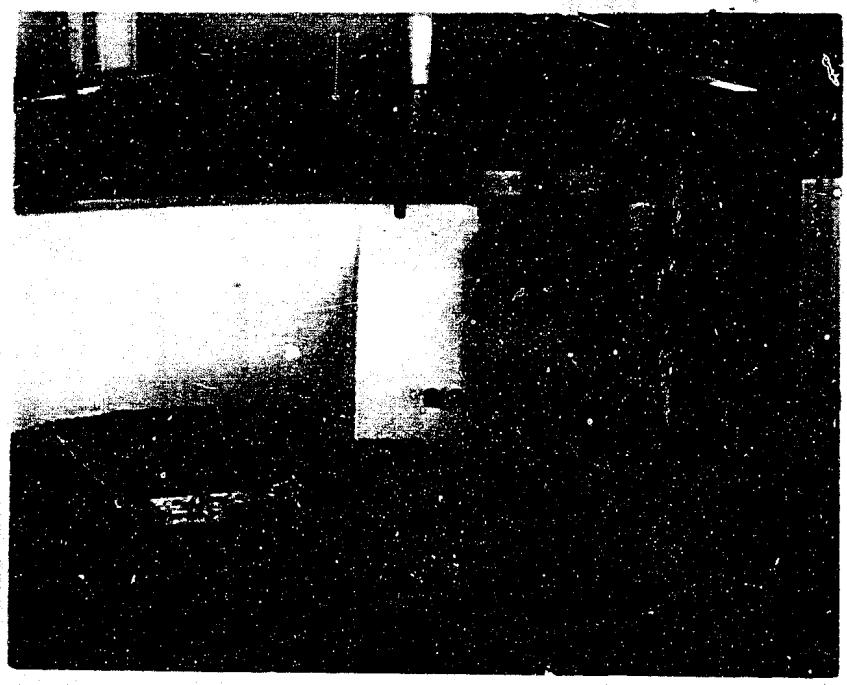
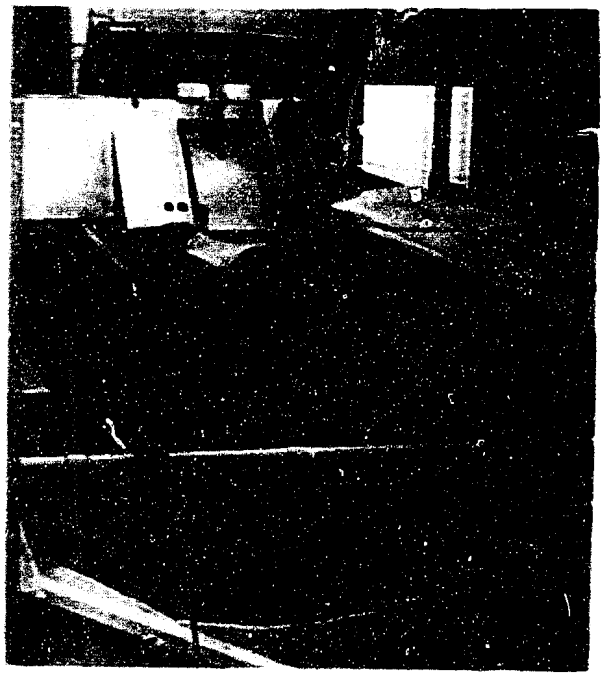

DOMINION OF CANADA
W H A T C O M
ROSS DAM
DIABLO DAM
GORGE HIGH DAM
BELLINGHAM
SKAGIT
Concrete
GORGE POWERHOUSE
ROCKPORT
Cedar Lake
Mt. Vernon
S K A G I T
Darrington
W A S H I N G T O N
S N O H O M I S H
EVERETT
Snohomish
SEATTLE
Fall City
North Bend
K I N G
TACOMA
N
SCALE OF MILES
CITY OF SEATTLE
LIGHTING DEPARTMENT
SKAGIT PROJECT
LOCATION MAP

Figure 2





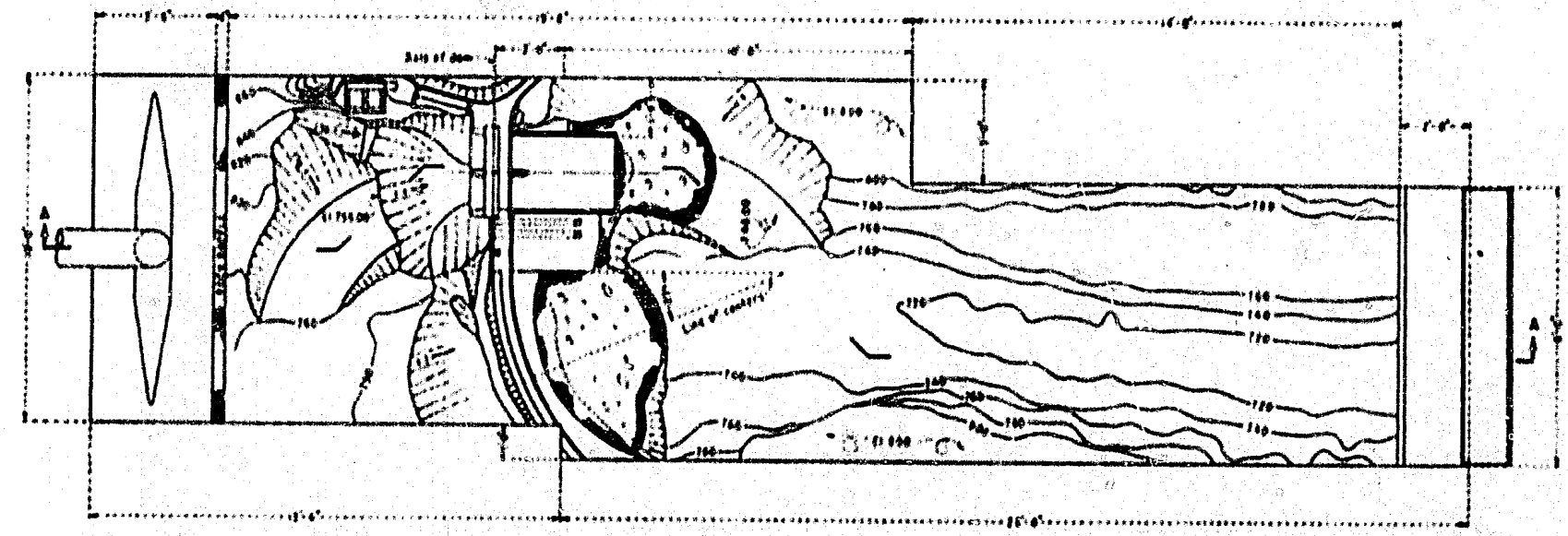
A. Gorge High Dam



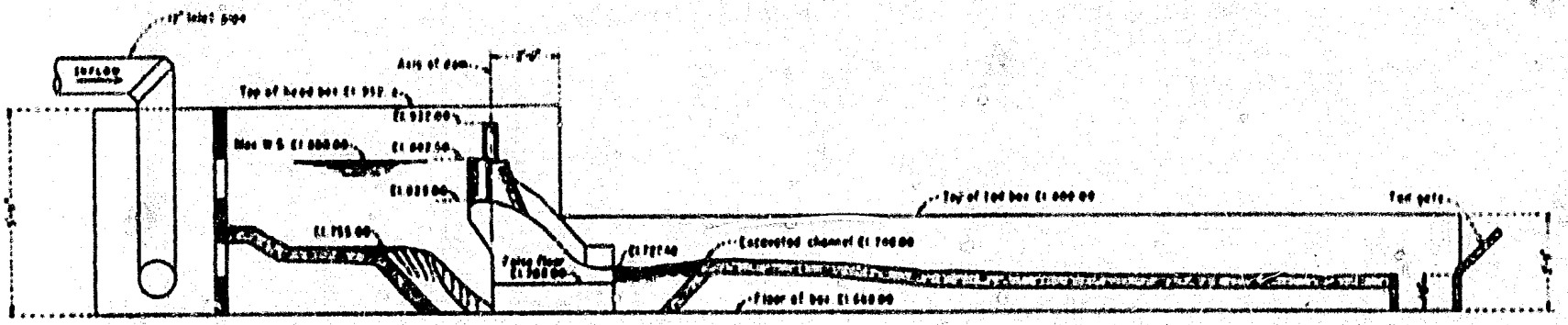
B. Downstream river channel

C. Power intake and spillway approach

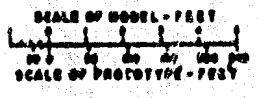
GORGE HIGH DAM
Preliminary Design
1:48 Scale Model



PLAN

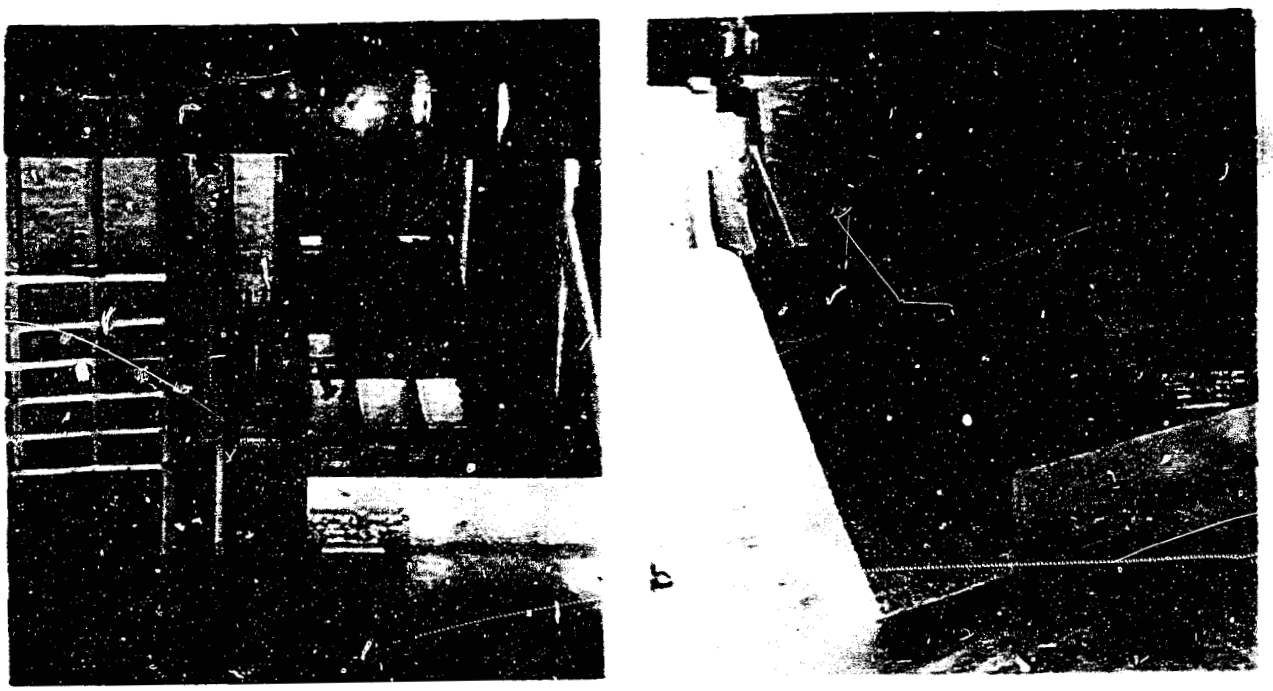


SECTION A-A

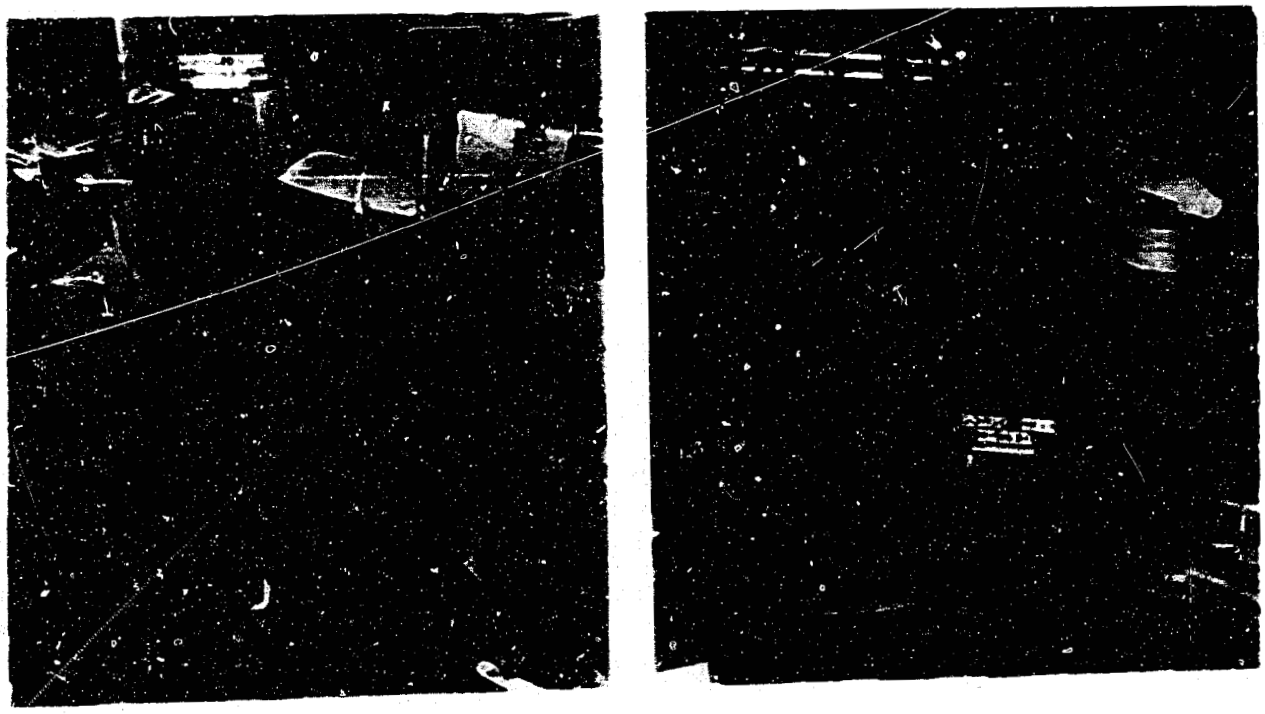


GORGE HIGH DAM SPILLWAY
MODEL LAYOUT - PRELIMINARY DESIGN
1:40 MODEL

FIGURE 4



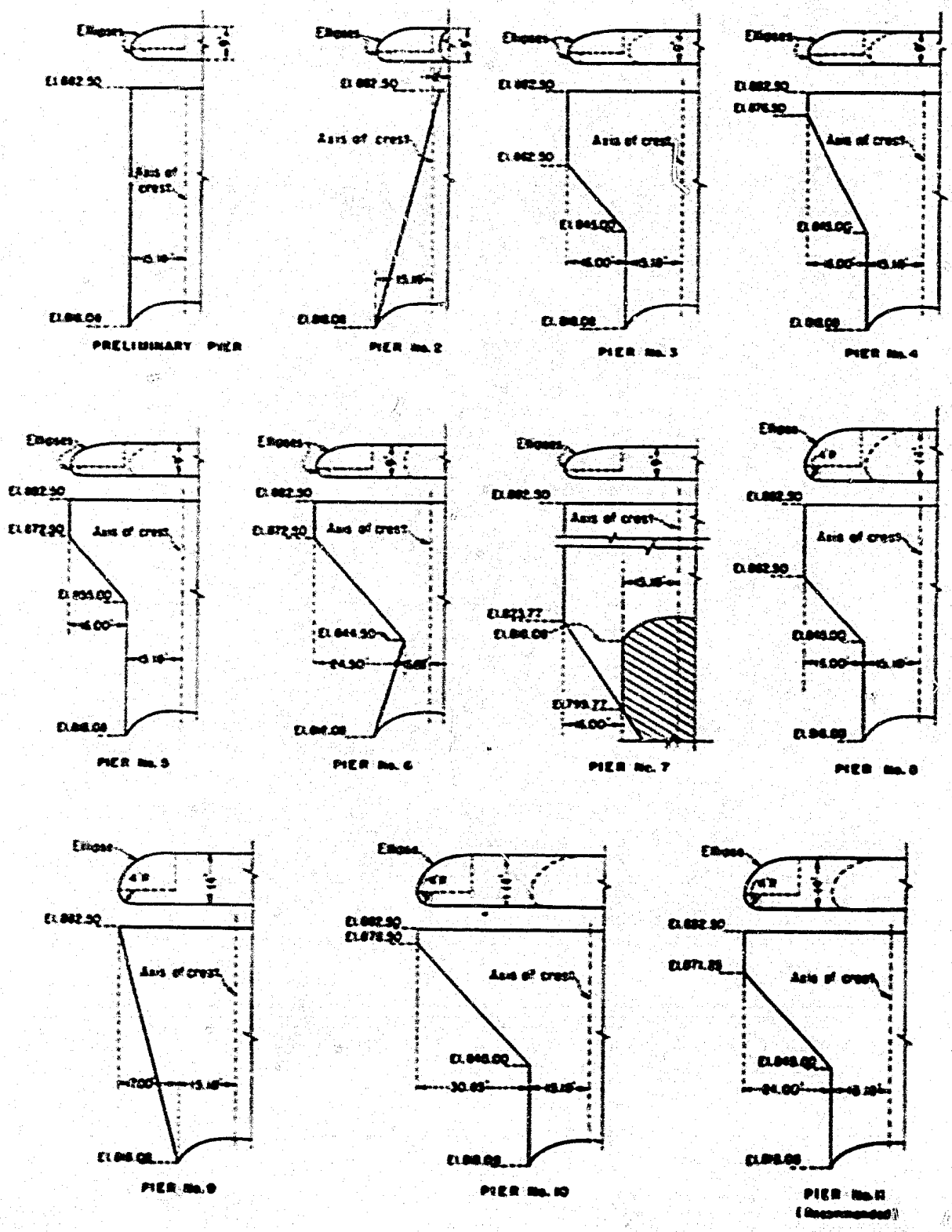
A. The spillway, outlets and log sluice



B. Placing topography in head and tail box

GORGE HIGH DAM
Construction of 1:48 Scale Model

FIGURE 6

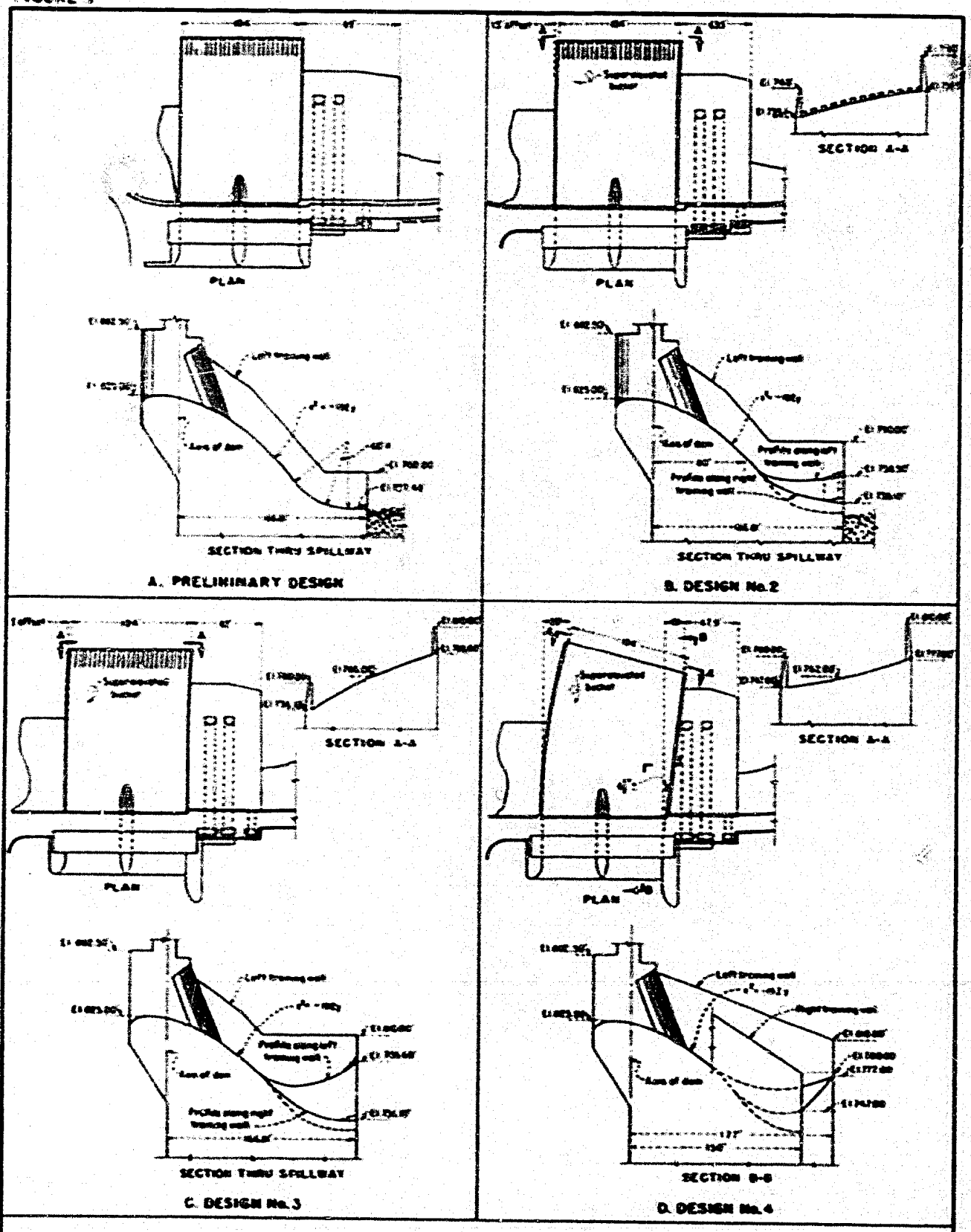


GORGE HIGH DAM

TEST SHAPES OF RIGHT SPILLWAY PIER

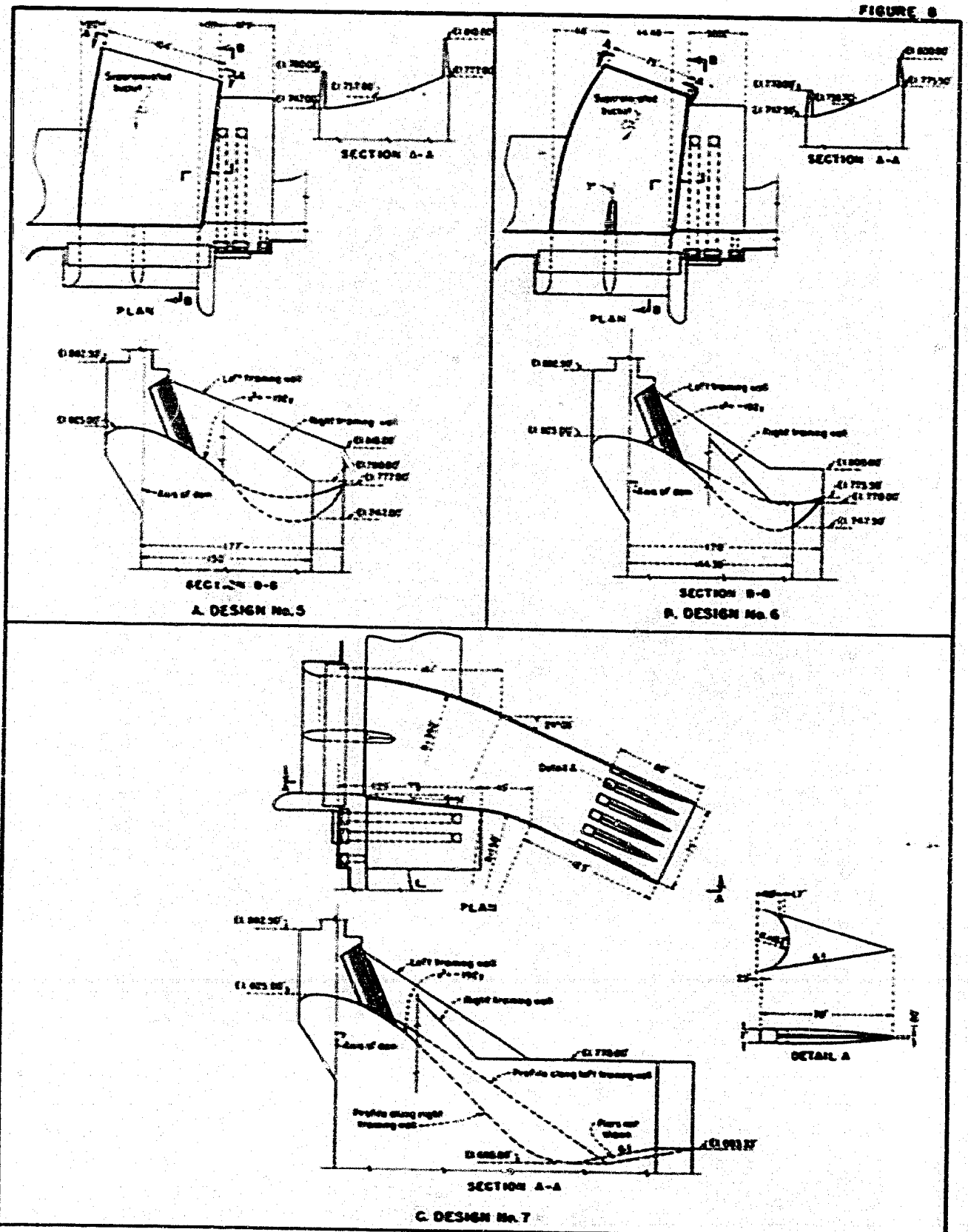
1:40 SCALE MODEL

FIGURE 7



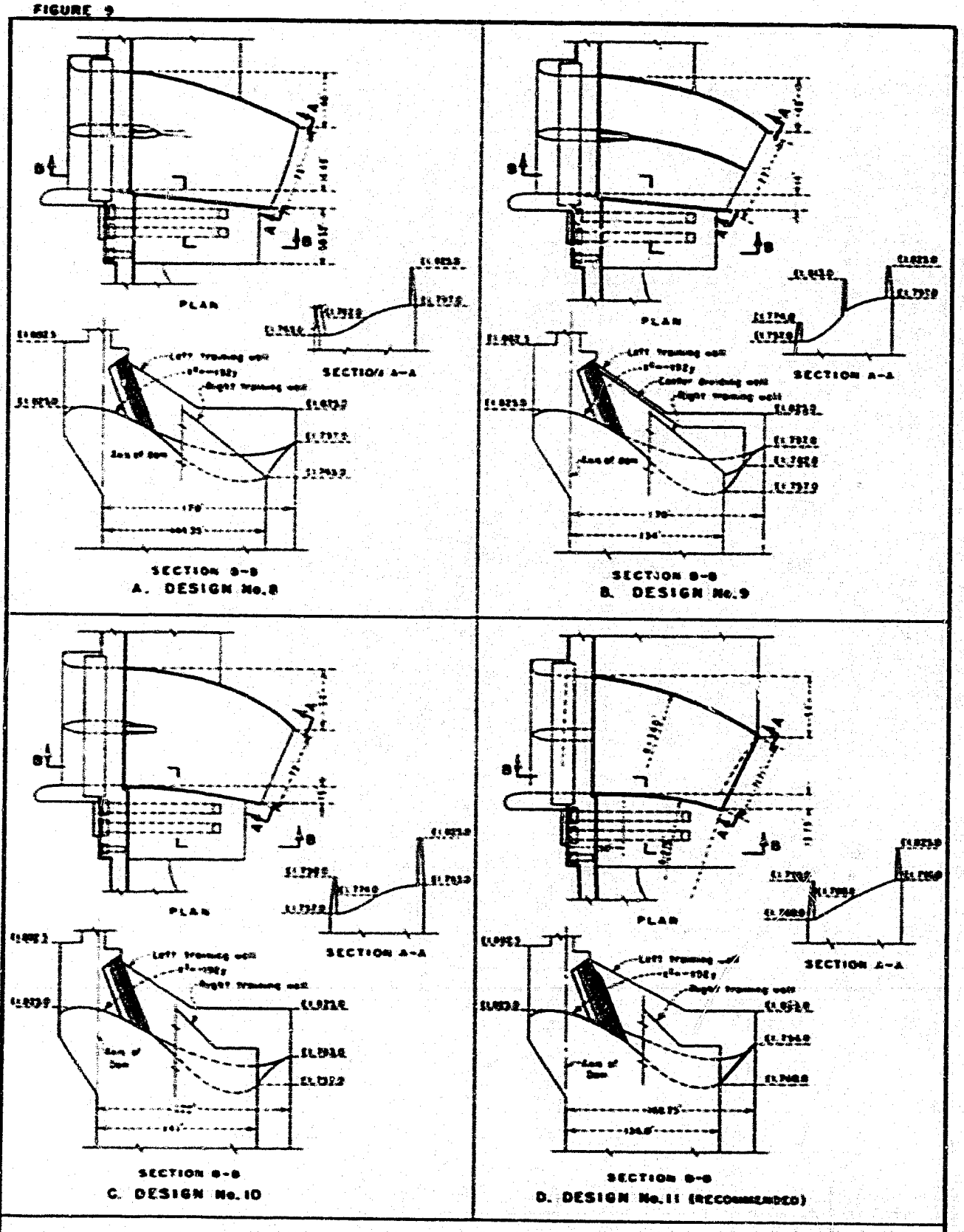
GORGE HIGH DAM SPILLWAY
SPILLWAY DESIGNS TESTED
(Continued on Figures 8 and 9)
1:48 SCALE MODEL

FIGURE 8

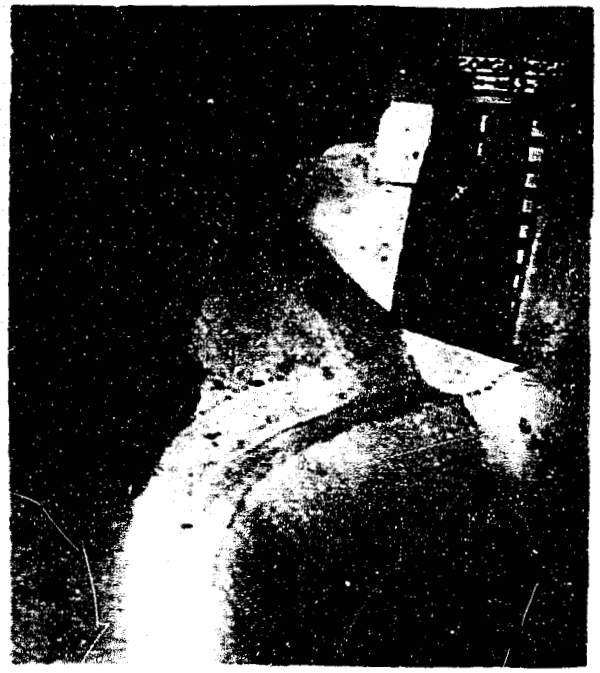


GORGE HIGH DAM SPILLWAY
SPILLWAY DESIGNS TESTED
(Continued on Figure 9)
1:48 SCALE MODEL

FIGURE 9



GORGE HIGH DAM SPILLWAY
SPILLWAY DESIGNS TESTED
(Continued from Figures 7 and 8)
1:40 SCALE MODEL

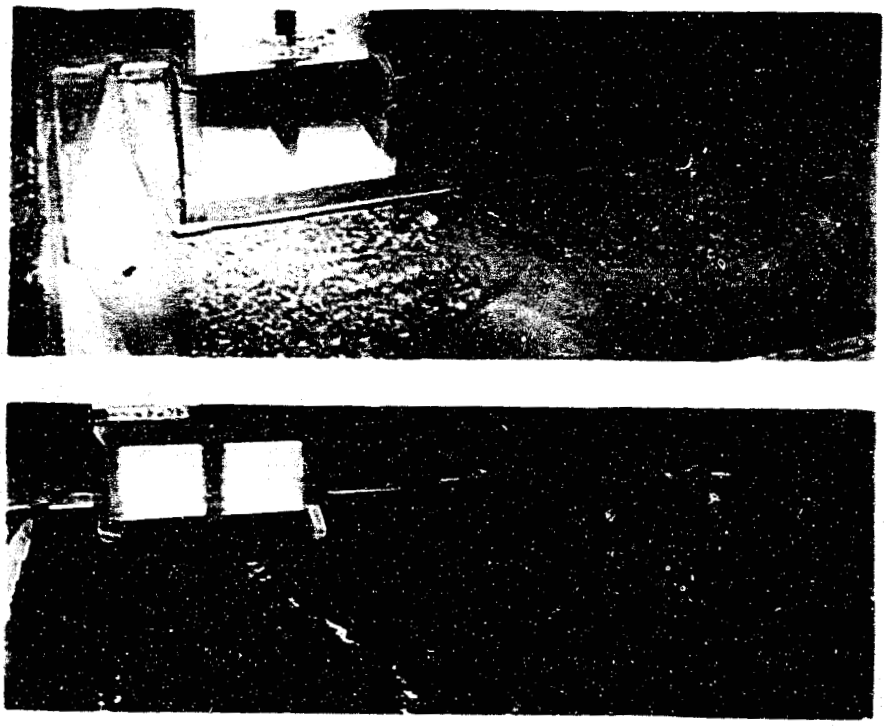


A. Preliminary Design. Note the "pile-up" of water at center pier and rough flow through left spillway bay.

B. Excavation upstream from power intake. Note improvement in flow through left spillway bay.

GORGE HIGH DAM
Spillway Approach Studies
Discharge 140,000 cfs
1:48 Scale Model

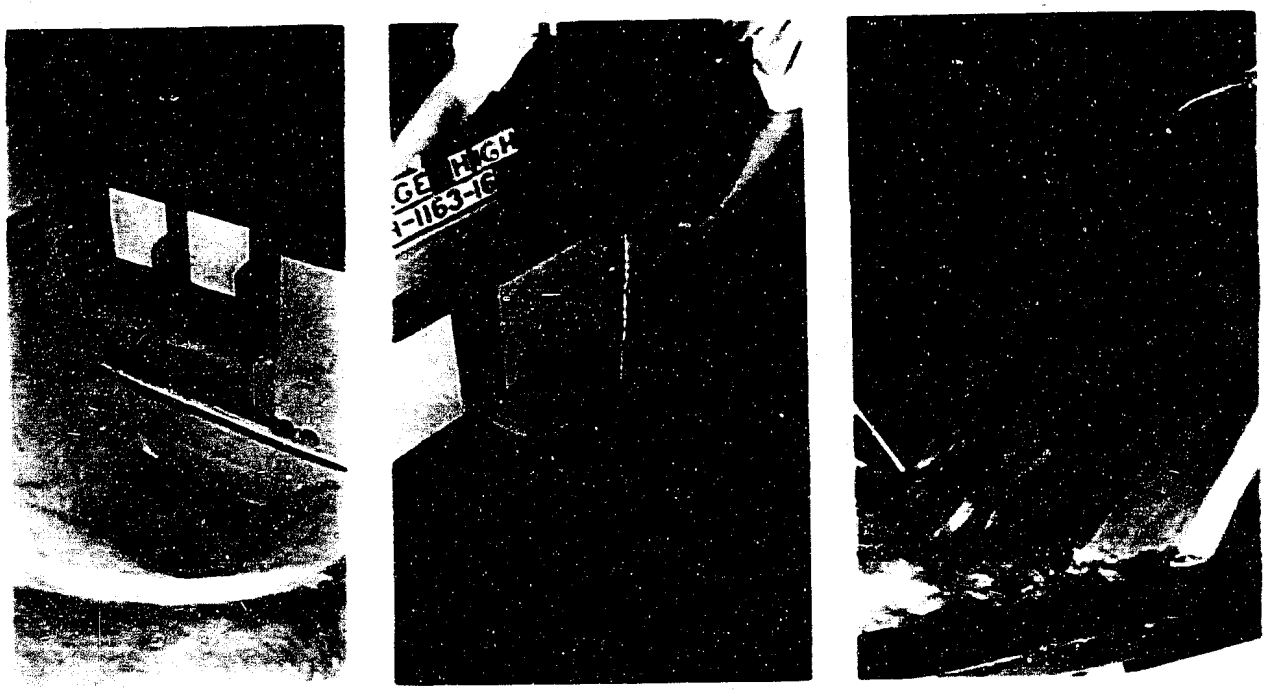
A. Spillway moved 20 feet to right and approach channel placed in model. Excavation holes unfilled.



B. Same as A above, except excavation holes backfilled to elevation 750 feet.

GORGE HIGH DAM
Spillway Approach Studies
Discharge 140,000 cfs
1:48 Scale Model

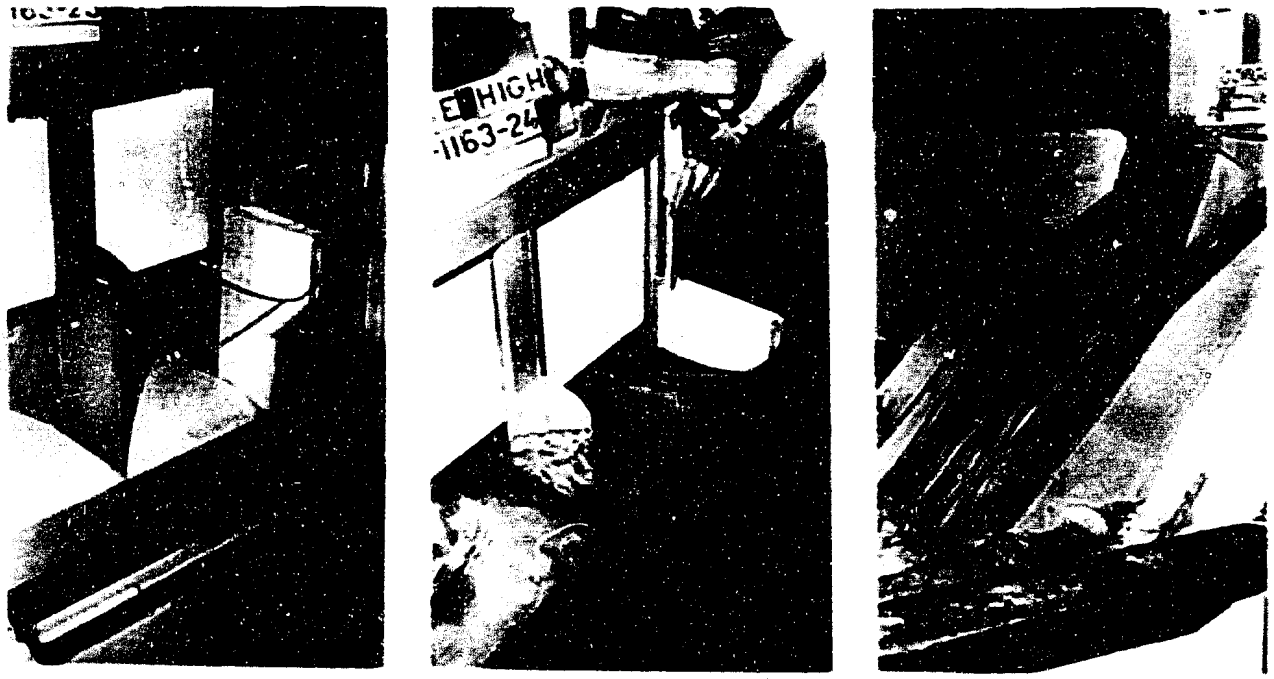
Figure 12



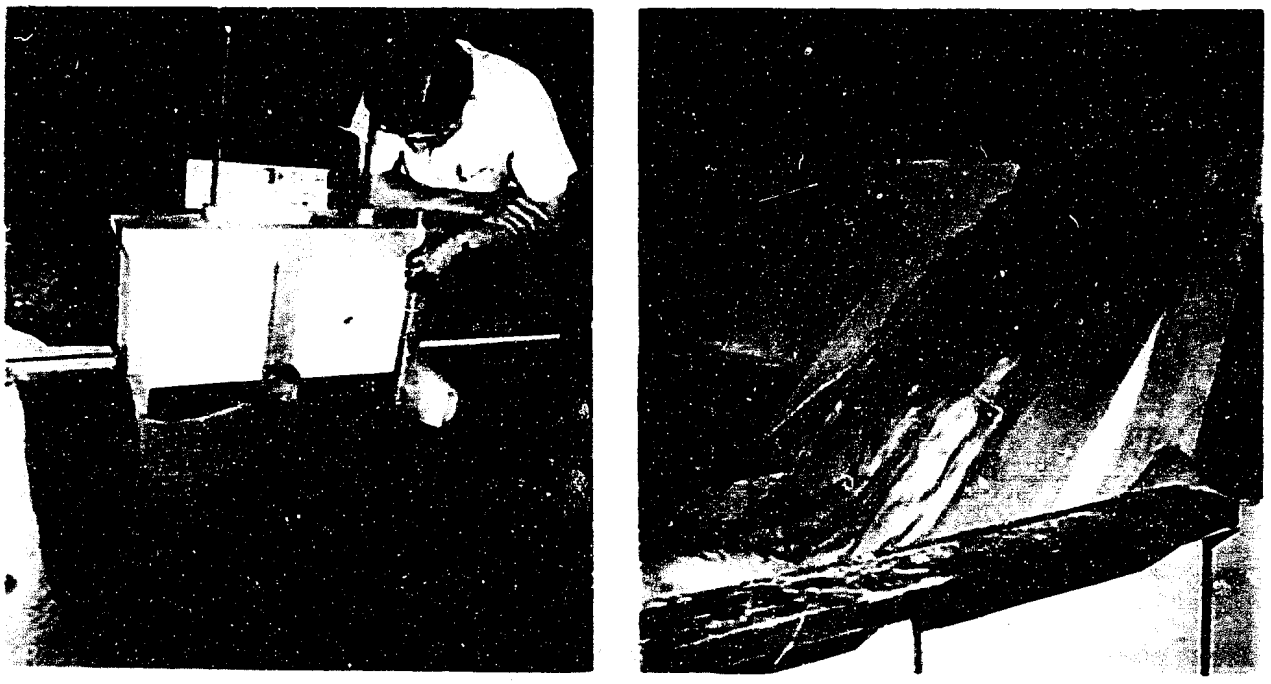
A. Preliminary Pier

B. Pier No. 2

GORGE HIGH SPILLWAY
Studies of Preliminary Pier and Pier No. 2
Discharge 140,000 cfs
1:48 Scale Model

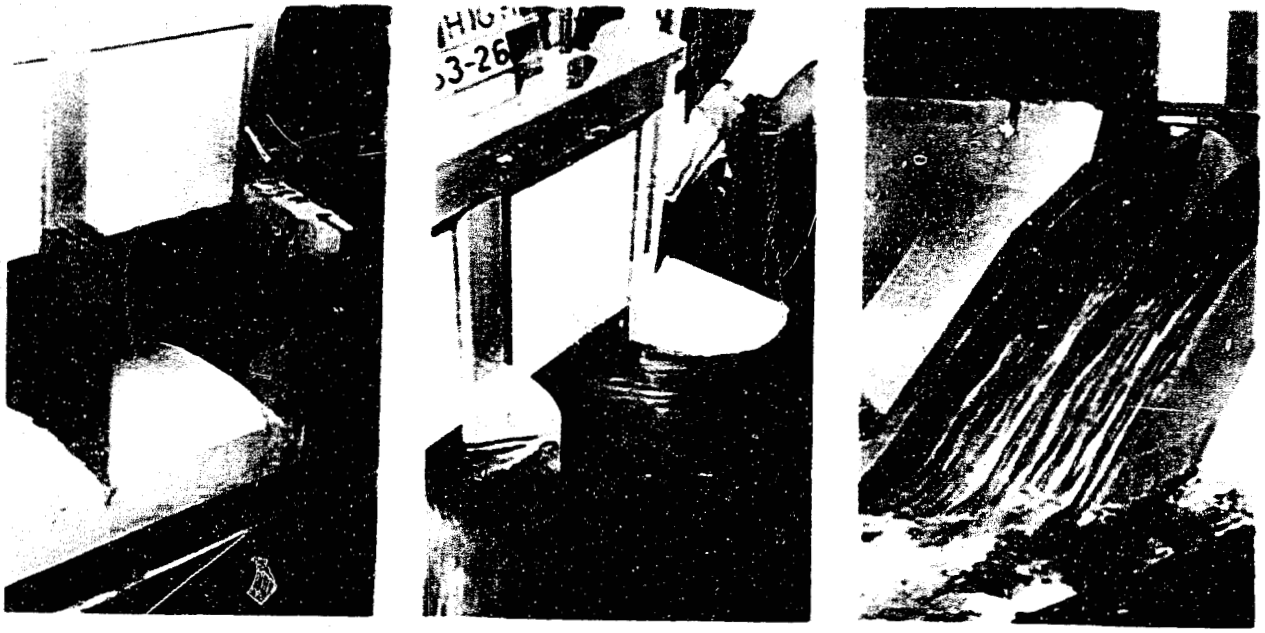


A. Minimum contraction at pier

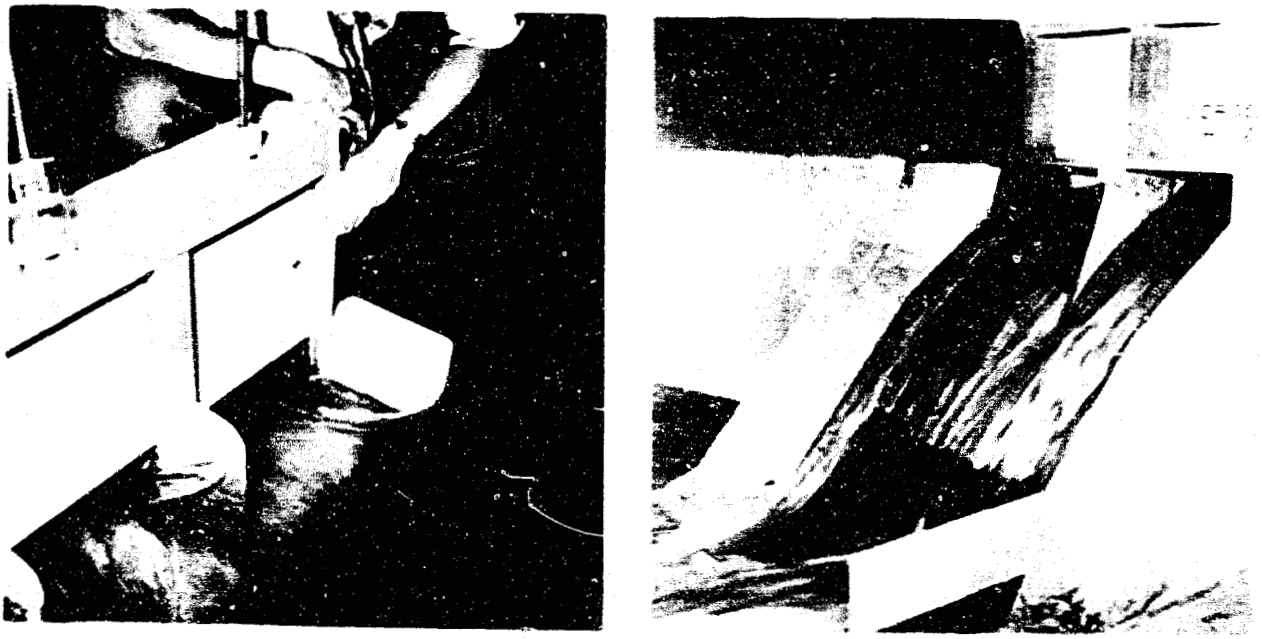


B. Maximum contraction at pier

GORGE HIGH SPILLWAY
Pier No. 3
Discharge 140,000 cfs
1:48 Scale Model

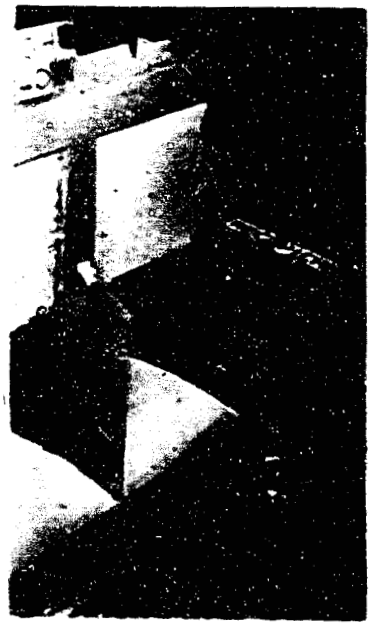


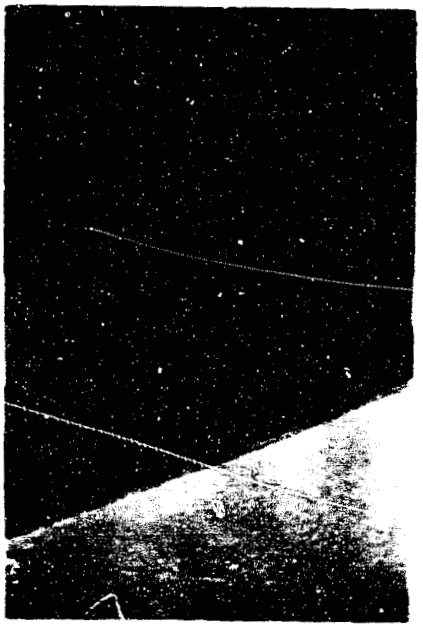
A. Pier No. 4 (minimum contraction)



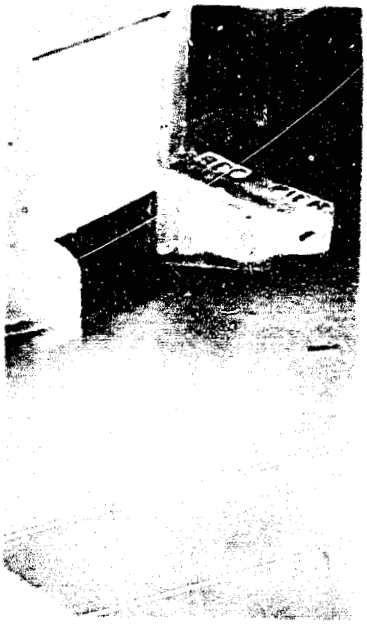
B. Pier No. 8

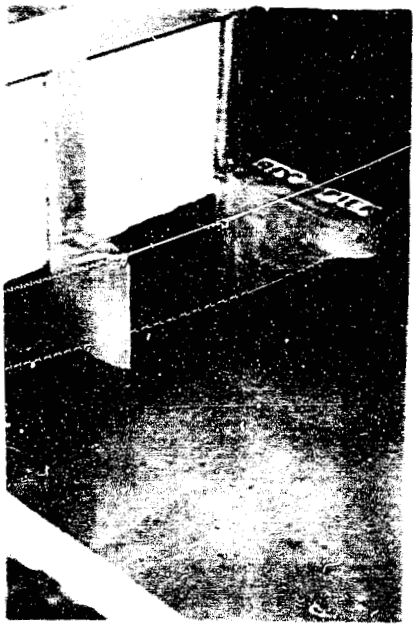
GORGE HIGH SPILLWAY
Pier Nos. 4 and 8
Discharge 145,000 cfs
1:48 Scale Model





A. Discharge 145,000 cfs



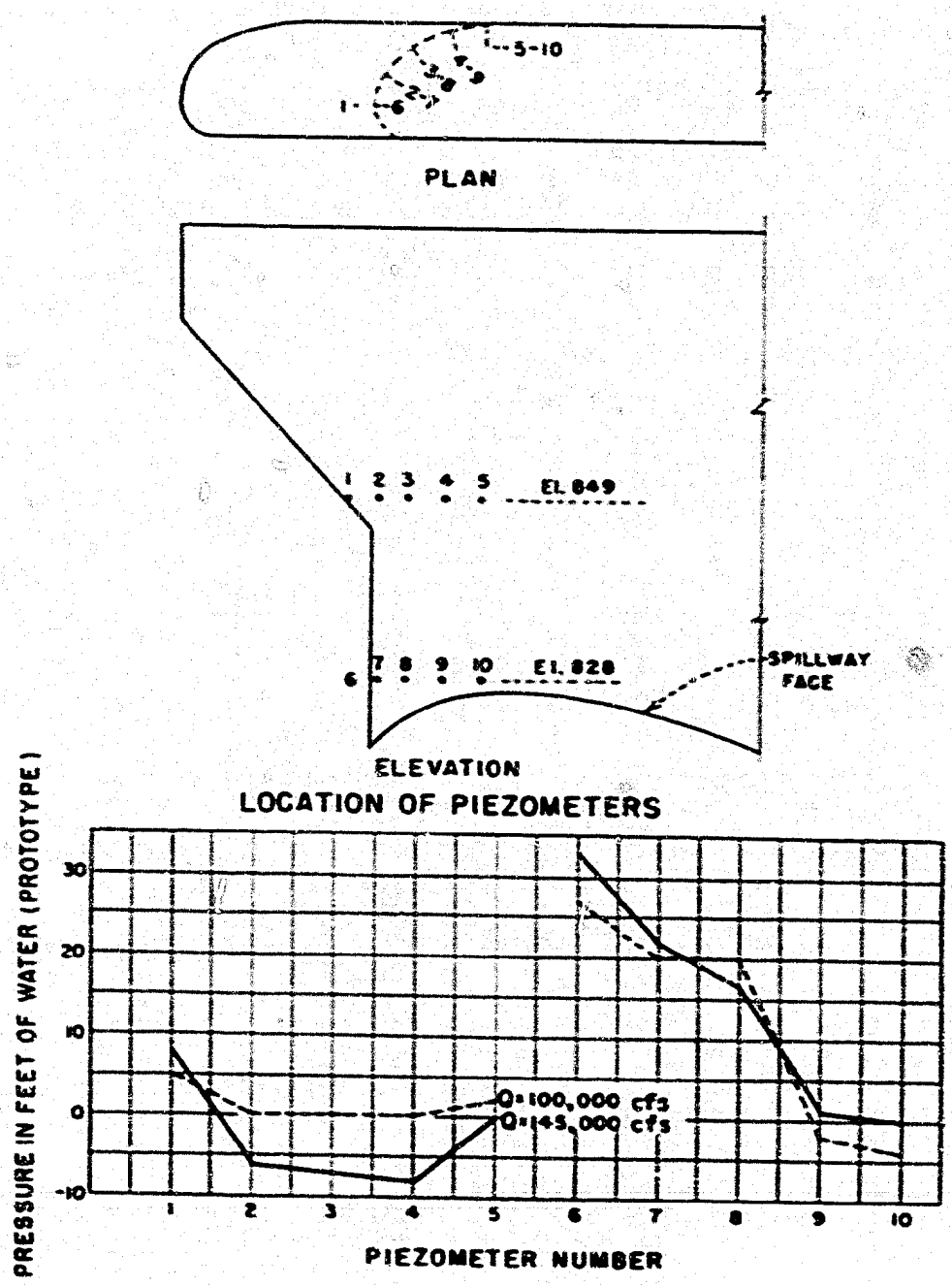


B. Gates fully open
100,000 cfs

C. Gates partly closed
100,000 cfs

D. 50,000 cfs

GORGE HIGH SPILLWAY
Recommended Right Pier
1:48 Scale Model

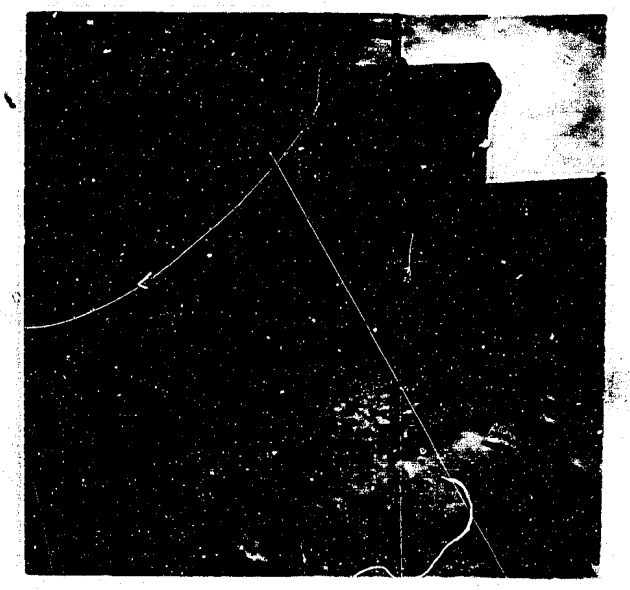


GORGE HIGH DAM SPILLWAY

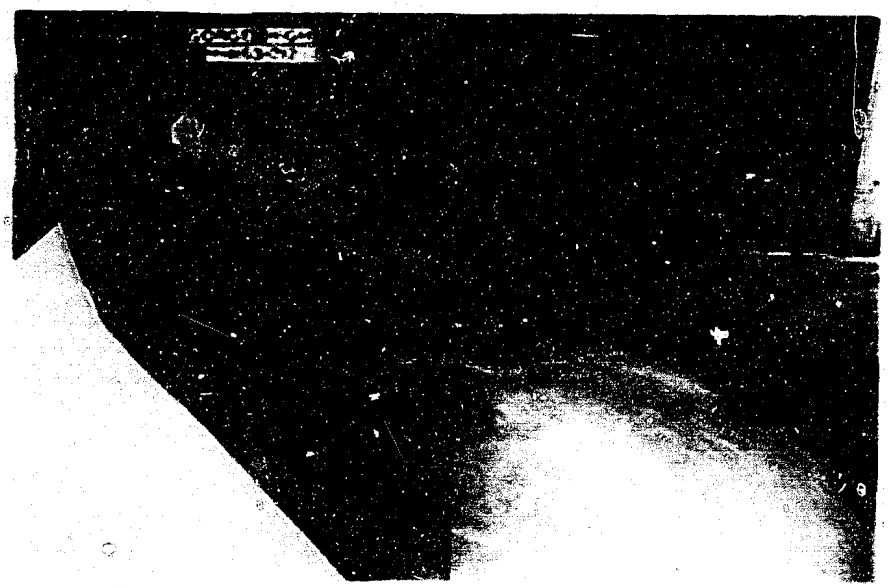
PIEZOMETRIC PRESSURES ON RIGHT SPILLWAY PIER

1:48 SCALE MODEL

A. Preliminary left pier and right pier No. 3, (spillway moved 20 feet toward river channel)



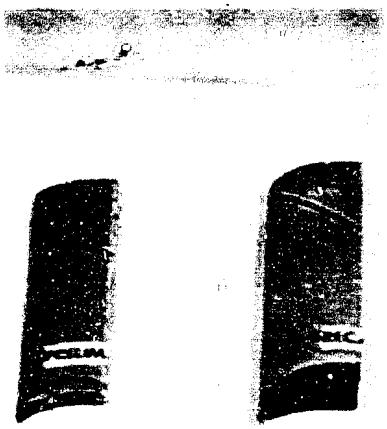
B. Recommended Piers



C. Recommended Piers - Flow along the right training

GORGE HIGH DAM
Left Spillway Pier Studies
145,000 cfs
1:48 Scale Model

A. Test shapes of right spillway pier

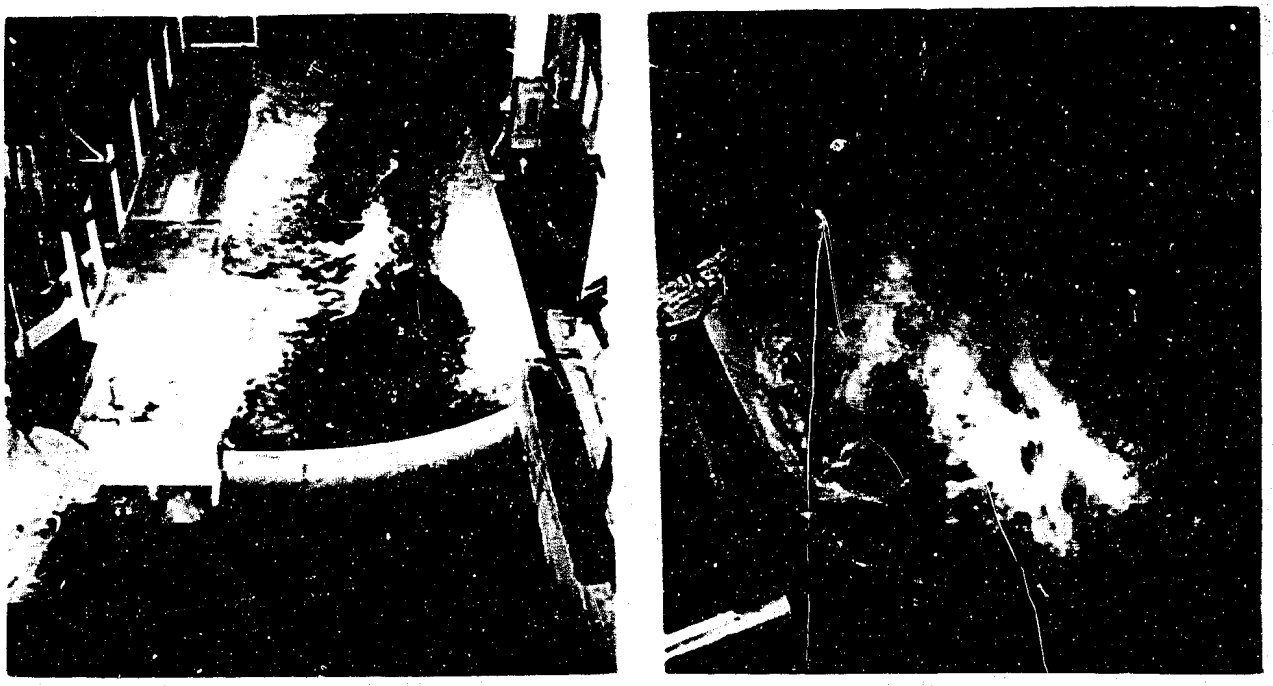


B. Left Pier

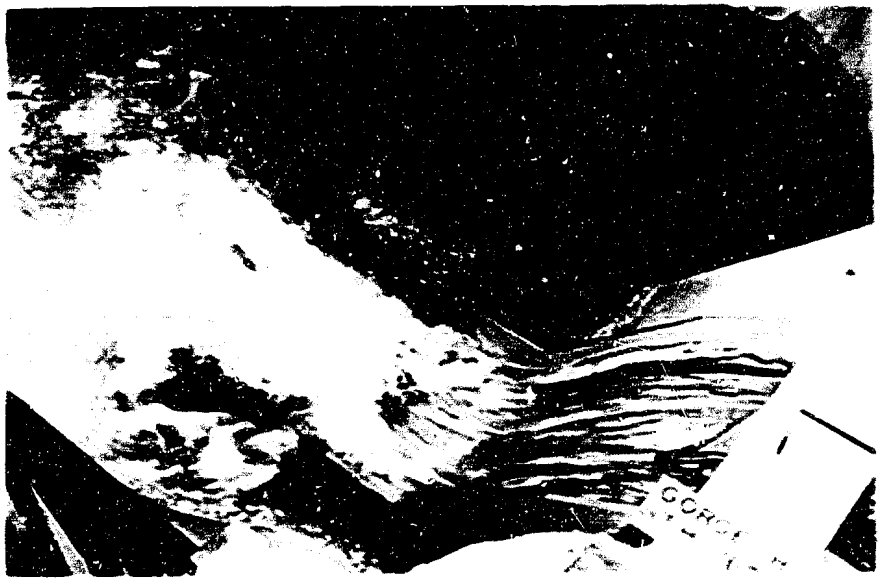
	PIER NO.	DEPTH OF CONTRACTION	% DECREASE IN DISCHARGE	REMARKS
RIGHT PIER	Prelim.	38 Ft.	5.9	High fin along right training wall.
	2	57	8.5	Spillway face exposed
	3	23–44	2.8	Contraction fluctuates between two extremes.
	4	24–41	3.0	
	5	—	—	No data taken
	6	40	5.3	High fin along right training wall.
	7	35	3.8	Contraction recovers to 21 feet.
	8	19	1.3	Fin along right training wall.
	9	24	—	No data on decrease in Q.
	10	14	1.5	Very good flow conditions
	11 (Rec.)	17	0.2	
LEFT PIER	Prelim.	23	—	Similar flow characteristics.
	Rec	21	—	

C. Recapitulation of spillway pier studies

GORGE HIGH SPILLWAY
Pier Studies
1:48 Scale Model

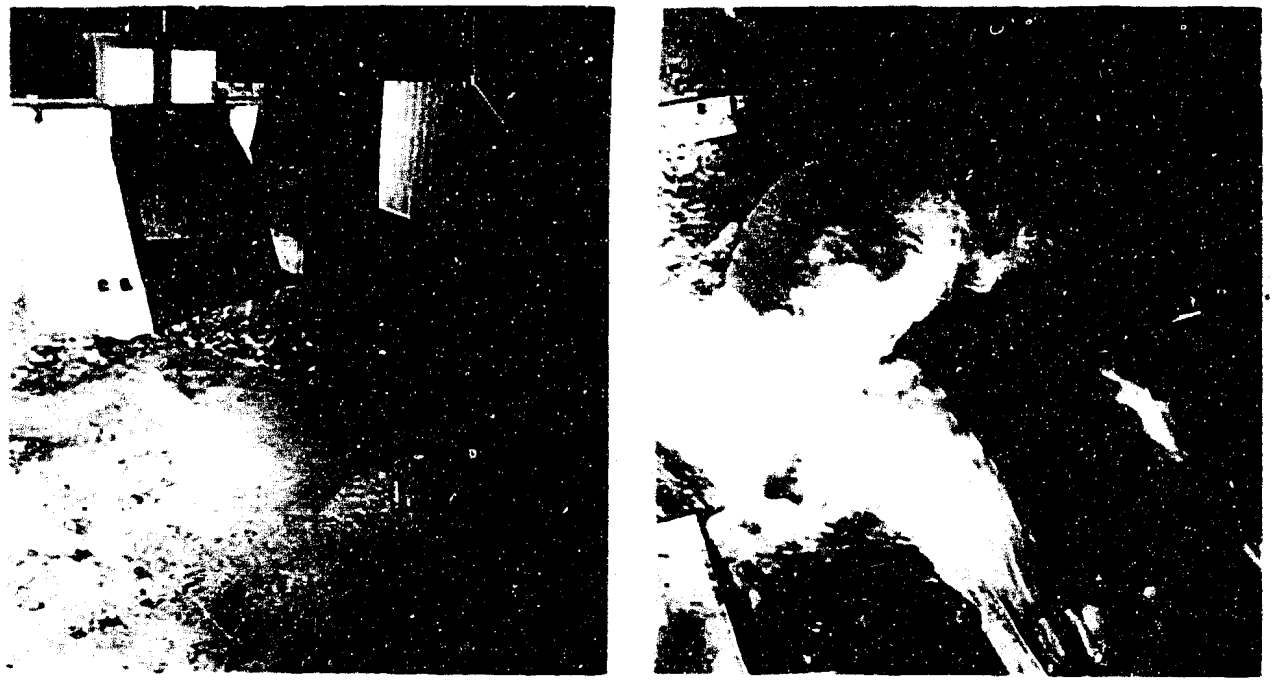


A. Discharge 140,000 cfs

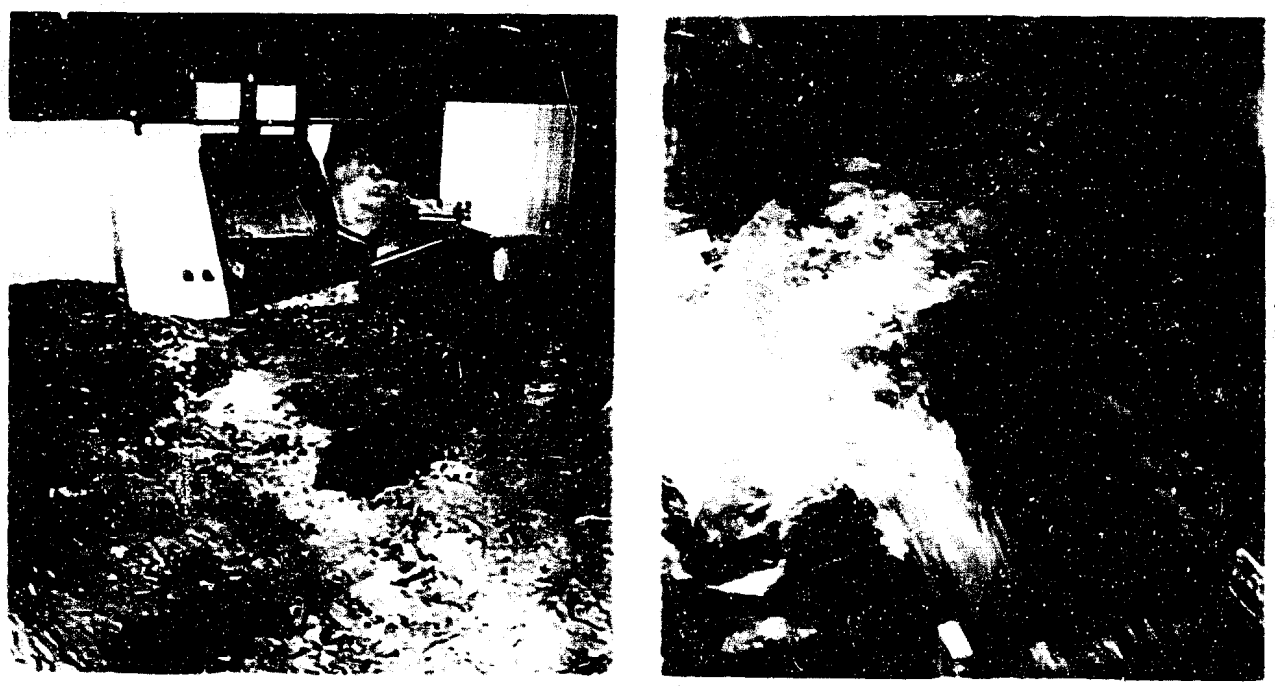


B. Discharge 100,000 cfs

GORGE HIGH DAM
Stilling Basin Studies
Preliminary Design
1:48 Scale Model

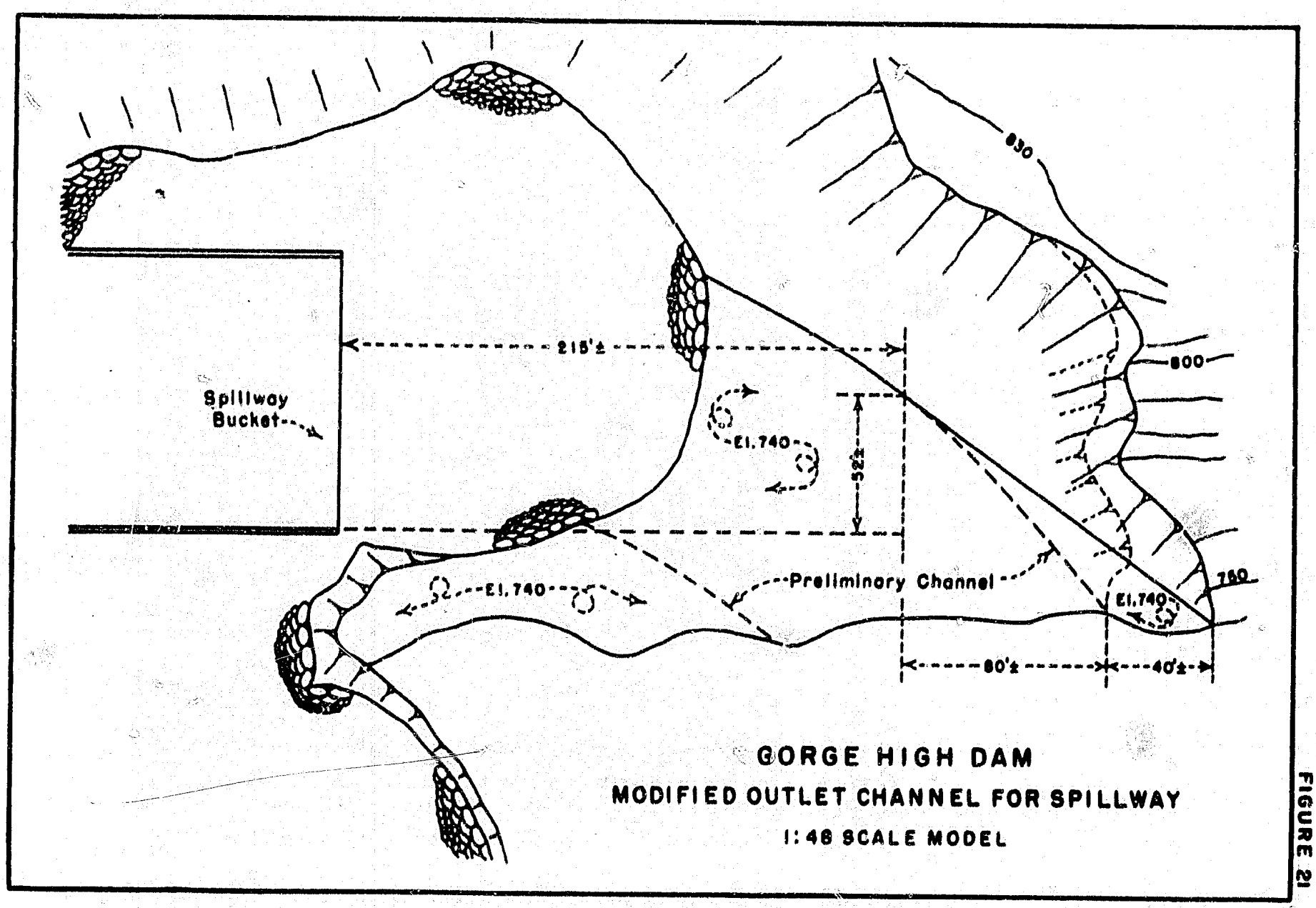


A. Before downstream end of excavated channel was widened as indicated by rule.



B. After channel was widened.

GORGE HIGH-DAM
Stilling Basin Studies
Design No. 2, 140,000 cfs
1:48 Scale Model



GORGE HIGH DAM

MODIFIED OUTLET CHANNEL FOR SPILLWAY

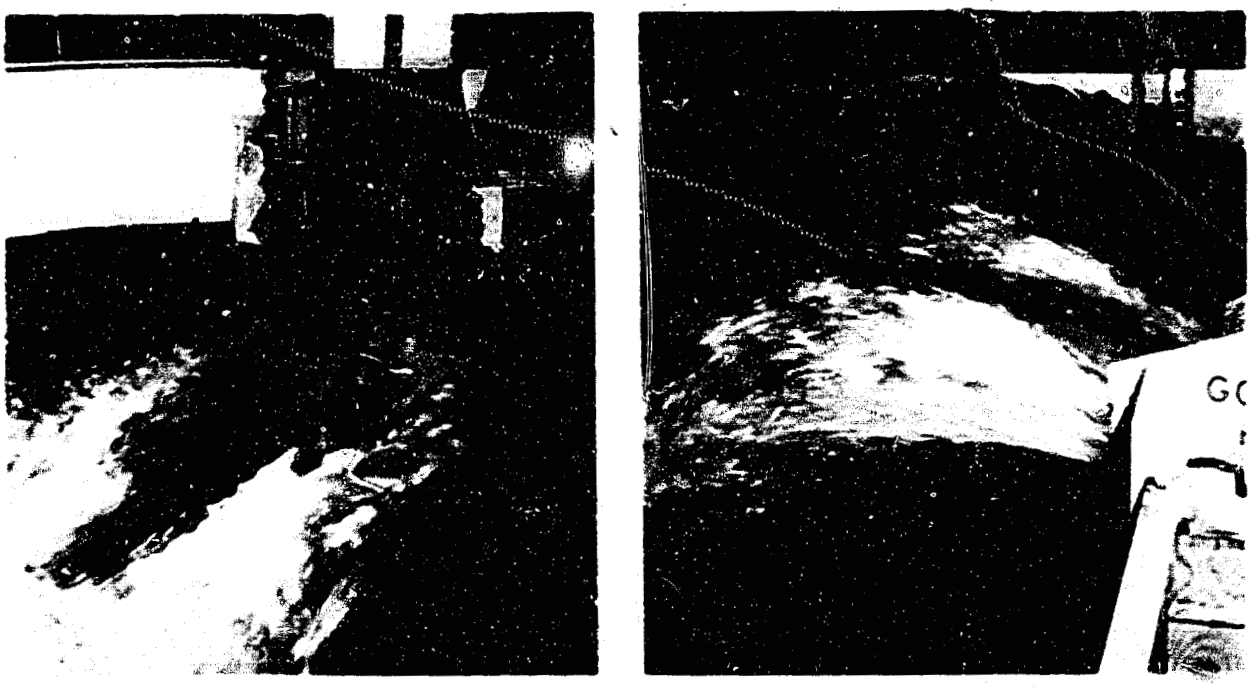
1:48 SCALE MODEL

FIGURE 21

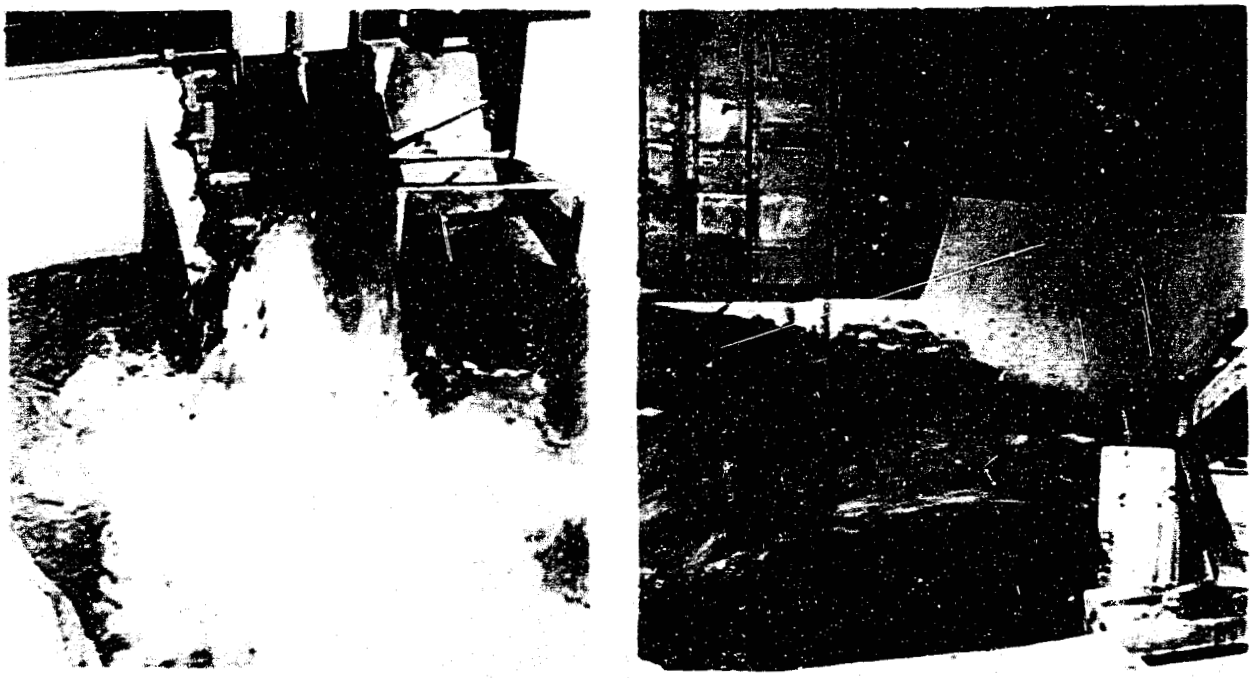
A. Design No. 3

B. Design No. 4

GORGE HIGH DAM
Stilling Basin Studies
Design Nos. 3 and 4, 145,000 cfs
1:48 Scale Model



A. Design No. 5



B. Design No. 6

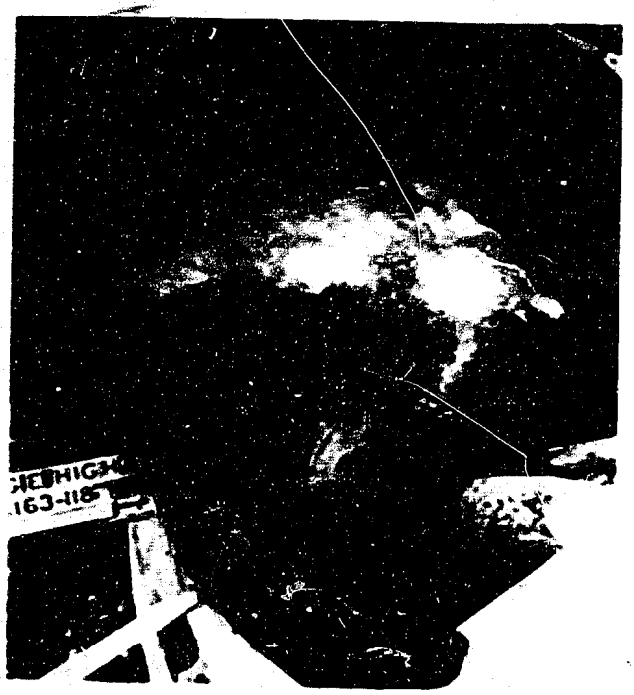
GORGE HIGH DAM
Stilling Basin Studies
Design Nos. 5 and 6, 145,000 cfs
1:48 Scale Model

A. Both gates equally open

B. Right gate closed

57,500 cfs - Reservoir El. 875

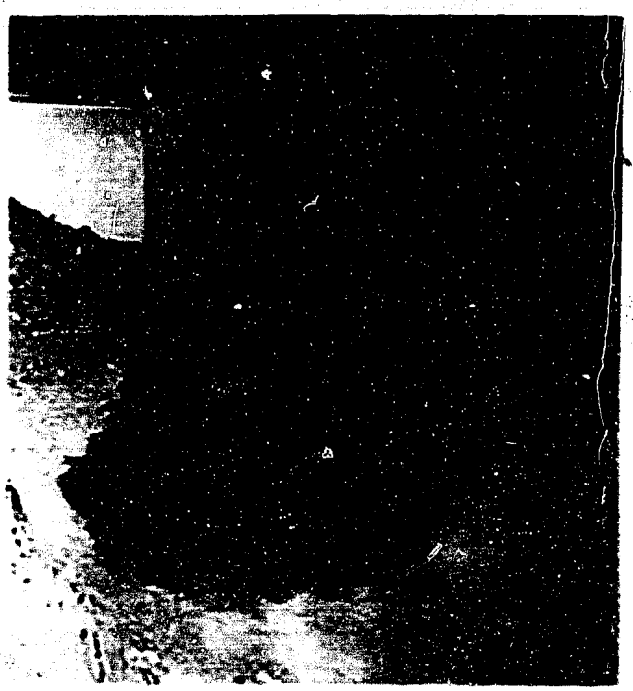
C. Both gates equally open



D. Right gate closed

25,000 cfs - Reservoir El. 875

GORGE HIGH SPILLWAY
Stilling Basin Studies
Design No. 6
1:48 Scale Model



A. Design No. 7

B. 25,000 cfs

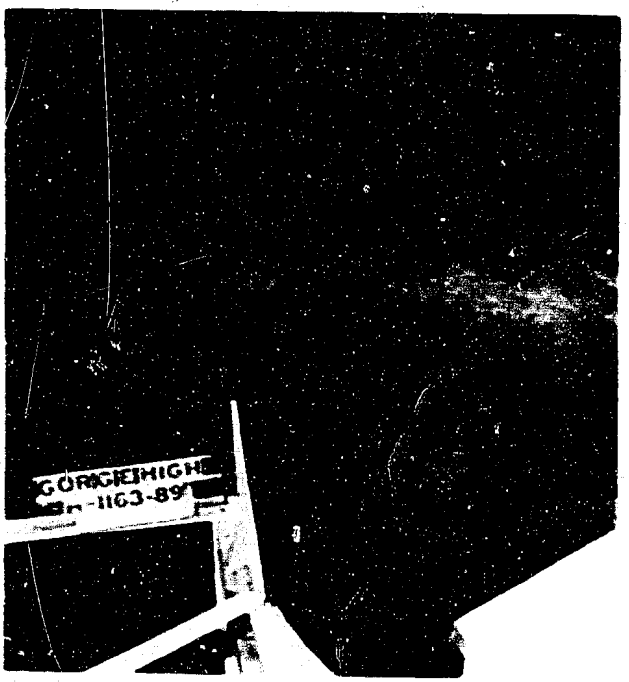
C. 75,000 cfs

D. 145,000 cfs

GORGE HIGH DAM
Stilling Basin Studies
Design No. 7
1:48 Scale Model

A. Discharge 145,000 cfs

B. Left gate discharging 57,500 cfs

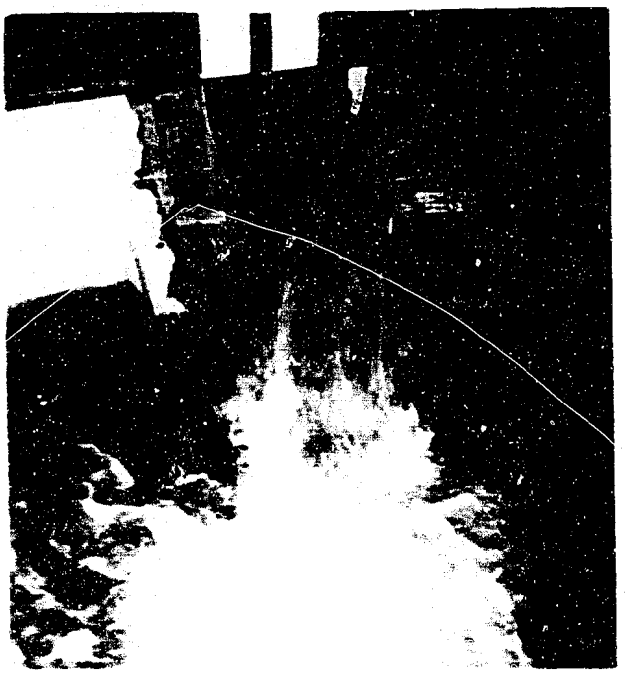


C. Left gate discharging 25,000 cfs

GORGE HIGH DAM
Stilling Basin Studies
Design No. 8
1:48 Scale Model

A. Design No. 9

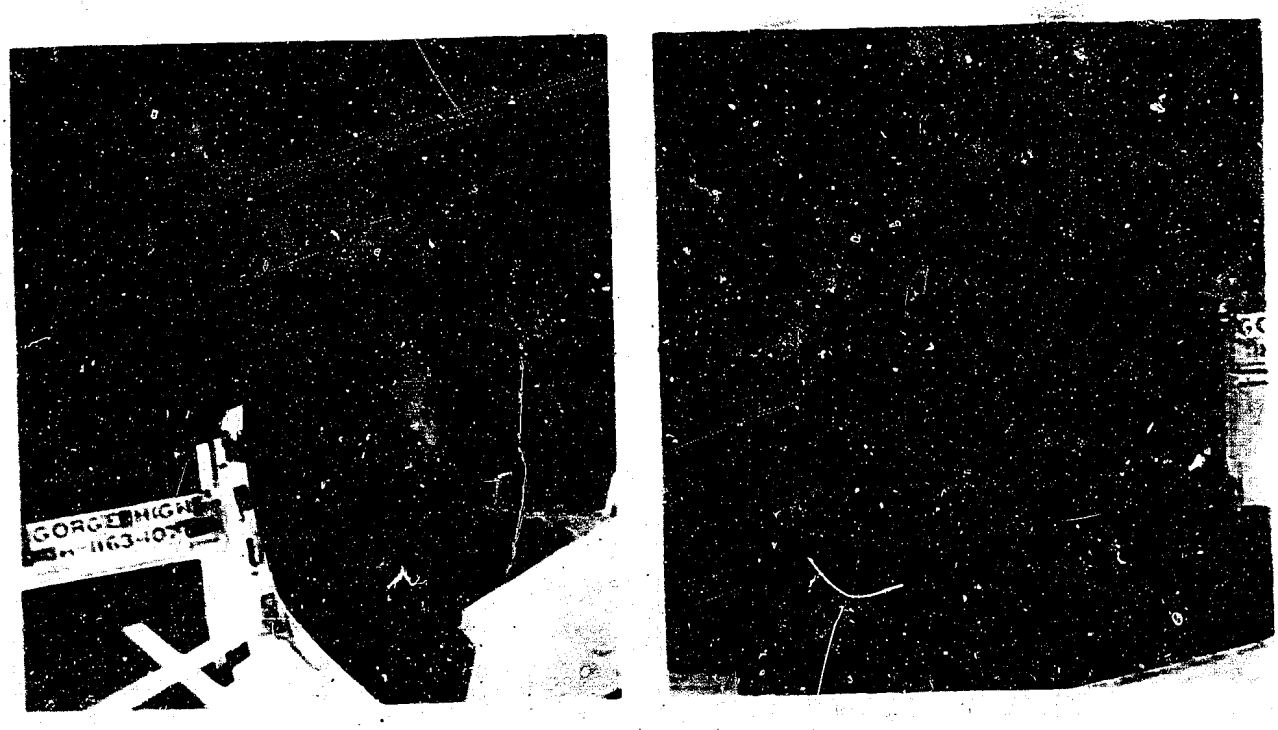
B. Design No. 9 with dividing wall cut back 50 feet



C. Design No. 9 with dividing wall cut back 50 feet from end of bucket

GORGE HIGH DAM
Stilling Basin Studies
Design No. 9, 145,000 cfs
1:48 Scale Model

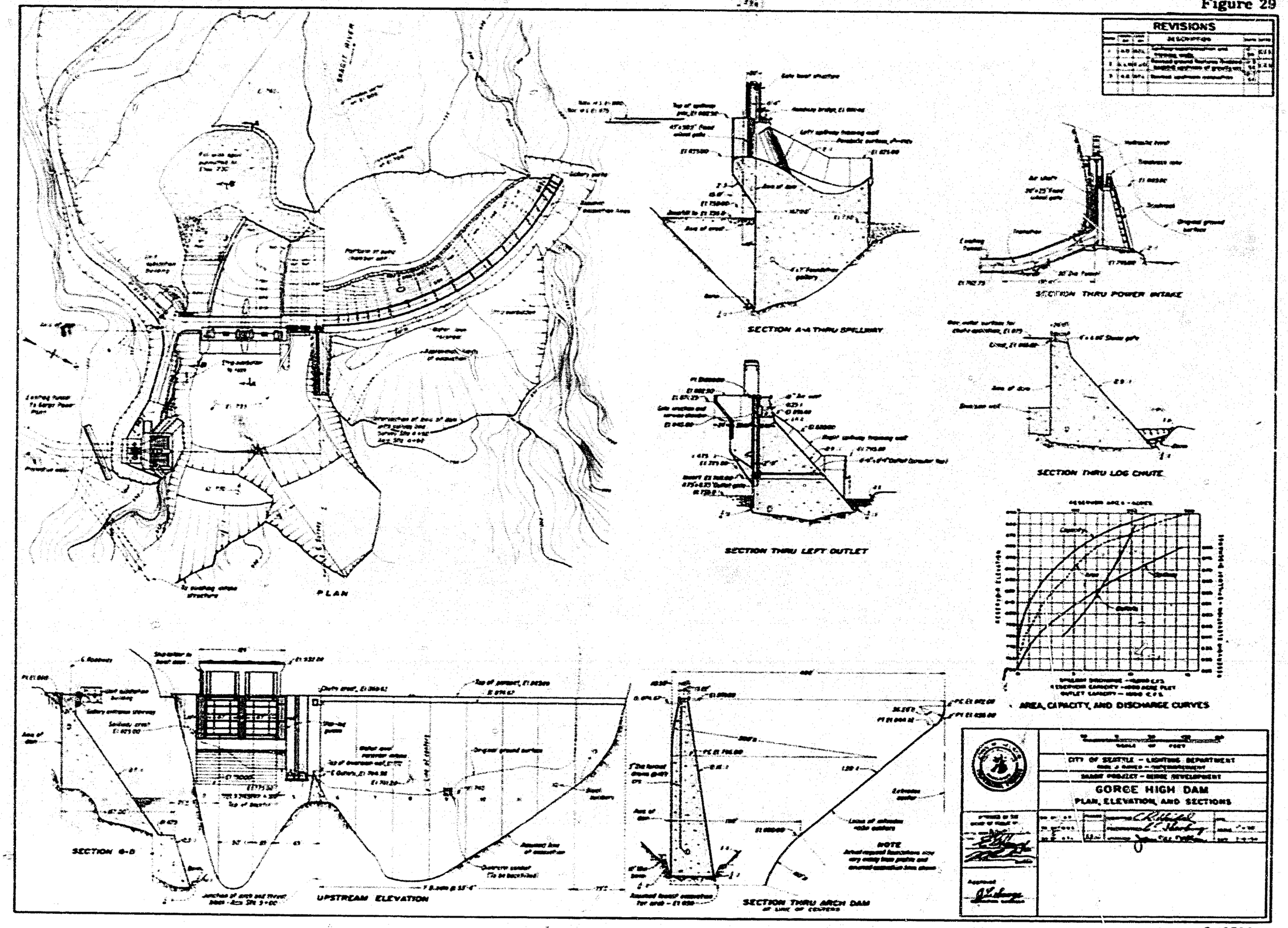
A. Design No. 10



B. Design No. 10 with left lip of bucket raised from elevation 783 to 791 feet

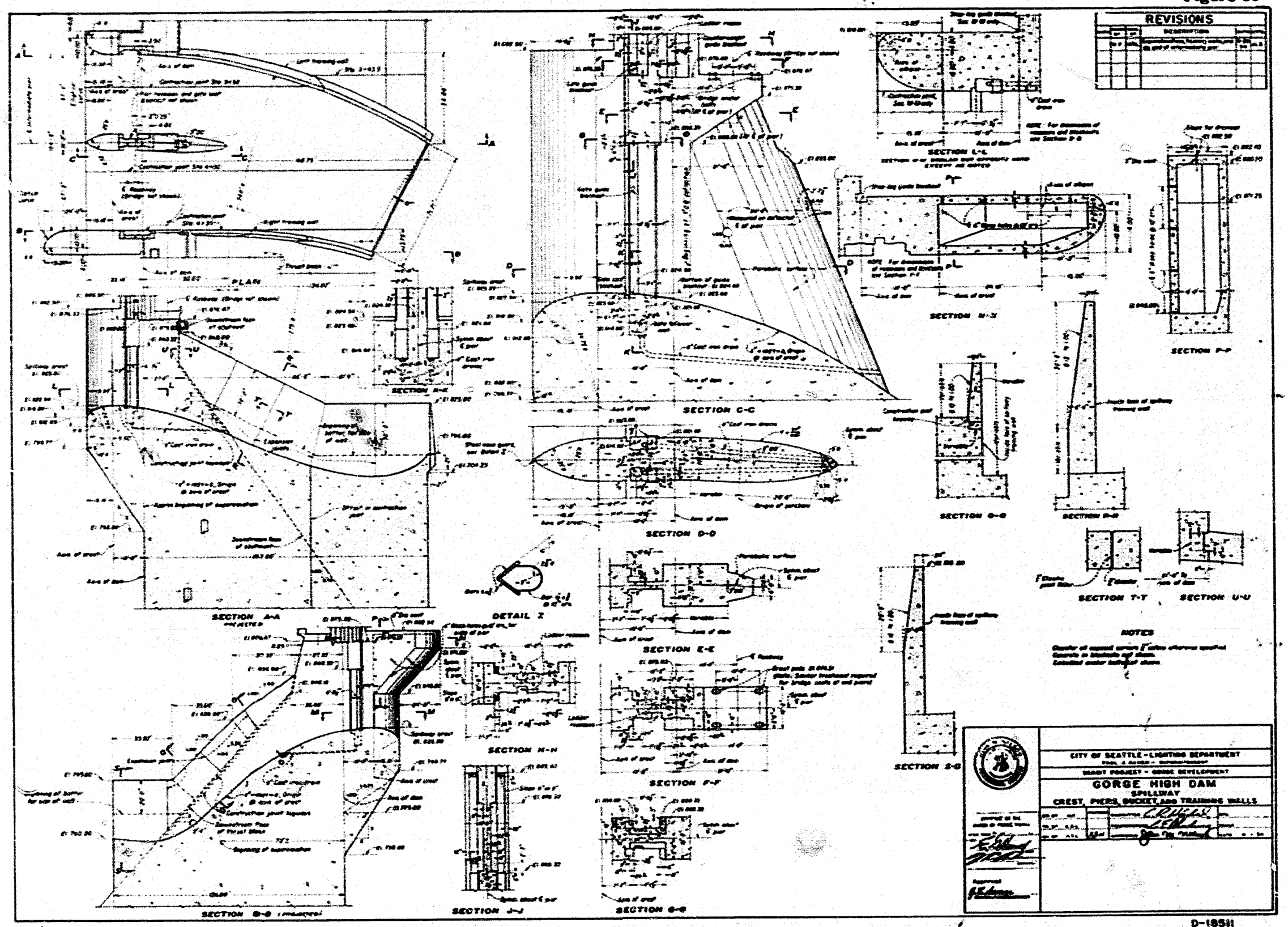
GORGE HIGH DAM
Stilling Basin Studies
Design No. 10, 145,000 cfs
1:48 Scale Model

Figure 29

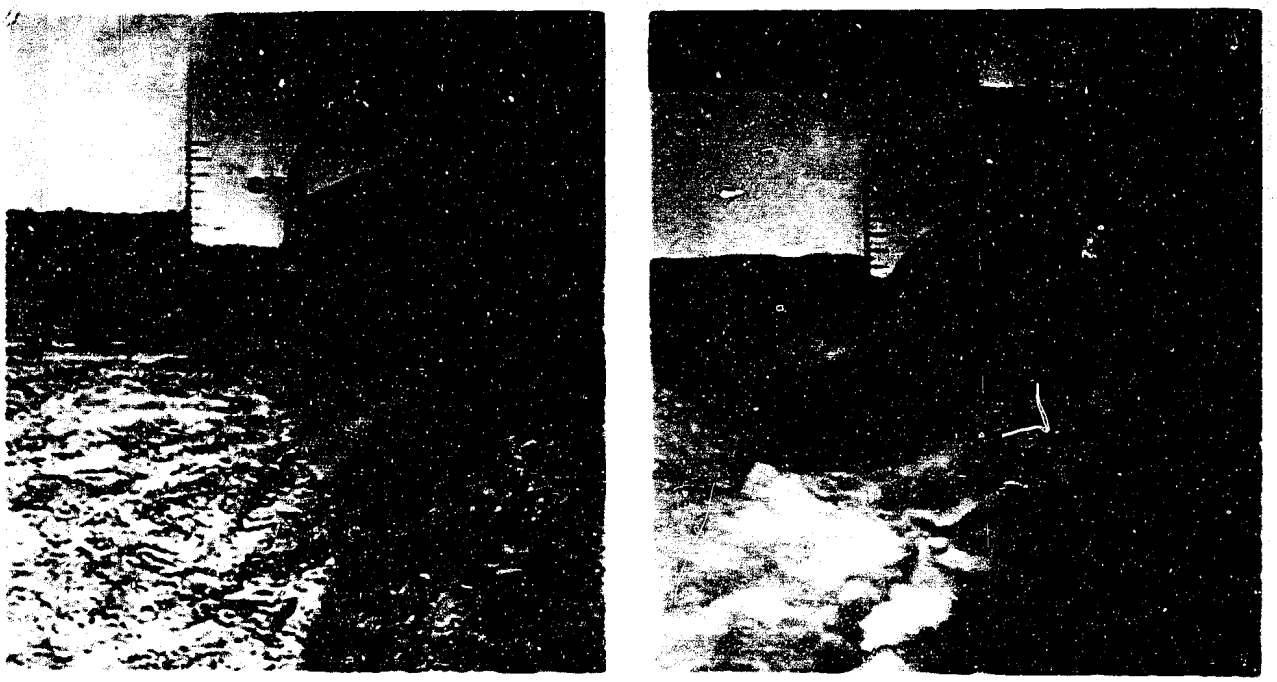


D-18506

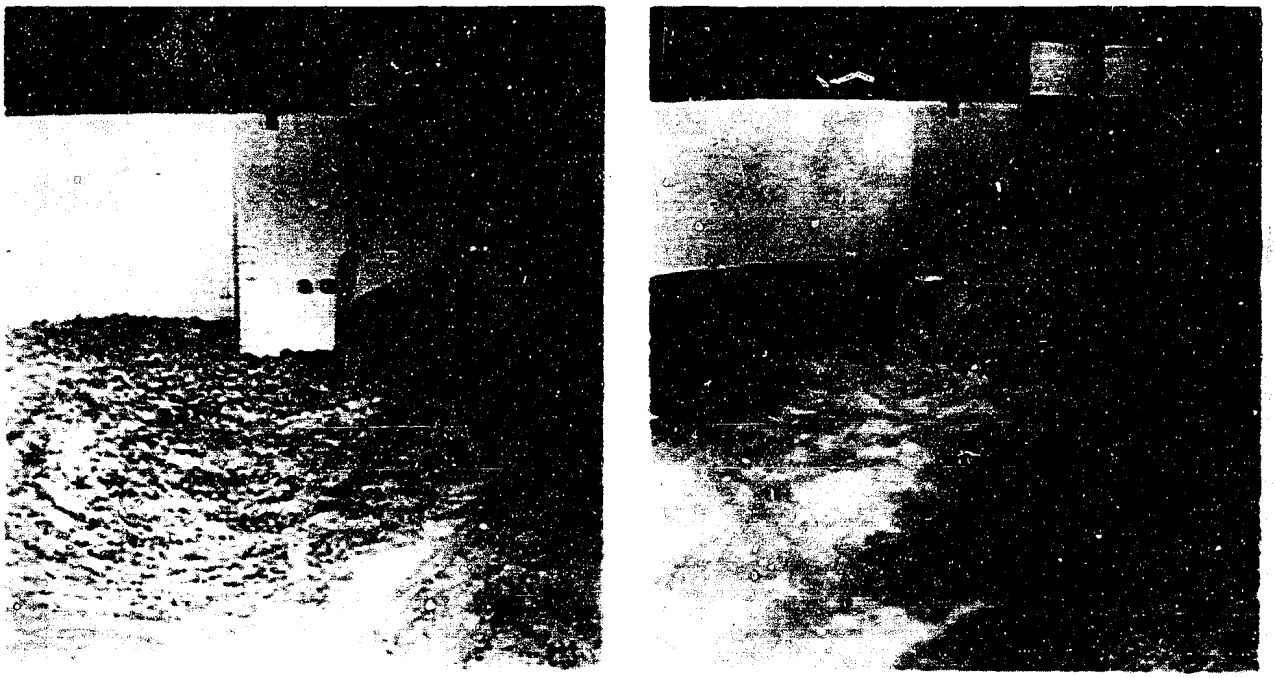
Figure 30



D-18511

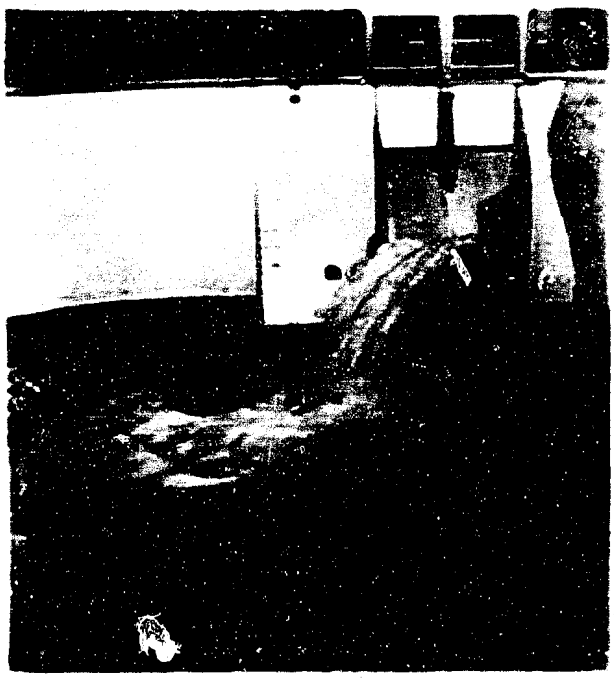


A. Stable river channel with no erosion

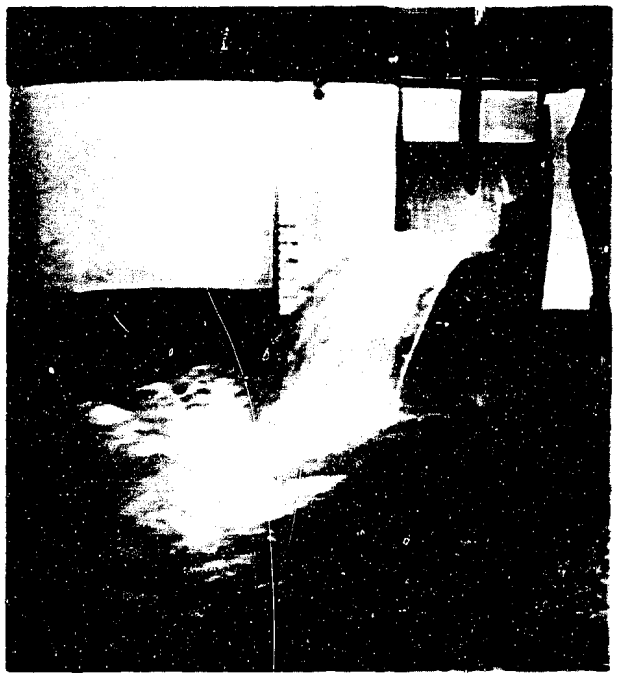


B. Composite eroded channel, stabilized after discharge of 57,500 cfs

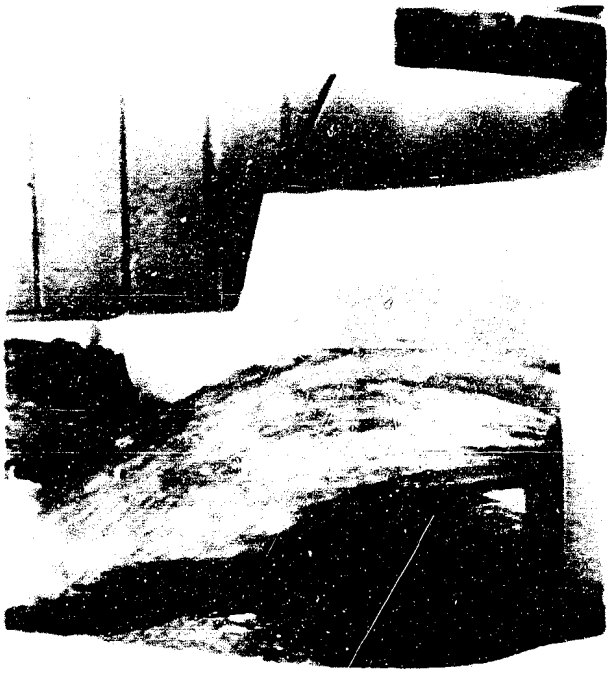
GORGE HIGH DAM
Effect of Stable Channels on Flow Distribution
Discharge 145,000 cfs
1:48 Scale Model



A. 10,000 cfs
Reservoir El. 877.6

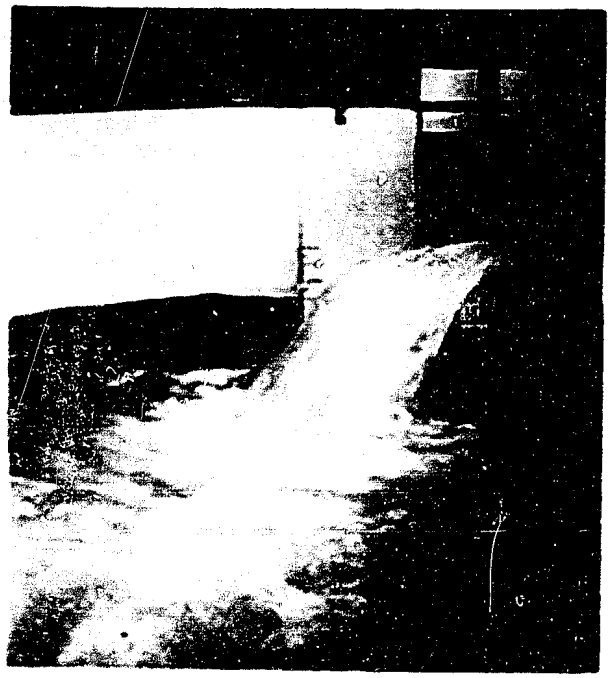


B. 25,000 cfs
Reservoir El. 881.6



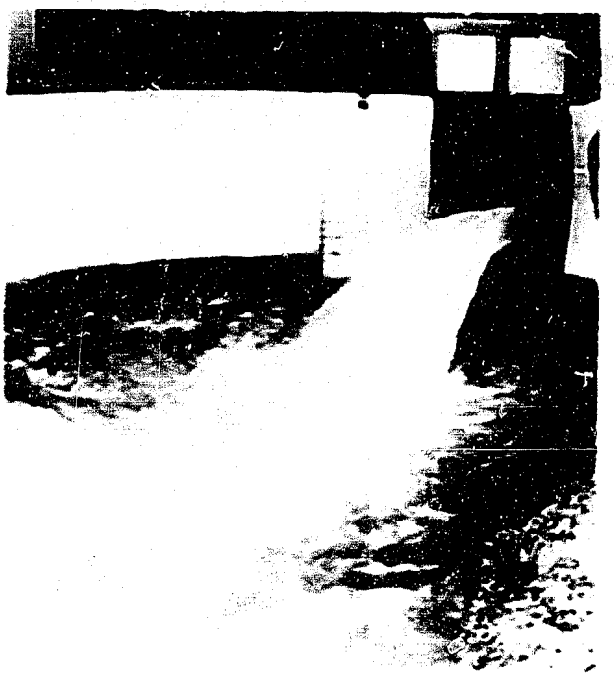
C. Discharge 60,000 cfs - Reservoir El. 882.5

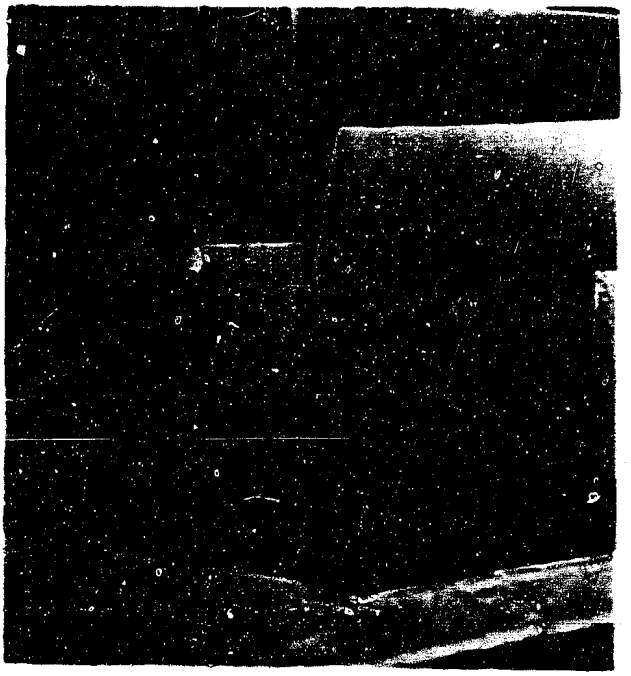
GORGE HIGH DAM
Recommended Design
Both Gates Open Equally
1:48 Scale Model



A. 100,000 cfs

B. 145,000 cfs



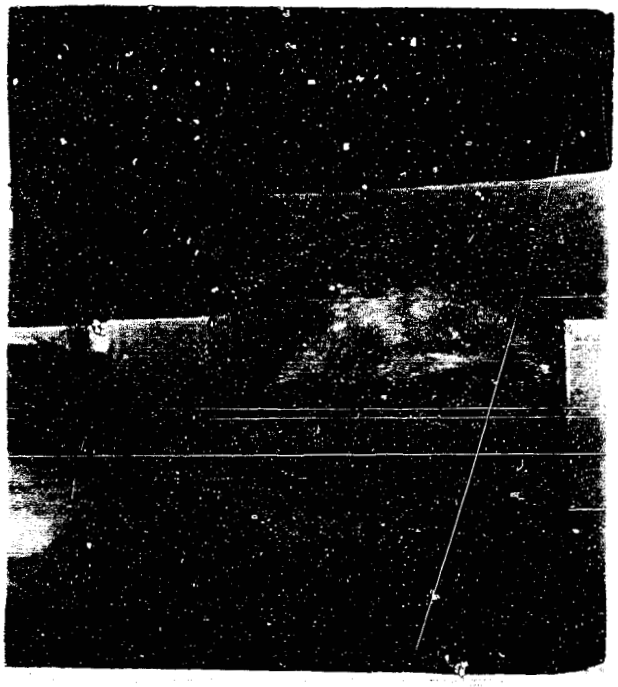


C. Discharge 145,000 cfs

GORGE HIGH DAM
Recommended Design
Both Gates Open Equally
Reservoir Elevation 881
1:48 Scale Model

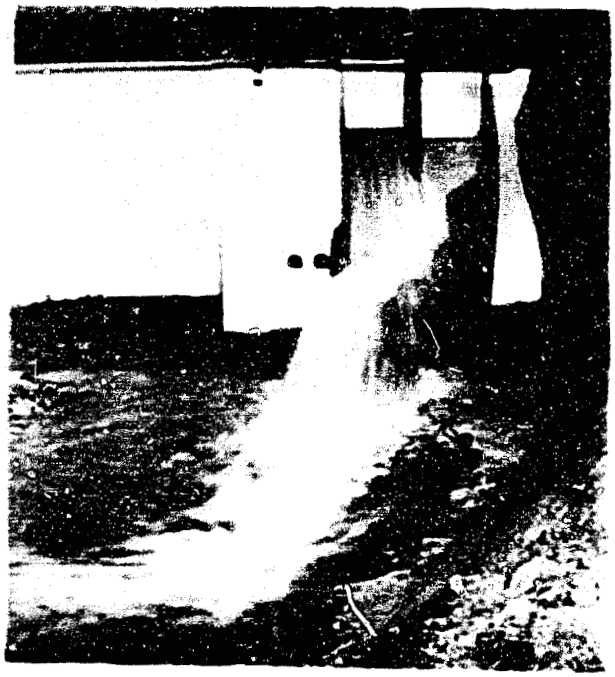
A. 10,000 cfs

B. 25,000 cfs

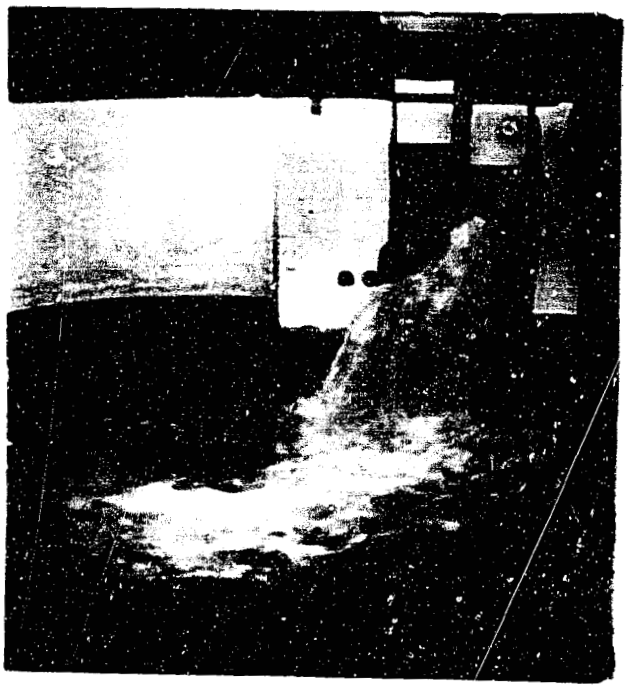


C. Discharge 57,500 cfs

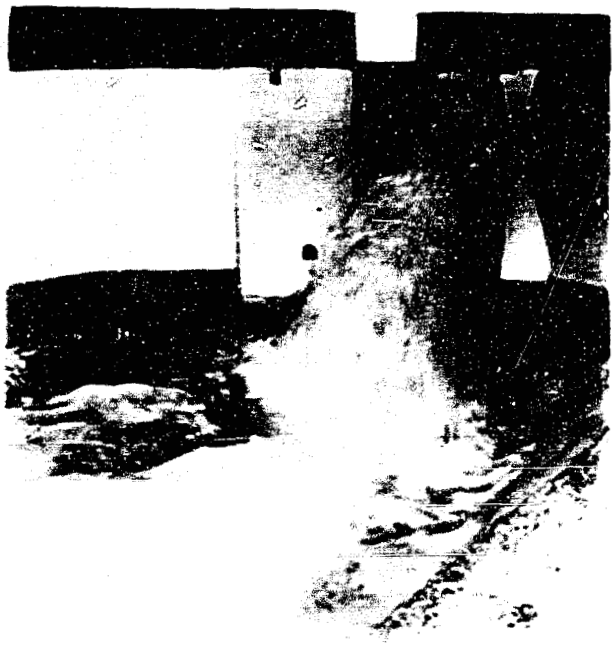
GORGE HIGH DAM
Recommended Spillway
Left Gate Operating - Right Gate Closed
Reservoir Elevation 875
1:48 Scale Model



A. 10,000 cfs



B. 25,000 cfs



C. Discharge 59,100 cfs

GORGE HIGH DAM
Recommended Spillway
Right Gate Operating - Left Gate Closed
Reservoir Elevation 875
1:48 Scale Model

A. Log sluice and outlet works

B. Outlets discharging 10,000 cfs

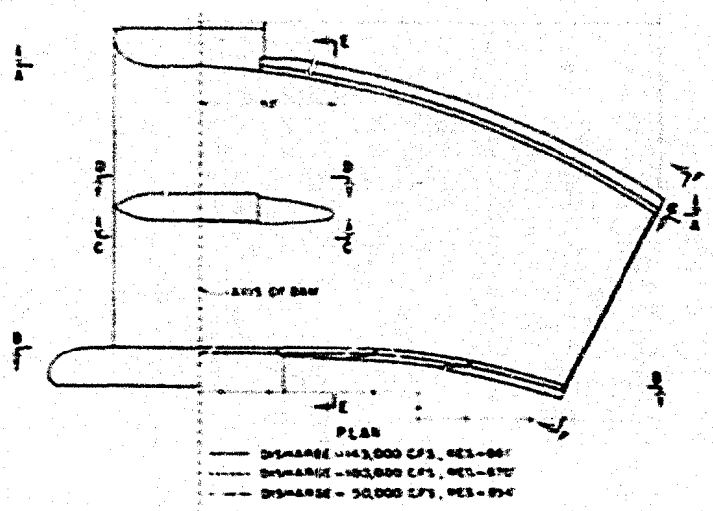
C. Log sluice and outlets operating

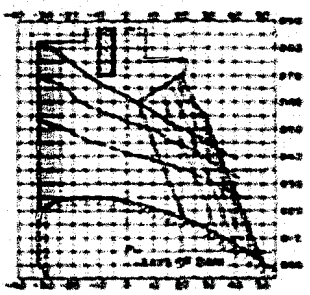
D. Approach conditions to log sluice

E. Logs entering sluice

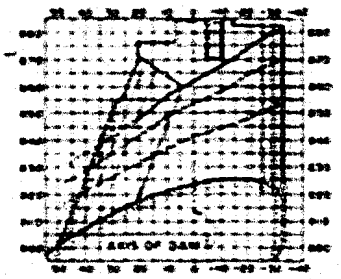
F. Logs passing through sluice

GORGE HIGH DAM
Log Sluice and Outlets Studies
1:48 Scale Model

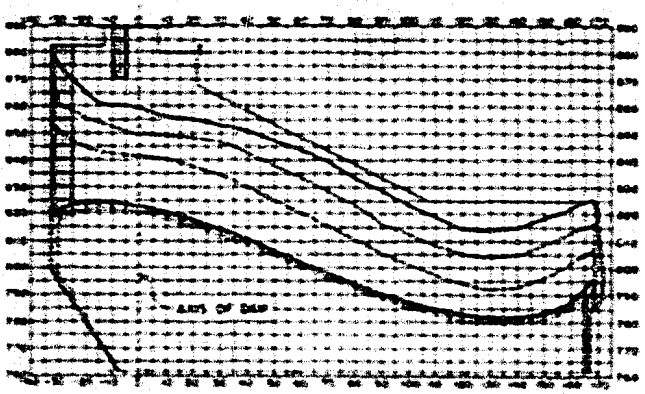




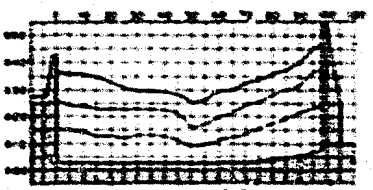
SECTION C-C (PROJECTED)
RIGHT SIDE OF CENTER PIER



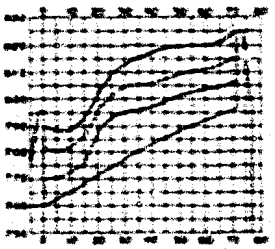
SECTION D-D (PROJECTED)
LEFT SIDE OF CENTER PIER



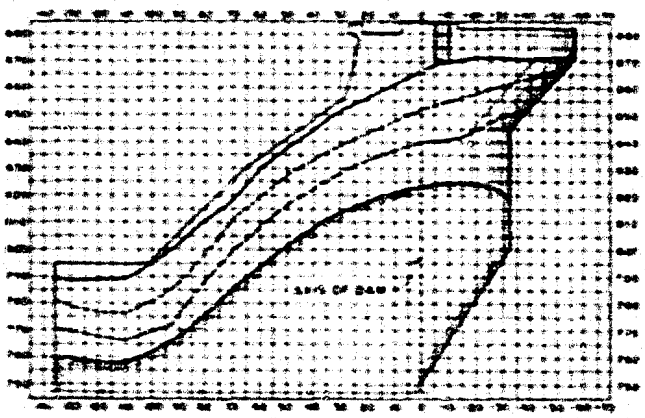
SECTION A-A (PROJECTED)
LEFT TRAINING WALL



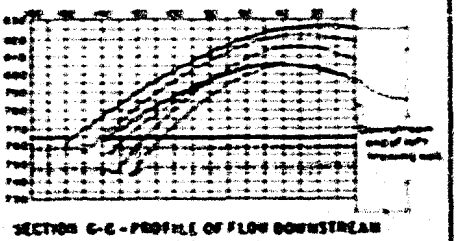
SECTION E-E - CROSS-SECTION OF FLOW
50 FEET DOWNSTREAM FROM AXIS OF DAM



SECTION F-F - CROSS-SECTION OF FLOW
AT DOWNSTREAM END OF BUCKET



SECTION B-B (PROJECTED)
RIGHT TRAINING WALL



SECTION G-G - PROFILE OF FLOW DOWNSTREAM
FROM LEFT TRAINING WALL

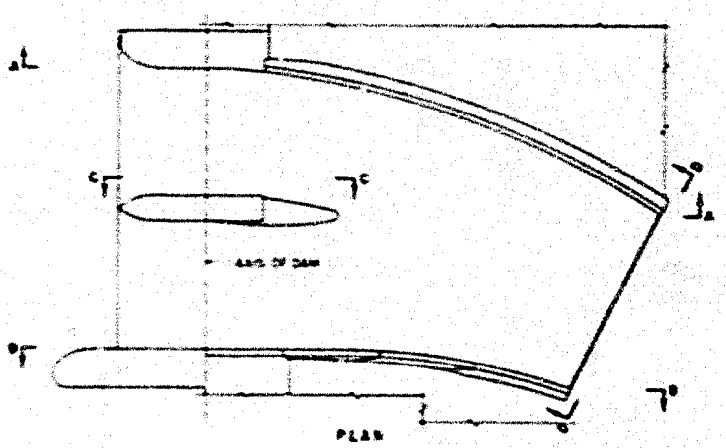
GORGE HIGH DAM SPILLWAY

WATER SURFACE PROFILES
WITH BOTH GATES FULLY OPEN

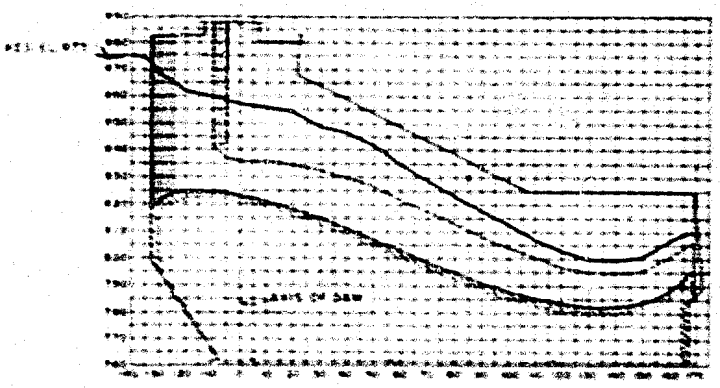
RECOMMENDED DESIGN

1:48 SCALE MODEL

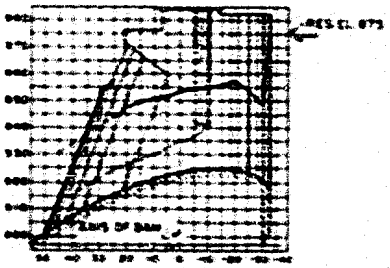
FIGURE 38



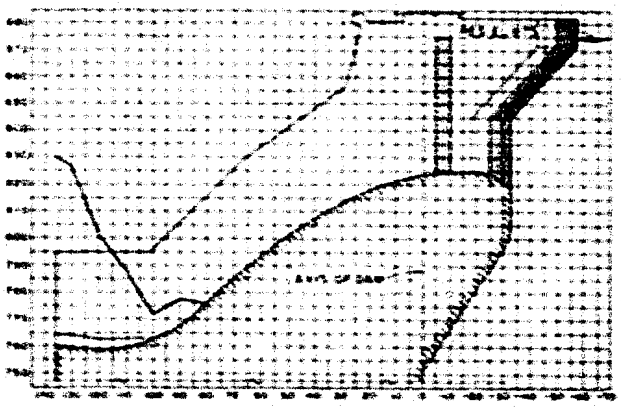
PLAN



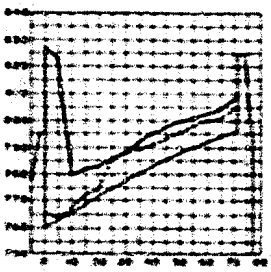
SECTION A-A (PROJECTED)
LEFT TRAINING WALL



SECTION C-C (PROJECTED)
LEFT SIDE OF CENTER PIER

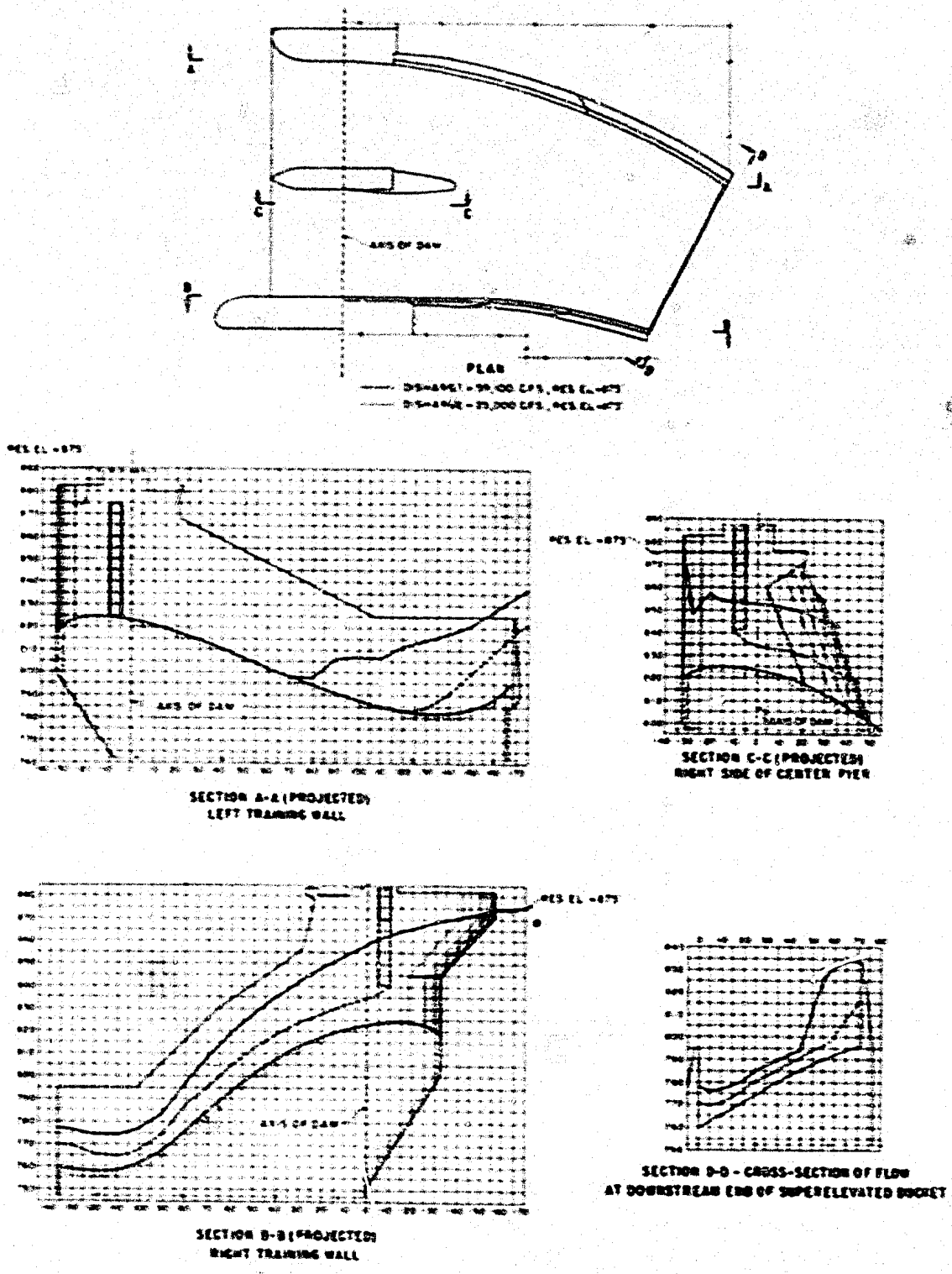


SECTION B-B (PROJECTED)
RIGHT TRAINING WALL



SECTION D-D
CROSS-SECTION OF FLOW AT DOWNSTREAM END
OF SUPER-ELEVATED BUCKET

GORGE HIGH DAM SPILLWAY
WATER SURFACE PROFILES
WITH RIGHT GATE CLOSED
RECOMMENDED DESIGN
1:40 SCALE MODEL

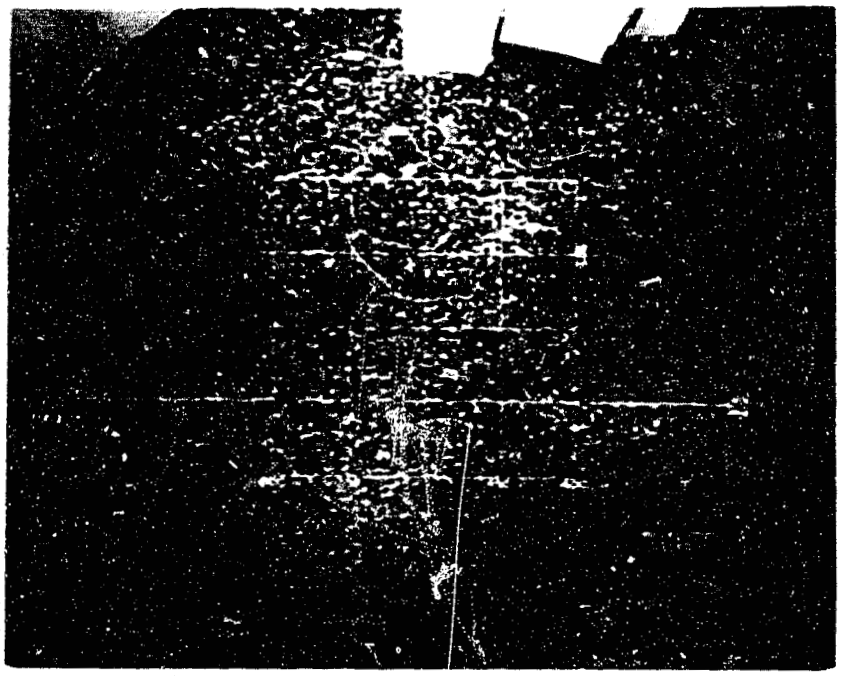


GORGE HIGH DAM SPILLWAY

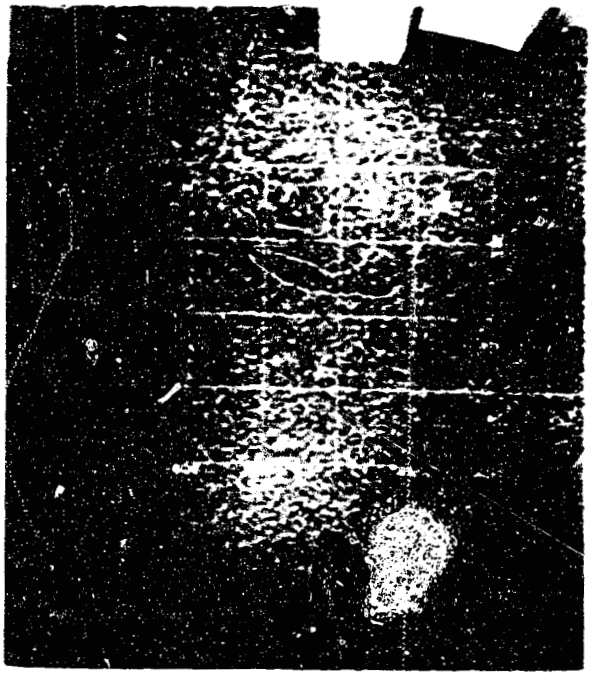
WATER SURFACE PROFILES
WITH LEFT GATE CLOSED

RECOMMENDED DESIGN

1:40 SCALE MODEL



A. After 10,000 cfs

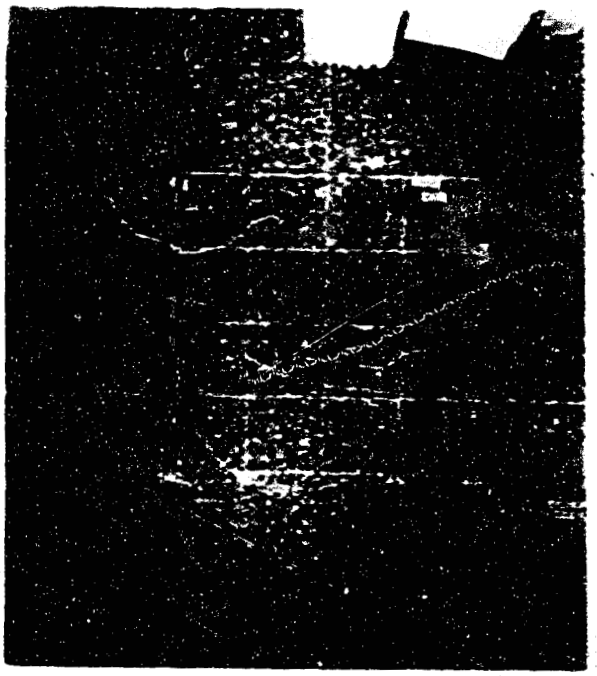


B. After 25,000 cfs

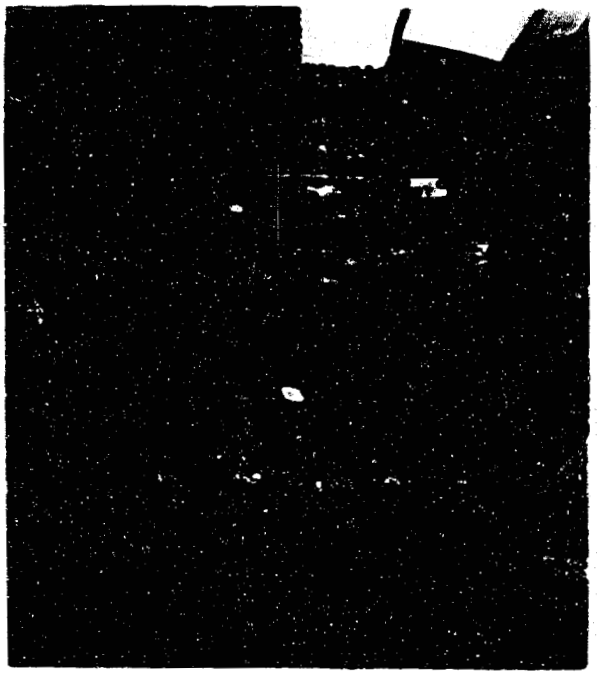
C. After 57,500 cfs

GORGE HIGH DAM
Erosion Studies
Discharge through Left Gate - Right Gate Closed
Reservoir Elevation 875
1:48 Scale Model

A. After 10,000 cfs

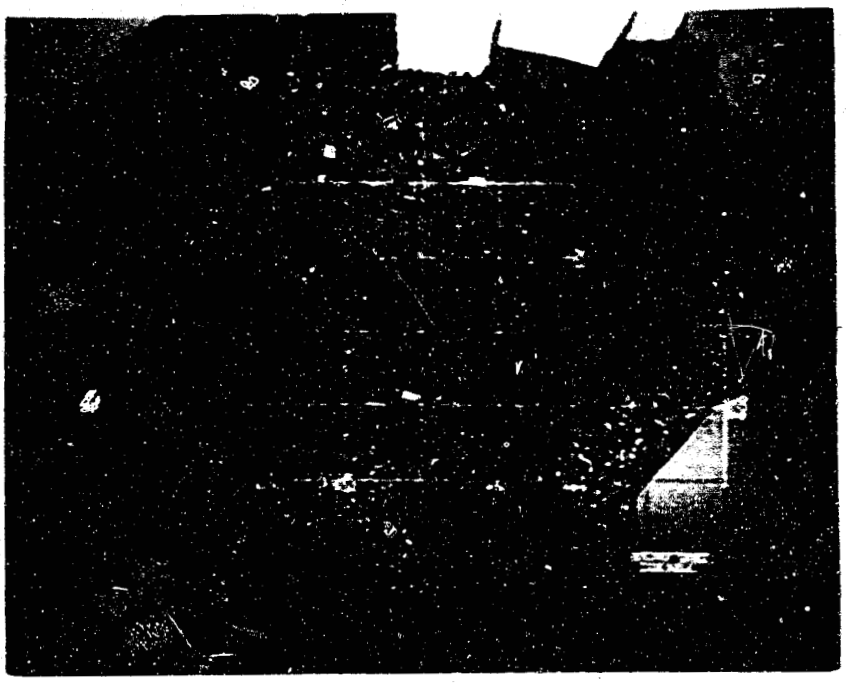


B. After 25,000 cfs

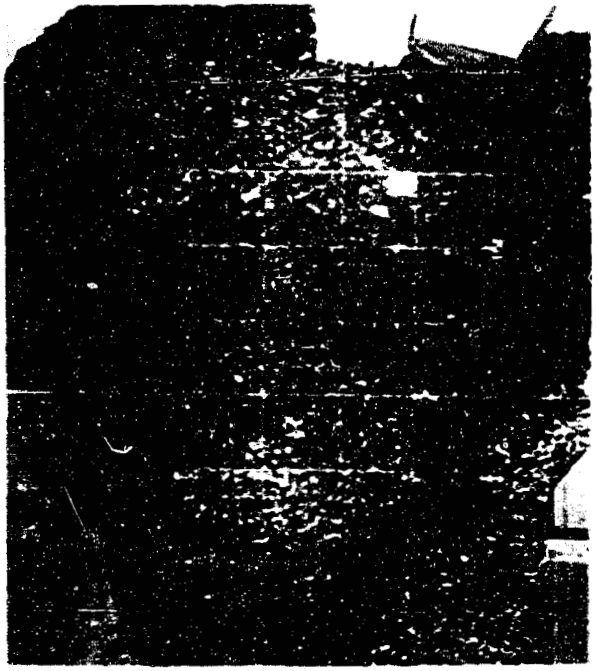


C. After 59,100 cfs

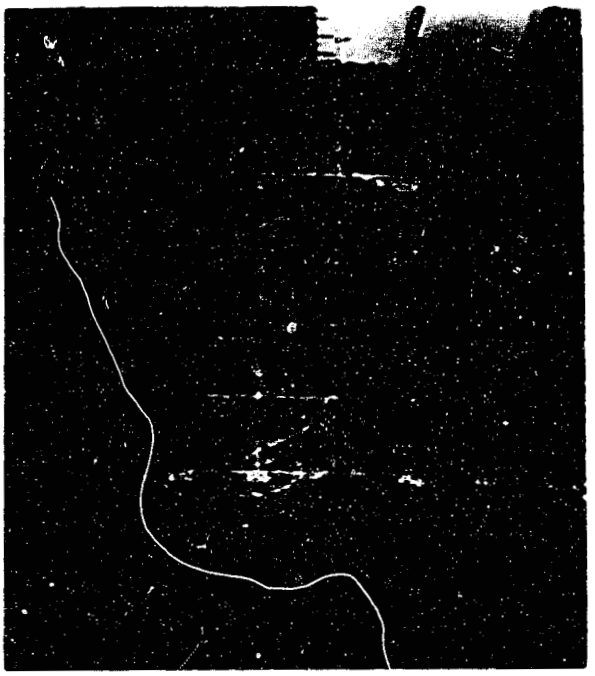
GORGE HIGH DAM
Erosion Studies
Discharge through Right Gate - Left Gate Closed
Reservoir Elevation 875
1:48 Scale Model



A. After 10,000 cfs - Reservoir El. 877.6



B. After 25,000 cfs
Reservoir El. 881.6



C. After 60,000 cfs
Reservoir El. 882.5

GORGE HIGH DAM
Erosion Studies
Discharge through Both Gates
1:48 Scale Model

A. Discharge 100,000 cfs and resulting erosion - Reservoir El. 882.5

B. Discharge 145,000 cfs and resulting erosion - Reservoir El. 881.0

GORGE HIGH DAM
Erosion Studies
Discharge through Both Gates
1:48 Scale Model

FIGURE 44

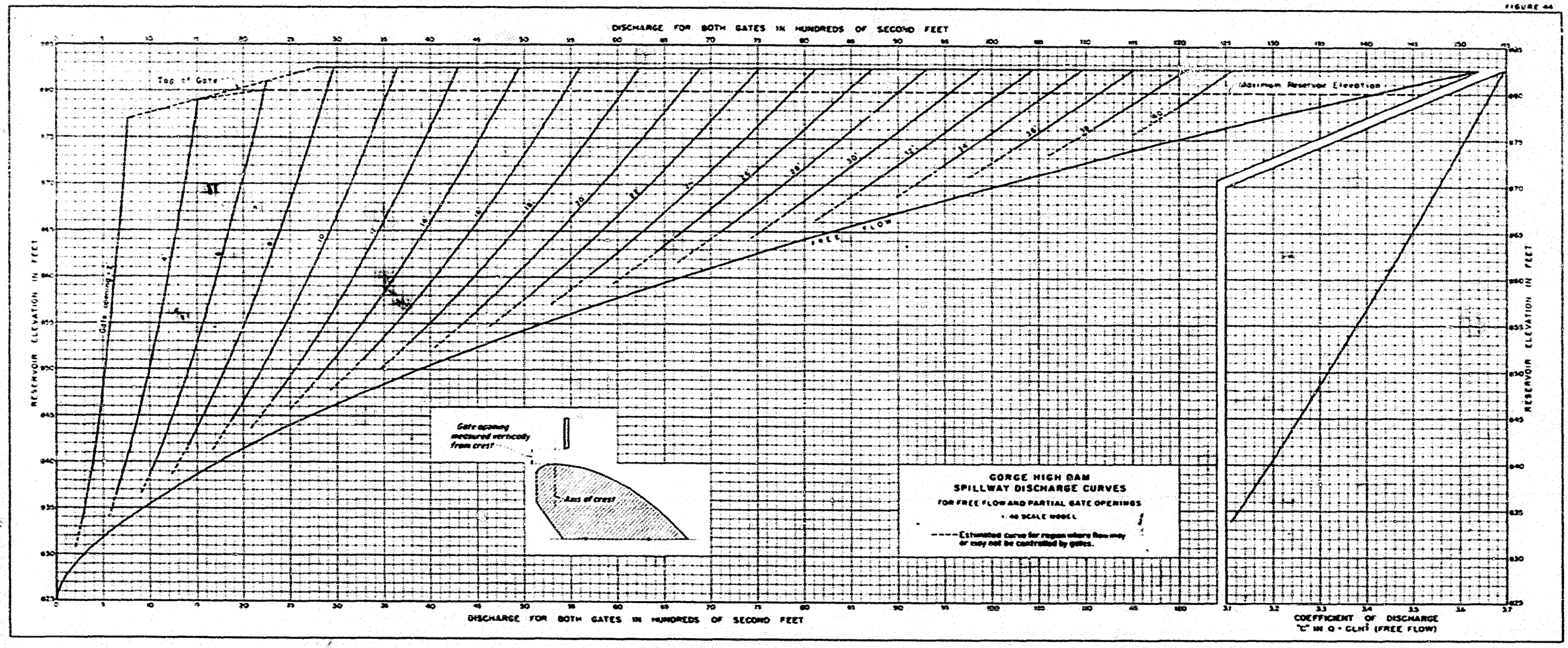
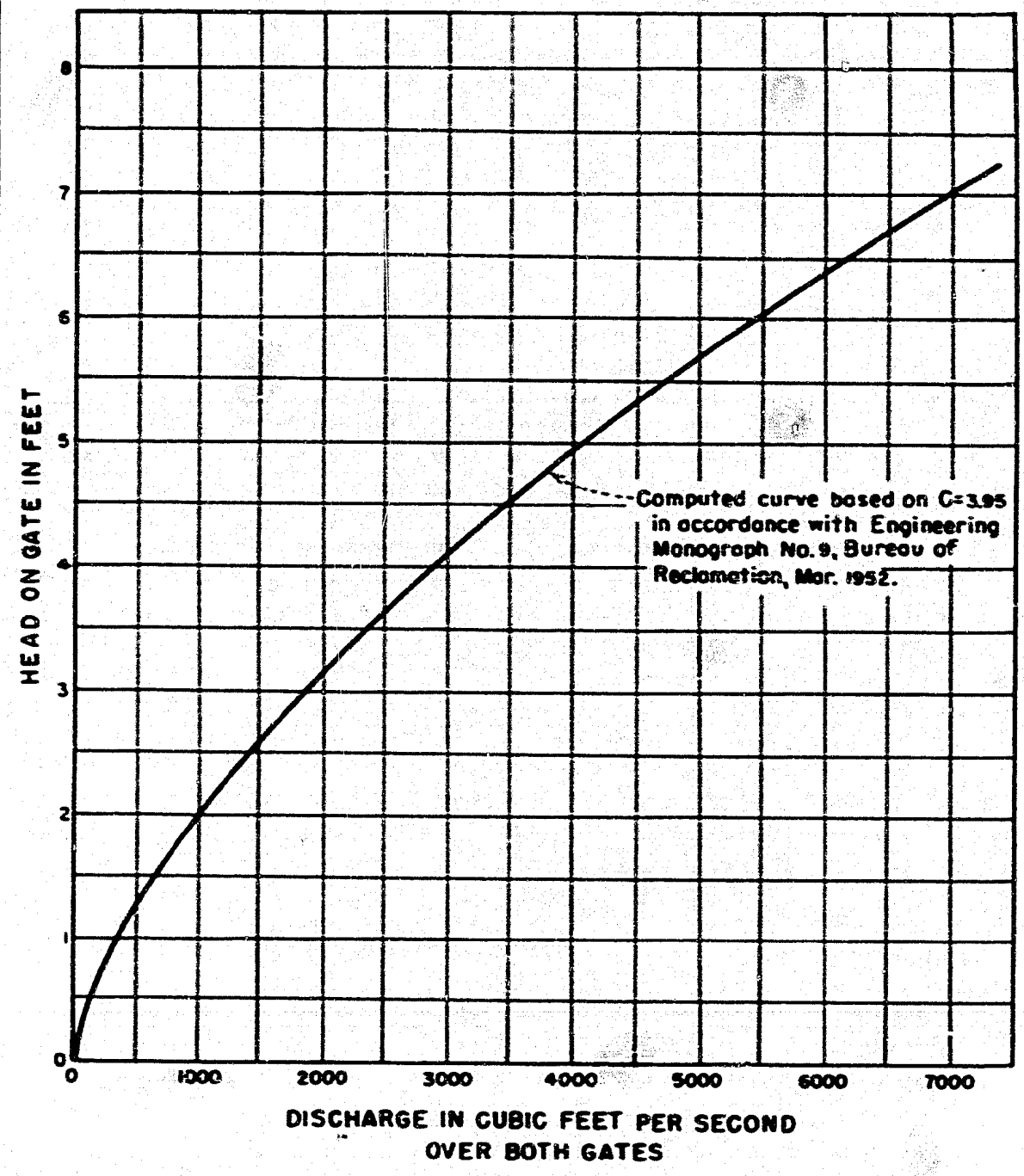
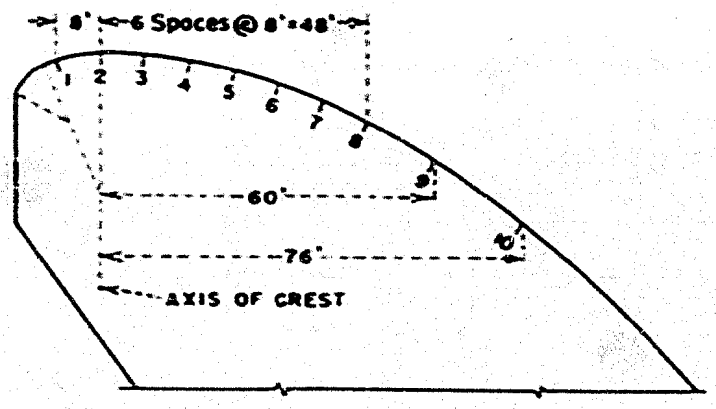

DISCHARGE FOR BOTH GATES IN HUNDREDS OF SECOND FEET
RESERVOIR ELEVATION IN FEET
Top of Gate
Maximum Reservoir Elevation
FREE FLOW
Gate opening measured vertically from crest
Axis of crest
GORGE HIGH DAM
SPILLWAY DISCHARGE CURVES
FOR FREE FLOW AND PARTIAL GATE OPENINGS
1:40 SCALE MODEL
----Estimated curve for region where flow may or may not be controlled by gates.
COEFFICIENT OF DISCHARGE
"C" IN Q = CLH³ (FREE FLOW)

FIGURE 45



GORGE HIGH DAM
HEAD-DISCHARGE CURVE
FOR FLOWS OVER TOP OF 47'x 50.32'
FIXED WHEEL GATES IN SEATED POSITION
1:48 SCALE MODEL

FIGURE 46



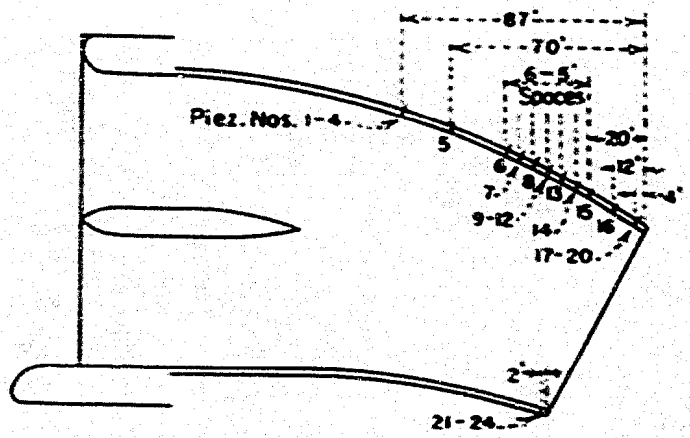
LOCATION OF PIEZOMETERS

PIEZO-METER No.	GATES FULLY OPEN DISCHARGE IN CFS			GATE OPENING AND DISCHARGE IN CFS								
				4"	8"	12"	16"	20"	24"	28"	32"	36"
	145,000	100,000	50,000	15,000	29,300	42,[illegible]0	55,800	69,000	80,000	91,400	104,600	114,900
1	27.0	26.5	21.0	54.6	56.5	52.3	49.7	47.6	44.0	41.7	39.3	37.2
2	21.8	21.8	17.4	51.5	49.2	44.7	40.8	39.1	35.8	33.5	31.9	30.5
3	20.8	19.4	15.1	27.4	28.3	28.3	27.4	27.7	26.6	26.1	25.6	25.2
4	21.0	18.8	14.0	2.3	6.7	17.8	14.7	17.1	18.7	19.8	20.5	21.3
5	16.8	15.5	11.5	1.0	1.2	3.4	5.4	8.0	9.9	12.0	13.2	14.8
6	9.4	9.8	7.6	-1.9	-2.4	-2.0	-2.2	-0.3	1.2	2.7	3.7	5.0
7	9.6	8.8	6.7	-0.5	-0.7	-0.3	-0.7	0.3	0.8	2.0	3.0	4.4
8	8.4	6.4	4.4	-0.5	-1.1	-1.3	-1.3	-1.3	-0.8	-0.4	0.6	2.0
9	6.6	2.1	0.7	-4.4	-5.0	-4.6	-5.0	-5.8	-4.4	-3.2	-2.7	-0.8
10	20.8	10.3	5.0	1.9	2.2	2.8	3.3	4.5	5.3	6.7	8.3	10.7

GORGE HIGH DAM SPILLWAY

PIEZOMETRIC PRESSURES ON SPILLWAY

1:48 SCALE MODEL



LOCATION OF PIEZOMETERS

PIEZOMETER		PRESSURE IN FEET OF WATER (PROTOTYPE)						
		BOTH GATES OPEN			RESERVOIR AT ELEVATION 875 FEET			
					POSITION OF SLIDE GATES			
No.	ELEV.	Q= 145,000	Q= 100,000	Q= 50,000	LT. CLOSED RT. OPEN Q=60,000	LT. CLOSED RT. PAR. OPEN Q=25,000	RT. CLOSED LT. OPEN Q=60,000	RT. CLOSED LT. PAR. OPEN Q=25,000
1	827.0	4.8	0	0	0	0	0	0
2	817.0	15.2	4.6	0	0	0	5.2	0
3	807.0	27.0	15.0	2.4	0	0	14.4	5.0
4	797.0	38.0	24.8	11.8	0	0	24.2	14.2
5	790.5	48.5	31.9	13.8	26.3	0	27.0	17.0
6	784.5	60.2	43.0	19.8	36.8	0	36.6	24.4
7	784.0	62.0	45.2	21.0	38.2	0	38.4	26.4
8	783.5	63.2	47.2	22.6	38.4	1.0	40.4	28.2
9	814.5	1.0	0	0	0	0	0	0
10	804.0	23.8	0	0	3.7	0	0	0
11	794.0	43.5	24.0	0	2.0	0	0	0.5
12	784.0	62.6	46.4	21.4	40.2	0	39.8	27.5
13	784.0	63.5	47.2	22.6	37.2	6.0	40.0	28.0
14	784.5	62.0	46.6	23.0	36.4	16.0	38.2	27.2
15	786.0	61.0	47.7	26.2	36.1	21.8	41.8	32.0
16	791.0	43.0	32.2	15.3	24.2	16.6	26.5	18.4
17	820.0	0	0	0	1.0	0	0	0
18	812.0	4.0	0.5	0	1.0	0.5	0	0
19	804.0	7.6	2.8	0	7.0	0.6	0	0
20	796.0	12.0	6.2	0.5	4.5	5.2	3.8	1.0
21	786.0	0	0	0	0	0	0	0
22	778.0	4.0	0	0	0	0	1.5	0
23	769.5	5.0	0	0	-3.0	0	25.0	0
24	761.5	5.0	-3.0	-8.0	-8.0	-9.0	9.0	3.0

GORGE HIGH DAM SPILLWAY

PIEZOMETRIC PRESSURES ON TRAINING WALLS

1:48 SCALE MODEL

A. Right gate closed B. Both gates equally open C. Left gate closed

Discharge 25,000 cfs - Reservoir El. 875

D. Right gate closed E. Both gates equally open F. Left gate closed

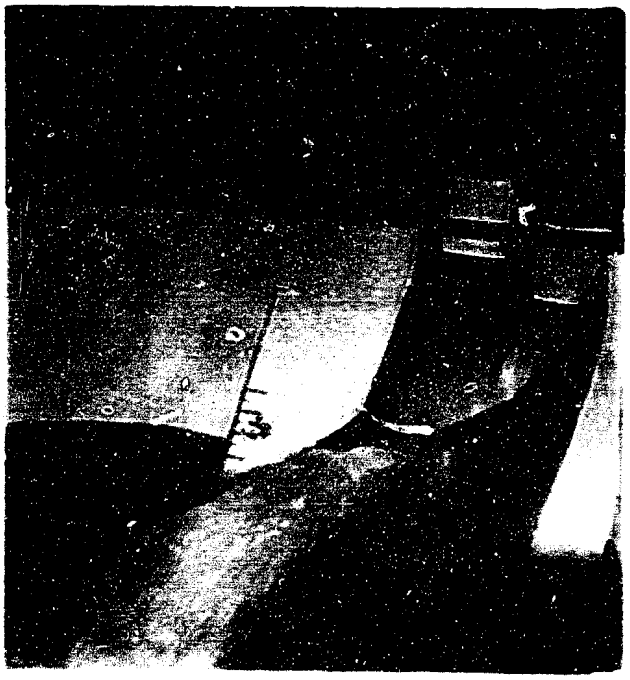
Discharge 57,500 cfs - Reservoir El. 875

GORGE HIGH DAM
Recommended Spillway
Shift of Jet Position by Manipulation of Gates
1:48 Scale Model

A. Right gate closed

B. Right gate raised 8 feet

C. Left gate closed

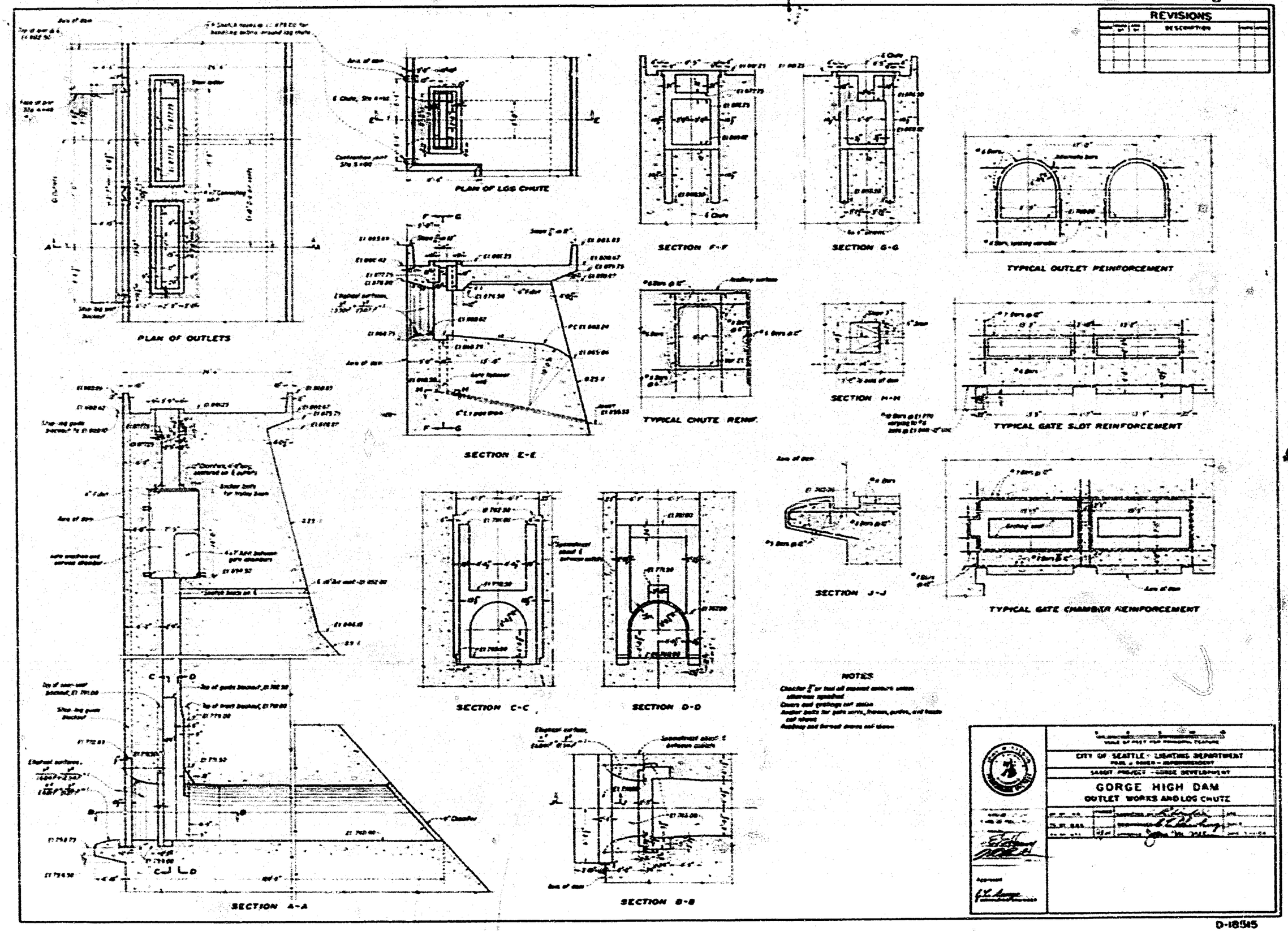


D. Left gate raised 7-1/2 feet

The fin of water can be eliminated by raising the closed gate 7 or 8 feet.

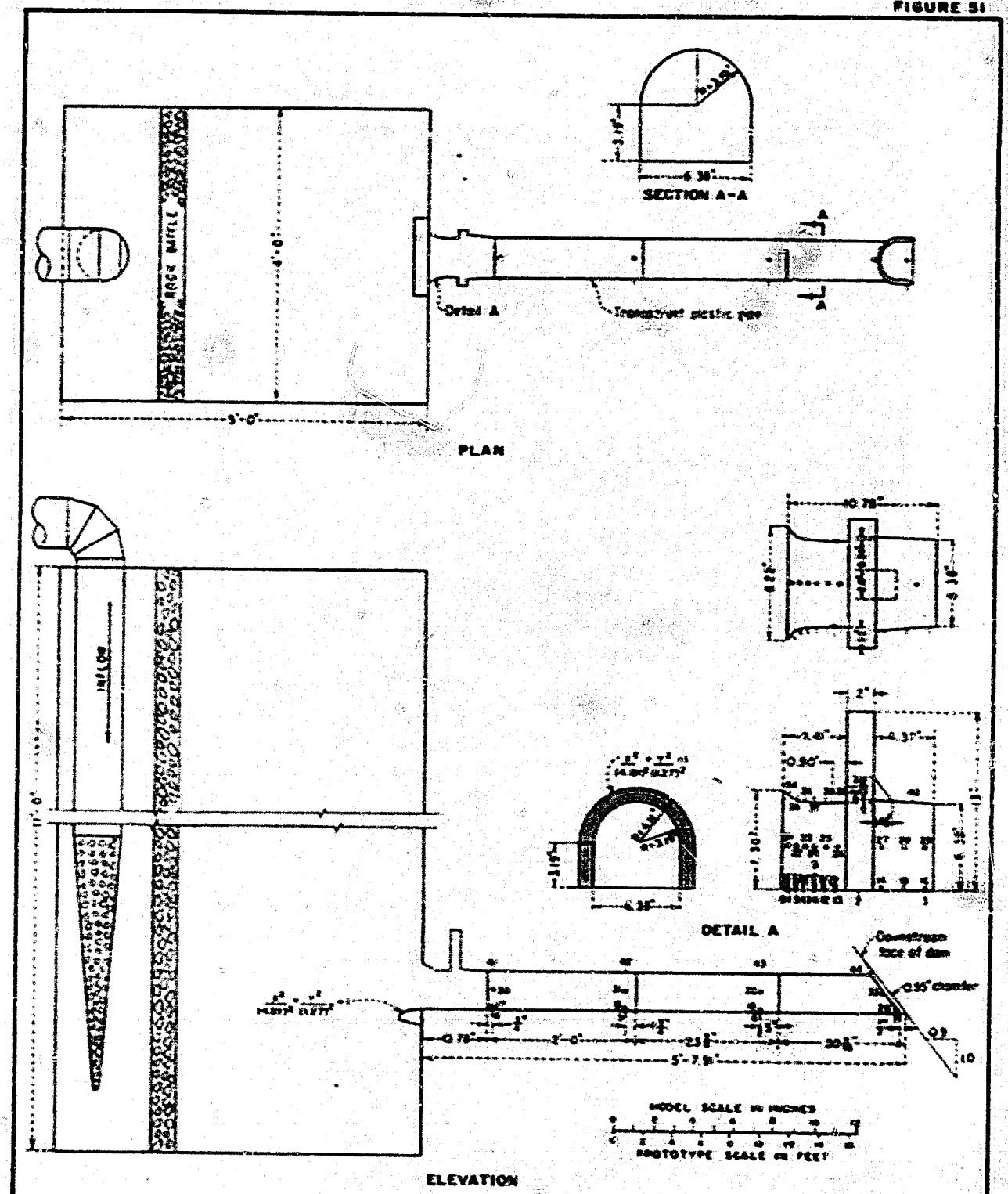
GORGE HIGH DAM
Recommended Spillway
Discharge 57,500 cfs
1:48 Scale Model

Figure 50



D-18545

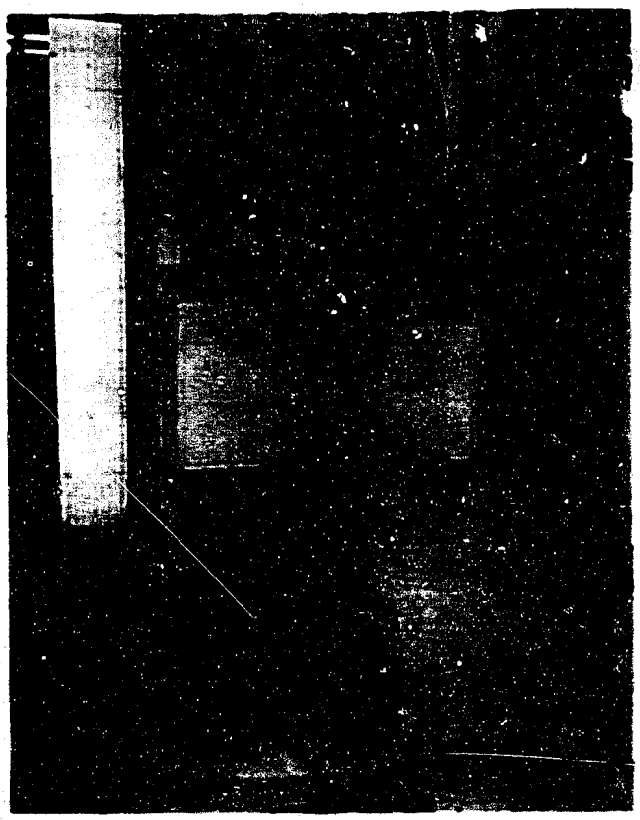
FIGURE 51



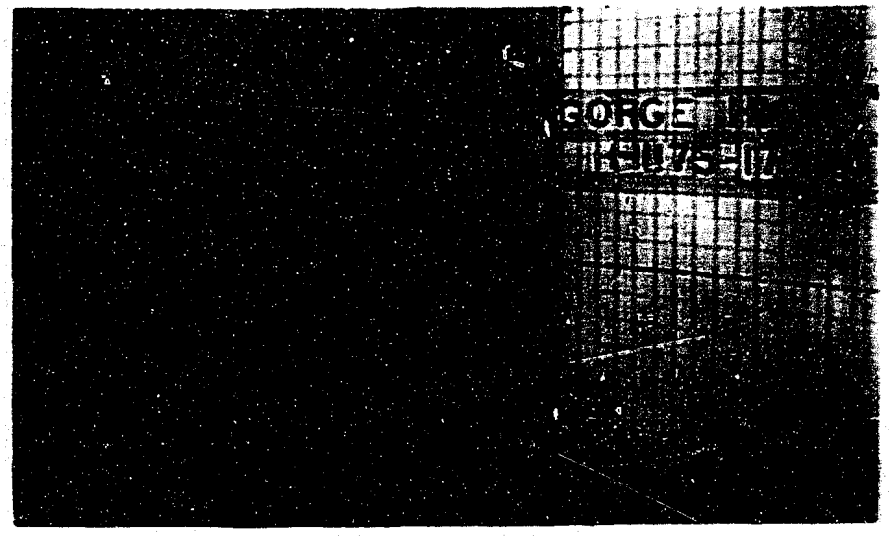
GORGE HIGH DAM

OUTLET WORKS

EXTENT AND LAYOUT OF THE 1:16.47 MODEL

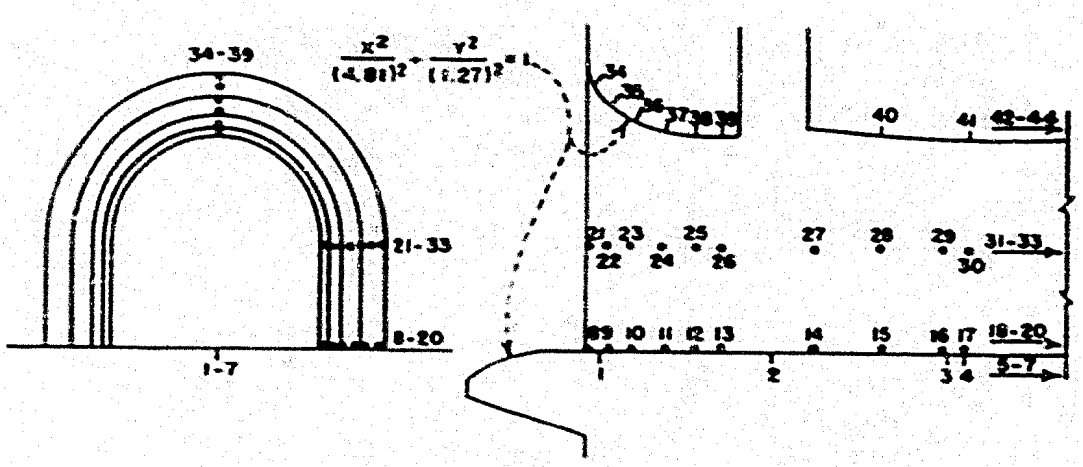


A. The 1:16:47 Model

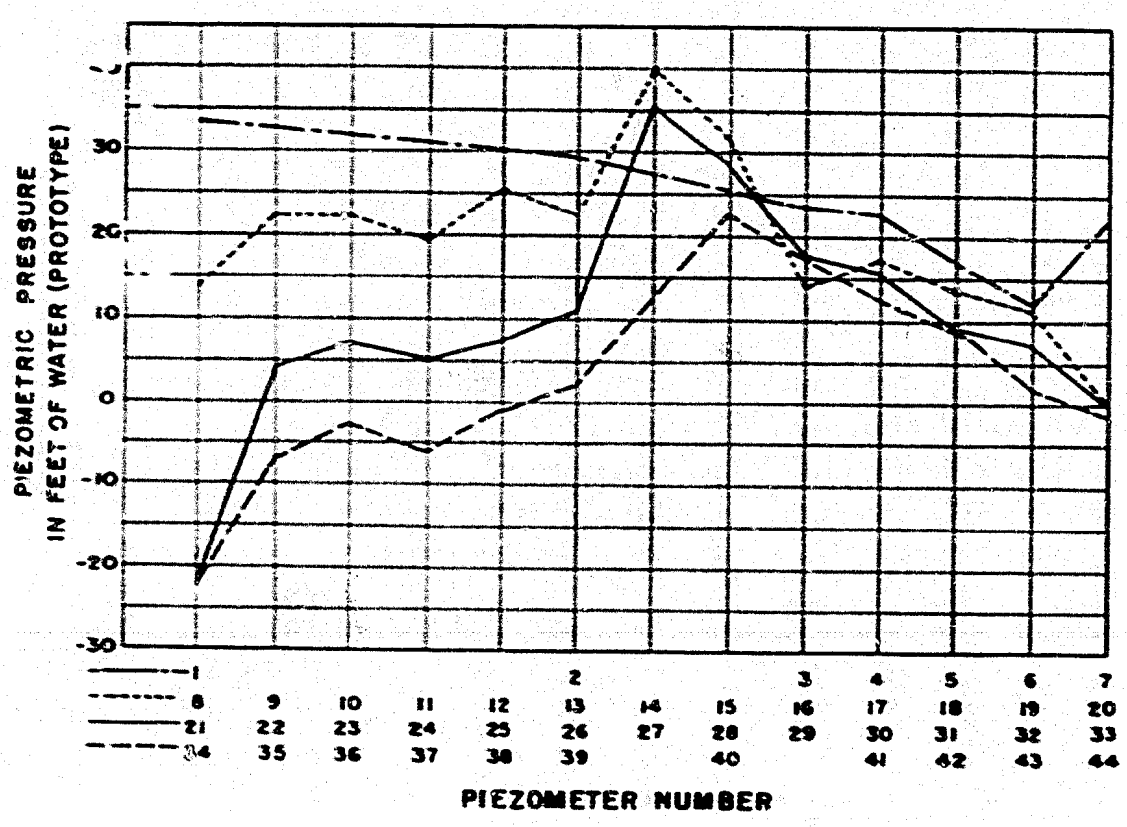


B. The gate chamber

GORGE HIGH DAM
The 1:16:47 Model of Outlet



LOCATION OF PIEZOMETERS

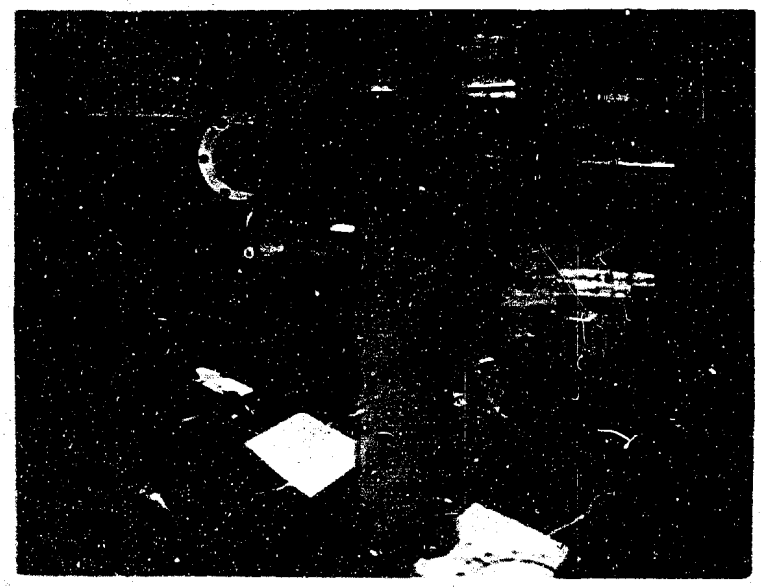


GORGE HIGH DAM OUTLET WORKS

PIEZOMETRIC PRESSURES IN PRELIMINARY DESIGN

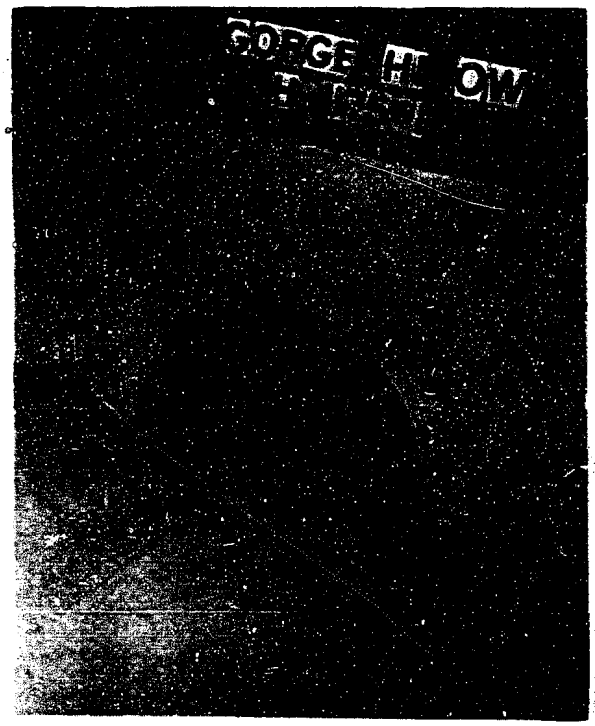
RESERVOIR ELEVATION = 880 FEET

1:16.47 SCALE MODEL



A. The Air Model

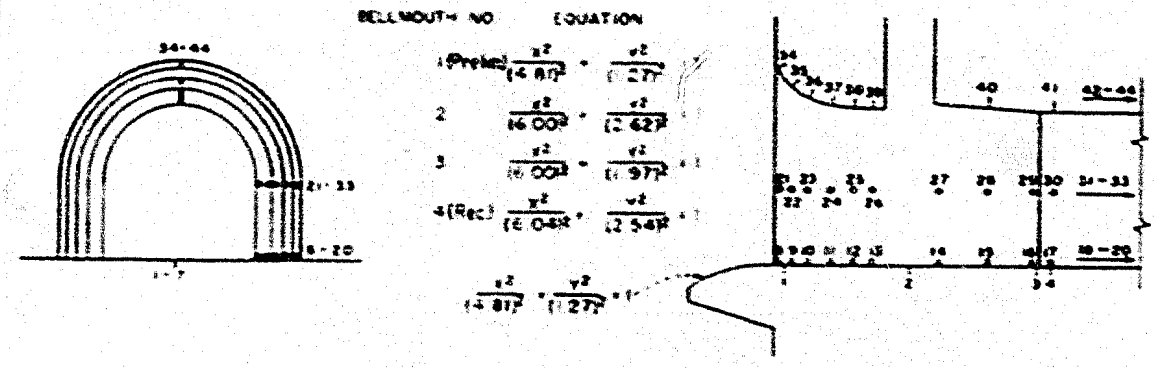
B. Preliminary bellmouth



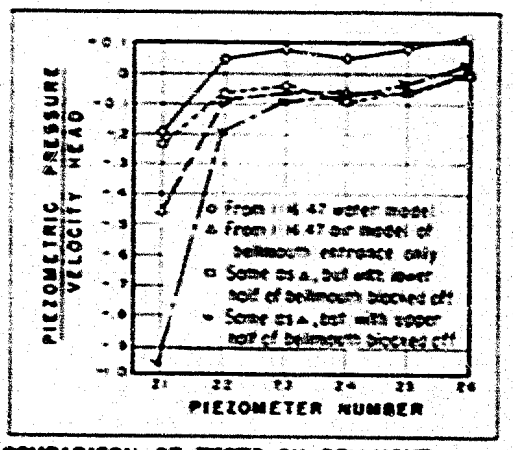
C. Preliminary bellmouth modified to represent a circular entrance.

GORGE HIGH DAM
Air Model Studies
1:16:47 Scale Model

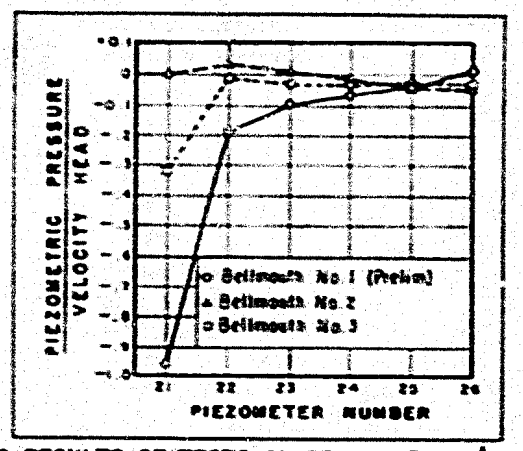
FIGURE 55



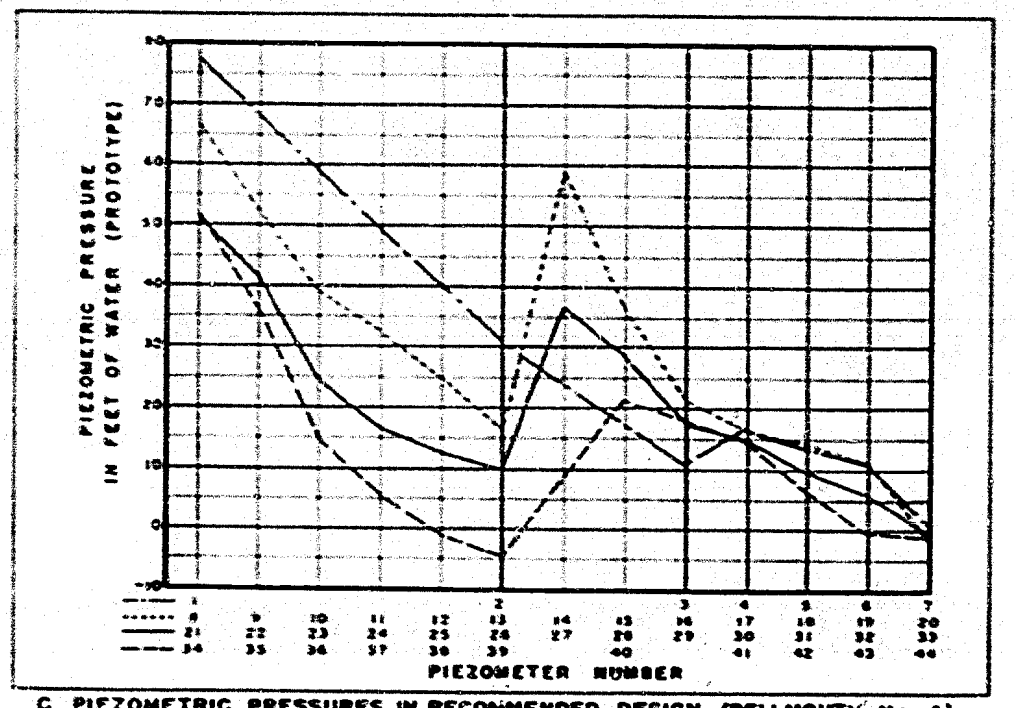
LOCATION OF PIEZOMETERS



A. COMPARISON OF TESTS ON BELLMOUTH No. 1 FROM AIR AND WATER MODEL

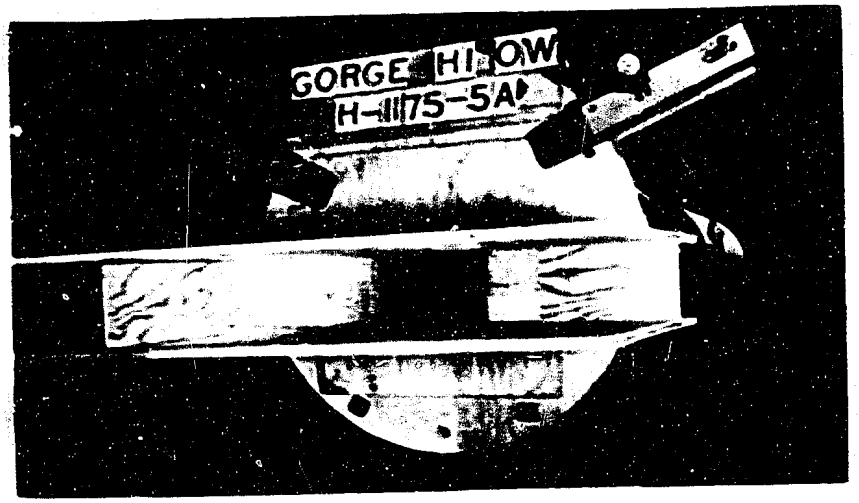


B. RESULTS OF TESTS ON BELLMOUTH No'S 1, 2, AND 3 FROM SECTIONAL AIR MODEL



C. PIEZOMETRIC PRESSURES IN RECOMMENDED DESIGN (BELLMOUTH No. 4)
RESERVOIR ELEVATION = 880 FEET

GORGE HIGH DAM OUTLET WORKS
BELLMOUTH ENTRANCE STUDIES
1:16.47 SCALE MODEL



A. Bellmouth studies in sectional air model

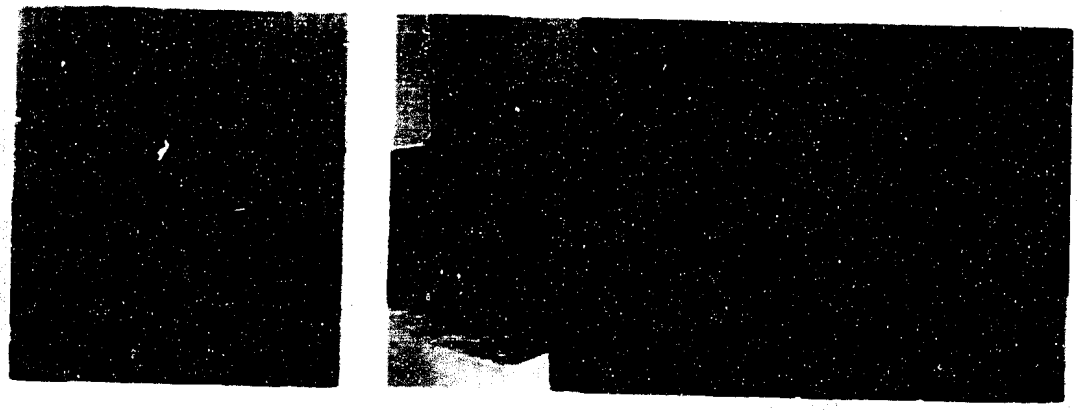
1,000 cfs

2,000 cfs

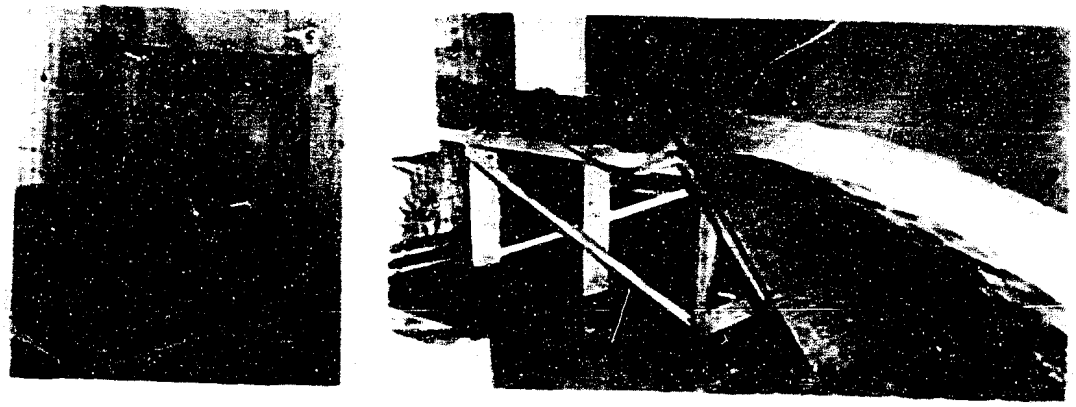
B. Flow in gate chamber and outlet
Recommended Design - 1:16:47 model

GORGE HIGH DAM
Outlet Works Studies

3,000 cfs



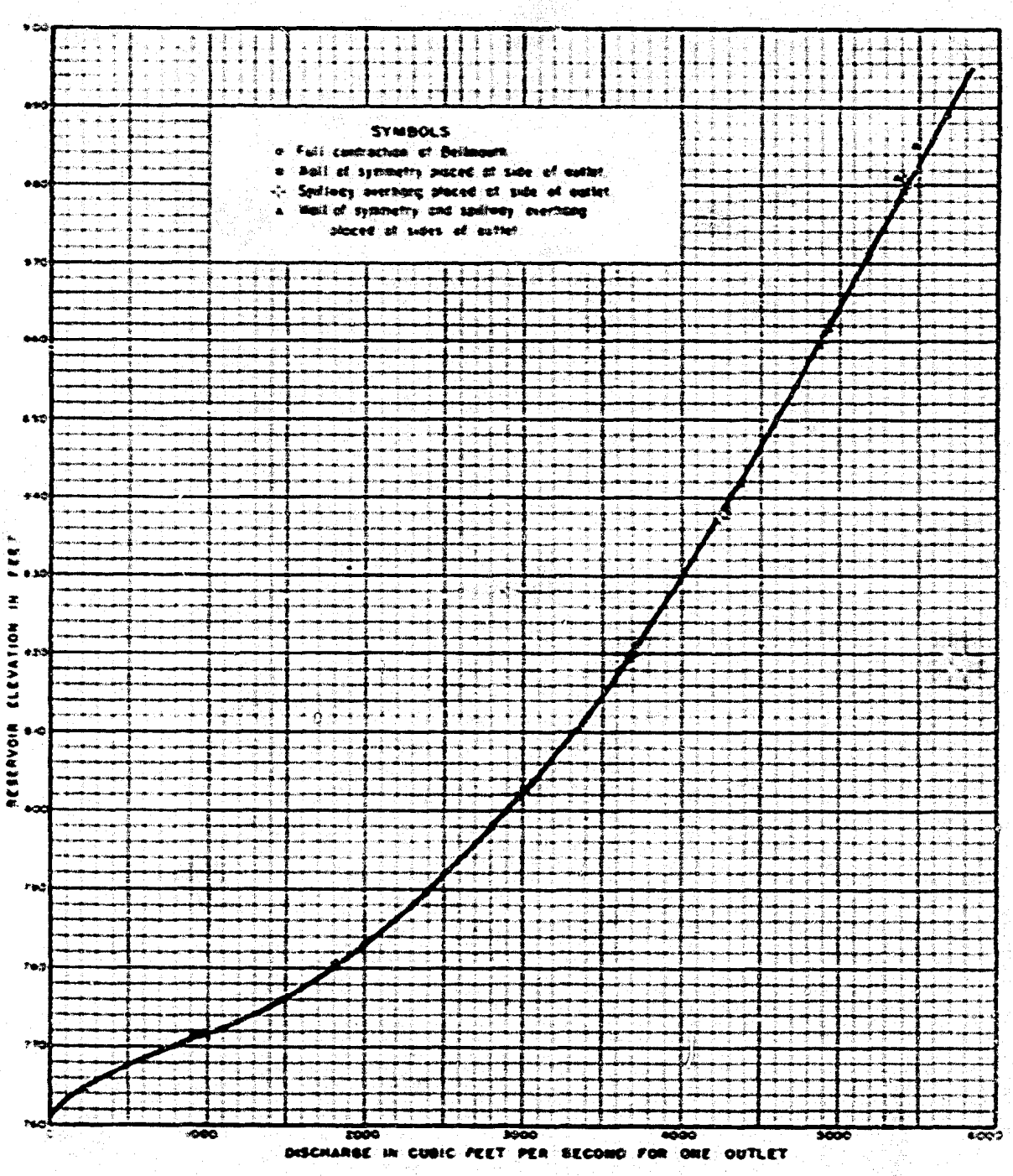
4,000 cfs



5,250 cfs

GORGE HIGH DAM
Flow in Gate Chamber and Outlet
Recommended Design
1:16:47 Scale Model

FIGURE 58



GORGE HIGH DAM OUTLET WORKS
HEAD-DISCHARGE CURVE

RECOMMENDED BELLMOUTH - $\frac{x^2}{(8.04)^2} + \frac{y^2}{(2.34)^2} = 1$

1:16.47 SCALE MODEL

Table 3

SUMMARY OF PERMANENT FRACTURES IN FRONT OF WATER (PERCENTAGE)
1:16.67 Water Model

Reservoir elev.	880.00					895.00	890.00				900.00								
CFs	5160	5610	5400	5361	5410	3007	3750	3871	3721	3877		2821	2809	2756	2820
Model arrangement / Pressure-meter No.	A with shallowness Ch. 1 (1)	A with shallowness Ch. 4 (2)	B with shallowness Ch. 4 (3)	C with shallowness Ch. 4 (4)	D with shallowness Ch. 4 (5)	A with shallowness Ch. 1 (6)	A with shallowness Ch. 4 (7)	B with shallowness Ch. 4 (8)	C with shallowness Ch. 4 (9)	D with shallowness Ch. 4 (10)	A with shallowness Ch. 1 (11)	A with shallowness Ch. 4 (12)	B with shallowness Ch. 4 (13)	C with shallowness Ch. 4 (14)	D with shallowness Ch. 4 (15)
1	31.6	79.6	75.9	76.6	77.9	26.1	41.2	39.2	38.7	40.2		25.8	25.5	25.2	15.8
2	25.5	32.6	30.8	30.1	30.8	34.3	28.6	28.8	28.5	28.6		20.2	20.4	22.2	20.2
3	23.7	28.3	27.2	26.0	28.9	22.6	27.1	27.1	26.8	26.6		9.7	9.4	9.7	9.9
4	20.6	28.6	15.8	27.1	26.5	12.1	12.4	11.9	11.7	11.5		8.6	8.6	8.6	8.7
5	26.8	--	--	--	--	20.2	--	--	--	--		--	--	--	--
6	11.9	11.6	9.7	10.4	10.4	8.2	8.4	8.2	8.6	8.4		7.2	7.2	7.4	7.4
7	22.3	1.3	0.7	0.5	0.7	6.7	1.2	1.2	1.2	0.8		1.6	1.8	1.8	1.8
8	11.8	57.2	48.4	54.0	47.2	10.2	31.3	27.3	25.3	30.0		12.8	12.2	12.3	12.9
9	20.1	43.8	37.6	39.3	32.4	12.2	26.5	22.1	23.6	26.6		11.7	11.3	11.1	11.7
10	22.1	32.2	29.2	31.2	28.3	12.6	19.9	18.2	19.2	20.5		10.6	10.5	10.5	10.6
11	19.4	32.0	25.7	28.3	32.4	11.4	17.3	16.2	16.6	17.5		10.0	10.0	10.0	10.13
12	25.4	26.9	22.7	20.7	25.0	12.8	16.7	13.6	13.8	15.9		9.8	9.5	9.5	9.8
13	22.2	16.0	15.3	11.8	16.1	11.9	10.5	10.1	10.7	10.0		8.9	8.9	8.5	8.9
14	--	59.7	59.7	57.2	73.6	--	33.4	30.8	32.6	32.1		12.8	11.8	12.8	12.7
20	--	43.6	37.7	41.6	58.1	--	23.5	22.4	24.2	25.8		11.8	11.0	11.0	11.7
108	--	36.7	30.8	38.5	45.6	--	22.6	20.6	20.1	22.4		11.0	10.2	10.3	10.7
118	--	31.1	27.3	27.5	35.1	--	18.0	16.3	17.6	18.8		10.0	9.9	9.9	10.4
128	--	22.9	18.6	20.1	25.9	--	14.5	13.0	13.7	16.7		9.4	9.4	9.2	9.4
138	--	18.9	17.0	16.5	21.1	--	13.3	12.9	12.2	12.7		9.1	9.1	9.1	9.1
14	46.0	55.8	42.4	42.6	55.0	21.1	29.5	25.1	26.6	29.9		13.8	13.4	13.4	14.7
15	31.8	35.6	30.7	34.4	36.2	15.5	20.3	20.4	3.9	19.9		10.7	10.7	10.5	11.0
16	13.3	19.9	20.9	20.4	21.4	9.2	13.3	13.6	13.6	12.1		9.7	9.2	9.4	9.2
17	17.1	14.8	14.8	14.2	16.5	10.7	10.9	10.9	11.0	10.9		8.6	8.7	8.4	8.6
18	11.1	14.6	12.8	12.9	14.0	9.2	10.4	9.9	10.1	10.4		8.2	8.1	8.2	8.2
19	11.5	11.0	11.7	10.4	11.0	8.1	8.6	8.6	8.9	8.6		7.4	7.0	7.7	7.7
20	1.0	-0.5	-0.5	-0.7	-1.7	1.0	0.0	-0.2	0	0		1.2	1.0	1.1	1.1
21	-32.2	36.1	46.4	35.6	31.4	-4.6	29.6	23.7	25.2	26.1		9.2	8.6	8.4	9.6
22	4.8	35.9	35.9	31.7	41.6	4.6	22.7	18.2	19.4	22.2		7.9	7.2	7.0	7.7
23	7.4	27.2	19.4	16.3	27.2	4.8	14.2	10.9	11.7	14.0		7.6	5.1	5.1	5.9
24	5.3	28.9	24.0	9.4	25.5	3.8	20.0	7.6	7.7	9.9		4.8	4.3	4.3	4.8
25	7.9	13.3	16.4	4.2	12.5	4.8	7.7	5.9	6.1	7.6		4.1	4.0	3.8	4.2
26	11.2	20.7	8.4	4.6	9.4	3.9	5.9	5.1	5.1	6.3		3.5	3.6	3.5	3.5
27	35.4	41.7	39.7	39.3	38.7	15.6	20.6	19.3	19.1	21.3		7.4	7.4	7.3	7.6
28	28.3	29.5	28.3	28.8	28.3	11.2	23.2	15.0	15.0	15.0		5.9	5.8	5.9	6.1
29	17.6	17.6	17.5	18.3	17.3	7.4	9.6	9.5	9.5	9.4		4.4	4.4	4.6	4.6
30	15.8	16.3	16.0	12.9	16.8	7.3	8.2	8.2	8.6	8.7		4.4	4.6	4.5	4.6
31	9.1	9.7	8.2	8.9	9.1	5.1	5.8	5.3	5.6	5.6		3.8	3.8	3.8	3.8
32	7.1	6.6	5.4	7.6	5.9	3.8	4.3	4.0	4.3	4.0		2.8	3.1	3.1	3.1
33	0	-0.5	-0.8	-1.7	-1.0	0	0.0	-0.3	-0.3	-0.3		0.3	0.7	0.7	0.7
34	-27.4	50.9	47.9	50.3	52.1	-20.7	23.2	20.2	20.3	23.8		3.4	3.2	3.1	3.2
35	-6.6	35.4	35.3	36.2	36.2	-2.6	16.5	13.7	14.3	15.8		1.6	1.8	1.8	1.8
36	-2.8	17.0	12.8	16.7	16.9	-1.5	6.6	5.5	6.0	6.6		-0.1	-0.3	-0.2	-0.1
37	-3.8	12.2	4.3	7.0	5.3	-2.8	2.3	1.9	2.0	2.2		-0.9	-0.9	-0.6	-0.9
38	-1.0	0.5	-2.3	0.4	-1.0	-1.1	-0.1	-1.4	-1.0	-0.9		-1.4	-1.2	-1.0	-1.1
39	-2.1	-1.6	-4.1	-2.5	-4.5	-0.3	-2.1	-2.1	-2.3	-2.3		-1.5	-1.7	-1.3	-1.5
40	20.4	21.7	19.6	21.4	21.1	3.1	9.7	9.6	9.7	9.7		1.3	1.5	1.8	2.0
41	22.7	15.0	14.0	15.2	16.7	2.8	5.9	5.9	6.1	5.8		0.3	0.3	0.3	0.5
42	8.7	6.9	6.6	6.4	6.3	1.3	2.1	2.0	2.3	2.0		-0.5	-0.5	-0.5	-0.3
43	1.8	0.2	0.2	-0.3	-0.5	-0.9	-1.0	-1.2	-1.0	-1.1		-1.3	-1.3	-1.5	-1.5
44	-1.5	-1.6	-2.0	-2.1	-1.7	-0.9	-1.5	-1.3	-1.2	-1.3		-0.7	-0.7	-0.7	-0.5

No data obtained.

See pages 15 and 17 for definition of model arrangements.
*The subscript "T" denotes pressure-meters located in the corner of the entrance immediately above Pressure-meter No. 5-13.