UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

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HYDRAULIC LABORATORY REPORT NO. 32

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MODEL STUDIES OF DOS BOCAS DAM SPILLWAY

BY

J. E. WARNOCK and J. H. DOUMA

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Denver, Colorado

February 28, 1938

Guayama, Puerto Rico
November 20, 1937.

Mr. R. F. Walter,
Chief Engineer,
Bureau of Reclamation,
Denver, Colorado.

Dear Sir:

At the time of the last visit of the Consulting Engineering Board to our work here, it was agreed that model tests and studies would be made by the Bureau of Reclamation of our spillway as soon as we were able to forward sufficient information to carry out this work. We now have sufficient Dos Bocas Project preliminary spillway drawings and information sufficiently completed for you to proceed, all of which, is now being forwarded to you.

We would have liked to have forwarded this information earlier in order to give you more time but were unable to complete this work previous to this week. We are anxious that these tests and studies for Dos Bocas be completed as promptly as possible and would appreciate anything you can do to aid us in keeping up with our construction schedule.

For your information, we expect to unwater our first cofferdam on or about January 1. This cofferdam includes slightly more than the left half of the spillway, including the complete apron for that section. We would like to commence placing concrete as soon as excavation is completed.for any one block and continue as rapidly as excavation progress permits.

The depth to be excavated is comparatively shallow and we would expect to be ready for our first concrete placing by about February 1, 1938. We will have some drafting room work in Ponce after receiving your recommendations and therefore urge that you make a report by air mail as soon after January 1 as possible.

The following drawings will give you information for the spillway model studies:

Drawing No. DB-73 - Topography of Dos Bocas Dam Showing General Plans of Dam.

Drawing No. DB-74 - Dos Bocas Downstream Elevation of Spillway.

Drawing DB-75 - Dos Bocas Dam, Section Thru Spillway.

The maximum flood condition is assumed to be 200,000 cubic feet per second. The spillway crest elevation is about 295 and there are no flashboards contemplated. The maximum flood is assumed to raise the reservoir water surface immediately above the dam to elevation 323, or 28 feet above the spillway.

Subsurface tests indicate that the foundation rock under the spillway apron is quite uneven, and generally lower under the right half of the apron than under the left half. The minimum foundation rock elevation is approximately 140. The tailrace water surface is at elevation 145.

The tailrace water surface curve furnished to you has no check from actual elevation under anything like maximum flood condition. Approximately, our greatest flood since our preliminary work was commenced is not over 10 percent of the assumed maximum. The jump curve is a result of computations without allowance for friction. Prints of this curve are included.

In order to keep concrete quantities as low as possible, we have assumed that the left half of the apron would be at elevation 146.5 and the right half at elevation 142. The concrete sill has an elevation of crest at 152.5. It is the intention of this sill to prevent cross current due to the varying apron elevation. The lower portion of the apron might be raised in elevation in case actual foundation rock elevations warrant such change and to an elevation not exceeding 146.5 which should, if fully carried out, make the entire apron at one elevation.

The training walls each have 1/4 to 1 slope on the sides facing the spillway below the bucket and a transition from a vertical face to this slope at the bucket curve. Our computations indicate that the vertical walls to the limits of the apron would increase the cost materially. It is intended to shape the rock hillside downstream

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from the spillway on the left bank to improve the conditions. The draft tube outlet is on the right side and downstream from the spillway, with a minimum floor elevation of 131.5.

We are also forwarding analysis sheets of the Dos Bocas gravity type concrete dam. These are titled as follows:

Sheet No. 1 - Explanatory Notes for Gravity Analysis Sheets.

Sheet No. 2 - Abutment Section - Maximum Flood W. S. Elevation 323 - No Earthquake Effects.

Sheet No. 3 - Abutment Section - Normal Full Reservoir Operation No Earthquake Effects.

Sheet No. 4 - Abutment Section - Normal Full Reservoir Operation with Horizontal and Vertical Earthquake Effects (Vertical Acceleration Upwards).

Sheet No. 5 - Abutment Section - Normal Full Reservoir Operation with Horizontal and Vertical Earthquake Effects (Vertical Acceleration Downward).

Sheet No. 6 - Spillway Section Maximum Flood W. S. Elev. 323 No Earthquake Effects.

Sheet No. 7 - Spillway Section - Normal Full Reservoir Operation with Horizontal and Vertical Earthquake Effects (Vertical Acceleration Upwards).

Sheet No. 9 - Spillway Section - Normal Full Reservoir Operation with Horizontal and Vertical Earthquake Effects (Vertical Acceleration Downwards).

We would appreciate your having these sheets examined and would like to receive your opinion particularly with reference to the adequacy of sliding factor, shear friction factor, location of resultant pressure, and relationship of water pressure to vertical foundation pressure. We would further appreciate your opinion as to the completeness of our various assumptions upon which the analysis is based.

We wish to particularly stress the need of your furnishing a report, recommendations and other correspondence in connection with this subject by air mail in order to avoid needless loss of time.

Very truly yours,

(Sgd.) A. Lucchetti
Anton Lucchetti
Chief, Rural Electrification Division.

AIR MAIL

December 23, 1937.

Mr. Antonio Lucchetti, Chief,
Rural Electrification Division,
Puerto Rico Reconstruction Administration,
Guayama, Puerto Rico.

Dear Mr. Lucchetti:

Reference is made to your letter of November 20, 1937, and office letter of November 30, 1937, concerning model studies of the Dos Bocas Dam spillway.

The model of the spillway (drawing No. 1) was constructed to a scale ratio of 1:60 and was completed ready for observation on December 15. The completed model is shown in plates 1 and 2. Investigation of the flow conditions around the spillway abutment piers and bridge piers has been completed and an analysis of the spillway crest characteristics and pressures on the spillway face has been made. Final recommendations on changes of the design of the abutments and bridge piers are contained herein. Preliminary tests on the stilling pool have been made and it is planned to complete those studies and mail them to you on or before January 8.

<u>Abutment entrance.</u> Operation with the original abutment entrance design was very unsatisfactory. The right-angle entrances caused the sheet of water to spring free from the spillway sides, resulting in a much reduced flow at the side-walls and an unbalanced flow condition in the stilling pool. The flow down the spillway was concentrated in the center, causing severe whirls to form on each side. These whirls cause disruption of the formation of the hydraulic jump. A new abutment entrance, shown as recommended design in drawing No. 2, completely eliminated this undesirable condition. Plates 3A and 3B show the original and recommended abutment entrance design. Plate 3C shows the sheet of water springing clear of the side-wall and plate 3D shows it remaining in contact with the side-wall. A smooth entrance

second-feet, the topography downstream on the left side will act as a control forcing the stream over into the main channel. As a result, a "dead" area was formed from which the water flowed back into the jump area, disrupting the jump and forming the whirl.

Various combinations of sills, dentates, and elevations of apron were tried in an effort to eliminate this whirl. Deeper excavation than the 8:1 slope shown was discarded because of the additional expense. The solution which is recommended for incorporation in the final design consists of shifting the entire spillway crest fifteen feet to the right and converging the spillway and stilling pool walls toward the downstream end of the apron. The step-by-step analysis of this solution is as follows: By rotating the left wall about its intersection with the crest line such that the downstream end was fifteen feet to the right of the original position, it was found that the deficiency of water along that wall was eliminated and the stream so directed toward the original channel that the "dead" area in which the whirl formed was eliminated. To again establish symmetry of design, the right wall was rotated a similar amount to the left. No adverse effect was noted on the flow conditions; in fact, there was some improvement, despite the fact that the downstream end of the apron had been constricted a total of thirty feet. However, the change in the right wall placed it out in the present channel fifteen feet. To bring it back to its original position, which is more desirable, the entire crest was shifted to the right the distance of fifteen feet, or in other words, the center-line of the spillway was shifted from Station 13+35 to Station 13+50. This change of position not only appreciably improves the hydraulic conditions in the stilling pool but will effect an appreciable economy by reducing the deep rock excavation on the left side. The depth of excavation at the left downstream end has been reduced from approximately 45 feet to approximately 20 feet.

All of the stilling basin changes recommended in this and previous correspondence have been incorporated in Drawing No. 7, a copy of which is attached. This drawing will clarify any questions

which occur concerning recommendations contained in the December 31 radiogram. The shifting of the entire spillway crest fifteen feet to the right effects two changes in the recommendations in the December 31 radiogram. The width of apron on the left side at Elevation 151.5 has been reduced from 85 feet to 55 feet and the slope between the two sections of apron has been changed from 1:7 to 1:8.

In the design submitted with your letter of November 20, 1937, the top of the spillway training walls was shown at Elevation 200.0 or four feet below the assumed downstream water surface for a discharge of 200,000 second-feet. With the recommended apron design the mean water height at the end of the pool is approximately elevation 196.0 with some splashing to a greater elevation. If the area back of each stilling pool training wall is back-filled and paved with sufficient slope to provide quick drainage of this splash, the top of these training walls can be lowered from Elevation 200.0 shown on your drawings to Elevation 196.0. In this case it will be necessary to provide a wing-wall between the power house and the mountainside to at least Elevation 204.0. Another wing-wall (Drawing No. 7, Section A-A) must be placed between the downstream end of the left stilling pool wall and the mountainside to prevent return flow over the paved section and into the stilling pool. This wall should slope from Elevation 196.0 at the pool wall to at least Elevation 204.0 at the mountainside. Some allowance on both of these wing-walls should be made for wave height.

If no back-fill or wing-wall is provided, it will be necessary to raise the stilling pool walls to Elevation 204.0 plus freeboard to prevent return flow between the stilling pool walls and the mountainsides, which would flow over the pool walls and tend to disrupt the hydraulic jump. In this case, it will be necessary to design the pool walls against full water load behind them. Hydraulically, there is no objection to this arrangement since there will be pools of quiet water behind the walls.

The stilling pool training walls are shown on your drawings with 1/4:1 slopes and transitions from that to vertical faces on the spillway training walls. It is recommended that the stilling pool walls be made vertical and in the same plane as the spillway training walls. It has been found on this and several previous experiments that sloping walls tend to allow water to return along the wall disrupting the formation of the hydraulic jump next to the walls.

With the reduction of length of the spillway apron, the power house can be moved upstream and in the final studies it has been so located.

Of the various combinations of sills and dentates studied on the downstream end of the stilling pool, the one shown on the drawing has proven to be the most satisfactory. In the December 23 letter under the heading of "Stilling Basin", it was stated "...but on the right side a very turbulent boiling action occurs over the end sill in line with the lower river channel. The combined action of a large ground roller in the deeper river channel behind the end sill and the action of the high end sill on this side gives the water flowing downstream on the stilling basin floor an excessive upward component of velocity resulting in the unfavorable boiling." This condition has been improved by omitting the sill entirely and placing a 1:1 slope from Elevation 146.5 to the channel bottom. This slope was not made flatter due to the possible obstruction of the tailrace in front of the power house draft tubes.

Excavation of the rock on an 8:1 slope downstream from the left side of the stilling pool showed satisfactory flow conditions on that side.

The alteration of the model to include all changes was completed today. Subsequent operation showed satisfactory conditions at all discharges. Photographs, 16 mm motion pictures, and hydraulic measurements will be taken immediately and sent to you at the earliest possible date.

The model of the spillway and stilling pool can be left intact for a few weeks so that any changes which you deem necessary can be studied. If you have occasion to be in Washington, D. C. during the next month or so, a trip to Denver to see the model in operation might be of interest to you.

Your discussion of the results obtained in these hydraulic model studies would be appreciated.

Very truly yours,

Chief Engineer.

Confirmation by
regular mail.

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condition existed at the side-walls and the flow distribution across the spillway was comparatively uniform, as shown by the depth distribution normal to the spillway at elevation 200.0 on drawing No. 3. By comparing plates 4A and 4B it is seen that the jump is less concentrated in the center of the pool with less return flow along the stilling basin walls with the recommended abutment entrance design.

Bridge piers. The design of the upstream end of the original bridge piers (plate 3A) was unsatisfactory, as can be seen by studying plate 3C. The square-end upstream edges caused water to rise against the front of the pier to a height of four feet (prototype) above the normal water surface. Due to the square corners, the sheet of water periodically sprang free from the sides of the piers. If this action would occur in the prototype, resulting vibration might damage the bridge. The recommended bridge pier design is shown on drawing No. 4 and in plate 3B. This design proved entirely satisfactory since the water remained in contact with the side-walls at all times. Since there are to be no gates on this structure, a square edge at the downstream end of the bridge pier is hydraulically satisfactory so no change is suggested.

Spillway discharge curve. The recommended abutment entrance and bridge pier designs produce a higher crest coefficient of discharge, which will result in lower reservoir elevations for corresponding discharges. The spillway discharge and crest coefficient curves are shown on drawing No. 5. For the maximum flood of 200,000 second-feet, the reservoir water surface will be Elevation 322.55, or 27.55 feet above the spillway crest. The assumed value of 28 feet compares favorably with the experimental value. The crest coefficient for the maximum discharge as computed by the formula $C - Q/LH^3/2$ is 3.95. The water surface on the draw-down curve at the upstream end of the bridge piers is at Elevation 318.0 (drawing No. 6) for the maximum discharge of 200,000 second-feet. This height may be of use in determining bridge clearance.

Spillway pressures. The pressures on the face of the spillway and on the stilling basin floor for discharges of 50,000, 100,000,

150,000, and 200,000 second-feet are shown on drawing No. 6. The piezometers (drawing No. 1) are located on a line 150 feet from the left wall. Piezometers 1 to 5, inclusive, show some pressure of impact due to velocity of approach. Small negative pressures are shown by piezometers 13, 16, and 17. The maximum negative pressure is 2.5 feet (prototype) at piezometer 17 for the discharge of 200,000 second-feet. High pressures due to centrifugal action are shown at the entrance to the stilling basin. Beyond the region of centrifugal action the pressures conform closely to the corresponding water depths.

Stilling basin. The original stilling basin design was tested for discharges of 50,000, 100,000, 150,000, and 200,000 second-feet. Mean water surface profiles through the jump are shown on drawing No. 6. The tailwater for each discharge was determined from the tailwater height curve for the Arecibo River at Dos Bocas Damsite furnished with your letter of November 20. Plates 4B, 5B, and 6B show the pool action for 200,000 second-feet with the revised design of the spillway crest entrance. An excellent hydraulic jump exists over the high-level floor, but on the right side a very turbulent boiling action occurs over the end sill in line with the lower river channel. The combined action of a large ground roller in the deeper river channel behind the end sill and the action of the high end sill on this side gives the water flowing downstream on the stilling basin floor an excessive upward component of velocity resulting in the unfavorable boiling. This may be remedied by decreasing the sill height on this side, or if this results in strong cross-currents due to the varying apron elevation, an apron sloping about 4:1 from the top of the sill to the channel bed may prove effective. The jump is satisfactory over most of the basin for a discharge of 150,000 second-feet (plate 9B) and the boiling on the right side is less serious but the condition still exists. The hydraulic jump is appreciably shorter for a discharge of 100,000 second-feet and most of the energy is dissipated before the end sill is reached so no boiling action is present (plate 10A) and a good jump exists over the entire basin. A peculiar condition exists with a discharge of 50,000 second-feet which can be

remedied by either a higher floor on the left side or a deeper excavation immediately downstream. The jump in this case is satisfactory on the low-level floor but the combination of the shallow tailwater on the left side with depth of pool causes a large whirl (plate 10B) to form, which might cause damage to the pool floor due to the grinding action of transported material.

Stilling basin floor elevation. While the studies have not progressed sufficiently to verify the conditions at all flows, the indications are that the floor may be raised provided the elevation of sound foundation rock warrants it. It has been found that a satisfactory jump forms with a flow of 200,000 second-feet and the tailwater at Elevation 195.0 which means that the right half of the apron could be raised to Elevation 146.5, as you mention in paragraph 9 of your November 20 letter. The indications are that a third floor level may be placed at Elevation 155.0 at the extreme left side of the pool. This would materially reduce the excavation. The feasibility of this proposal is being studied at the present time.

Stilling basin length. The length of the stilling basin appears to be satisfactory with the original floor levels. Other shapes of end sill and baffle blocks will be tested in an attempt to shorten the length. It is proposed to determine a stilling basin length and sill combination which will show the least scour in a sand bed at the downstream end of the pool and still contain the complete jump within its length.

Training walls. The spillway training walls have seven feet of freeboard at the downstream edge of the bridge abutment and five feet at the point where the sheet of water enters the pool. The waves reach Elevation 210.0 on the side-walls at the end of the stilling basin for 200,000 second-feet. The 1/4:1 slope of the training walls and the transition from the vertical face to the 1/4:1 slope appear to be satisfactory.

Very truly yours,

R. F. Walter
Chief Engineer.

Confirmation by
regular mail.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
Customhouse
Denver, Colorado

Office of the Chief Engineer

February 28, 1938.

Mr. Antonio Lucchetti, Chief,
Rural Electrification Division,
Puerto Rico Reconstruction Administration,
Guayama, Puerto Rico.

Dear Mr. Lucchetti:

Reference is made to office letter of January 12, 1938, concerning model studies of Dos Bocas Dam, in which it was stated that photographic records and hydraulic measurements of the final recommended design would be taken immediately and sent to you at the earliest possible date. Due to the urgency of other work in the laboratory, the final work on the Dos Bocas model was postponed several weeks. Still photographs, results of hydraulic measurements on the final design and a statement of the cost of the model study are contained in this letter and one reel of moving pictures which becomes your property will be mailed under separate cover as soon as the titles are inserted.

To present all of the findings resulting from the hydraulic model studies, which were contained in office letters of December 23, 1937, and January 12, 1938, and office radiogram of December 31, 1937, and those resulting from the final measurements in one group, the following summary and conclusions are given:

SUMMARY

Features of the design developed from the model tests.

1. Shape of abutment entrances.
2. Shape of bridge piers.
3. Elevations and length of spillway apron.
4. Height and shape of sill at end of apron.
5. Shape of sidewalls down spillway face.
6. Height and shape of stilling basin training walls.

7. Treatment of left bank excavation.
8. Height of wing-walls between the power house and the right mountainside and the left training wall and left mountainside.
9. Extent of back-filling and paving from the top of the training walls to the mountainsides.

Points of design checked by results from the model tests.

1. Reservoir elevation for the maximum discharge.
2. Maximum water-surface elevation at the bridge.
3. Spillway capacity. Heads and coefficients at various discharges over the spillway.
4. Amount and extent of pressures and vacuum on spillway crest.

Other data obtained.

1. Position and water surface profiles of jump on apron through all stages of the maximum flood.
2. Pressures on apron at various stages of flow.
3. Nature of sand and gravel erosion in river bed below spillway.
4. Minimum tailwater required for a satisfactory hydraulic jump.
5. Tailwater at which jump sweeps from apron.
6. Comparison of flow conditions with no downstream scour and with downstream scour.
7. Photographic record, both motion and still pictures, of flow conditions at various discharges.

RECOMMENDED DESIGN AND CONCLUSIONS

1. Abutment entrances. The right-angle abutment entrances caused the sheet of water to spring free from the spillway sides, resulting in a much reduced flow at the side-walls and the formation of severe whirls on each side of the stilling basin. The recommended design on drawing No. 2 improved the flow conditions.

2. Bridge piers. The original square-ended upstream pier noses resulted in a rough water surface immediately in front of the

piers and aerated areas on the sides of the piers. The type of pier nose shown in drawing No. 4 and used on the upstream end gave satisfactory conditions.

3. Discharge Capacity. The discharge capacity of the crest is ample. For a maximum reservoir elevation of 323.00, the crest will discharge 205,000 second-feet, which exceeds the requirement of 200,000 second-feet.

4. Bridge clearance. For the maximum discharge of 200,000 second-feet the water surface on the draw-down curve at the upstream end of the bridge piers is at elevation 318.0, which will be useful in determining bridge elevations.

5. Crest pressures. There is a slight vacuum developed on the downstream face of the crest amounting to about 2.5 feet of water at the maximum point for a discharge of 200,000 second-feet over the crest. It is possible that pressure measurements are in error by as much as 3 feet due to the difficulty in obtaining exact pressure measurements in the model. However, should the small vacuum exist and no account of it made in the stability design no alarm should be shown since the negative pressure is slight and only extends over a small area.

6. Stilling basin floor elevation. The spillway stilling basin floor must be so located with respect to elevation that the tailwater will produce correct depths for the formation of a good hydraulic jump over the entire range of discharges. The relationship of curves A and B on drawing No. 8 indicates that the floor placed at elevation 146.5 is slightly low for discharges below 180,000 second-feet and slightly high for discharges above 180,000 second-feet. Assuming that the ground line shown on submitted drawings limits the pool floor to a position at or below elevation 146.5 on the right side, studies were made to determine the maximum satisfactory elevation on the left side. Tests showed that with maximum discharge a comparatively satisfactory jump formed over the entire basin with elevation 146.5 on the right and 151.5 on the left, with an 8:1 slope between. The raising of the left floor level above

elevation 151.5 will produce poor hydraulic jump conditions. The lack of agreement between the experimental D_2 and theoretical D_2 as determined by the momentum formula may be attributed to the presence of an end sill which is effective in reducing the required D_2 for a satisfactory jump formation. The small cross-flow on the inclined floor between the two level sections is not sufficient to produce any unbalanced condition.

7. Stilling basin length. The length of the stilling basin should cover the range of velocity retardation until a point is reached where the velocities are low enough not to cause scour of the river bed below. The apron length may be shortened by placing baffle blocks, or an end sill, or both at the end of the basin to lift velocities just clear of the river bed until they are retarded in the natural course of the jump action. The length of the jump or distance downstream required for full recovery varies as its magnitude, increasing with the discharge. The recommended basin length, drawing No. 7, does not extend the full length of the jump occurring at the higher flows but covers the range of greatest turmoil characteristic of the beginning. The velocity leaving the end of the apron may exceed 30 feet per second at the minimum flow but it is lifted by the sill from the river bottom below until retarded in the jump. The hillside along the left bank is exposed to comparatively high velocities during maximum floods, but since the bank is of good rock little erosion is to be expected. However, if the bank erodes a better jump is formed and the velocities are reduced.

8. End sill. The dentated sill or any arrangement of baffle blocks and end sill placed across the apron will, of course, check the velocity of the overflowing sheet of water. Blocks and sills with vertical upstream faces are more effective than those with sloping upstream faces so that smaller sizes may be used to produce the required results. However, the vertical upstream face blocks and sills will receive tremendous impact as the high velocity carries to the floor with little retardation. This calls for heavy blocks with reinforcement and anchorage. In a stilling basin design

with one or more rows of baffle blocks upstream from the end of the apron, considerable dependence for energy dissipation is placed on the blocks, a small break in which might endanger the entire scheme. The whirls formed behind the steps increase the possibility of cavitation effects and pitting of the concrete. The form work required would be additional expense. Altogether the advantages to be gained by their use in this scheme were so small that no recommendation was made for their adoption in the design. They may be used to greater advantage in an installation where there is a greater variation between the depth required for the hydraulic jump and those actually furnished by the natural tailwater. To eliminate any dependence on baffle blocks or end sills, only a sloping upstream end sill is recommended, drawing No. 7. The sill should be of proper height. If too high, it will receive excessive impact, lift the high velocity flow from the bottom to the surface where retardation is much slower and also expose the river banks to higher erosive forces. If made only high enough to deflect the flow just off of the river bed the velocity is quickly retarded by the static pressure of the tailwater. Tests showed the best sill height to be 4 feet on the 151.5 elevation level and 7 feet on the 146.5 elevation level, except on the right side in line with the deeper river channel where the ground roller is effective in lifting the high velocity flow to the surface, eliminating the end sill.

9. Training Walls and Wing Walls. To have a balanced, effective hydraulic jump at the stilling pool side walls, the training walls must be vertical. The recommended design includes this feature. With the downstream rock slope of 8:1 on the left side and the deep river channel on the right side, a severe whirl formed on the high level floor for flows of less than 75,000 second-feet and with parallel training walls 360 feet apart. By changing the spillway centerline station from 13+35 to 13+50 and converging the training walls from a spacing of 360 feet at the spillway crest to 330 feet at the end of the stilling basin, the whirl on the high-level floor is completely eliminated, and the hydraulic jump on the right

side is improved. This has the additional advantage in reducing the rock excavation on the left side. The change, recommended for design, results in a balanced, effective hydraulic jump for all flows. The height of these vertical side-walls should be such as to prevent excess inflow of the adjacent tailwater. It is better if the walls are high enough to exclude the adjacent tail-water altogether. The training wall height of 196.0 as recommended is ample to exclude all but splash water from overtopping the wall. A 10:1 slope concrete covering from the top of the training walls to the mountain side, on each side is recommended to allow the splash water to drain back into the stilling basin. Wing-walls as shown on drawing No. 7 are recommended to prevent return flow from the adjacent tailwater over the paved section and into the stilling basin.

10. Treatment of downstream excavation. Excavation of rock to an 8:1 slope downstream on the left side results in satisfactory flow conditions for all discharges. Excessive cross-flow occurs with less excavation. Deeper excavation is not warranted because of the additional cost and the small improvement in the jump conditions.

11. Hydraulic jump performance. The hydraulic jump is very effective in dissipating the energy of the overflow, the resultant velocities after completion of the jump being little greater than those normally present in the river channel during a given flow, were no dam present. The elevation of the apron is such that the hydraulic jump will form well up on the toe of the dam at all stages of the flood, and the dissipation of the tremendous energy of the falling sheet occurs by the natural action of impact of water against water (drawing No. 10). Assuming the rock downstream from the apron to be of such soundness as to resist scour completely, the tailwater may be lowered from 3 to 7 feet to curve C, drawing No. 8, without appreciably changing the conditions of flow. If the rock does scour, after maximum scour has been attained, the downstream ground roller is effective in keeping the jump in the stilling basin so that then the tailwater may be lowered from 7 to 11 feet to

curve E, drawing No. 8, and still have a satisfactory jump. When the tailwater is lowered to curves D and F the jump will sweep out of the stilling basin for no scour and maximum scour, respectively. Curves C, D, E, and F show that for flows less than 40,000 second-feet, scour does not influence the jump performance, and for flows less than 25,000 second-feet, the end sill gives sufficient D_2 depth to produce the jump on the apron without the aid of downstream tailwater. When the maximum scour has been attained the water-surface is not as smooth as with no downstream scour due to the effect of the ground roller in directing the flow upward at the end of the stilling basin, drawing No. 11. However, this condition exists only for flows greater than 100,000 second-feet and even then the condition is very much less severe at the sides where the bank scour might occur. The higher water surface at the end of the stilling basin for the maximum scour condition results in a larger surface jump roller, which is more efficient in dissipating the energy, so that a lower back pressure due to tailwater is required to maintain a satisfactory jump.

12. Stilling basin pressures. High pressures due to centrifugal action are shown at the entrance to the stilling basin. Beyond the region of centrifugal action the pressures conform closely to the corresponding water depths except near the end sill where centrifugal action again occurs. The maximum scour condition gives the highest pressures, due to the greater depth of water, so these pressures should be used in designing the stilling basin floor (drawing No. 11).

13. Scour. It may be expected that the gravel deposits in the river channel and the earth overburden will be washed a considerable distance downstream during a flow of any magnitude. These deposit as a bar across the channel downstream, through which the receding flood waters cut a passage along the right bank. Scour tests in the model were made under the most adverse conditions and these tests indicate that the maximum depth of scour will be

approximately to elevation 136 and about 120 feet beyond the end of the stilling pool. The crest of the downstream bar will be at about elevation 166 and 350 feet downstream from the end of the basin. The sand remained piled well against the stilling basin end sill, giving assurance against undermining at the end of the basin (drawing No. 11 and plate 18B). Sand deposits in the deeper river channel on the right almost completely cover the 1:1 slope apron from the elevation 146.5 floor to the river channel so that this apron serves no purpose. It is recommended that a vertical cut-off wall down to rock in the river channel be used instead of the 1:1 sloped apron recommended heretofore and shown on drawing No. 7.

14. Power house. The recommended 50 feet shortening of the stilling pool allows the placing of the power house 50 feet nearer the crest of the dam, consequently, resulting in 50 feet shorter power conduits.

15. Prototype performance. Prototype performance similar to that of the model may be reasonably assured due to the well established theory and existing proofs of the hydraulic jump action at various velocities and depths. The Dos Bocas model was built and tested with precision and care. The surfaces exposed to the flow of water were practically frictionless so that any friction such as that caused by the entrainment of air existing in the prototype structure will be on the safe side.

16. Justification of model study. These studies indicated the difficulty of stilling basin design by use of a theoretical or empirical method. Minor factors often control pool action and designs should be checked by model studies whenever efficient operation is desired. The experiments developed a much better form of spillway with a smaller cost than the original design. The saving in cost alone is many times the cost of the investigation.

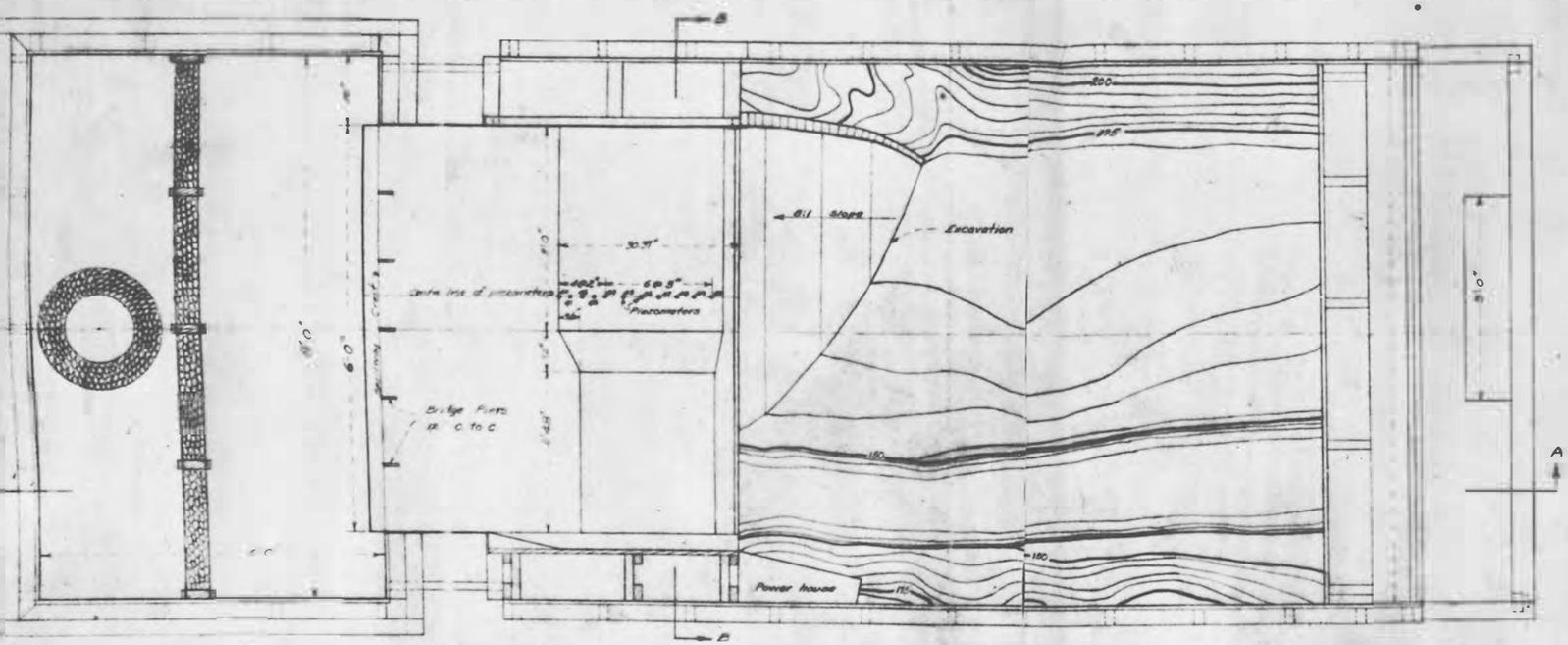
The following is an itemized statement of the cost of the model study:

Preliminary investigations at Fort Collins	$ 331.56
Materials of construction	150.00
Labor (construction, operation, and dismantling model)	452.44
Engineering (model design, supervision of construction, analysis of data and reports)	442.36
Total, less overhead charges	1,376.36
Overhead charges, 10%	137.64
Total cost	$1,514.00

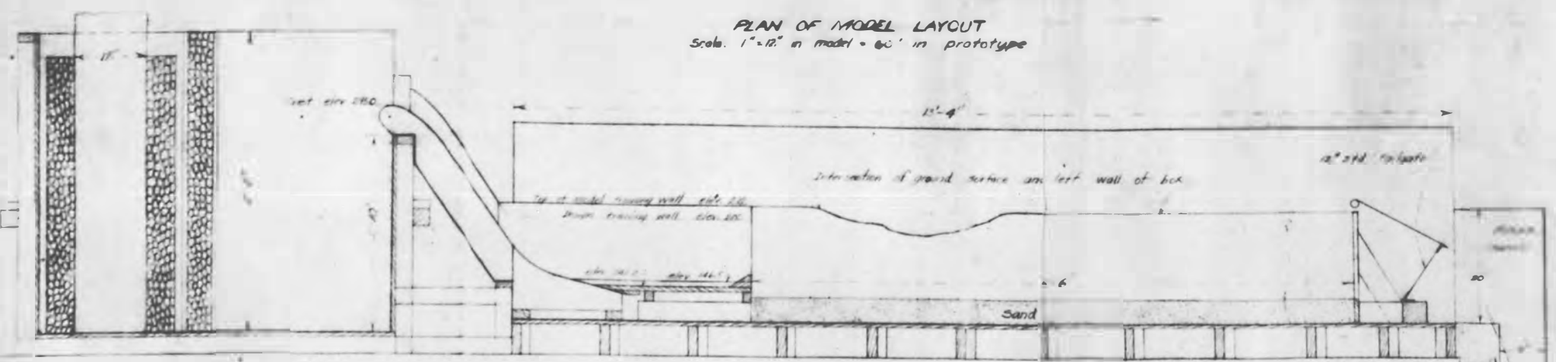
Previous experience on similar problems has shown that optimum results on model studies can be obtained only by the close cooperation of the design and laboratory engineers. In this case, unfortunately, time and distance prevent such cooperation. As a result, the following suggestion is offered. Demands for space in the laboratory have made it necessary to remove the model; however, if any changes of design are made which are at variance with the recommendations of the laboratory staff, it is suggested that they be discussed by letter before being finally incorporated.

Very truly yours,

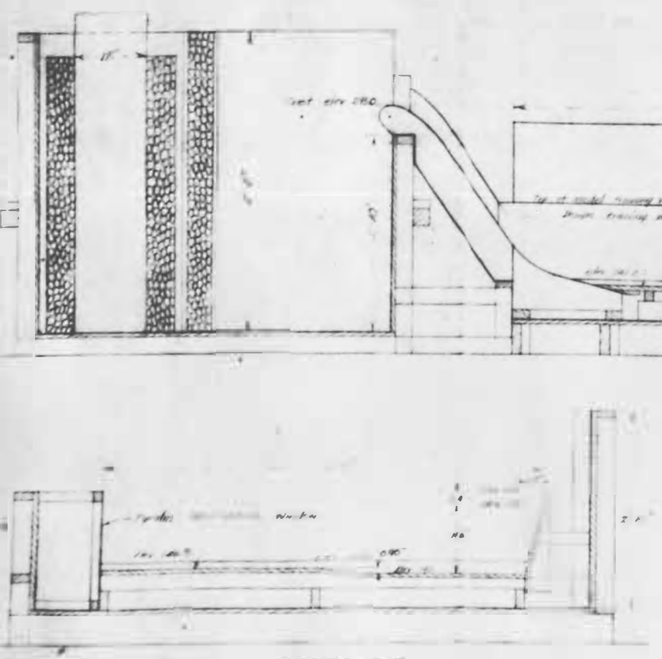
(Sgd.) R. F. Walter
Chief Engineer.



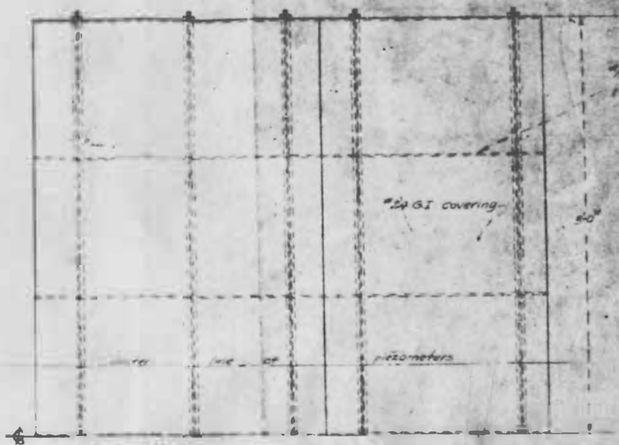
PLAN OF MODEL LAYOUT
Scale. 1" = 12" in model = 60' in prototype



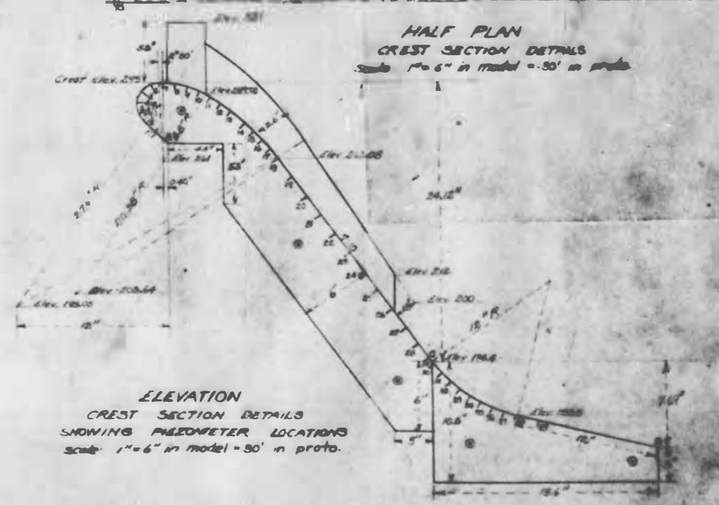
SECTION A-A



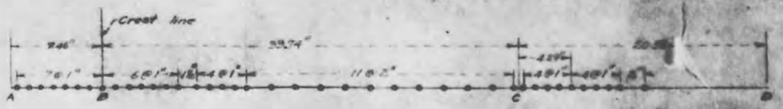
SECTION B-B



HALF PLAN
CREST SECTION DETAILS
Scale 1" = 6" in model = 30' in proto



ELEVATION
CREST SECTION DETAILS
SHOWING PIEZOMETER LOCATIONS
Scale 1" = 6" in model = 30' in proto.



PIEZOMETER LOCATIONS ALONG THE SPILLWAY CREST
Scale: 1" = 6" in model = 30' in proto.

PLAN

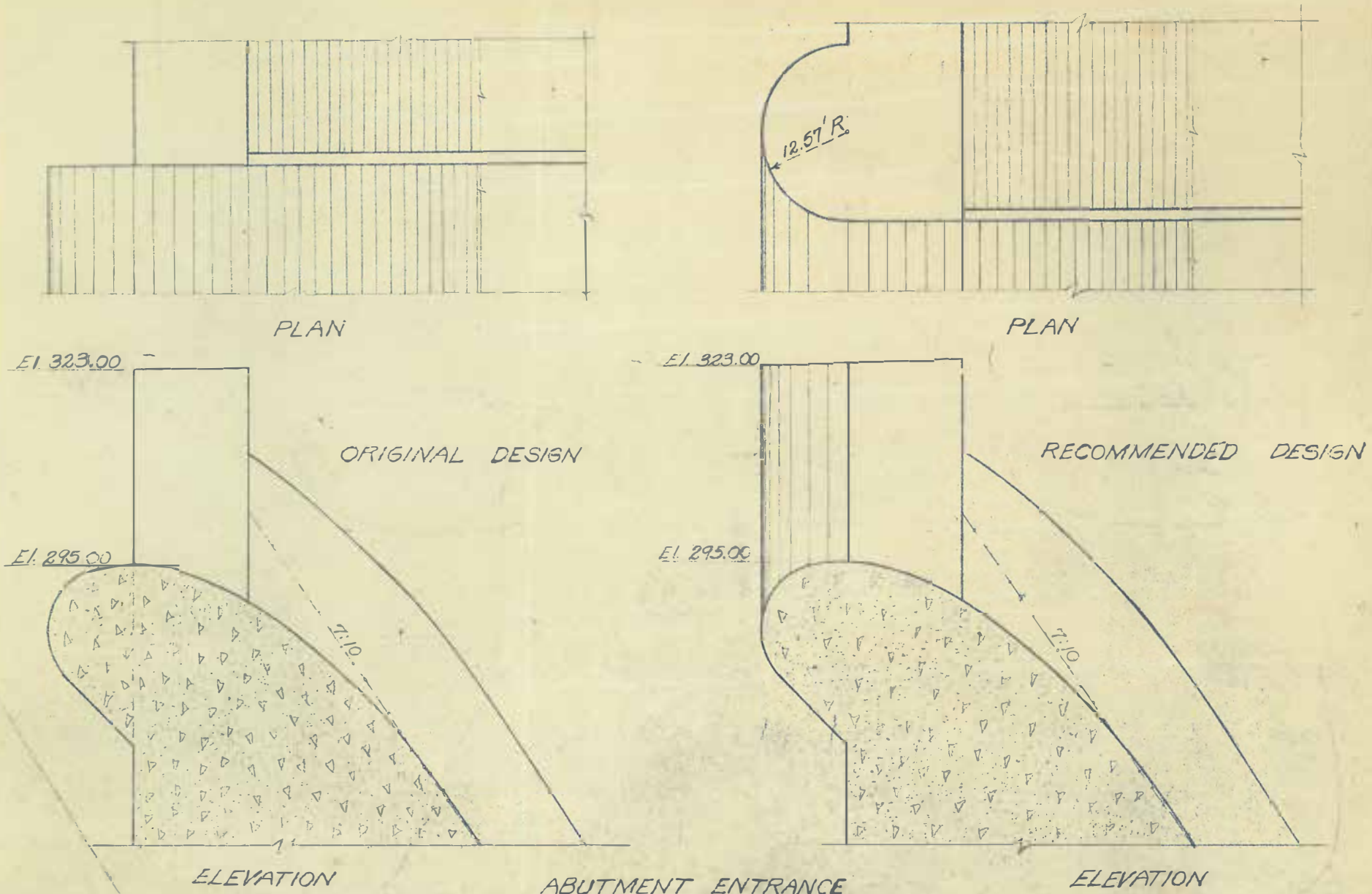
END ELEVATION
RIGHT SIDE

ELEVATION
END SILL
Scale 1" = 6" in model = 30' in proto

END ELEVATION
LEFT SIDE

DEPARTMENT OF INTERIOR
BUREAU OF RECLAMATION
FOR
PUERTO RICO RECONSTRUCTION ADMINISTRATION
RURAL ELECTRIFICATION DIVISION
DOS BOCAS DAM
HYDRAULIC MODEL STUDIES
GENERAL MODEL LAYOUT
MODEL SCALE FOR [illegible]
DRAWN J.H.D. SUBMITTED
TRACED RECOMMENDED
CHECKED J.H.D. APPROVED
DENVER, COLORADO

HYD 32

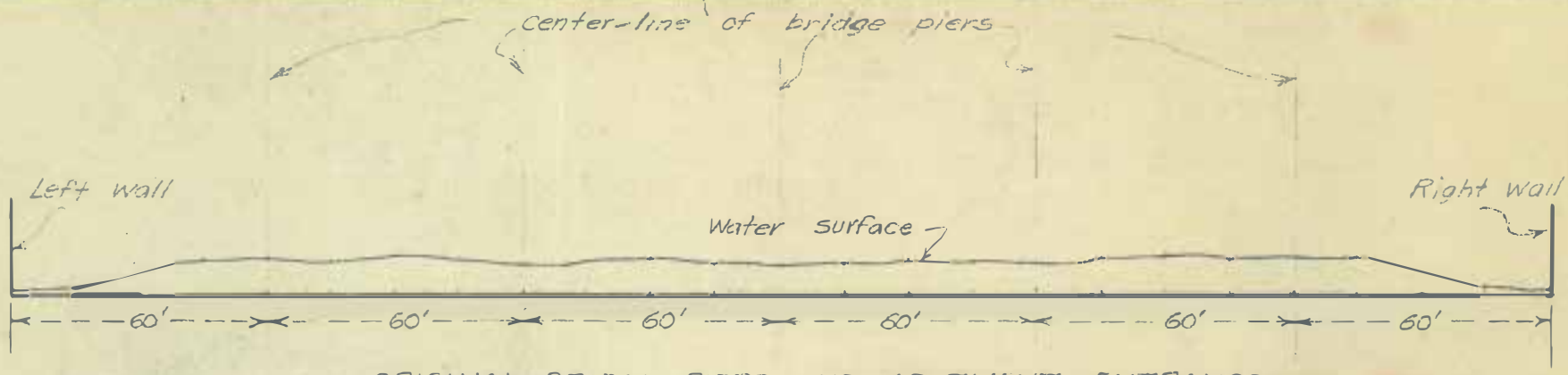


ABUTMENT ENTRANCE
DOS BOCAS DAM Spillway — Puerto Rico

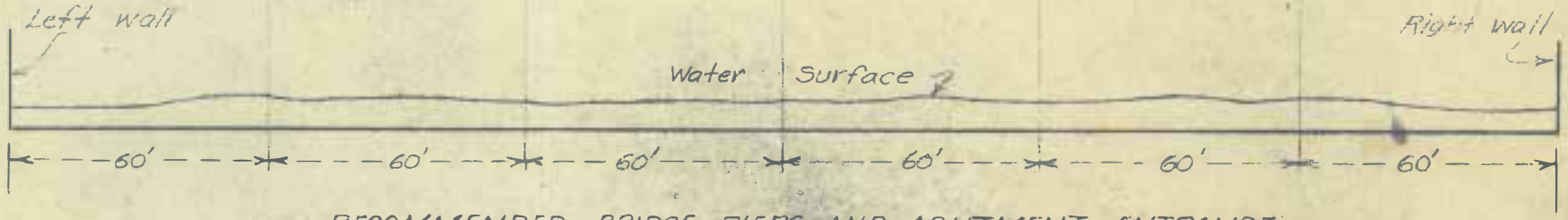
DRAWING No. 2

Job 24

10-17-57



ORIGINAL BRIDGE PIERS AND ABUTMENT ENTRANCE



RECOMMENDED BRIDGE PIERS AND ABUTMENT ENTRANCE

DEPTH DISTRIBUTION
NORMAL TO THE SPILLWAY AT ELEV 200.0

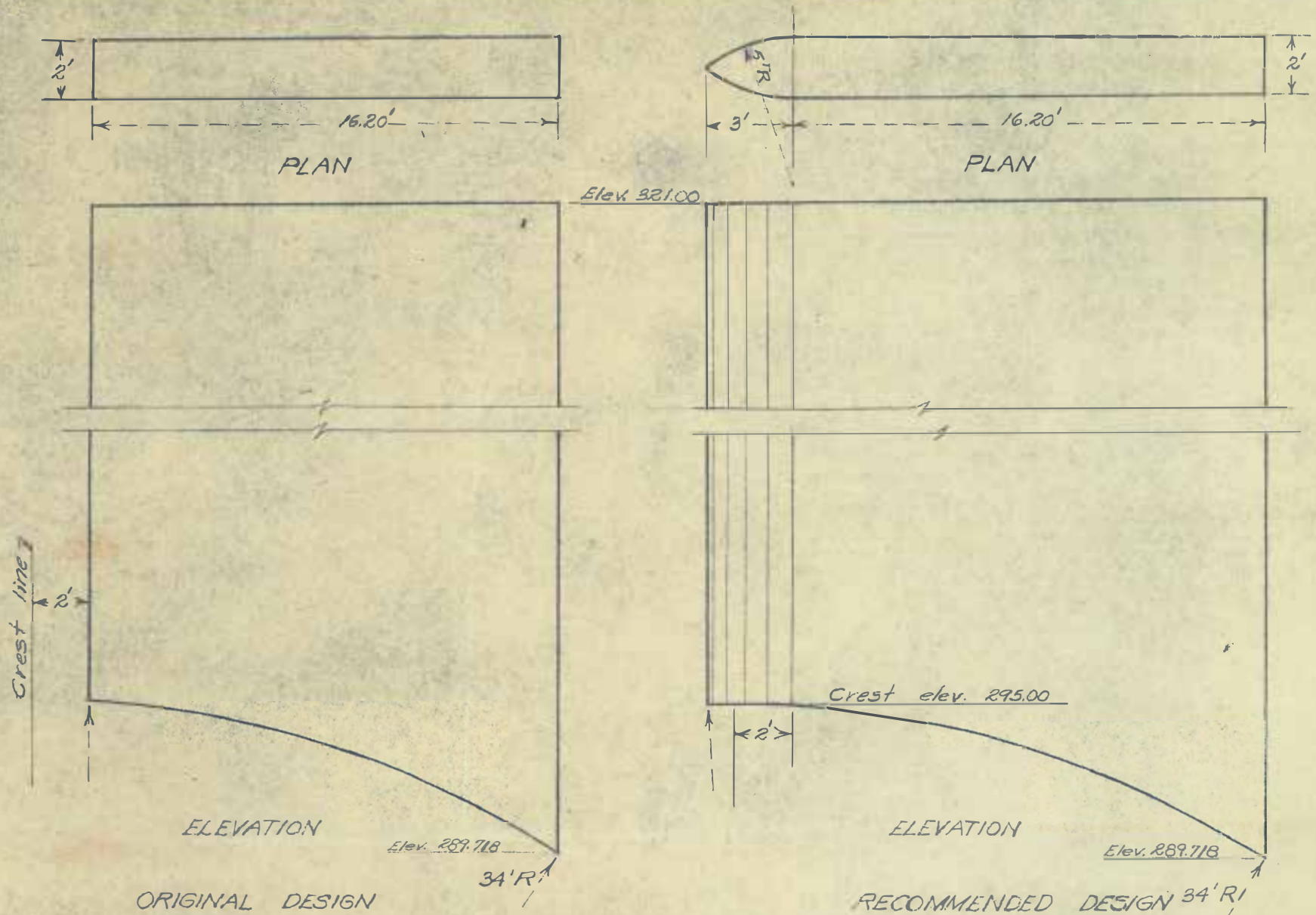
DOS BOCAS DAM

SCALE 1"=40'

DRAWING No 3

Job No 24

12-21-37



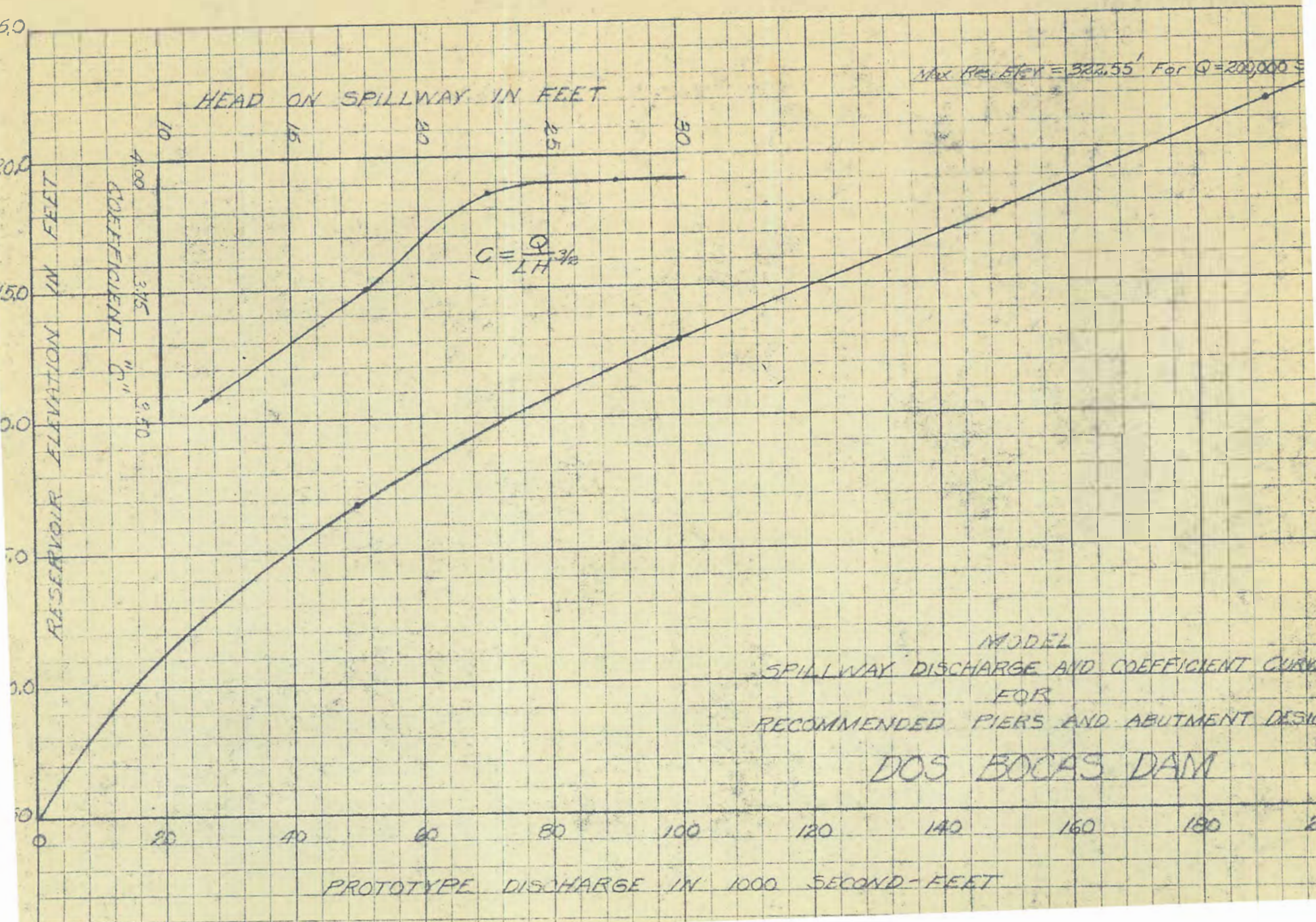
BRIDGE PIERS

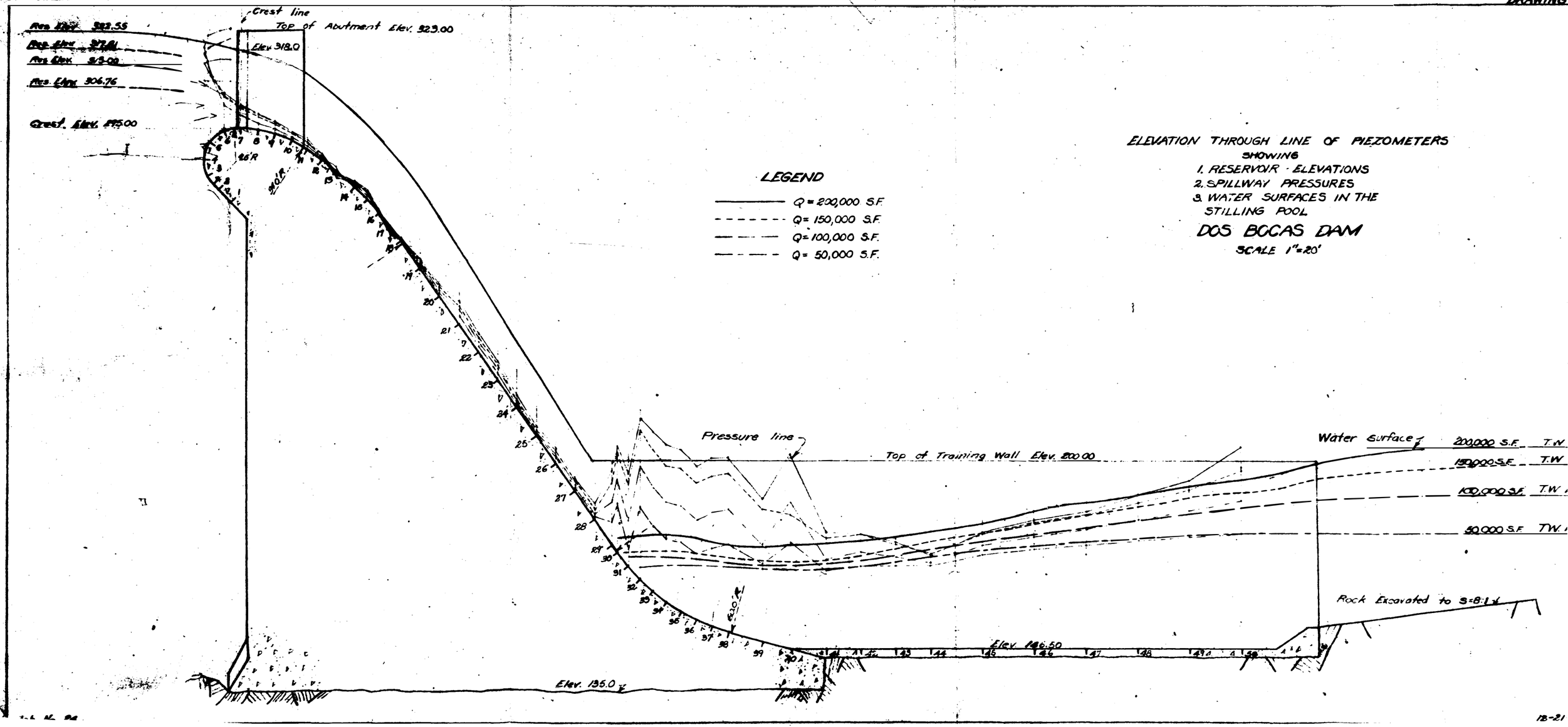
DOS BOCAS DAM

SCALE 1"=20'

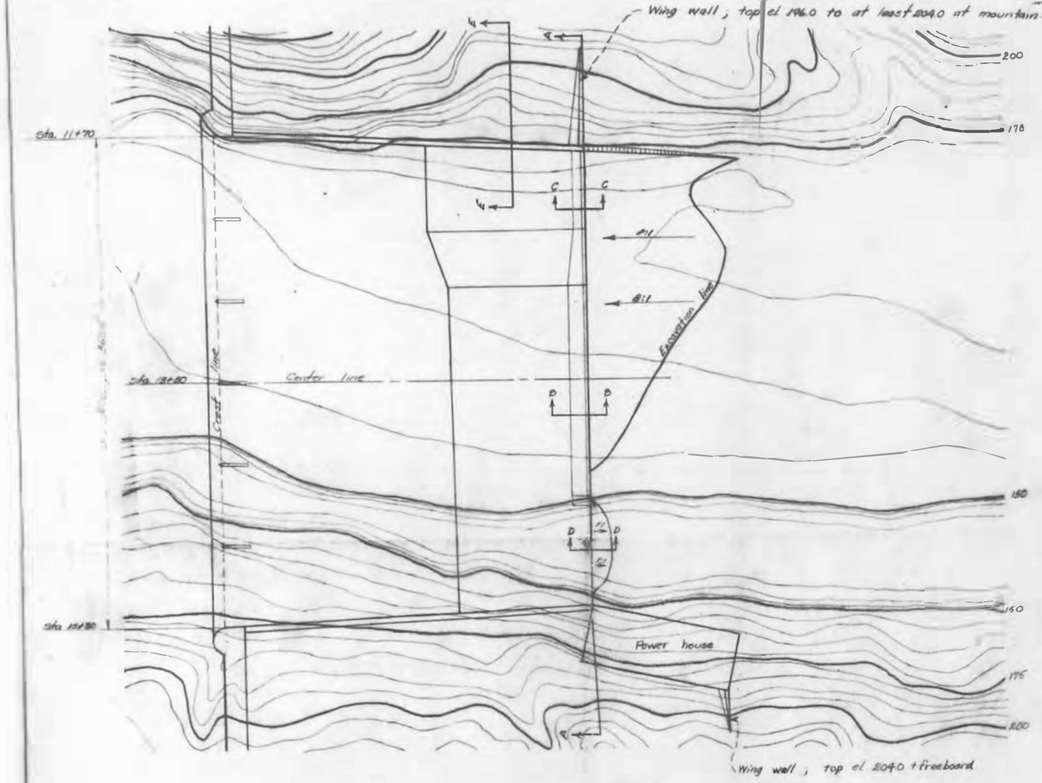
DRAWING No 4

12-20-37



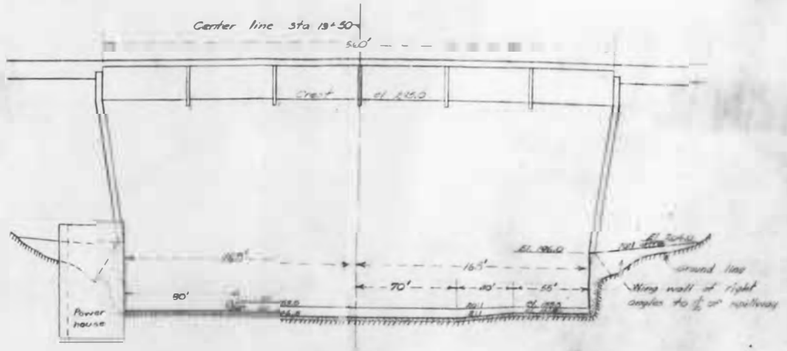

Res. Elev. 323.55
Res. Elev. 317.61
Res. Elev. 313.00
Res. Elev. 306.76
Crest. Elev. 295.00
Crest line
Top of Abutment Elev. 323.00
Elev. 318.0
46R
40R
LEGEND
Q = 200,000 S.F.
Q = 150,000 S.F.
Q = 100,000 S.F.
Q = 50,000 S.F.
ELEVATION THROUGH LINE OF PIEZOMETERS
SHOWING
1. RESERVOIR ELEVATIONS
2. SPILLWAY PRESSURES
3. WATER SURFACES IN THE STILLING POOL
DOS BOCAS DAM
SCALE 1"=20'
Pressure line
Top of Training Wall Elev. 200.00
Water Surface
200,000 S.F. T.W.
150,000 S.F. T.W.
100,000 S.F. T.W.
50,000 S.F. T.W.
Rock Excavated to S=8:1
Elev. 146.50
Elev. 135.0
42R
HYD 82

DRAWING No. 7

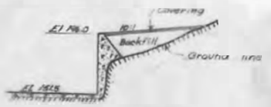


PLAN
SCALE 1" = 50'

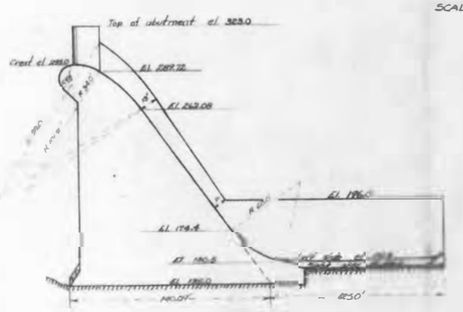
SECTION D-D
SCALE 1" = 10'



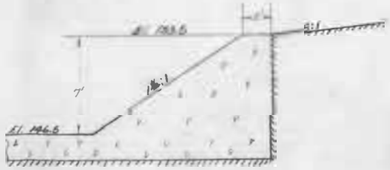
DOWNSTREAM ELEVATION
SECTION A-A
SCALE 1" = 50'



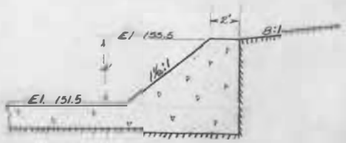
SECTION E-E
SCALE 1" = 50'



SECTION ON CENTER LINE
SCALE 1" = 50'



ELEVATION OF END SILL
SECTION B-B
SCALE 1" = 5'



ELEVATION OF END SILL
SECTION C-C
SCALE 1" = 5'

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

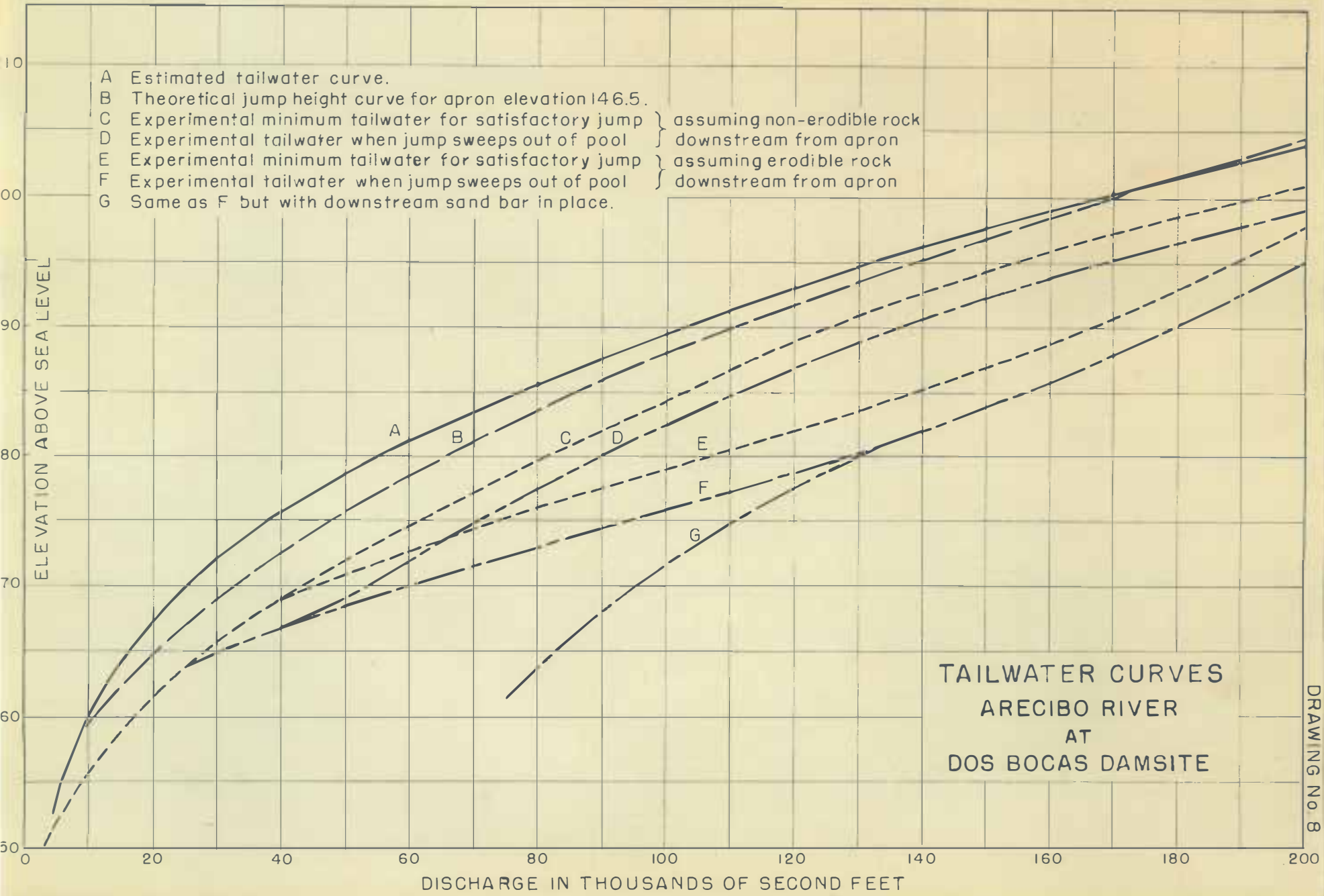
DOS BOCAS PROJECT - PUERTO RICO

DOS BOCAS DAM

RECOMMENDED SPILLWAY DESIGN

DRAWN J.H.D. SUBMITTED
TRACED RECOMMENDED
CHECKED J.E.W. APPROVED

DENVER, COLO. JAN. 18, 1951


A Estimated tailwater curve.
B Theoretical jump height curve for apron elevation 146.5.
C Experimental minimum tailwater for satisfactory jump } assuming non-erodible rock
D Experimental tailwater when jump sweeps out of pool } downstream from apron
E Experimental minimum tailwater for satisfactory jump } assuming erodible rock
F Experimental tailwater when jump sweeps out of pool } downstream from apron
G Same as F but with downstream sand bar in place.
ELEVATION ABOVE SEA LEVEL
DISCHARGE IN THOUSANDS OF SECOND FEET
A
B
C
D
E
F
G
TAILWATER CURVES
ARECIBO RIVER
AT
DOS BOCAS DAMSITE
DRAWING No 8

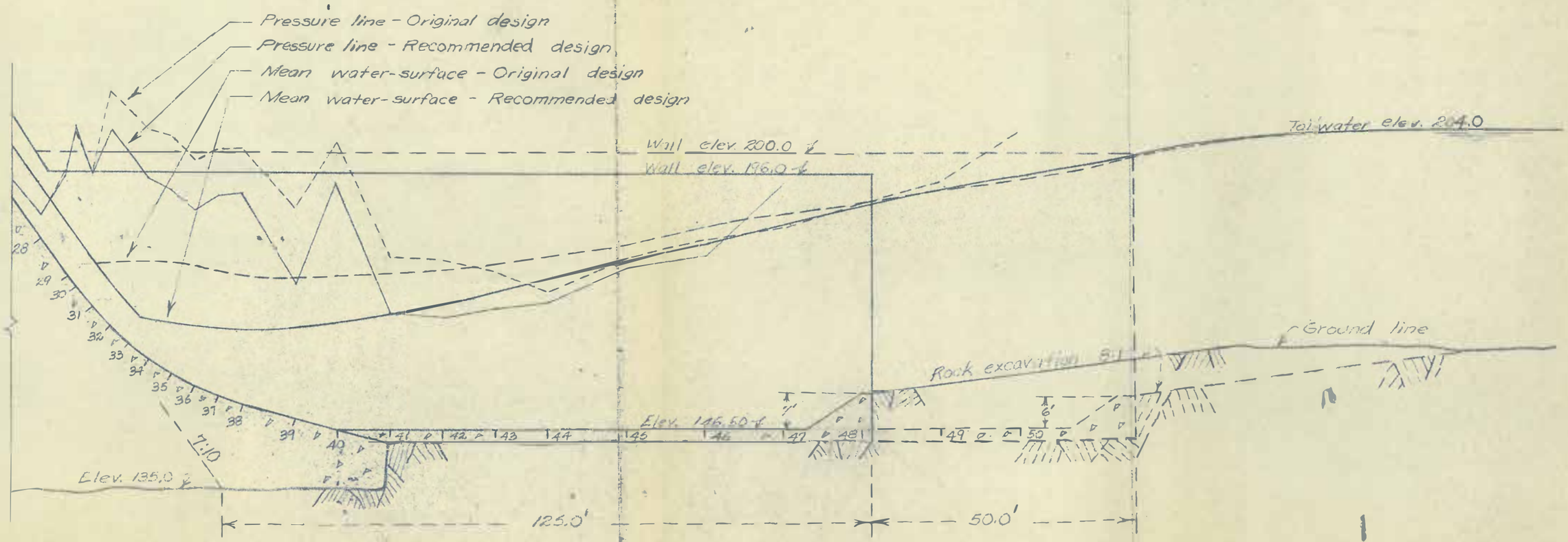
COMPARISON OF ORIGINAL AND RECOMMENDED STILLING BASIN DESIGNS

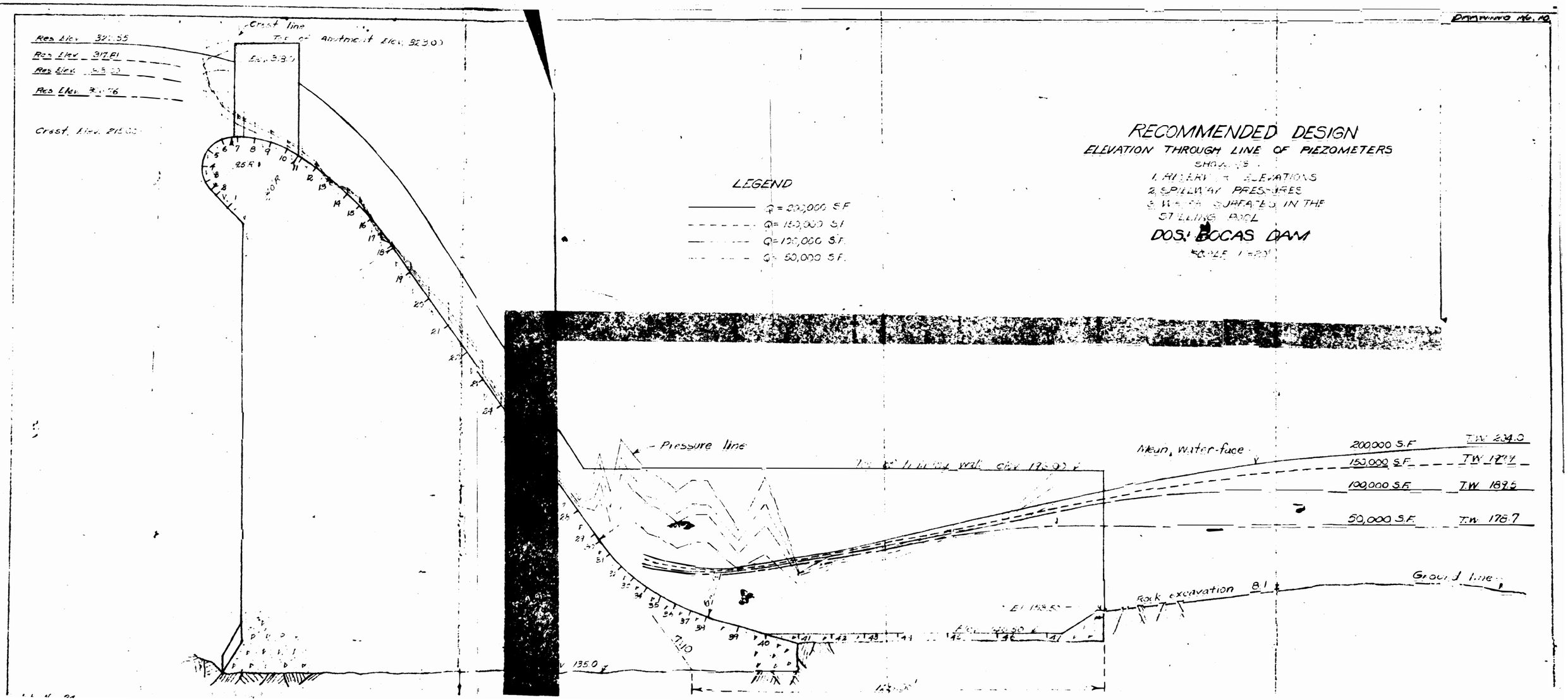
PRESSURE AND MEAN WATER-SURFACE PROFILES ALONG THE CENTER-LINE

Q = 200,000 S.F., TAILWATER ELEVATION 204.0

DOS BOCAS DAM

SCALE 1" = 20'




DRAWING No. 10
RECOMMENDED DESIGN
ELEVATION THROUGH LINE OF PIEZOMETERS
SHOWING:
1. RESERVOIR ELEVATIONS
2. SPILLWAY PRESSURES
3. WATER SURFACES IN THE STILLING POOL
DOS BOCAS DAM
SCALE 1"=20'
LEGEND
Q = 200,000 S.F.
Q = 150,000 S.F.
Q = 100,000 S.F.
Q = 50,000 S.F.
Crest line
Top of Abutment Elev. 323.00
Res. Elev. 321.55
Res. Elev. 317.81
Res. Elev. 313.00
Res. Elev. 307.76
Crest Elev. 312.00
Pressure line
Mean water-face
200,000 S.F. T.W. 204.0
150,000 S.F. T.W. 197.1
100,000 S.F. T.W. 189.5
50,000 S.F. T.W. 178.7
Ground line
Rock excavation
El. 138.50
135.0

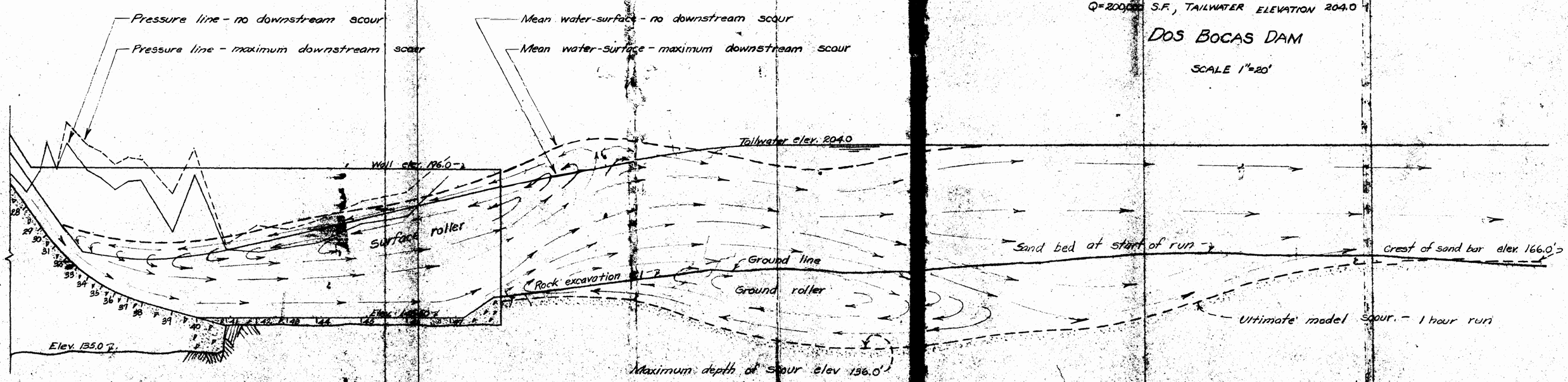
COMPARISON OF STILLING POOL ACTION WITH NO DOWNSTREAM SCOUR AND WITH MAXIMUM DOWNSTREAM SCOUR

ELEVATION ALONG THE CENTER-LINE

Q=200,000 S.F., TAILWATER ELEVATION 204.0

DOS BOCAS DAM

SCALE 1"=20'

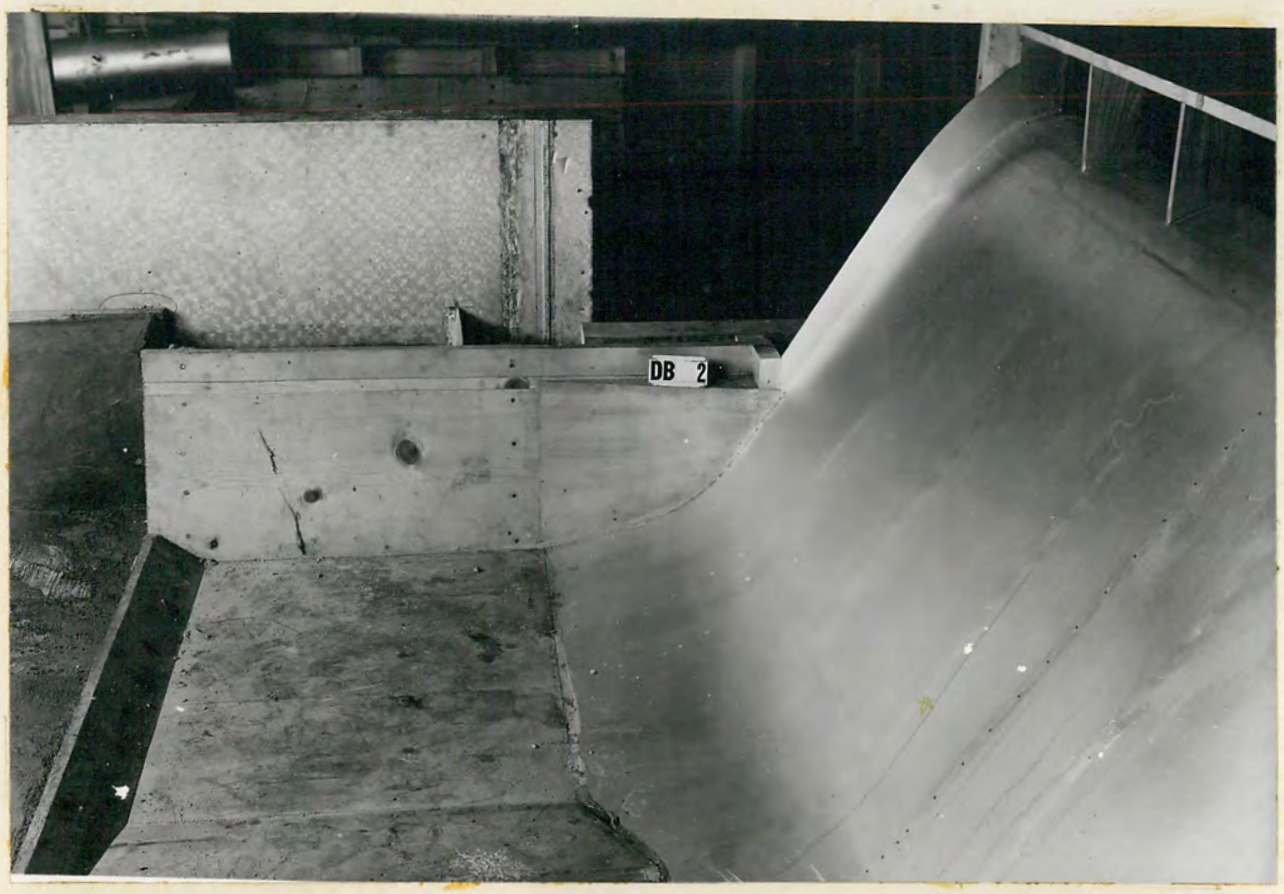


Job 24

2-11-38

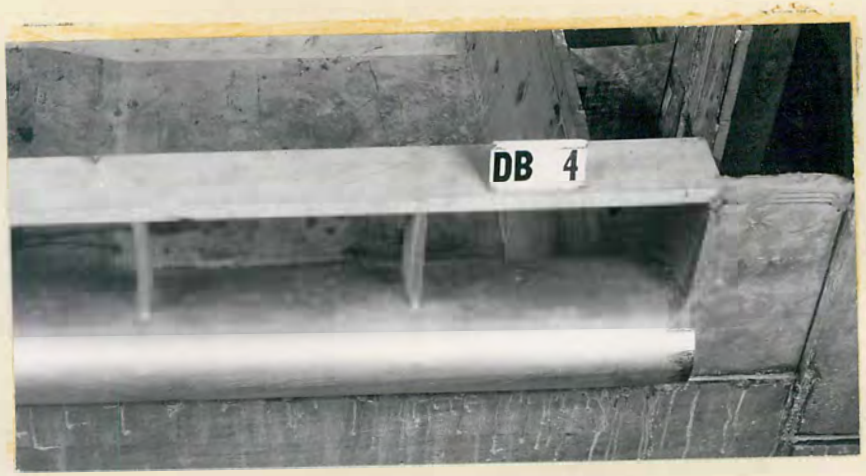
HYD 82

View looking upstream showing model of original design

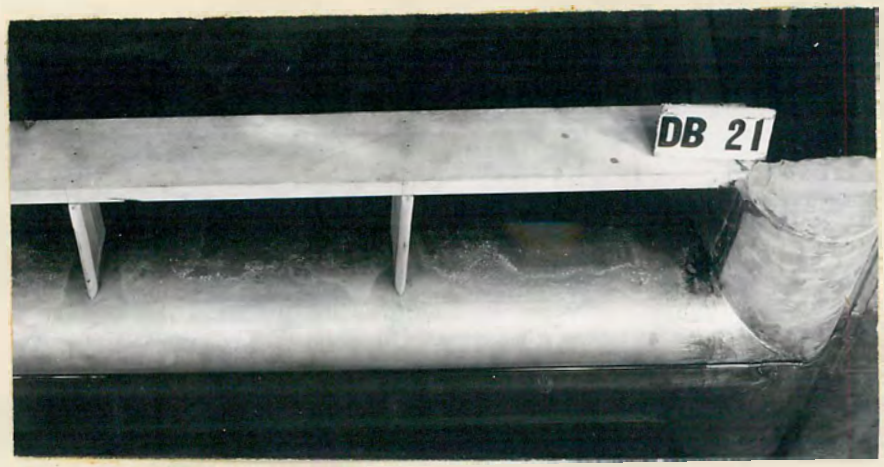


A. Right side view for original design

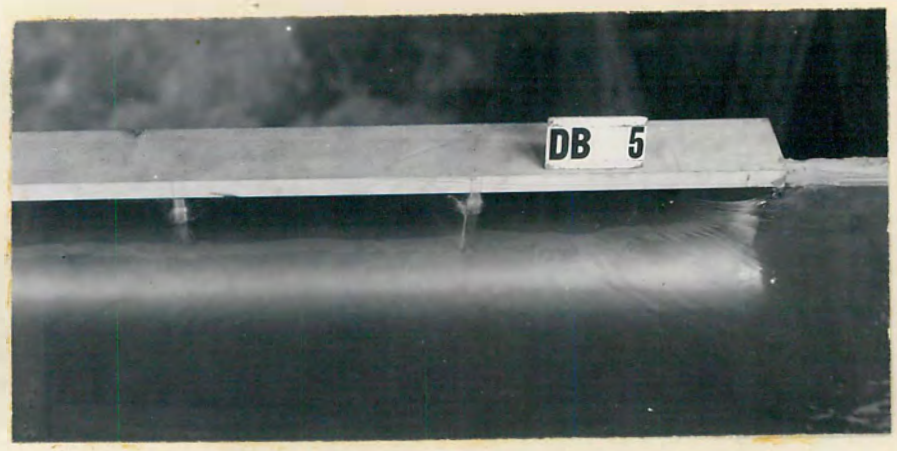
B. Left side view for original design



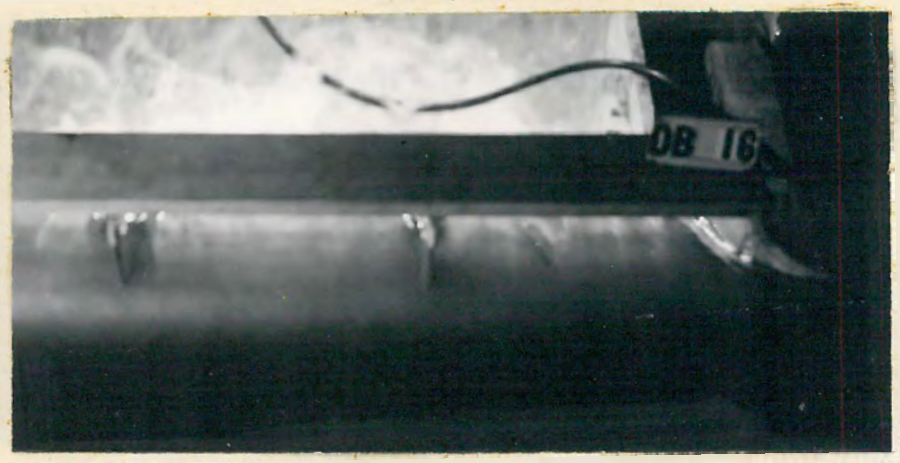
A. Original bridge piers and abutment entrance design



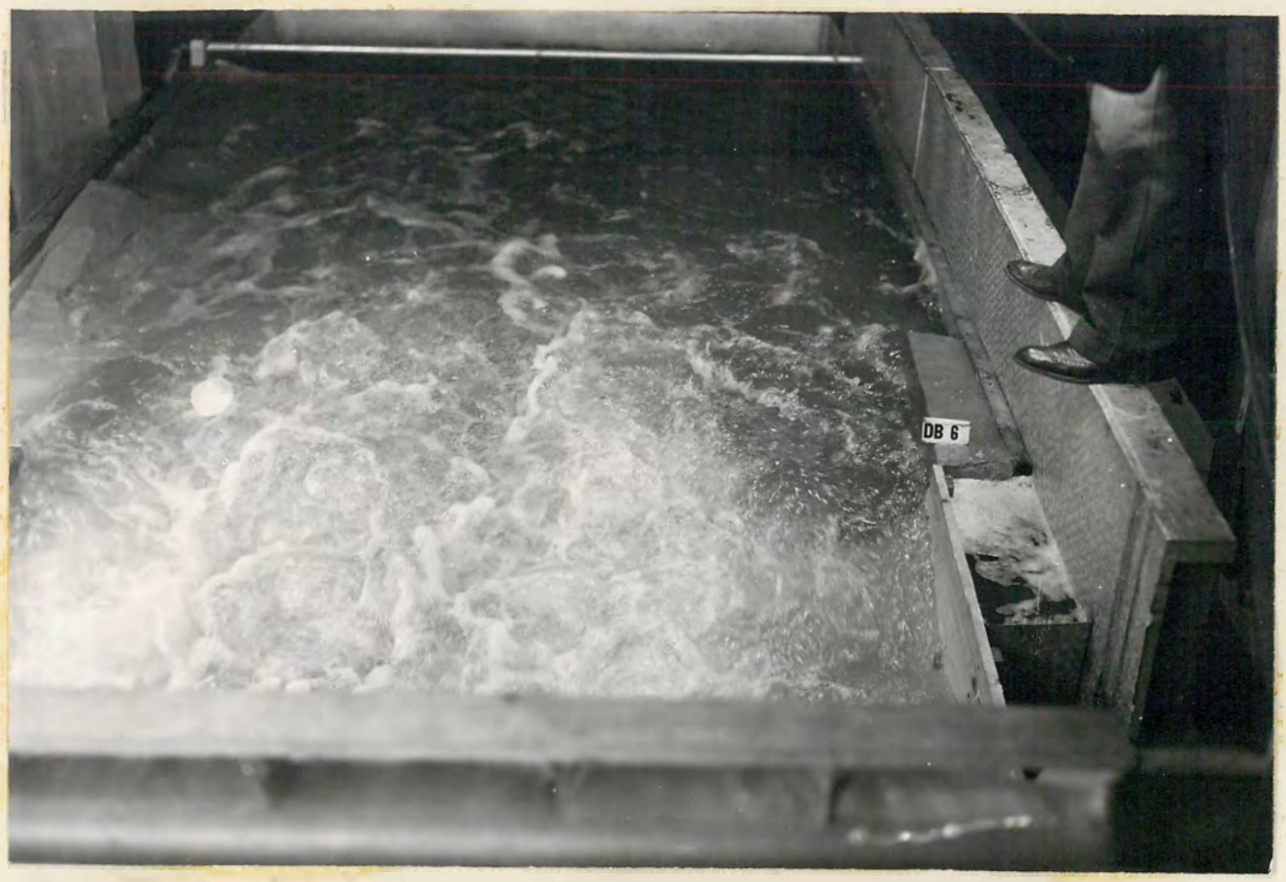
B. Recommended bridge piers and abutment entrance design



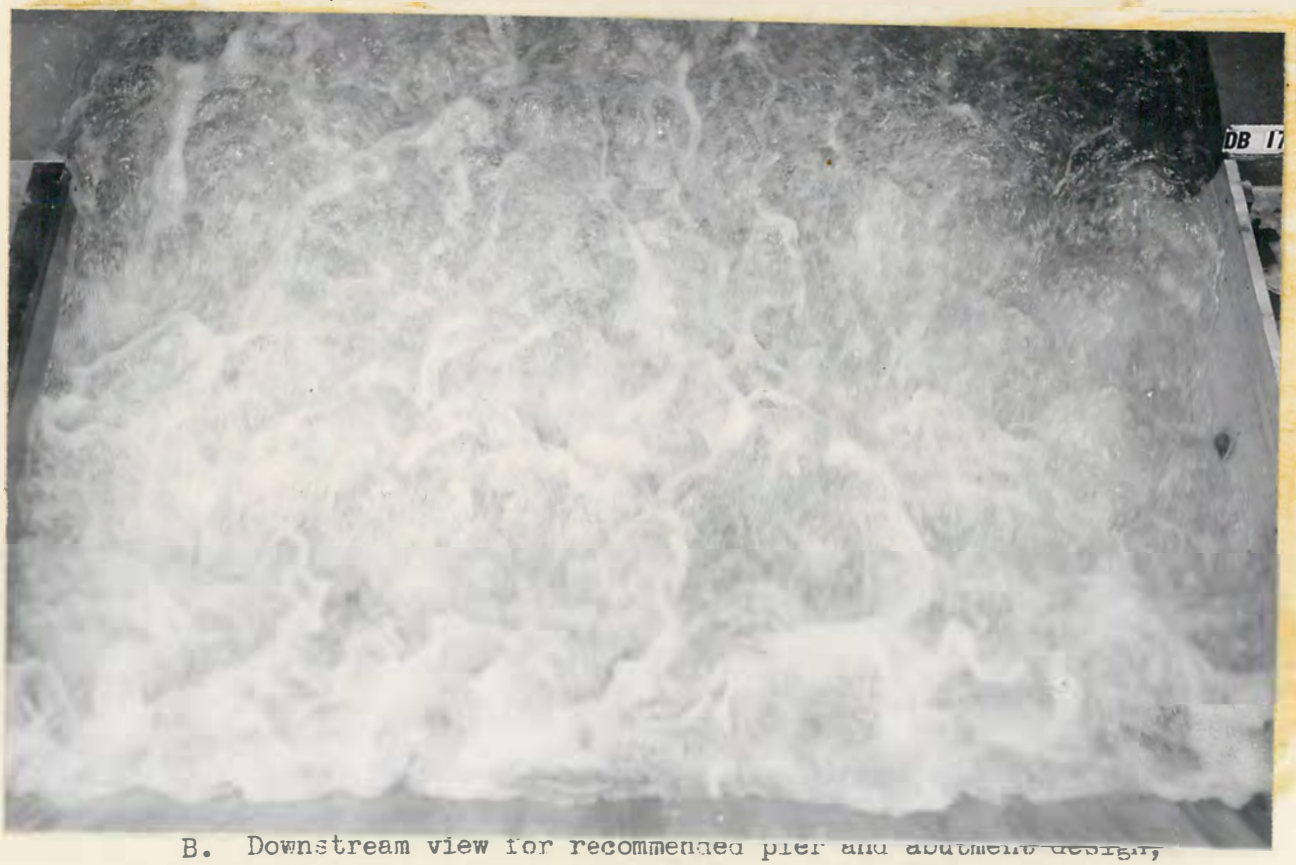
C. Crest conditions for 200,000 second-feet and original design



D. Crest conditions for 200,000 second-feet and recommended design



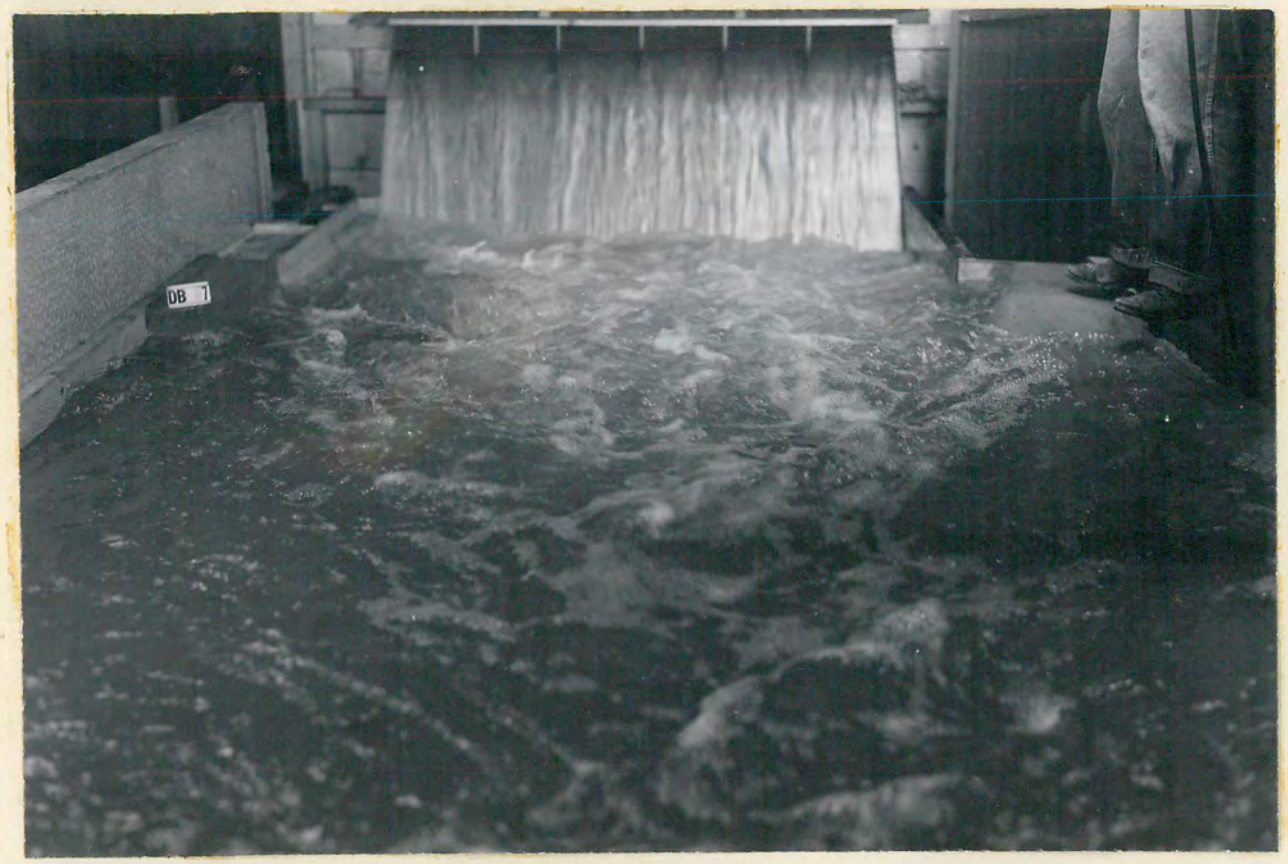
A. Downstream view for original pier and abutment design, 200,000 second-feet and tailwater elevation 204.0



B. Downstream view for recommended pier and abutment design, 200,000 second-feet and tailwater elevation 204.0

A. Right side view for original pier and abutment design, 200,000 second-feet and tailwater elevation 204.0

B. Right side view for recommended pier and abutment design, 200,000 second-feet and tailwater elevation 204.0



A. Upstream view for original pier and abutment design, 200,000 second-feet and tailwater elevation 204.0

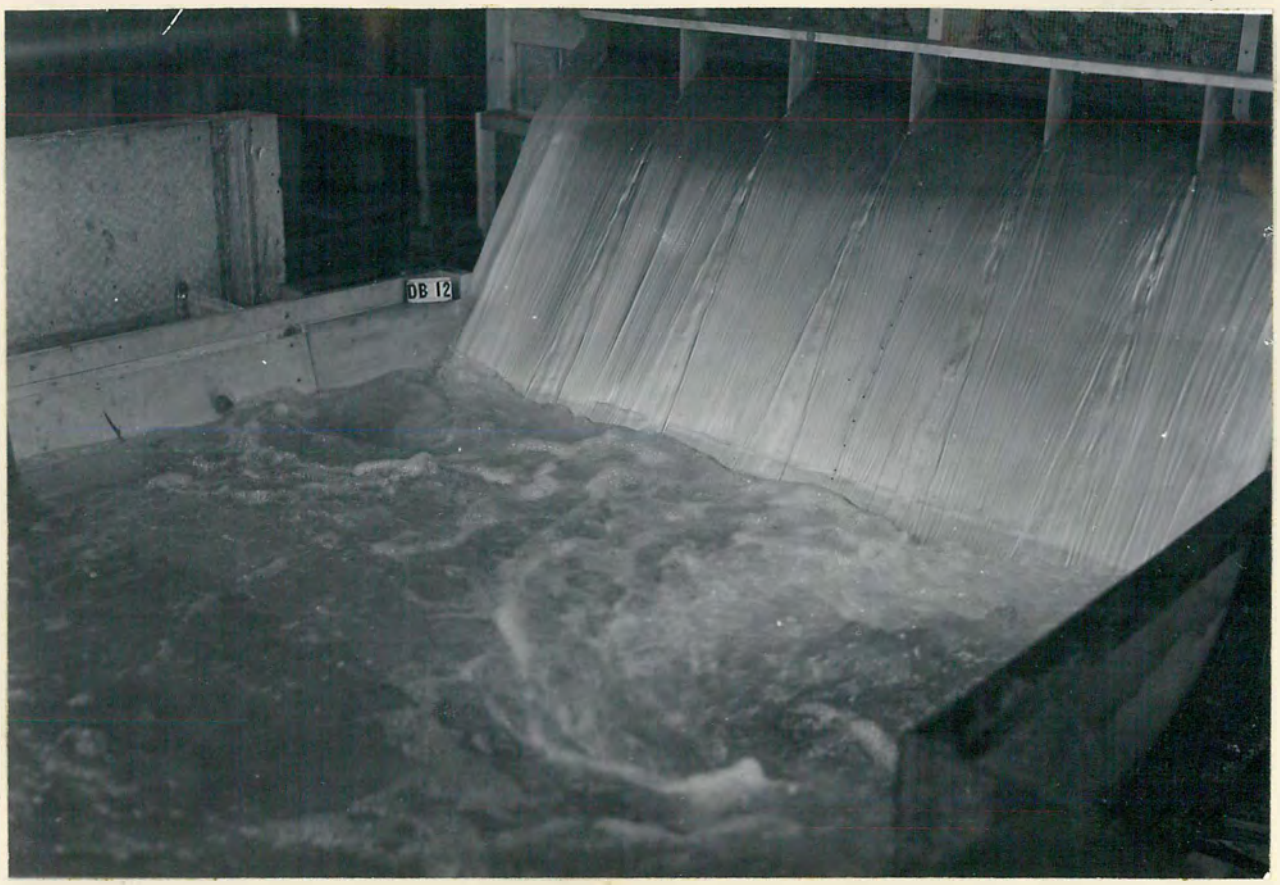
B. Upstream view for recommended pier and abutment design, 200,000 second-feet and tailwater elevation 204.0

A. Right side crest flow for original design and 200,000 second-feet

B. Right side crest flow for recommended design and 200,000 second-feet

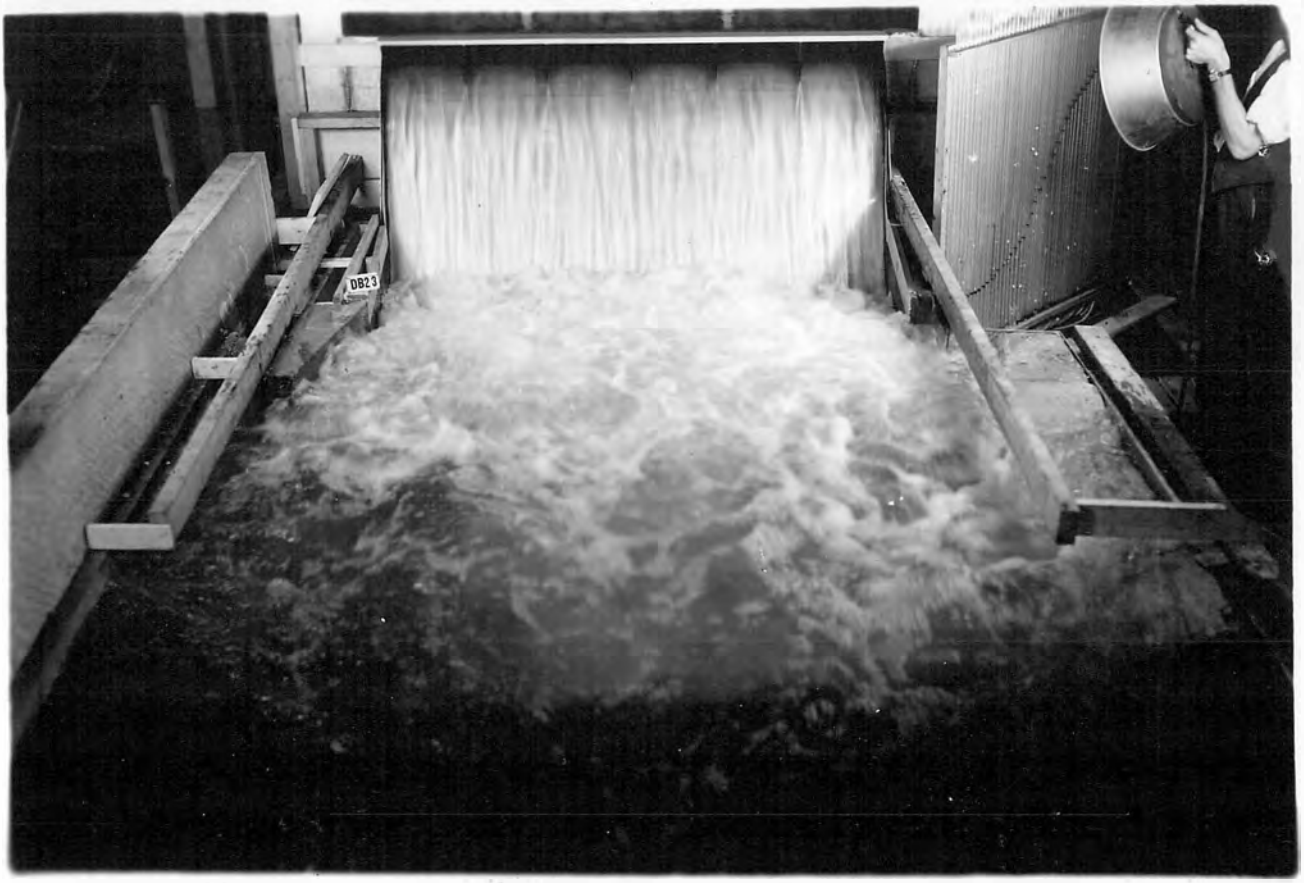

DB 10

A. Right side view for original pier and abutment design, 150,000 second-feet and tailwater elevation 197.7



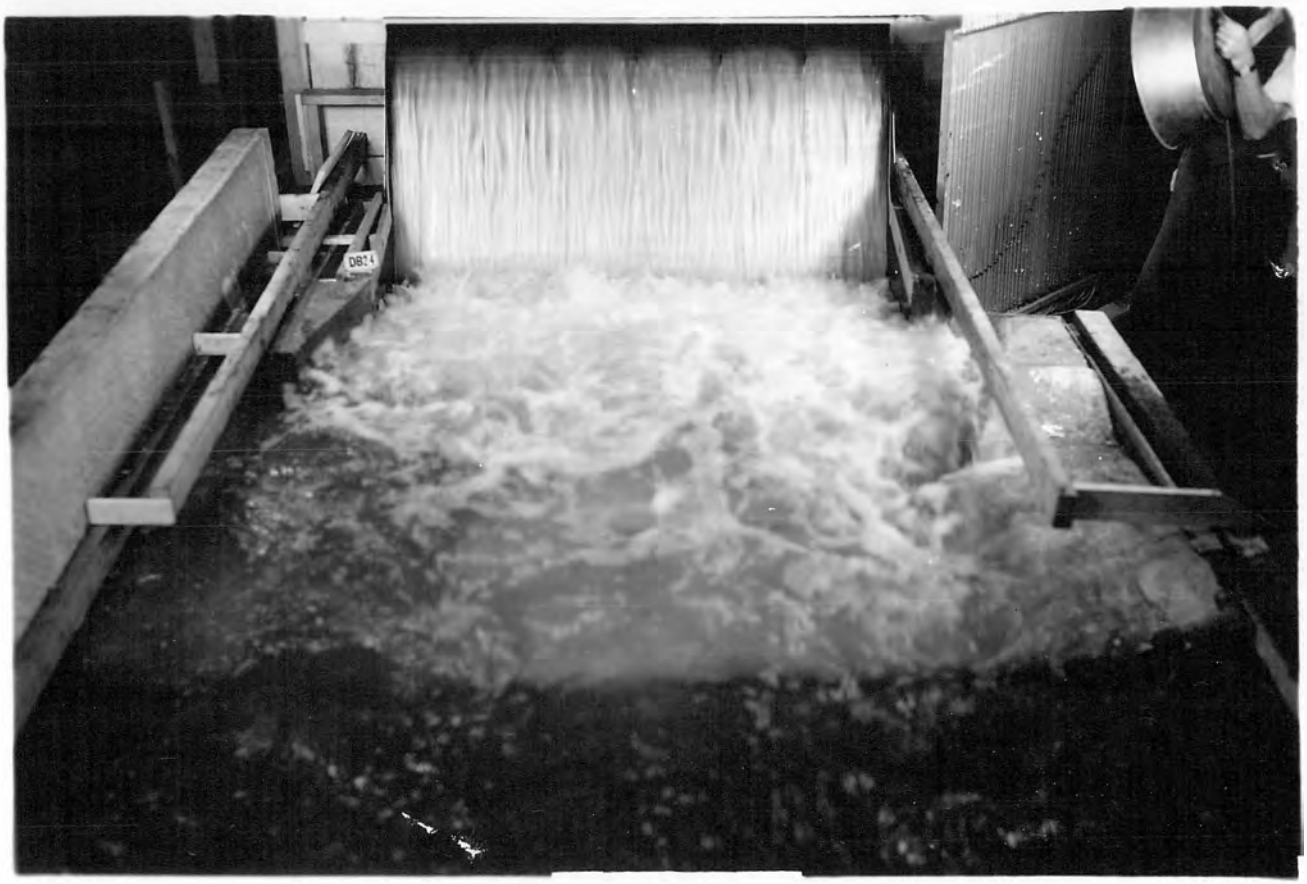
A. Right side view for original design, 50,000 second-feet and tailwater elevation 178.7

pier and abutment design,
ter elevation 197.7



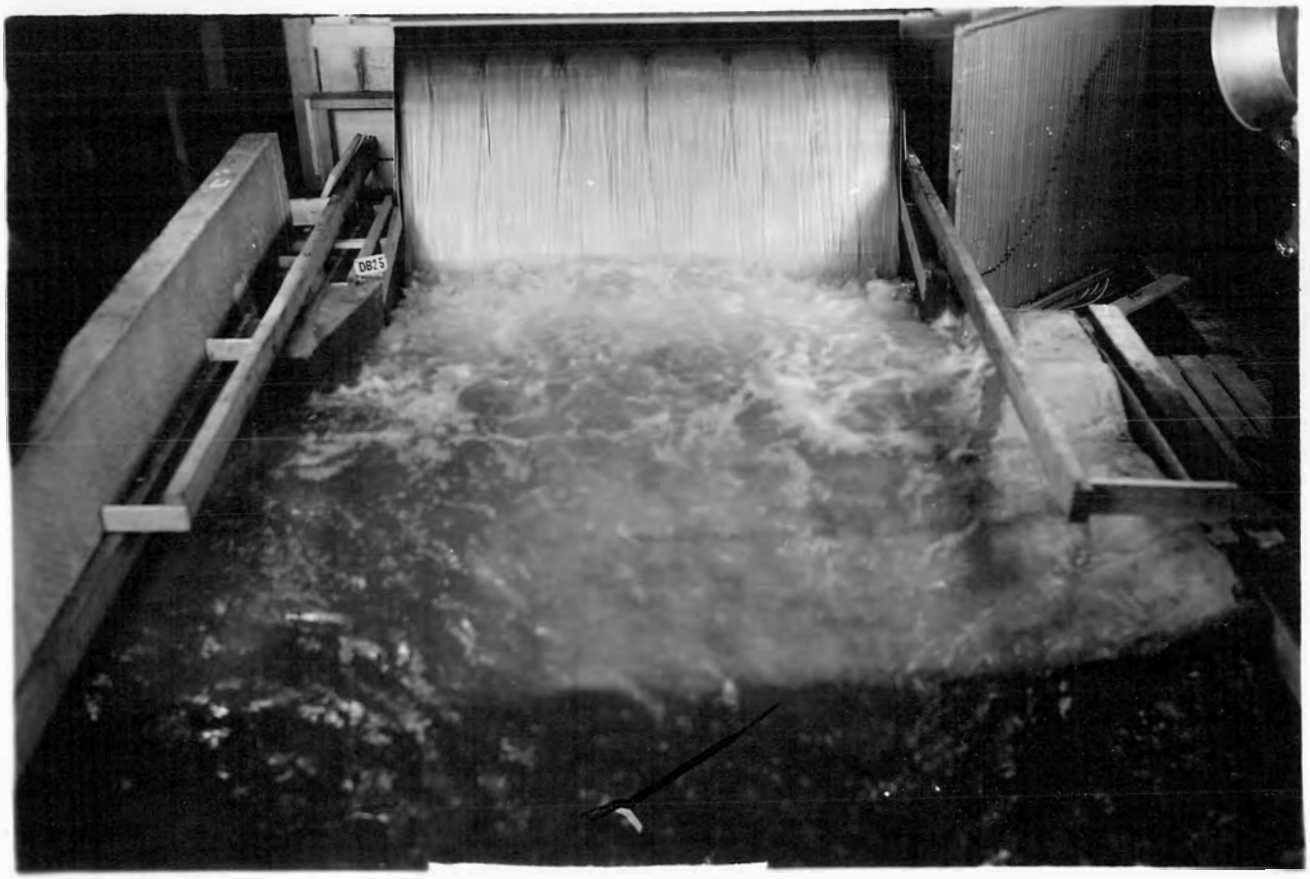
A. RECOMMENDED DESIGN - 200,000 SECOND-FEET AND TAILWATER ELEVATION 204.0

B. RECOMMENDED DESIGN - 200,000 SECOND-FEET AND TAILWATER ELEVATION 204.0



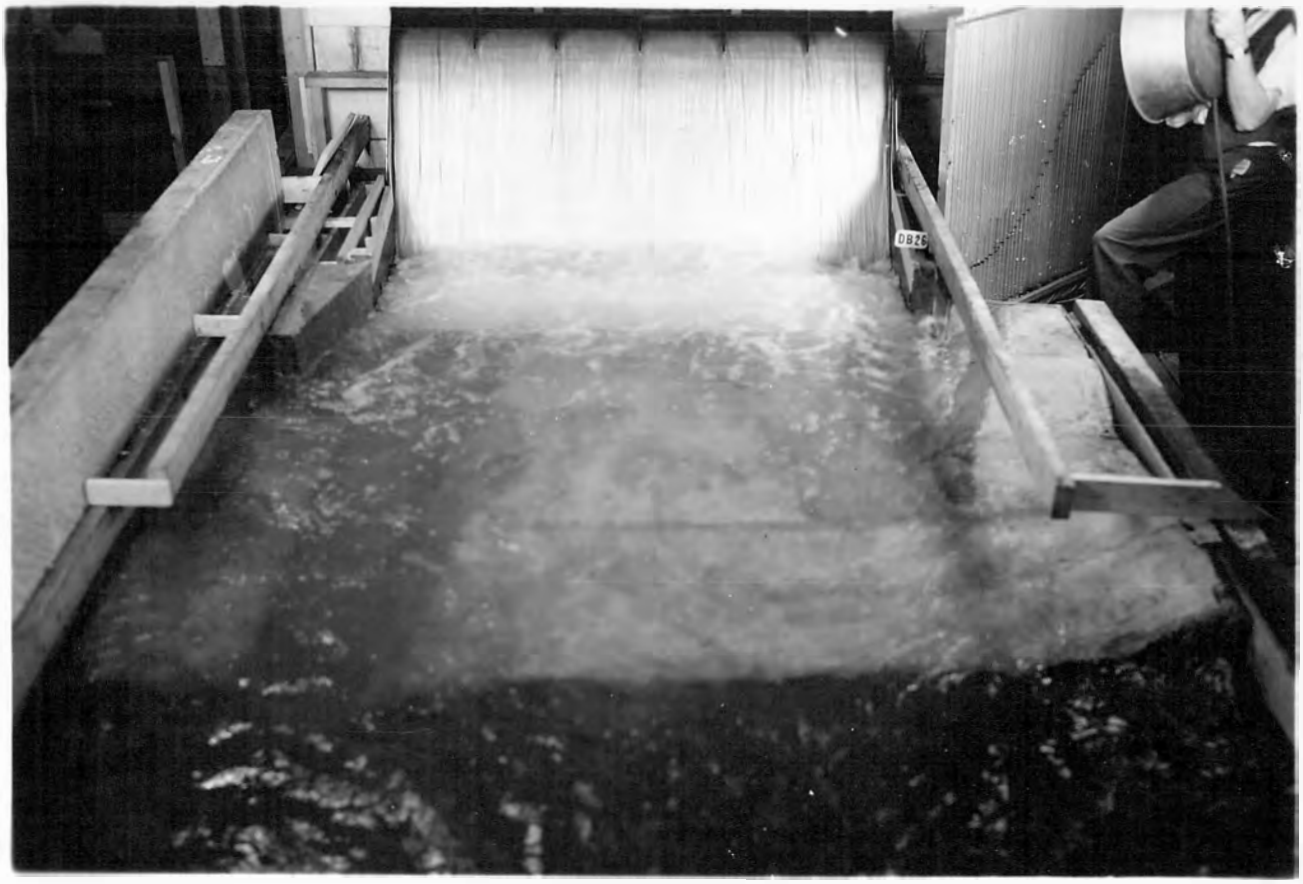
A. RECOMMENDED DESIGN - 150,000 SECOND-FEET AND TAILWATER ELEVATION 197.5

B. RECOMMENDED DESIGN - 150,000 SECOND-FEET AND TAILWATER ELEVATION 197.5

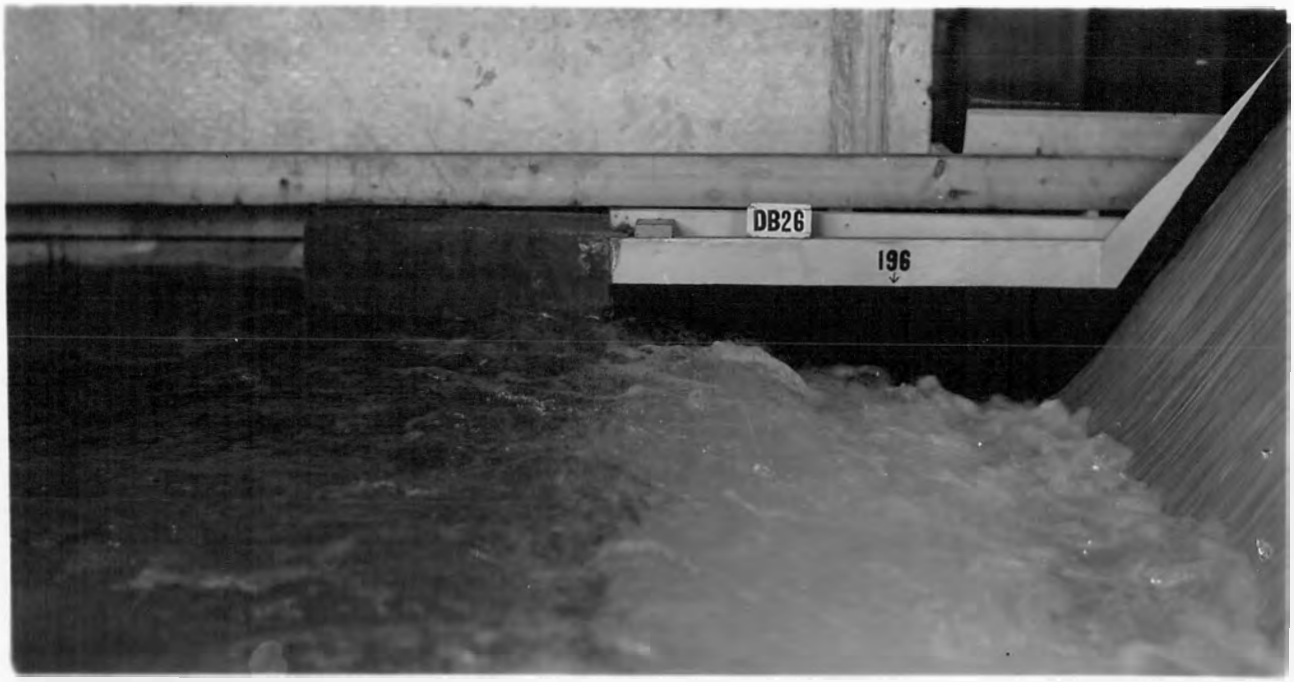


A. RECOMMENDED DESIGN - 100,000 SECOND-FEET AND TAILWATER ELEVATION 189.5.

B. RECOMMENDED DESIGN - 100,000 SECOND-FEET AND TAILWATER ELEVATION 189.5

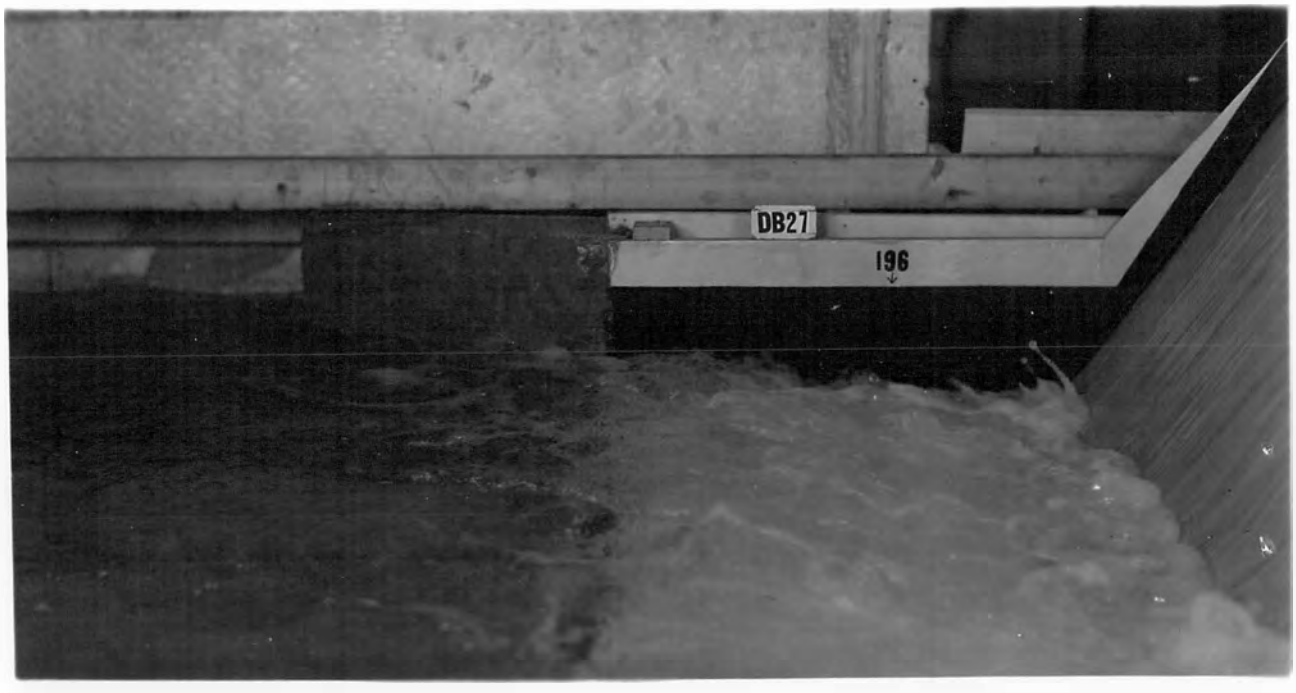


A. RECOMMENDED DESIGN - 75,000 SECOND-FEET AND TAILWATER ELEVATION 184.6.

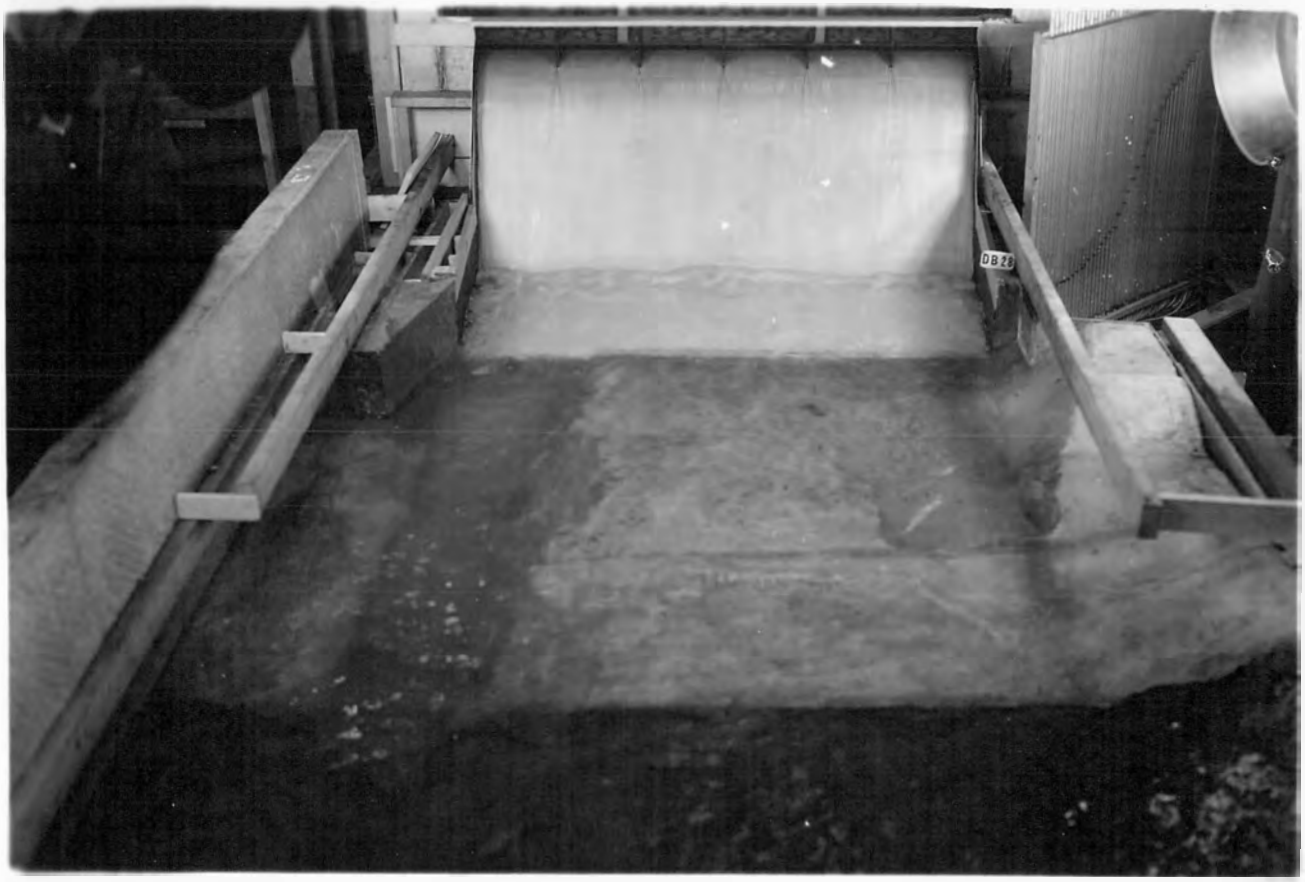


B. RECOMMENDED DESIGN - 75,000 SECOND-FEET AND TAILWATER ELEVATION 184.6.

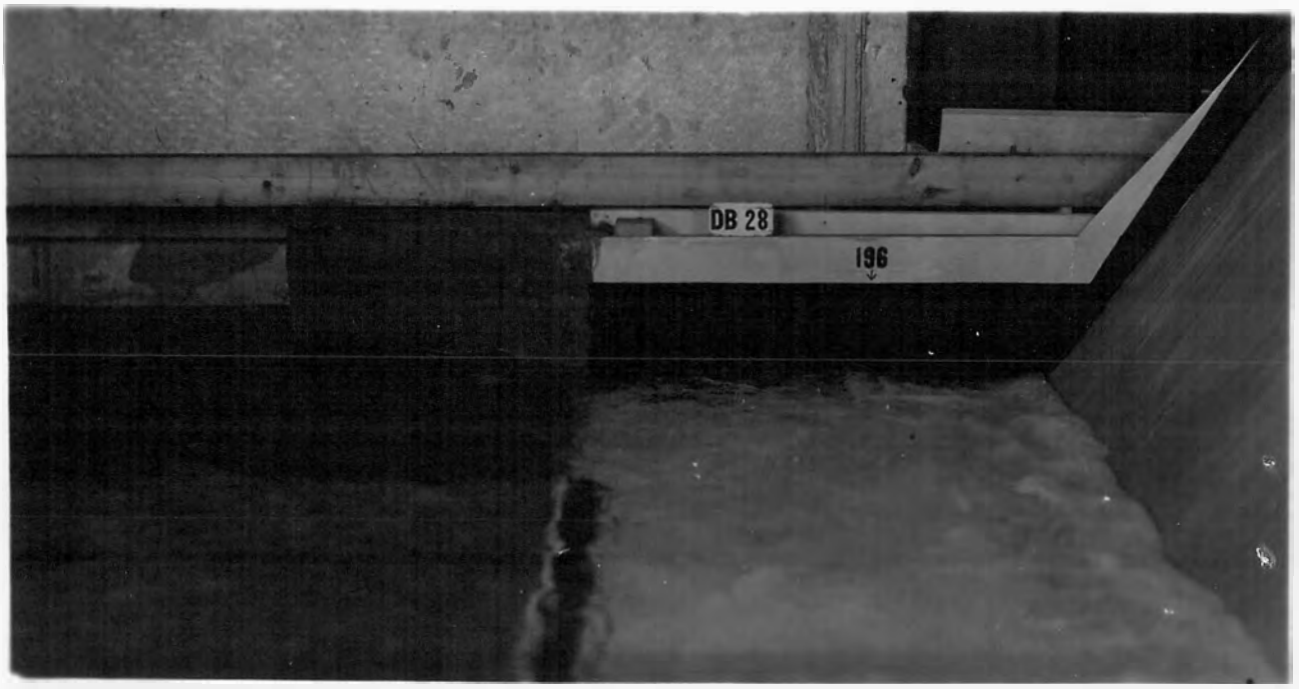
A. RECOMMENDED DESIGN - 50,000 SECOND-FEET AND TAILWATER ELEVATION 178.7.



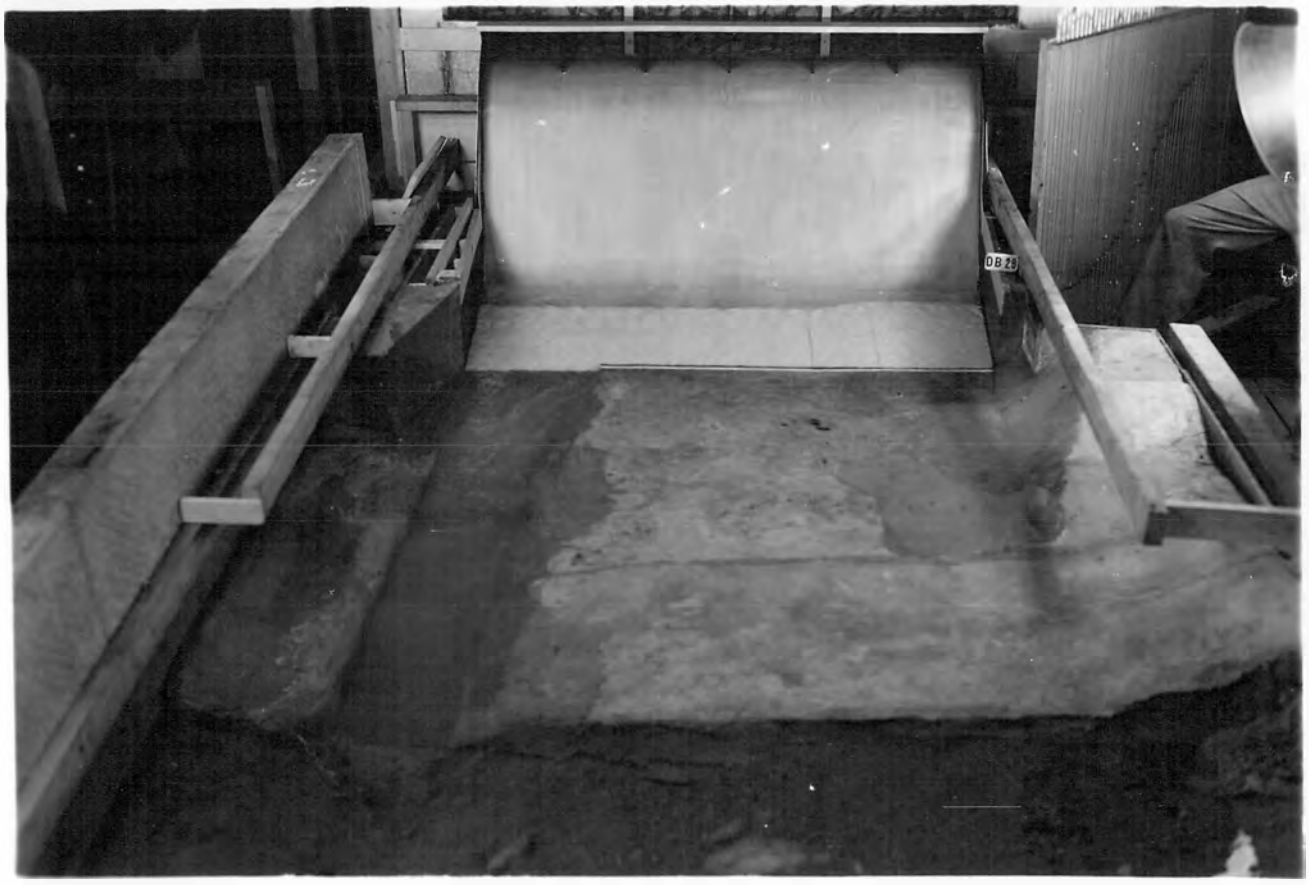
B. RECOMMENDED DESIGN - 50,000 SECOND-FEET AND TAILWATER ELEVATION 178.7.



A. RECOMMENDED DESIGN - 25,000 SECOND-FEET AND TAILWATER ELEVATION 170.5



B. RECOMMENDED DESIGN - 25,000 SECOND-FEET AND TAILWATER ELEVATION 170.5



A. RECOMMENDED DESIGN - 10,000 SECOND-FEET AND TAILWATER ELEVATION 160.5.

B. RECOMMENDED DESIGN - 10,000 SECOND-FEET AND TAILWATER ELEVATION 160.5.

A. RECOMMENDED DESIGN - SCOUR AFTER ONE HOUR RUN - 200,000 SECOND-FEET.

B. RECOMMENDED DESIGN - SCOUR AFTER ONE HOUR RUN - 200.000 SECOND-FEET.

[redacted] DESIGN - SCOUR AFTER ONE HOUR RUN
200,000 SECOND-FEET.