Hyd-301

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

HYDRAULIC MODEL STUDIES OF THE TRENTON DAM SPILLWAY--FRENCHMAN-CAMBRIDGE DIVISION--MISSOURI RIVER BASIN PROJECT

Hydraulic Laboratory Report No. Hyd-301

ENGINEERING LABORATORIES BRANCH

DESIGN AND CONSTRUCTION DIVISION
DENVER, COLORADO

March 7, 1952

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FIGURE LIST

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Division of Design and Construction
Engineering Laboratories Branch
Denver, Colorado
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Laboratory Report No. 301
Hydraulic Laboratory
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Subject: Hydraulic model studies of the Trenton Dam spillway--Frenchman-Cambridge Division--Missouri River Basin Project

PURPOSE OF STUDY

(a) Investigate the flow conditions in the approach channel, the overflow section, the chute, and the stilling basin, and determine the capacity of the Trenton Dam spillway.

(b) Investigate the pressures in and the capacity of the river outlets, which pass through the overflow section of the spillway.

CONCLUSIONS

1. The end pier upstream extension, Design F, Figure 7F, which is elliptical in section between elevation 2787.8 and 2775.0, will cause the least water surface contraction at the ends of the overflow section of the spillway for all discharges through the maximum design capacity of 126,500 cfs, Figure 13.

2. Vortices occurring on both sides of the spillway entrance under the roadway may cause noise but will not have any detrimental effect on spillway operation.

3. Flow conditions were improved in the spillway chute by streamlining the trailing edges of the center piers (Figure 14) but improvement was not sufficient to justify structural changes.

4. Pressures on the overflow section of the recommended spillway will be satisfactory as evidenced by a maximum recorded subatmospheric pressure of 4.5 feet of water (Figure 21).

5. Water will not overtop the spillway chute training walls at flows up to and including the maximum discharge with the wall height shown on Figure 2.

6. The hydraulic jump will form satisfactorily in the stilling basin with the 1 to 42.2 sloping apron.

7. Stilling basin freeboard is sufficient for all discharges including the maximum design discharge.

8. Protection of the riverbed by riprap of 1 cubic foot to 1 cubic yard size will prevent serious erosion without the aid of downstream wing walls (Figure 15).

9. Spillway capacity curves for free discharge and gate control are on Figure 26.

10. Water discharged from the river outlets into the spillway chute will spread and flow evenly to the stilling basin (Figure 22).

11. Pressures on the bellmouth entrance of the river outlets will be above atmospheric for all flows (Figure 21).

12. Capacity curves for the river outlets are on Figure 26.

RECOMMENDATIONS

1. Use End Pier F with an undercut upstream extension and an elliptical section between elevations 2787.8 and 2775.0, for the Trenton Dam spillway (Figure 7F).

2. Use the training wall with the top slopes and elevations shown on Figure 2.

3. Slope the stilling basin floor (1 on 42.2) from elevation 2655.50 at Station 37+75.00 to elevation 2653.0 at Station 38+80.50.

4. Use riprap of 1 cubic foot to 1 cubic yard size immediately downstream from the stilling basin (Figure 2).

5. Discharge equal amounts of water through each of the three radial gates to assure even flow distribution through spillway chute and stilling basin and to prevent overtopping of the training walls and undue erosion of the river channel.

ACKNOWLEDGMENT

The design of the hydraulic features of the Trenton Dam spillway were evolved jointly by engineers of the Dams Branch and the Hydraulic Laboratory of the Engineering Laboratories Branch.

INTRODUCTION

Trenton Dam and Reservoir are located near Trenton, Nebraska, on the Republican River (Figure 1). The reservoir is linked with Enders, Red Willow, and Medicine Creek reservoirs for flood control and irrigation storage. The dam is a compacted earth structure having a length of approximately 9,000 feet and a height of 103 feet above the stream bed.

Water from the reservoir is passed through the spillway at the left abutment of the dam or through the canal outlet works located approximately 6,300 feet to the right of the spillway. Flood discharges are released by three 42-foot wide by 30-foot high float-controlled radial gates over a 126-foot wide spillway crest at elevation 2743, Figure 2. The 126,500 cfs maximum discharge from the overflow section of the spillway will flow through the spillway chute into a concrete-lined stilling basin having a constant width of 266 feet. The 133,000 cfs discharge given in Figure 2 was a preliminary design criteria. Chute blocks 5 feet high by 5 feet wide were placed at the upstream end of the stilling basin, and a dentated sill 9 feet high was used at the downstream end. The floor elevation of the stilling basin was 2653 with a training wall height of 57 feet to provide 9.4 feet of freeboard above normal tailwater elevation of 2700.6 for the maximum design discharge.

Normal river discharges flow through two 6- by 7-foot 6-inch river outlets with an invert elevation of 2710 which pass under the overflow section on the longitudinal center line of the two 8-foot-wide center piers. The river outlet entrance is a rectangular bellmouth 14 feet wide by 14.5 feet high which reduces in 15 feet by elliptical curves to the 6- by 7-foot 6-inch conduit. The flow through the river outlets is controlled by two 6-foot by 7-foot 6-inch high pressure slide gates. The top of the river outlet exit coincides with the downstream end of the center piers at Station 30+54.18 and the bottom blends into the spillway chute 26.5 feet downstream: The two river outlets discharge a maximum of 4,100 cfs which flows through the spillway chute to the stilling basin.

Irrigation water from the reservoir passes to the canal outlet works through a 56-inch pipe. A 4- by 4-foot high pressure slide gate controls the discharge into a 16-foot hexagonally-shaped vertical stilling-well. A report of the hydraulic studies of the canal outlet works is published separately as Hydraulic Laboratory Report No. Hyd. 300.

DESCRIPTION OF MODEL

The hydraulic model of Trenton spillway was built to a scale of 1:54 (Figure 3). The model consisted of a portion of the reservoir, the upstream approach channel, all of the spillway including the stilling basin, and a section of the channel downstream. Topography upstream of the spillway was constructed of concrete mortar and the topography downstream was formed in sand. The spillway crest, chute, and stilling basin apron were formed of mortar using metal templates for profile guides. The radial gates were made of metal and the piers of wood. Water for the model was supplied by a 12-inch centrifugal pump and measured by venturi meters. The flow approaching the crest was uniformly distributed by passing it through a rock baffle. A point gage with vernier graduations to thousandths of a foot was used to measure the reservoir elevation and a staff gage scaled in feet prototype for tail-water elevations. Piezometers were located in both the overflow section and in the river outlets to determine water pressures.

INITIAL OBSERVATIONS AND TEST PROCEDURE

When the model of the preliminary design was placed in operation, the over-all hydraulic action of the spillway was generally good but the overflow section and the stilling basin indicated possible improvement. At the inside of both end piers a water surface depression (pier contraction) occurred which resulted in unequal flow per foot width across the two outside overflow sections. Heavy fins of water formed downstream from each of the two center piers which indicated streamlining to be desirable. The flow through the spillway chute topped the training wall in the vicinity of Station 36+15 (Figure 2), but the efficiency of the stilling basin indicated that the floor could be raised to require less excavation.

As a result of the initial observations and consultations with the designers, it was agreed that the two end pier upstream extensions should be studied for the purpose of increasing the capacity of the overflow section and of obtaining equal lateral distribution of the flow entering the spillway chute. After a solution had been obtained for the overflow section, the stilling basin was studied in regard to depth and to the elimination of erosion occurring at the exit corners. On the completion of the spillway revisions, calibration of the overflow section including the radial gates and river outlets concluded the tests.

INVESTIGATION OF THE SPILLWAY OVERFLOW SECTION

Preliminary Upstream Extension of the Two End Piers--End Pier A

A calibration of the spillway overflow section (Figure 3A) with the two preliminary end piers (End Pier A Figure 7A) showed a maximum flow of 125,500 cubic feet per second at the maximum reservoir elevation of 2785. This was below the preliminary design maximum of 133,000 cfs. Flow contractions at the end piers were responsible for a part of the deficiency in discharge. The flow disturbance for discharges of 50,000, 100,000, and 125,500 cfs, which originates with the contractions and passes through the overflow section of the spillway, is shown in Figures 4, 5, and 6. Water spilled over the walls immediately downstream from the end piers where a fin was formed by the flow contraction (Figure 6A and B). It was apparent that if the flow contractions were reduced or eliminated, the water surface along the pier wall would be lowered and a more uniform flow distribution without overtopping of the walls would be obtained.

The foundation material in the approach channel to the spillway beyond the upstream end of the concrete of the overflow section was not suitable for supporting the weight of either an end-pier extension or a wing wall. Thus, the use of a wing wall for Trenton Dam spillway was not feasible and consideration was given to the use of pier extensions supported by the crest and kept even with the two center piers at Station 29+67.44 (Figure 2). Based on a similar flow distribution problem whose solution resulted in wing walls extending into the reservoir where approach velocities were low (see Report Hyd.-279 on Medicine Creek Dam spillway), above restrictions imposed a more difficult flow distribution problem.

Determination of End Pier Shape

For determining the size and the shape of the end pier extension, end pier A was removed and a series of thin sheet-metal plates were attached in a vertical position to the side-wall of the overflow section (Figure 8A). The flow distribution in the spillway with these plates was similar to that which would occur for the end pier. Metal and modeling clay were used to form the general shape of the flow contraction void as the water flowed past the plate. A series of plates having a vertical leading edge (not shown in Figure 8A) were extended into the reservoir 5, 10, 15, and 22.25 feet measured from Station 29+89.69. This type of end pier was unsatisfactory for improving the flow disturbance.

End Pier B

A study of the flow distribution and conditions in the immediate vicinity of End Pier A and the sheet-metal plates led to the following conclusions: The water entering at the ends of the overflow section of the spillway flowed at right angles to the spillway center line and caused flow contractions because of the abrupt change of direction, and conversion of pressure head to velocity head. If the velocity and pressure reduction were gradual, as is the case for wing walls that are uniformly curved and extended into the reservoir, the flow contraction would be reduced. Application of this knowledge to the Trenton end pier extensions led to an undercutting of the end pier as shown in Figures 7B and 8.

A comparison of Figures 6 and 9, end pier A and end pier B, shows a 500 cfs increase in spillway capacity and some improvement of the flow conditions through the spillway even though end pier B was roughly shaped in metal. The undercutting caused the water acceleration and pressure reduction to occur along the sloping upstream end of the extension instead of occurring at the vertical upstream end of end pier A. A further reduced flow contraction was thought possible by proper shaping and sloping this type of extension. Further tests were made to accomplish this.

End Pier C

Since there was a decrease in the flow contraction for end pier B, a second extension with the outside face at 30^{o} with respect to the spillway center line was constructed of metal (Figure 7C and 10A). Approximate dimensions taken from the metal end pier are shown on Figure 7C. A comparison of the water surfaces on End Pier B (Figure 9) and on End Pier C (Figure 10) shows the decrease in flow contraction and the decrease in water surface height as the flow enters the chute.

Two factors were responsible for this improvement, (1) the 30° angle which decreased the required change of direction for the water to enter the spillway, and (2) the decrease of slope from top to bottom of the end pier. However, End Pier C was not used for Trenton spillway because it was believed that the pier extension could be reduced in size and the outer side could be maintained parallel to the spillway center line.

End Pier D

In determining the shape for the end pier extension, it was decided to make the end pier no more than 8 feet wide and to maintain the outer side parallel to the spillway center line to facilitate construction. Several shapes were formed in metal from which dimensions were taken to shape the wooden End Pier D (Figures 7D and 11A).

The plan view of the end pier at elevation 2787.79 was defined by 1/4 of the ellipse $X^2/(22.25)^2 + Y^2/(7)^2 = 1$, and a 1-foot radius arc which was tangent to the upstream end of the ellipse and to the outer side of the pier. The maximum width of the pier was 8 feet, with 7 feet representing one half the ellipse minor axis. One half the ellipse major axis, 22.25 feet, was the maximum allowable because of the location of the 5-feet wide by 8-feet 8-inch long radial gate float wells in the end piers. The center of the float well is at Station 29+93.40. The section through the end pier was constant from elevation 2787.79 to 2785 where the nose was sloped toward the spillway on a 1 to 1.7 slope to elevation 2754. At elevation 2754, the shape of the end pier was a semicircle of 4-foot radius which was extended down below the floor of the approach channel. Sections through the end pier between elevations 2785 and 2754 were defined by the trace of the ellipse on the spillway side. An arc with a radius, which varied from 1.0 foot at elevation 2785 to 4.0 feet at elevation 2754, was tangent to the ellipse and to the flat, outer side of the end pier. The arc also intersected the 1 to 1.7 slope.

Flow distribution (Figures 11B and C) through the overflow section was improved for all discharges except from approximately 120,000 to 126,000 cfs. Through this range, the flow past the extension became unstable. Water flowing from the reservoir into the spillway did not continuously follow the curve of the end pier. The water was springing clear of the inside face of the end pier for short periods of time and then moved back in contact, which would cause vibration. The primary source of this instability was found to be the 1 to 1.7 slope of the end pier extension. With the upper end of the sloped portion of the pier at elevation 2785, the water change of direction was still too abrupt on the upper portion of the sloped section. The upper end of the slope of the end pier was lowered in elevation by using modeling clay, to begin at elevation 2775 and end at 2745, which kept the slope approximately the same as on end pier D but at a different elevation. The flow past this revised extension was stable.

Streamlining of Center Pier Trailing Edge

Impingement of the water flowing through adjacent sections of the overflow portion of the spillway resulted in fins of water downstream of the two center piers (Figure 13C). Because of structural considerations, the downstream edges of the piers could not be readily streamlined. As noted from Figures 2 and 3, the center piers bridge the river outlets, and any tapering of the downstream end of the piers would require added reinforcement in the top of the outlet conduits. Moreover, the radial gate counterbalances with guides made of steel I-beams operate up and down the ends of the piers and tapering of the pier ends would make it more difficult to attach the steel I-beam guides.

Nevertheless, reshaping of the downstream ends of the piers was studied to determine the influence on the flow distribution in the spillway chute. Some improvement of the distribution immediately below the overflow section was obtained by tapering the downstream ends of the piers to 1 foot, Figure 14A and B. The fins at the ends of the tapered piers were approximately one-eight the width of those with square pier ends, although rising to the same height. A second fin occurring at Station 13+95 in the chute downstream was thin although at times rising above the top of the training wall. Flow distribution downstream of the disturbance caused by the fins was essentially the same for both the square and the tapered pier ends. Slightly more wave action at the training walls throughout the spillway chute was noted for the tapered ends. Because no outstanding improvement was accomplished with the tapered pier ends and they introduced structural problems, the designers decided in favor of the pier without the taper.

INVESTIGATION OF SPILLWAY CHUTE AND STILLING BASIN

Spillway Chute

Satisfactory flow conditions were established through the overflow section by using the recommended end pier, design F, Figure 7. The flow was nearly equal per foot width within the spillway chute except for the fins that formed downstream of the center piers. There was slight evidence of the water overtopping the training walls of the model which had not been increased in height. With a 5-foot increase in training wall height at Station 36+15.00, near the point of intersection of the chute slope and the vertical curve that enters the stilling basin (Figure 2), no overtopping will occur. A decision was made by the designers to increase the height of the spillway training wall at this station, from elevation 2718.64 to elevation 2724.04. This change in the training wall height decreased the slope of the training wall top from Station 31+65.00 to Station 36+15.00 and increased the training wall slope from the latter station to the stilling basin, Station 37+75.00.

Spillway Stilling Basin

Preliminary inspection of the flow conditions for various discharges into the stilling basin showed satisfactory operation. Formation of the jump started well up on the sloping floor to the basin and the chute blocks remained covered for all discharges. At maximum discharge the jump used the full length of the basin, with the surface roughness caused by the jump extending approximately 30 feet downstream. Eddies were formed on both sides of the basin exit near the 2 to 1 sloped bank and the retaining wall. A maximum water velocity of approximately 7 feet per second was attained by these eddies at a point 5 feet downstream of the retaining wall and at elevation 2660, which was approximately the top of the dentated sill.

Erosion of the Spillway Exit Channel with Sand and Riprap

Specifications for Trenton spillway require riprap for a distance of 110 feet downstream of the stilling basin and includes the 2 to 1 slopes to elevations above maximum tail-water elevation, 2700, on both sides of the basin exit. The riprap blanket is 3 feet thick and composed of rock varying from 1 cubic foot to 1 cubic yard in size. The model channel was formed according to Figure 2 in sand representing an erodible material with a 5/8-inch and smaller screened gravel for riprap (Figure 15A). Operation of the model for 4 hours at a maximum discharge of 126, 500 cfs and at the maximum tail-water elevation, 2700. 6, resulted in the erosion pattern of Figure 15B. An erosion depth of 2 feet occurred on the right side of the basin near the training wall but without seriously disturbing the gravel. A general downstream movement of the gravel occurred across the channel width, but some gravel was moved upstream by the ground eddy and deposited on the downstream side of the dentated sill. Riprap on the 2 to 1 slopes adjacent to the retaining walls (at the end and at right angles to the spillway training walls) was not moved except that at the downstream edge of the blanket on the left side of the basin. The riprap material in this area was moved upstream toward the erosion pocket by the eddy. The eddy tended to carry gravel down the slope opposite the end of the training wall. The extent and depth of the riprap blanket was considered sufficient, and the design was satisfactory. For comparison the same test was made using only sand as a riverbed material.

Erosion of the Spillway Exit Channel with Sand

Channel contours were re-formed in sand after screening out the majority of the gravel. Two erosion tests were made with the use of sand, one for 2 hours (Figure 16A) and one for 4 hours (Figure 16B) at 126, 500 cfs and a tail-water elevation of 2700. 6. The erosion pattern formed at the end of 2 hours showed a maximum erosion depth of 9 feet which progressed to a depth of 13 feet at the end of 4 hours, both depths referred to the elevation of the basin floor 2653. The eddy velocity was great enough to erode away the 2 to 1 side slopes (sand) in both the 2 and 4 hour tests. Without the use of riprap this amount of erosion was considered unsatisfactory for safety of the structure.

Wing Wall to Reduce Erosion

In the past, several ways have been determined to reduce the type of erosion described in the previous paragraph. The best way when feasible, is to gradually expand the stilling basin width by means of wing walls downstream of the sill. These walls have proved costly in spillways the size of Trenton when they must be 50 to 60 feet long and must be of the same height as the basin training wall. Therefore, a relatively small wing wall similar to a buttress was set at right angles to the retaining wall (Figure 17). The position selected in Trenton was 11 feet 6 inches from the stilling basin corner. A test was made with the wing wall extended downstream 20 feet which was the convenient length to permit the wall to rest on the same footings as the retaining wall. The wing wall reduced the erosion at the corners of the basin but it did little to protect the slope. The maximum depth of erosion at 126,500 cfs and a tailwater elevation of 2700.6 was 8 feet, located at the end of the downstream wing wall (Figure 17A). Sand completely covered the cut-off wall at the end of the basin, and with the exception of the extreme corners sand covered the downstream side of the sill. The 2 to 1 slopes near the retaining walls were eroded to approximately 5 to 1. A second wall of 15-foot length was added at a distance of 24 feet 6 inches from the basin corner without material improvement to the erosion pattern (Figure 17B), although the depth of erosion was reduced from 8 to 5 feet. The buttress wing wall reduced the amount of riprap required since only the slopes need be covered but the additional cost of the wall for Trenton was not considered warranted. Thereforé, a spillway exit channel protected by 1-cubic-foot to 1-cubic-yard riprap and with retaining walls at right angles to the spillway center line was recommended (Figure 2).

Stilling Basin Velocity Measurements

Concurrent with the sand erosion tests, maximum velocities were measured at several locations in the stilling basin and spillway channel and plotted in Figure 18. The velocity profile at the end of the basin was good with the highest velocities near each training wall. The velocities across the basin exit were sufficiently equal that no recommendations were made in reshaping of the spillway chute training wall.

Sloping of Stilling Basin Floor

The preliminary stilling basin was designed for a horizontal floor at elevation 2653. To withstand anticipated uplift pressures under the stilling basin floor, the designers increased the thickness of the concrete to approximately 2.5 feet at Station 37+75 (upstream end). This was done by incorporating a 42.2 to 1 slope up from elevation 2653 at the dentated sill to the intersection of the basin floor with the 3 to 1 slope of the spillway chute (Figure 19B). Triangular fillets 5 feet high, 15 feet wide and full length of the basin were added to the basin training wall and floor junction. An erosion test made with this change showed no appreciable differences with the horizontal floor (Figure 16A). Sand was used in this test because it is more easily moved by the water and therefore differences would be more readily apparent than by using riprap. Since good flow conditions were obtained, the stilling basin was considered satisfactory and was recommended for Trenton spillway (Figure 2).

RECOMMENDED SPILLWAY AND APPURTENANCES

Flow Condition in Approach Channel and Overflow Section

With the completion of the stilling basin studies, the Trenton spillway was considered to be hydraulically satisfactory (Figure 19). Further studies were made to record operational characteristics, such as water-surface profiles, pressures on the overflow section, and flow capacity of the spillway.

Figure 20 shows the water surface profiles on the pier walls of the recommended overflow section (Figure 19A). With the recommended End Pier F, nearly equal flow per foot across the overflow section existed with water surface contractions at the end piers only slightly greater than those on the center piers. These contractions were acceptable.

Overflow Section Pressures

Pressures on the overflow section were recorded at the maximum reservoir elevation, 2785, at gate openings of 5, 10, 15 feet and full gate opening (Figure 21). Slight subatmospheric pressures were found for all flows. The maximum was minus 4.5 feet of water.

River Outlets Through Overflow Section

The previously described river outlets could not be studied in detail because of the small scale of the Trenton model. Since no operation difficulties were anticipated up to and including the design head corresponding to reservoir elevation 2773, which is the elevation of the top of the radial gates when they are closed, a larger model was not considered necessary. With the outlets closed at the entrances by metal plates, water flowed from the gate air vents on the downstream side of the two center piers at maximum discharge and reservoir elevation 2785 (Figure 11C). The flow was from water entrapped under pressure within the outlet. The pressure source was apparently a pressure rise which accompanied the turning of water on the curve of the overflow section between Station 30+54.18 and Station 30+80.97. This condition will probably occur without detrimental effect for maximum discharge with the outlet gates shut in the full size structure.

Outlet Flow Distribution on Spillway Chute

Outlet flow distribution across the spillway chute for reservoir elevation 2720, and a discharge of 875 cfs, is shown in Figure 22A, and for reservoir elevation 2752 and a discharge of 3,570 cfs is shown in Figure 22B. The distribution was considered satisfactory.

Outline Capacity and Pressures

Capacity of the two river outlets has been plotted in Figure 26 and pressures occurring in the bellmouth entrance have been plotted in Figure 21. The model outlets were constructed without control gates which limited the calibration to full open gates at various heads. The maximum discharge for the two outlets is 4,280 cfs at reservoir elevation 2773. Pressures in the bellmouth were positive and only slightly negative in the outlet downstream of the slide gates for the two reservoir elevations of 2773 and 2743 which cover the upper range of operation.

Pressures at Float-Well Intakes

For radial-gate operation at Trenton Dam, water is supplied from the reservoir to six float wells located in the four piers. Rate of water flow to the wells is dependent upon the pressure at the well intakes. With the rate of gate opening proportioned to the rate of reservoir rise, the relationship of the pressure at the intakes to the reservoir elevation is important. It is desired that the pressure head at the intakes correspond to reservoir elevation. The intakes should be located where velocity of flow does not reduce the intake pressure below reservoir head. Pressures were obtained for two intake proposals:

1. Two intakes located on the outsides of the end piers at Station 30+01.00 and one intake on the right center pier at Station 20+67.44, all at elevation 2766.36. These intakes were to supply water to all float wells.

2. One intake on each of the center lines of the right and left center piers, and one five feet to the left of the center line of the right center pier, all at elevation 2728 and over the entrances to the river outlets at Station 29+67.44. All of these intakes were protected by the river-outlet trashracks and supply water to all float wells.

For Proposal (1), as the radial gate opening was increased, the reservoir elevation indicated by piezometers located at the float-well intake positions deviated lineally from the actual reservoir elevations. This deviation varied from 0 at reservoir elevation 2773 and the radial gates closed to 0.5 foot of water at reservoir elevation 2778 and the gates 25 feet open. This was considered acceptable; however Proposal (1) was considered unsatisfactory because debris could collect under the roadway, cover the intakes and this may cause improper regulation of the radial gates. Also, wave action with the reservoir at gate top elevation 2773 could cause a pressure fluctuation at the intake elevation 2766.36 that could open the gates and waste water

For Proposal (2), the deviation of indicated reservoir elevation to that of the actual reservoir elevation varied from 0 with the gates closed to 0.1 foot at a 20-foot radial gate opening. With the intakes protected by the river outlet trashracks and at an elevation low enough to eliminate wave effect, Proposal (2) was considered satisfactory. It was planned that the river outlets will be closed while the radial gates are operated; thus flow through the outlets does not enter the problem.

Flow Conditions Through Spillway

Figures 23 and 24 show satisfactory flow conditions in the spillway chute and stilling basin for discharges ranging from 25,000 cfs to the maximum of 126,500 cfs.. Normal control of the flood discharge will be maintained by the float-controlled radial gates. Entrance and exit flow distribution at the overflow section was shown in Figure 13B and C for the maximum discharge of 126,500 cfs. With radial gate control of discharges of 25,000, 50,000, and 100,000 cfs, entrance conditions to the overflow section were tranquil except for the normal vortices that formed at the junction of the gates and piers. A discharge of 160,000 cfs, 26 percent over design maximum, was passed through the spillway. Flow distribution at this discharge evidenced no serious overtopping of training walls. The jump remained in the stilling basin, moving to the middle of the basin only when the tail water was lowered 5 to 6 feet below elevation 2702, which is approximately the tail-water elevation corresponding to a discharge of 160,000 cfs.

Water-Surface Profiles in Stilling Basin

Maximum water-surface profiles in the stilling basin for a discharge of 126,500 cfs and tail-water elevation 2700.6 have been plotted in Figure 25. The minimum freeboard on the basin walls is 0.5 feet.

Spillway Capacity

The discharge capacity curves of Figure 26 are for the recommended spillway design shown in Figure 2. Figure 26 includes capacity curves for the radial gates at various openings. The capacity of the spillway exclusive of the river outlets with the reservoir at elevation 2785 was 126,500 cfs, the designed value. The capacity of the spillway including river outlets with reservoir at elevation 2785 was approximately 130,000 cfs.

Interior - Reclamation - Denver, Colo.

FIGURE 1.
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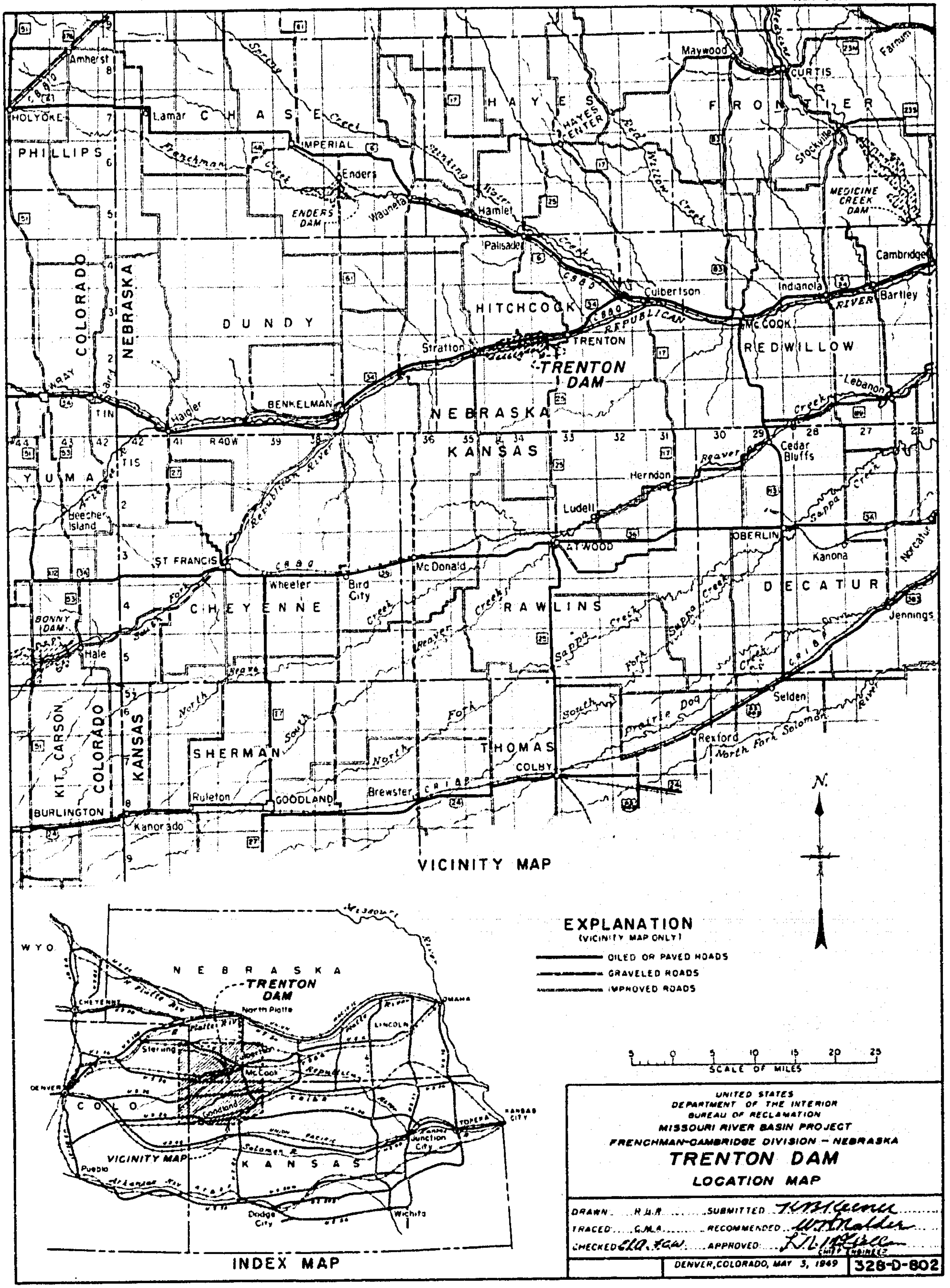
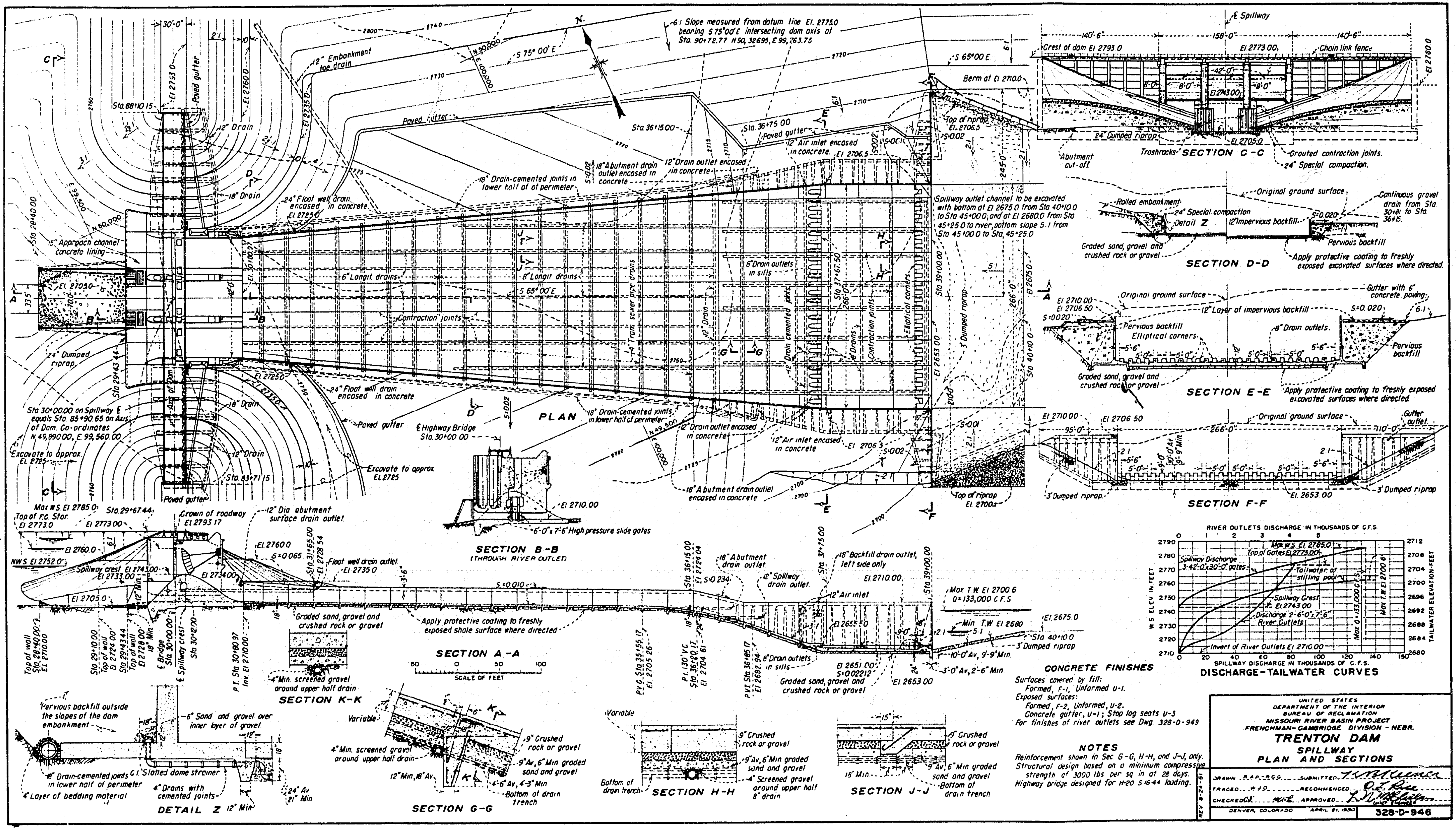
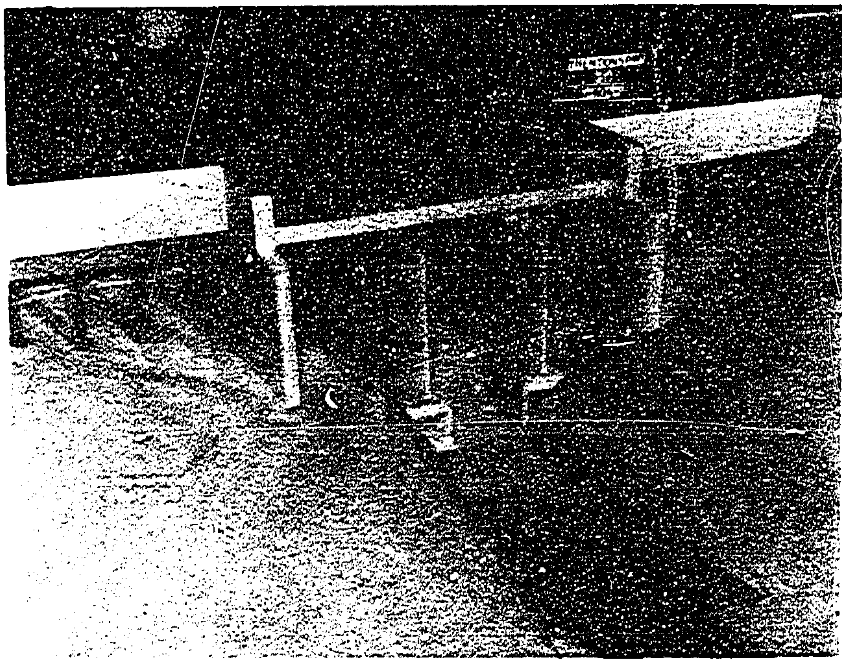
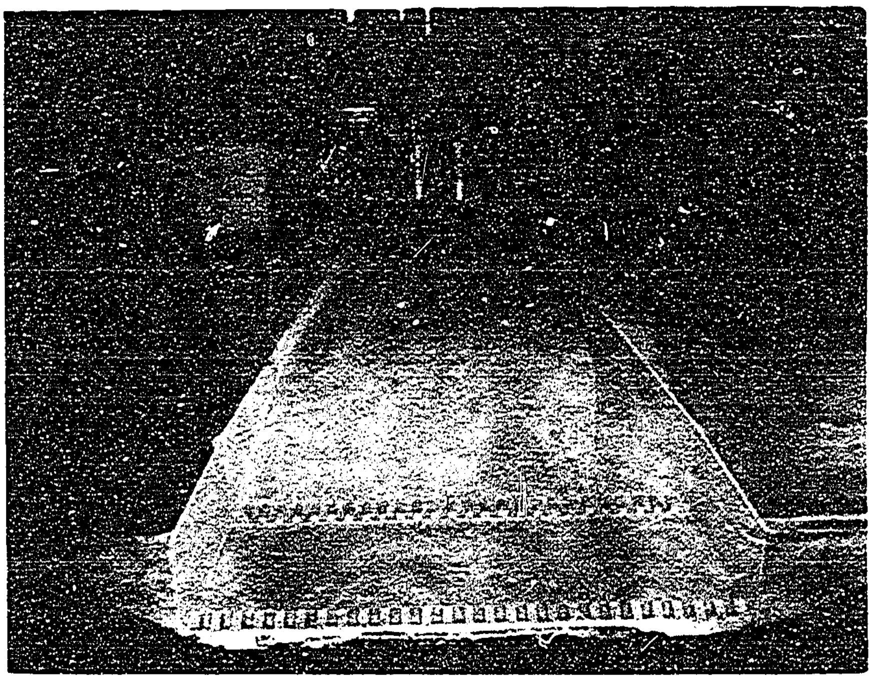



PLAN
SECTION A-A
SECTION B-B
(THROUGH RIVER OUTLET)
SECTION C-C
SECTION D-D
SECTION E-E
SECTION F-F
SECTION G-G
SECTION H-H
SECTION J-J
SECTION K-K
DETAIL Z
DISCHARGE-TAILWATER CURVES
RIVER OUTLETS DISCHARGE IN THOUSANDS OF C.F.S.
SPILLWAY DISCHARGE IN THOUSANDS OF C.F.S.
W.S. ELEV IN FEET
TAILWATER ELEVATION-FEET
CONCRETE FINISHES
Surfaces covered by fill:
Formed, F-1, Unformed U-1.
Exposed surfaces:
Formed, F-2, Unformed, U-2.
Concrete gutter, U-1; Stop log seats U-3
For finishes of river outlets see Dwg. 328-D-949
NOTES
Reinforcement shown in Sec. G-G, H-H, and J-J, only.
Structural design based on a minimum compressive strength of 3000 lbs per sq in at 28 days.
Highway bridge designed for H-20 S16-44 loading.
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MISSOURI RIVER BASIN PROJECT
FRENCHMAN-CAMBRIDGE DIVISION - NEBR.
TRENTON DAM
SPILLWAY
PLAN AND SECTIONS
DENVER, COLORADO APRIL 21, 1950
328-D-946

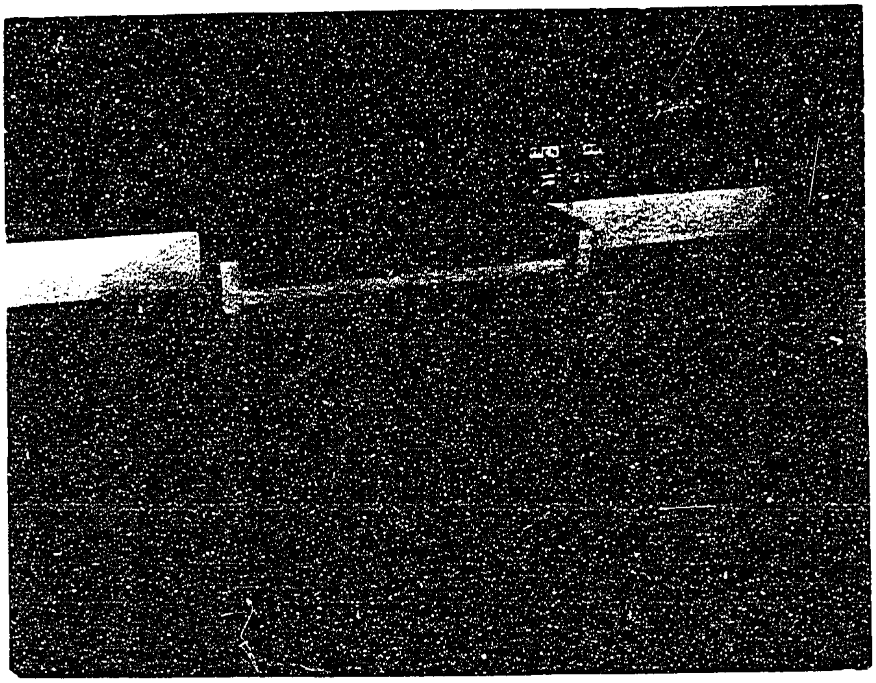


A. Spillway entrance

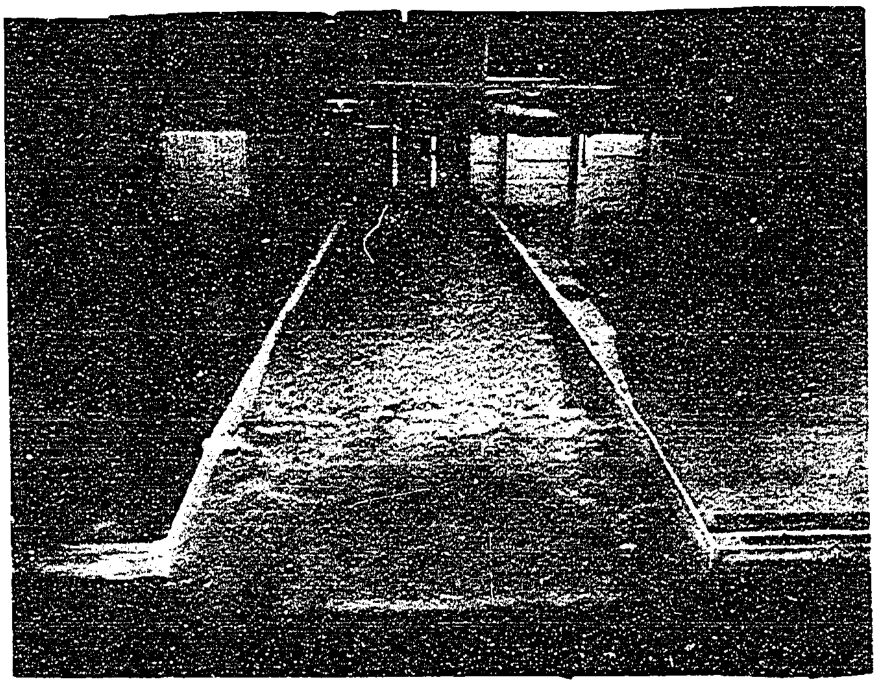


B. Spillway chute and stilling basin

1:54 MODEL OF PRELIMINARY SPILLWAY DESIGN
Trenton Dam

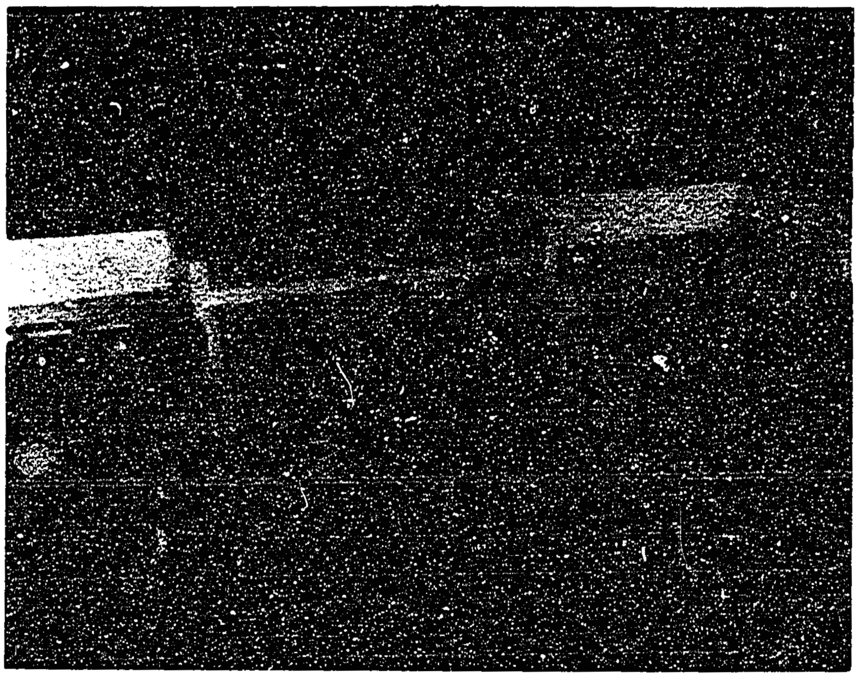


A. Spillway entrance

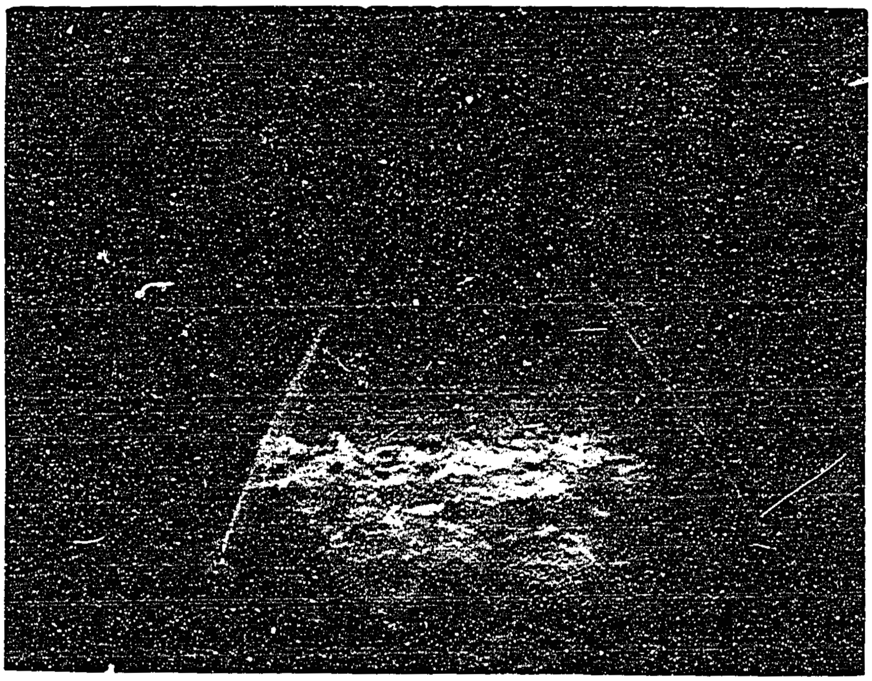


B. Spillway chute and stilling basin

PRELIMINARY SPILLWAY DESIGN - DISCHARGE OF 50,000 CFS
Trenton Dam

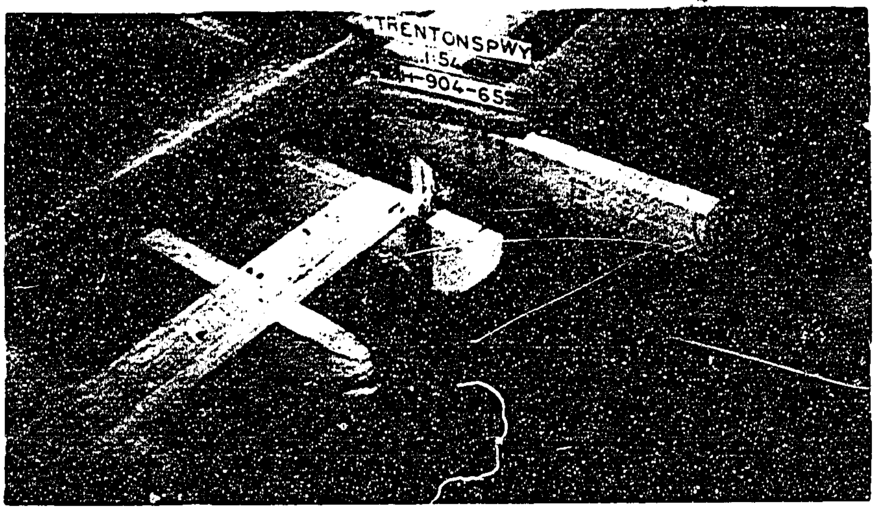


A. Spillway entrance

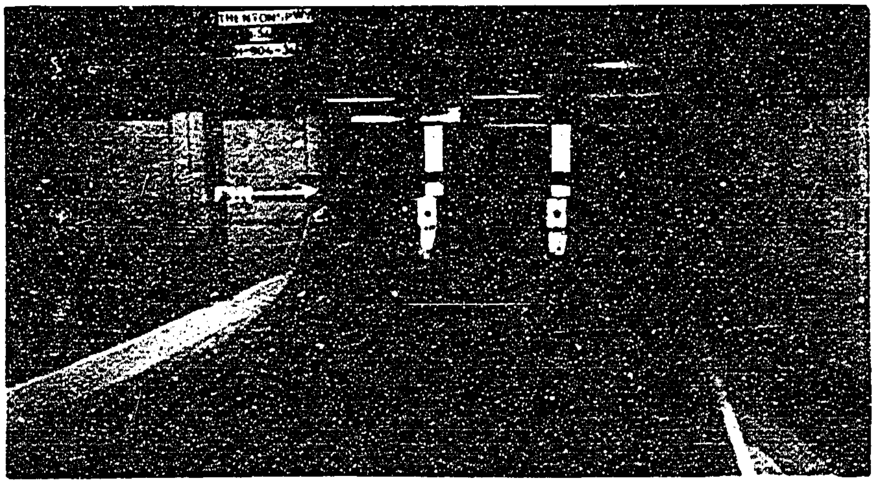


B. Spillway cute and stilling basin

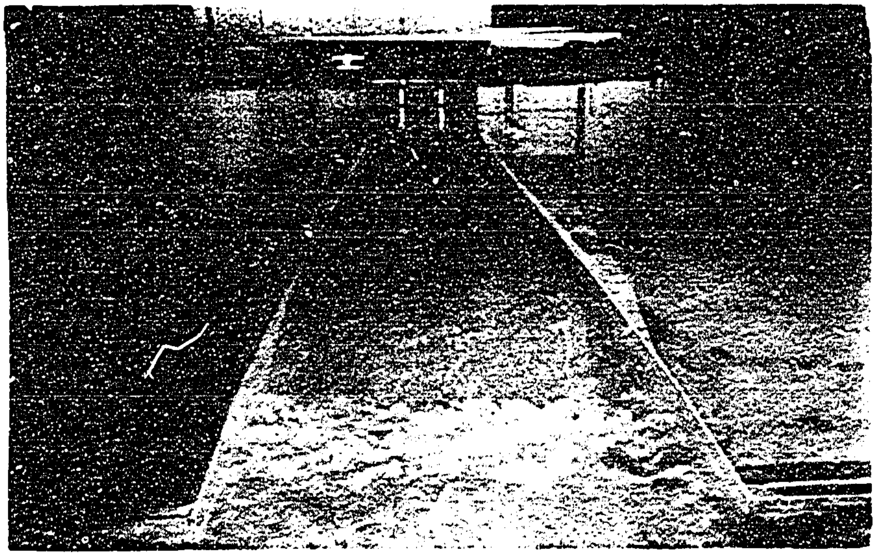
PRELIMINARY SPILLWAY DESIGN - DISCHARGE OF 100,000 CFS
Trenton Dam



A. Flow distribution and flow contraction at spillway entrance (Flow right to left)

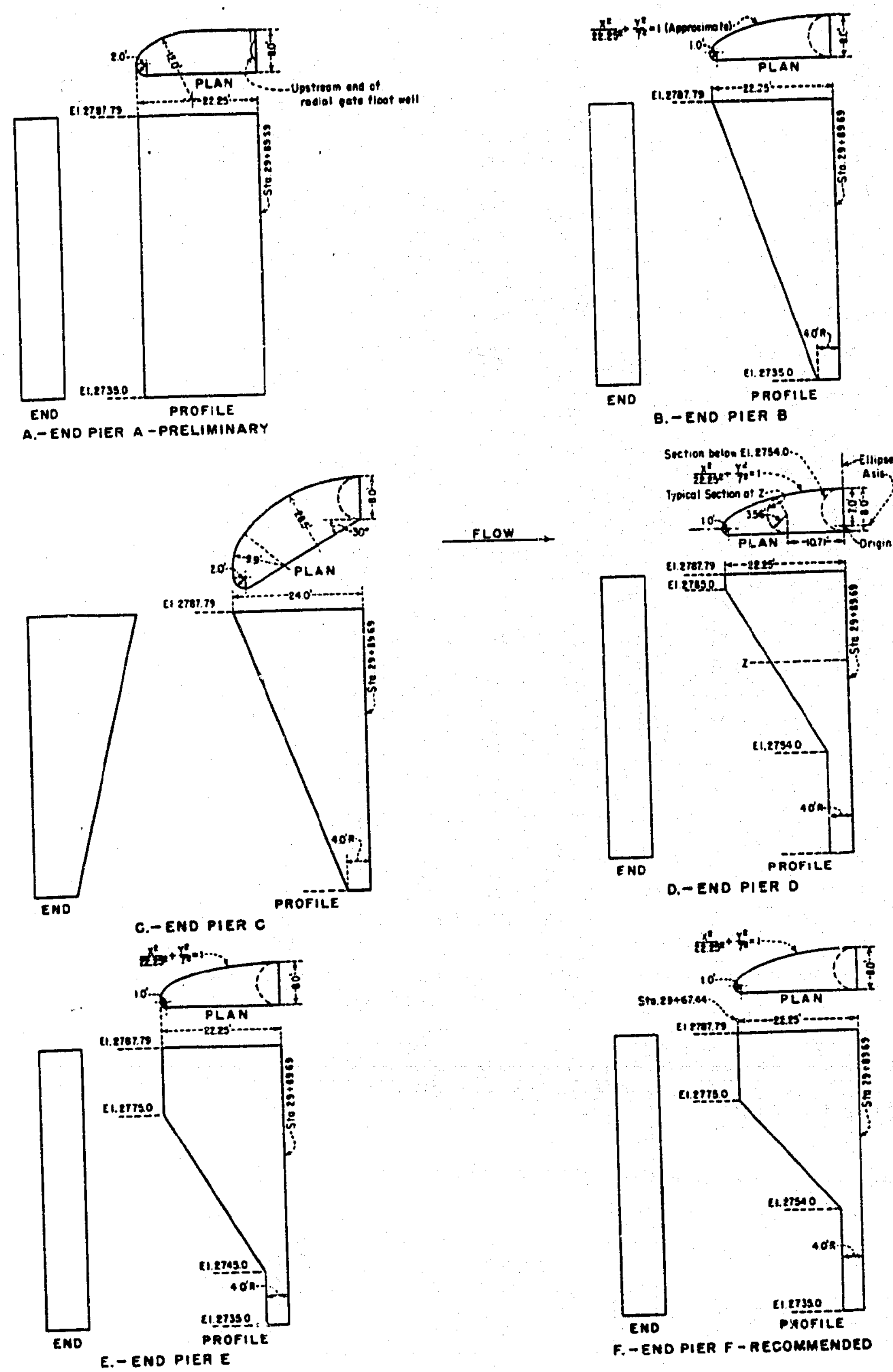


B. Flow distribution and waves on end piers in overflow section



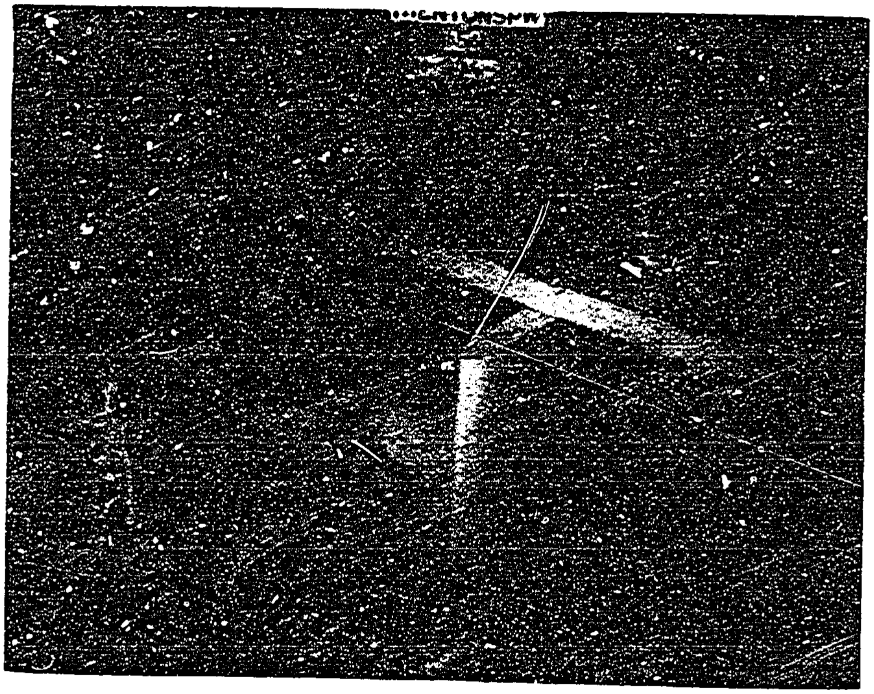
C. Spillway chute and stilling basin

PRELIMINARY SPILLWAY DESIGN - DISCHARGE OF 125,000 CFS - AT MAXIMUM RESERVOIR ELEVATION 2785
Trenton Dam



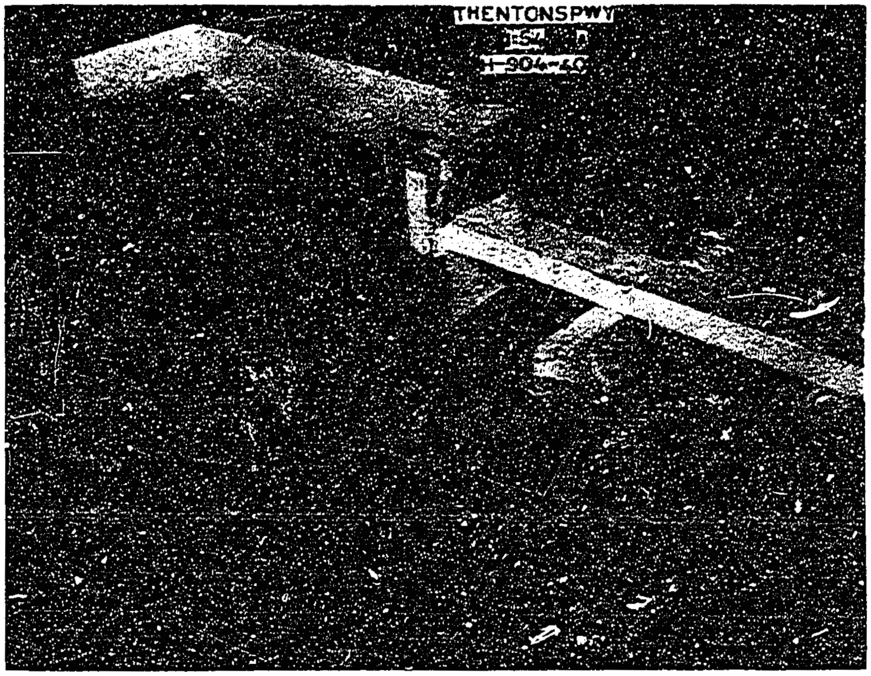
UPSTREAM EXTENSIONS OF END PIERS
SPILLWAY OVERFLOW SECTION
TRENTON DAM

A. Sheet metal plate used for determining the end pier shape

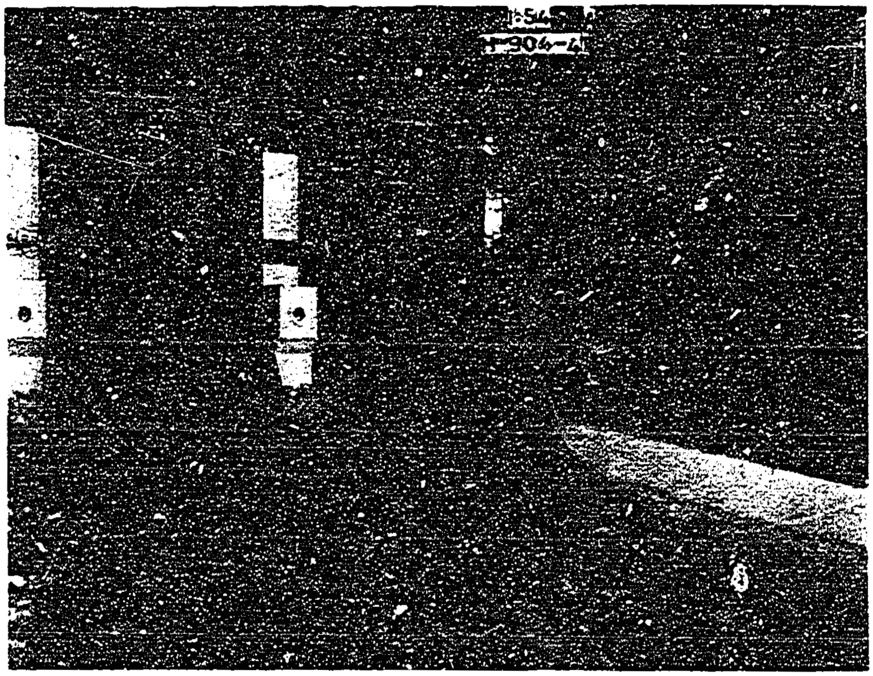


B. End pier B

SPILLWAY ENTRANCE-END PIER B
Trenton Dam

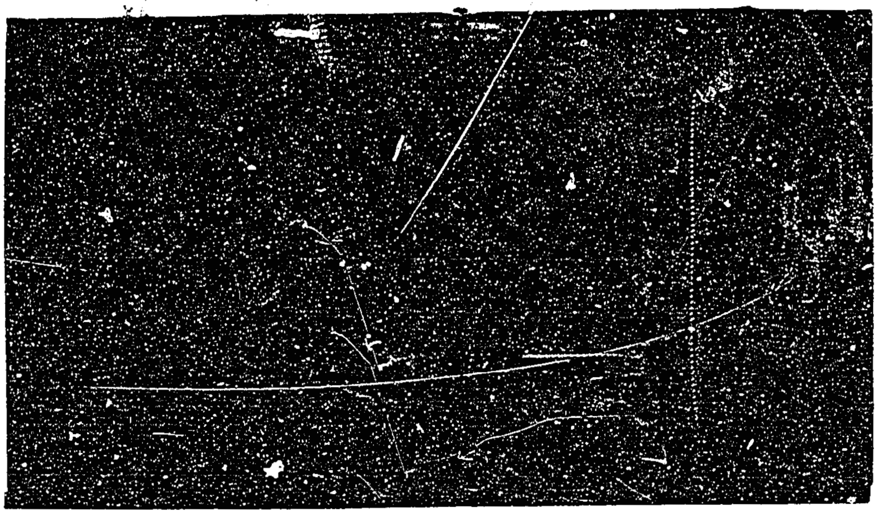


A. Flow distribution at spillway entrance for end pier B

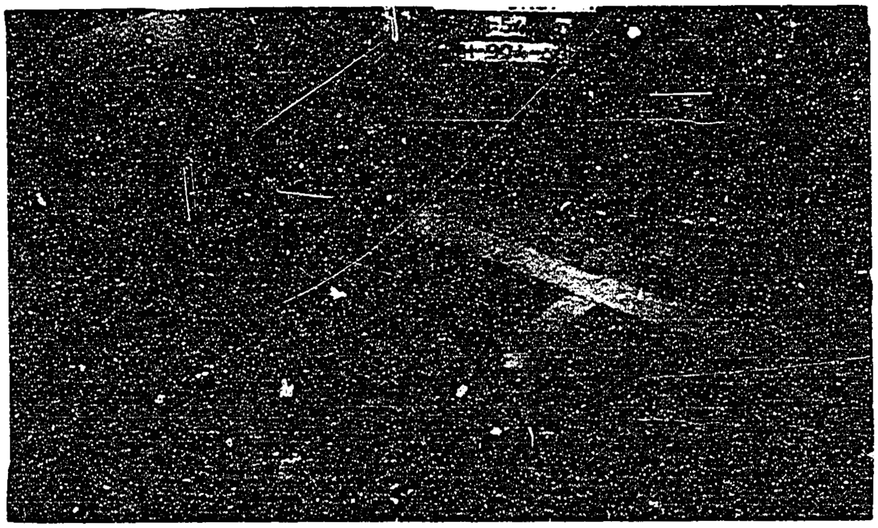


B. Flow distribution in overflow section for end pier B

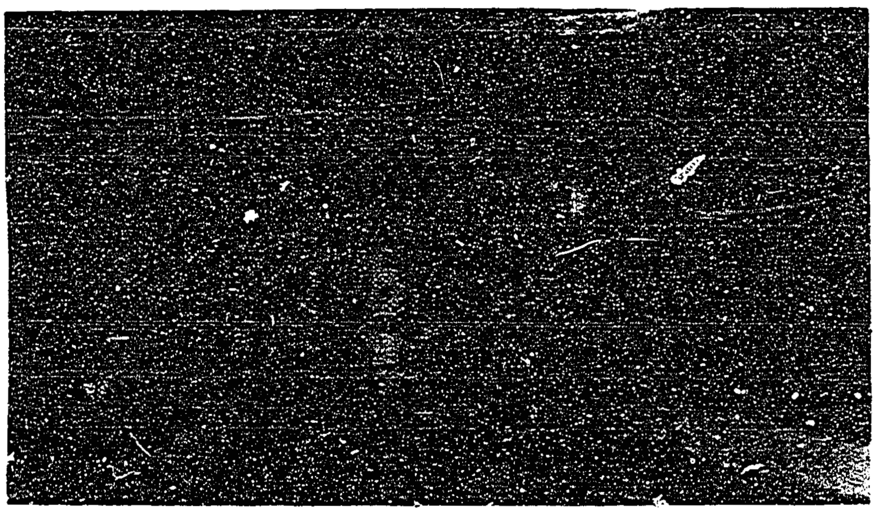
SPILLWAY END PIER B - DISCHARGE 126,000 CFS AT MAXIMUM
RESERVOIR ELEVATION 2785
Trenton Dam



A. End pier C

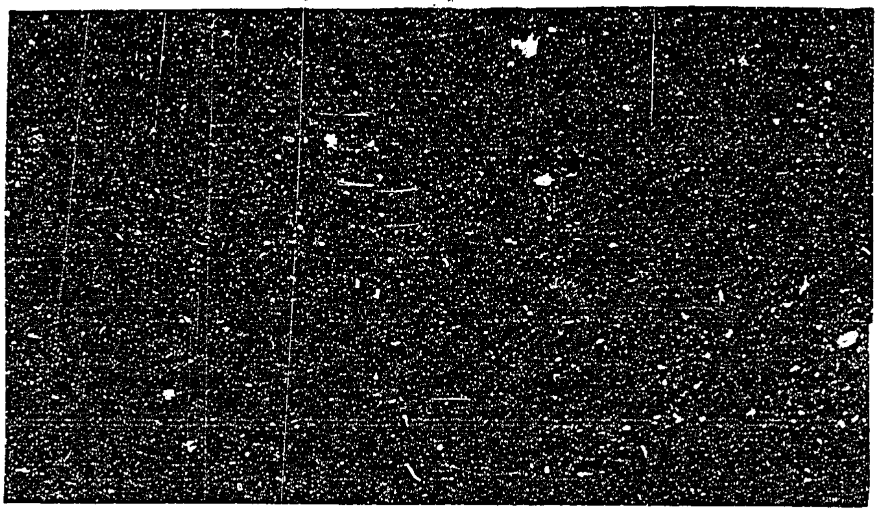


B. Flow distribution at spillway entrance for end pier C

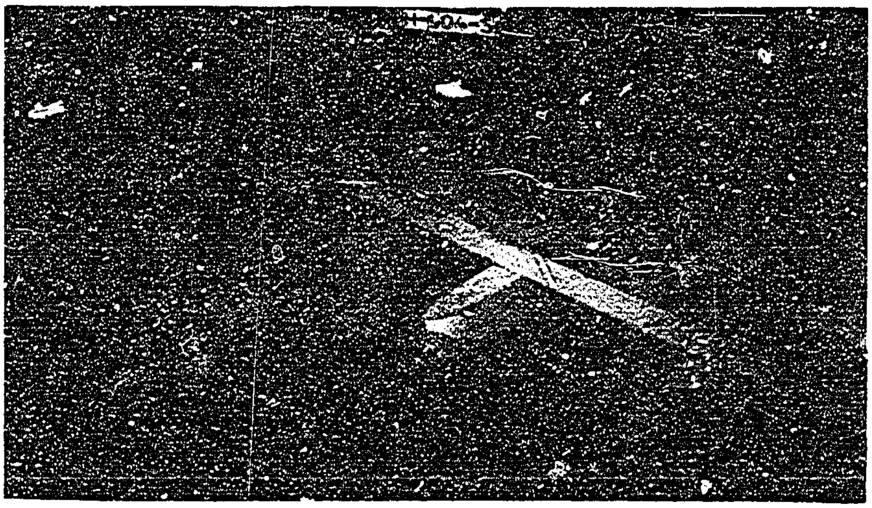


C. Flow distribution in overflow section for end pier C

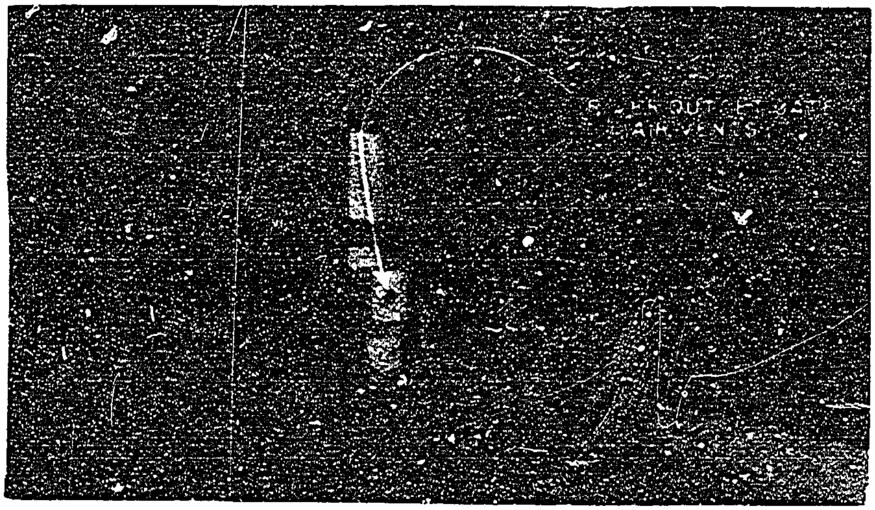
SPILLWAY END PIER C - DISCHARGE 126,000 CFS AT MAXIMUM
RESERVOIR ELEVATION 2785
Trenton Dam



A. End pier D

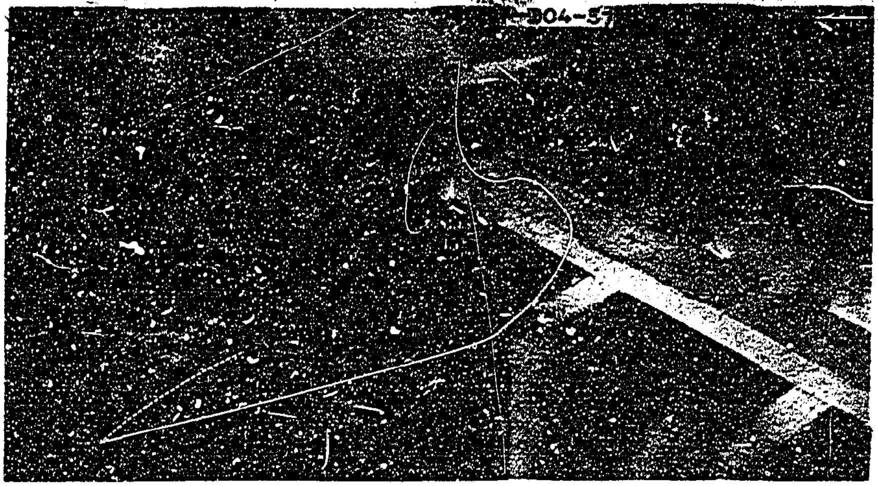


B. Flow distribution at spillway entrance for end pier D

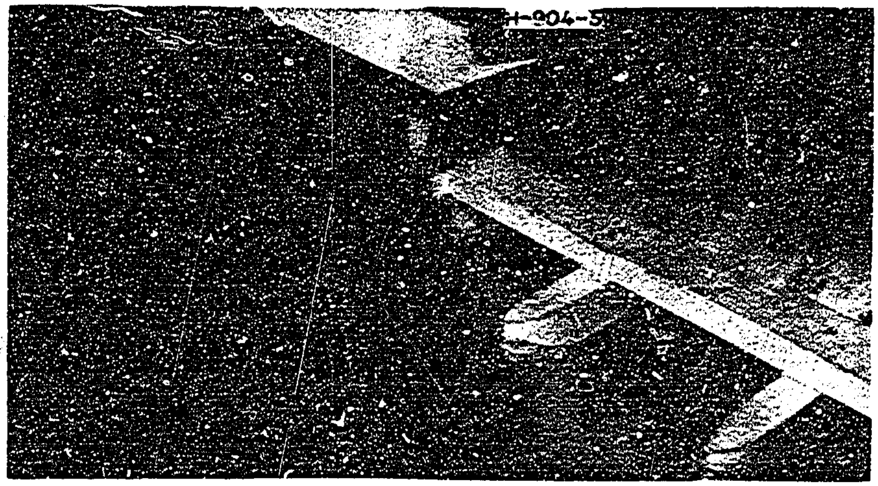


C. Flow distribution in overflow section for end pier D

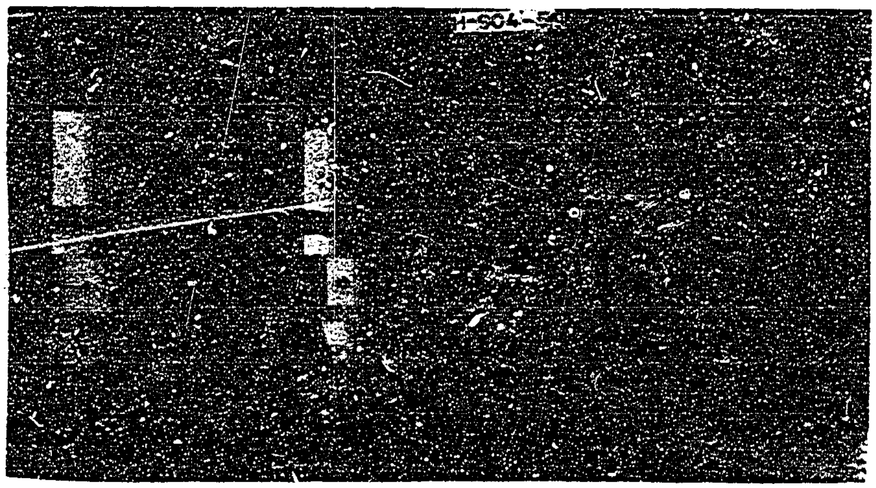
SPILLWAY END PIER D - DISCHARGE 126,000 CFS AT MAXIMUM
RESERVOIR ELEVATION 2785
Trenton Dam



A. End pier E

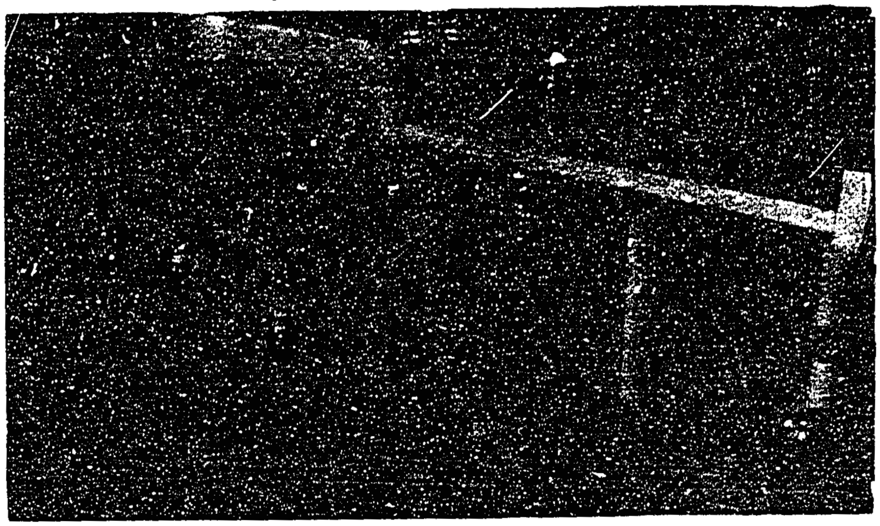


B. Flow distribution at spillway entrance for end pier E

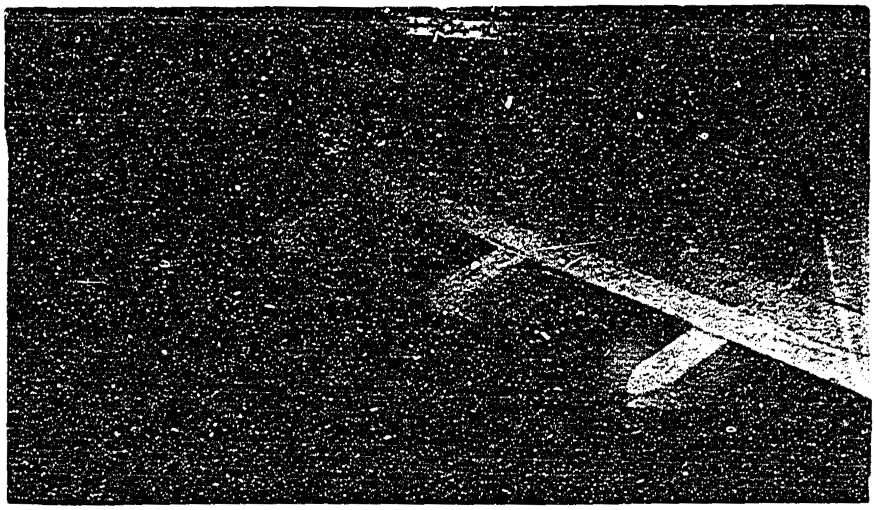


C. Flow distribution in overflow section for end pier E

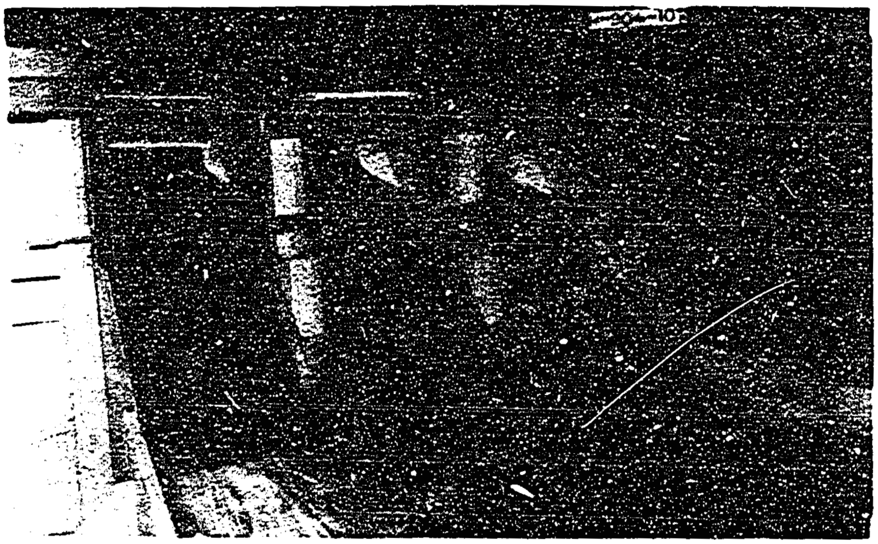
SPILLWAY END PIER E - DISCHARGE 126,000 CFS AT MAXIMUM
RESERVOIR ELEVATION 2785
Trenton Dam



A. End pier F - Recommended



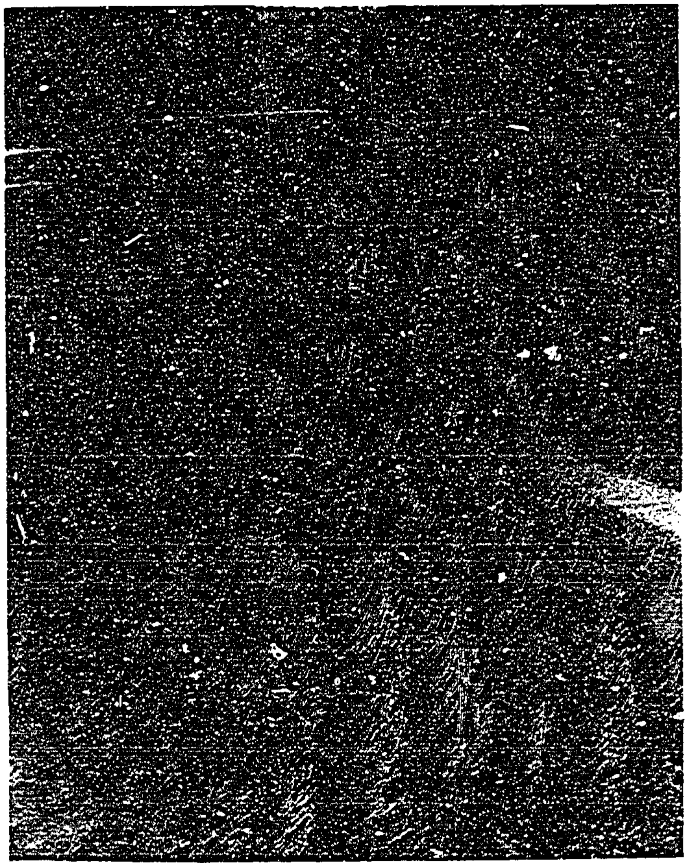
B. Flow distribution at spillway entrance for end pier F



C. Flow distribution in overflow section for end pier F

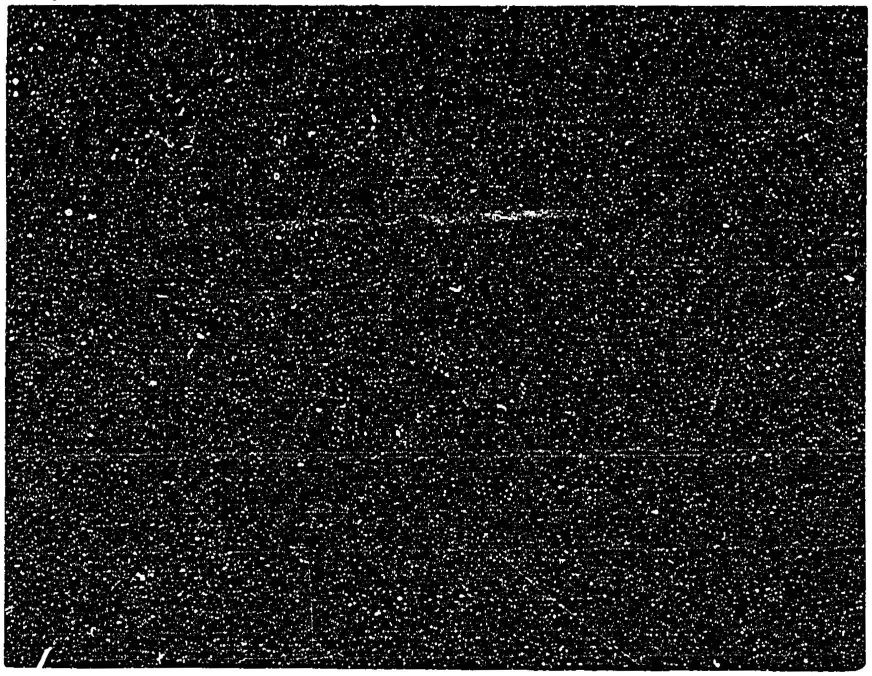
RECOMMENDED SPILLWAY END PIER F - DISCHARGE 126, 500 CFS
AT MAXIMUM RESERVOIR ELEVATION 2785
Trenton Dam

A. Streamlining of trailing edge of center piers

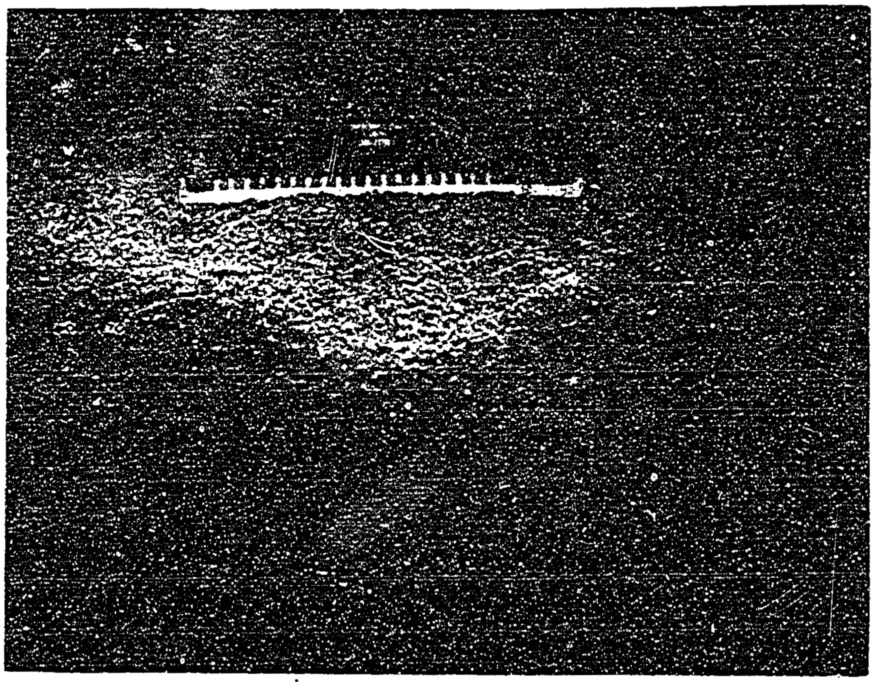


B. Flow distribution downstream of overflow section showing 3 fins of water, one from the end of each center pier and one at Station 31+95.8 - Discharge 126,000 cfs

SPILLWAY FLOW WITH STREAMLINED TRAILING EDGE OF CENTER PIERS
Trenton Dam

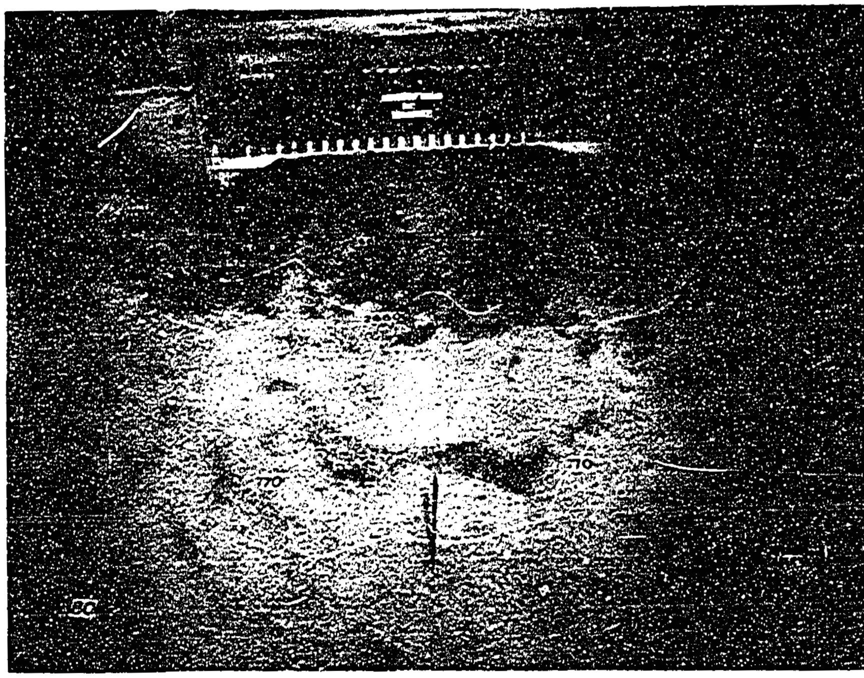


A. Preliminary stilling basin and spillway exit channel with representative 1 cubic foot to 1 cubic yard riprap 3 feet in thickness

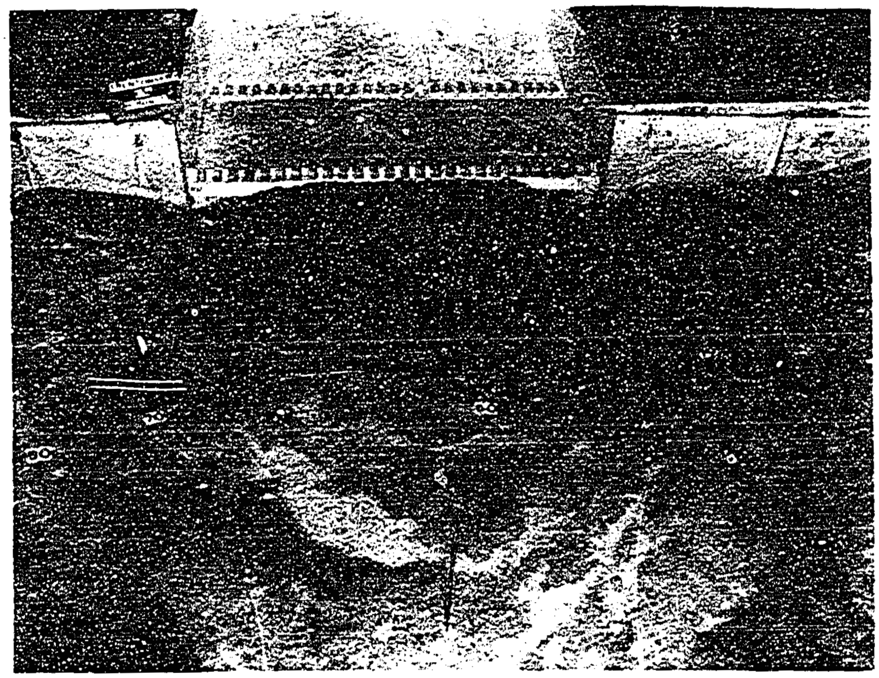


B. Erosion after a 4-hour discharge of 126,500 cfs at a tailwater elevation of 2700.6. Basin floor elevation 2653

EROSION IN SAND AND RIPRAP SPILLWAY EXIT CHANNEL
Trenton Dam

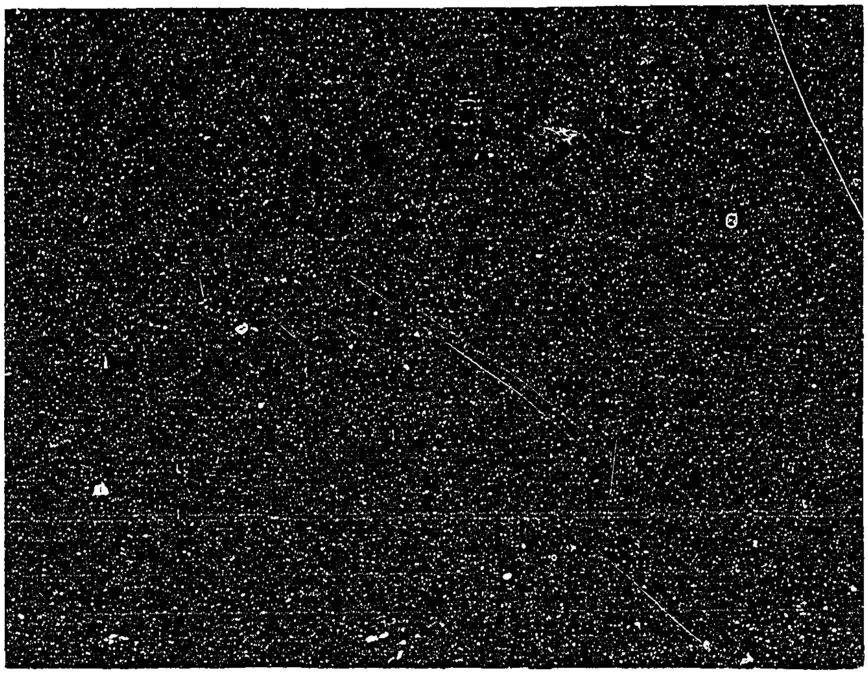


A. Preliminary stilling basin and spillway channel after 2-hour sand erosion

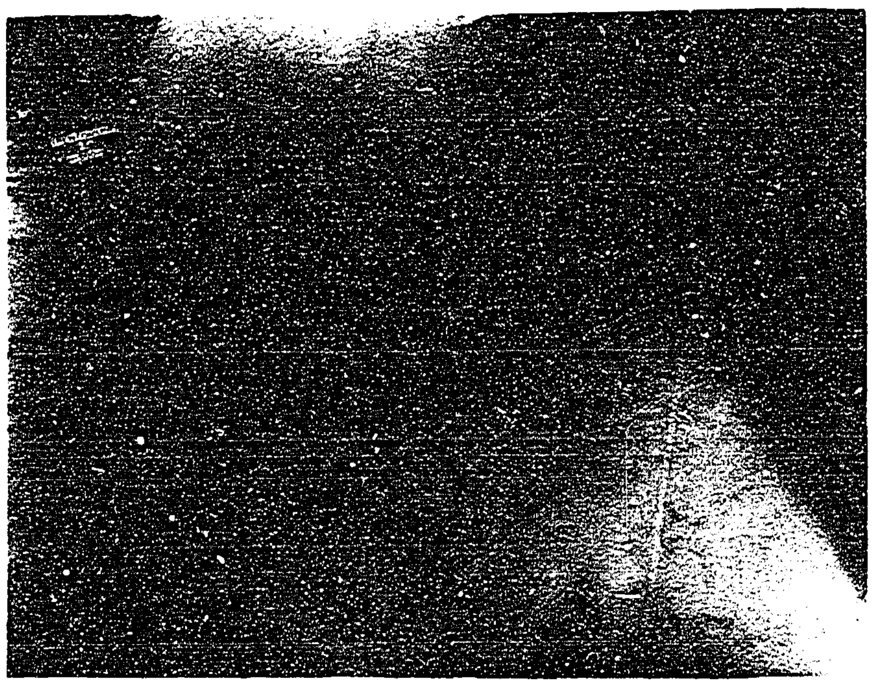


B. Preliminary stilling basin and river channel after 4-hour sand erosion

EROSION IN SAND OF SPILLWAY EXIT CHANNEL
DISCHARGE 126,500 CFS
TAIL WATER ELEVATION 2700.6
Trenton Dam

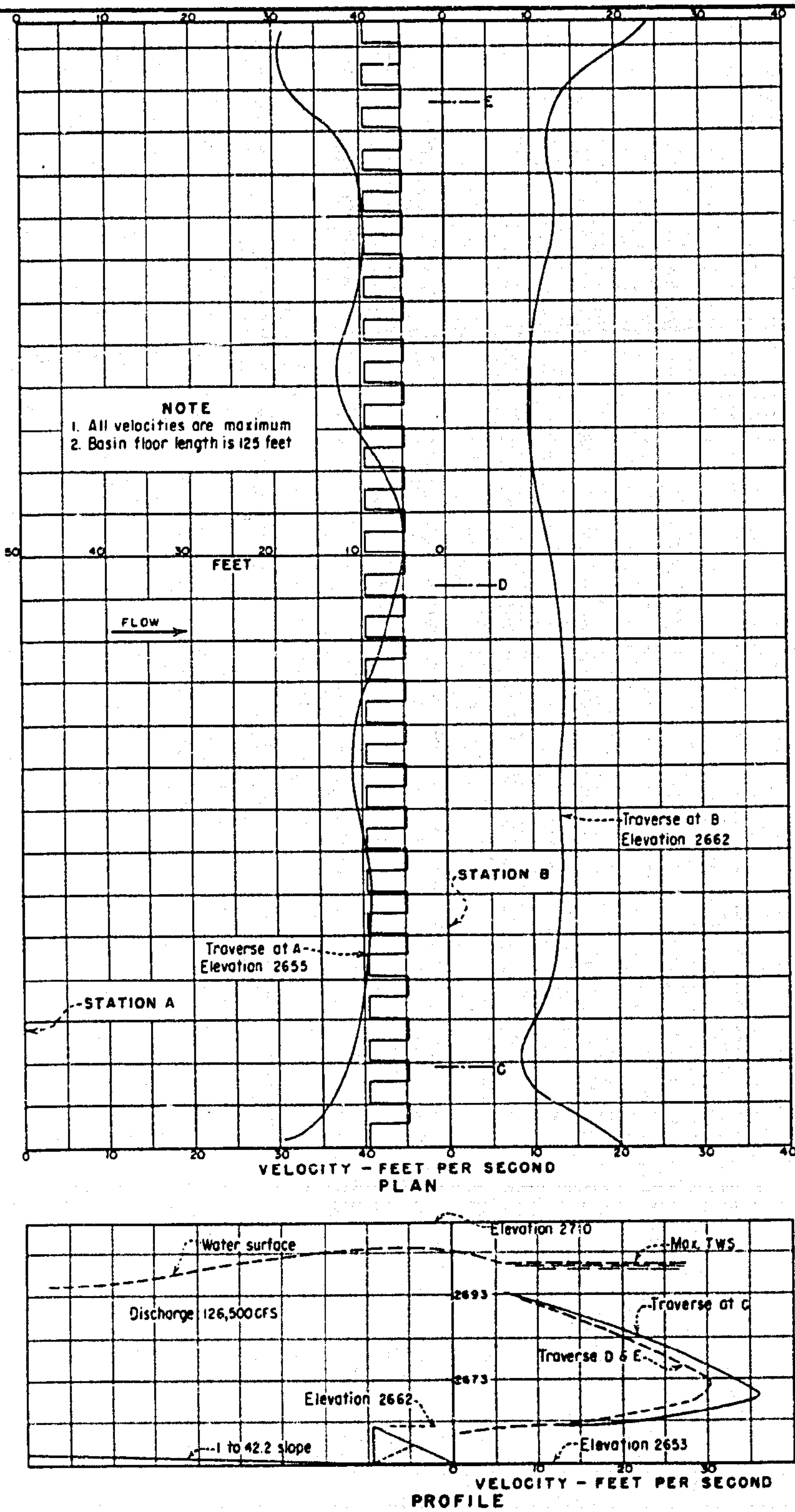


A. Four hour erosion results using one 20 foot long buttress at a distance of 11.5 feet from inside of training wall on both sides of spillway



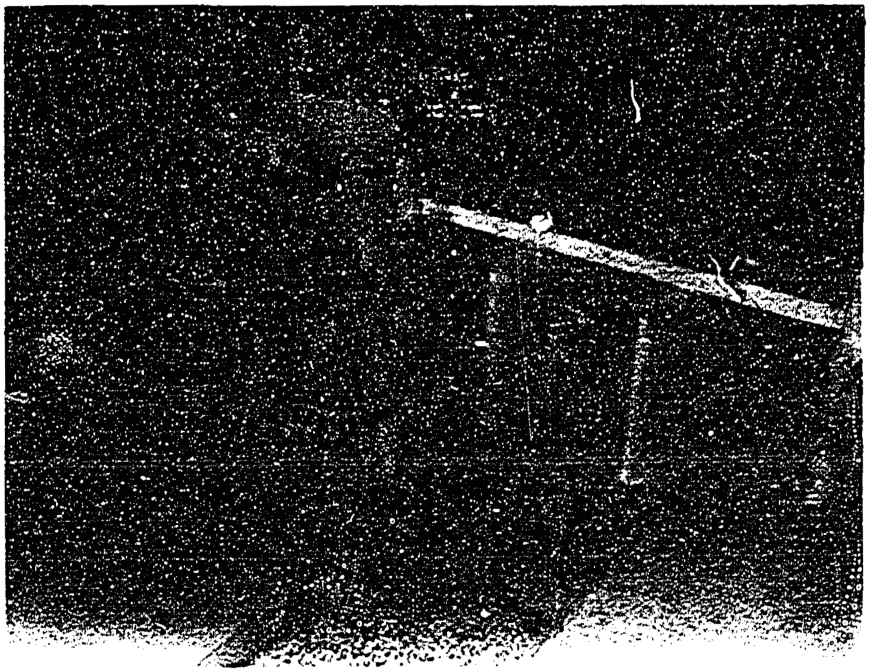
B. Four hour erosion results using one 20 foot and one 15 foot long buttress at respective distances of 11.5 and 24.5 feet from inside of training wall on both sides of spillway

EROSION IN SAND OF SPILLWAY CHANNEL - WING WALLS ON RETAINING WALLS - DISCHARGE 126,500 CFS
TAIL WATER ELEVATION 2700.6
Trenton Dam

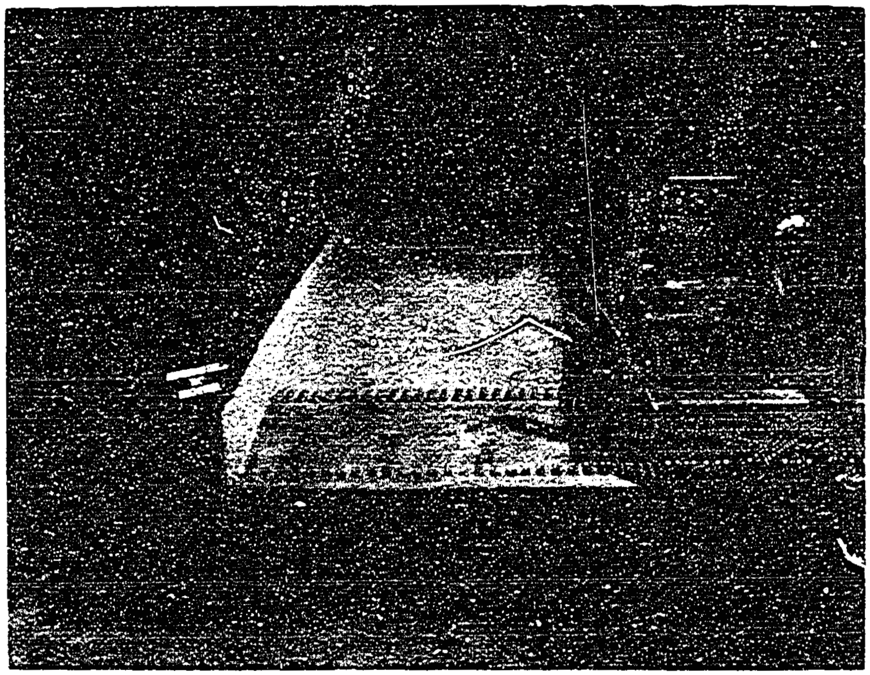


VELOCITY TRAVERSES AT SPILLWAY STILLING BASIN

TRENTON DAM

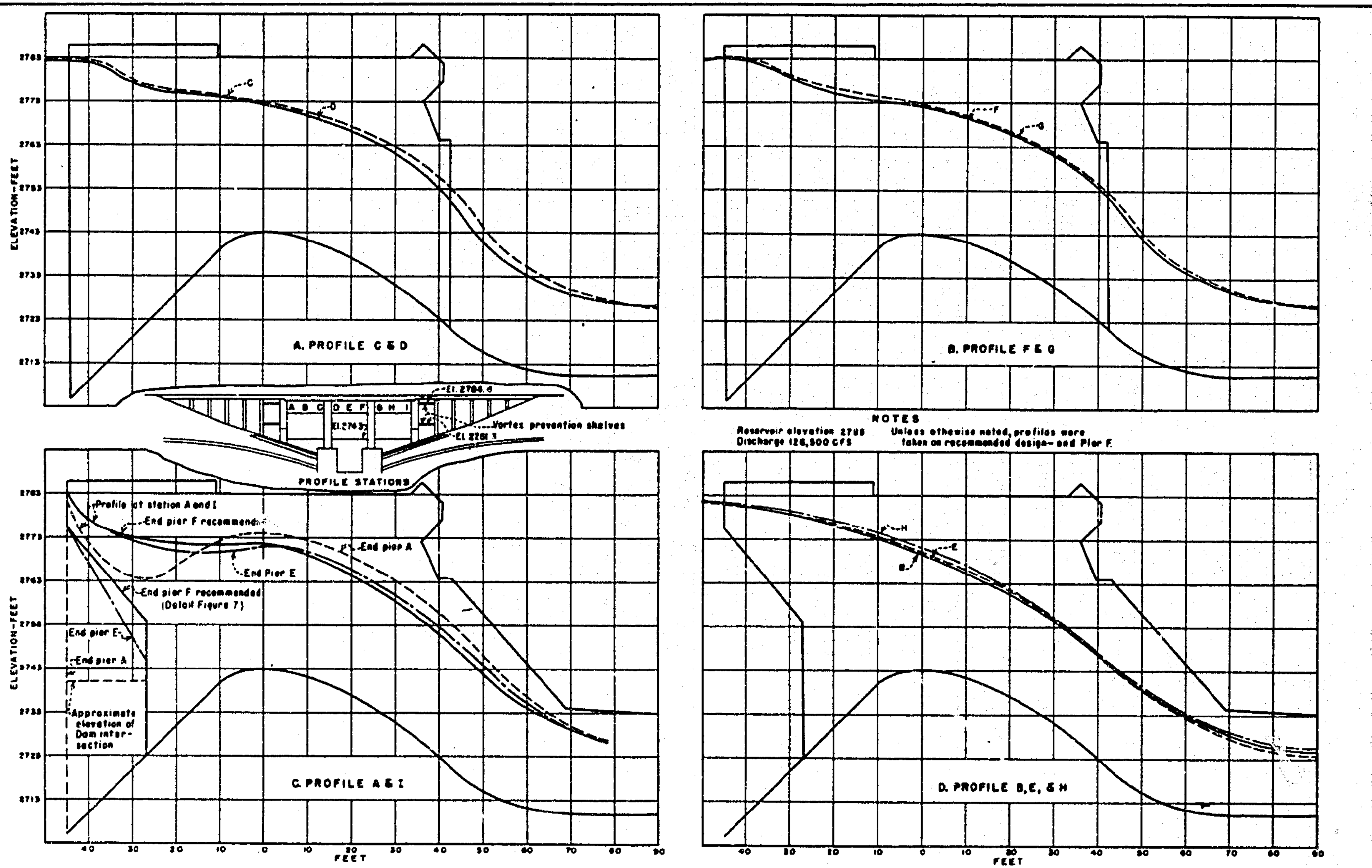


A. Spillway entrance and abutments - recommended design



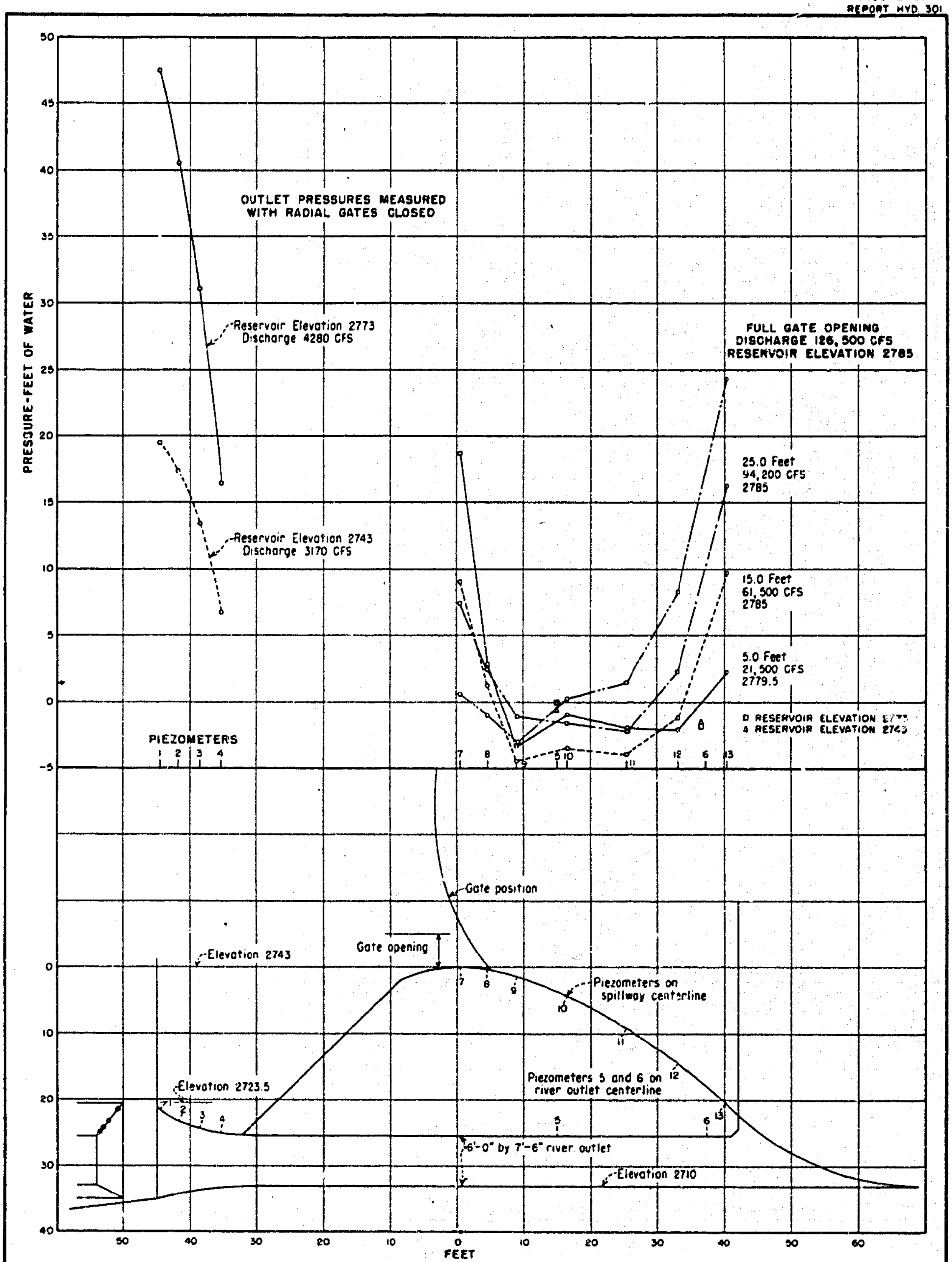
B. Spillway chute and stilling basin - recommended design
1 on 42.2 slope up in direction of arrow

1:54 MODEL OF RECOMMENDED SPILLWAY DESIGN
Trenton Dam



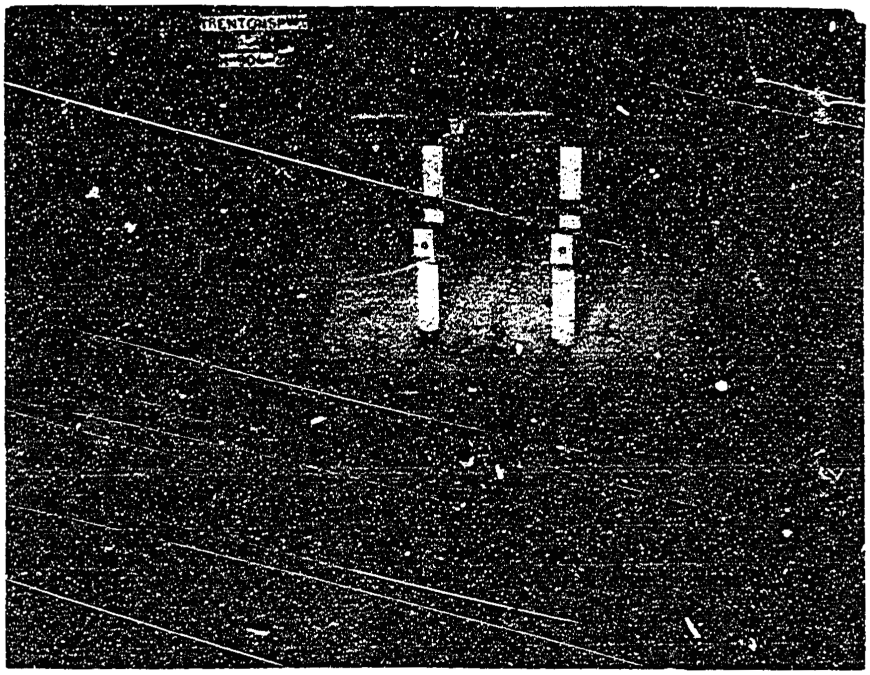
WATER SURFACE PROFILES
IN OVERFLOW SECTION OF SPILLWAY
TRENTON DAM

FIGURE 20
REPORT HYD 291

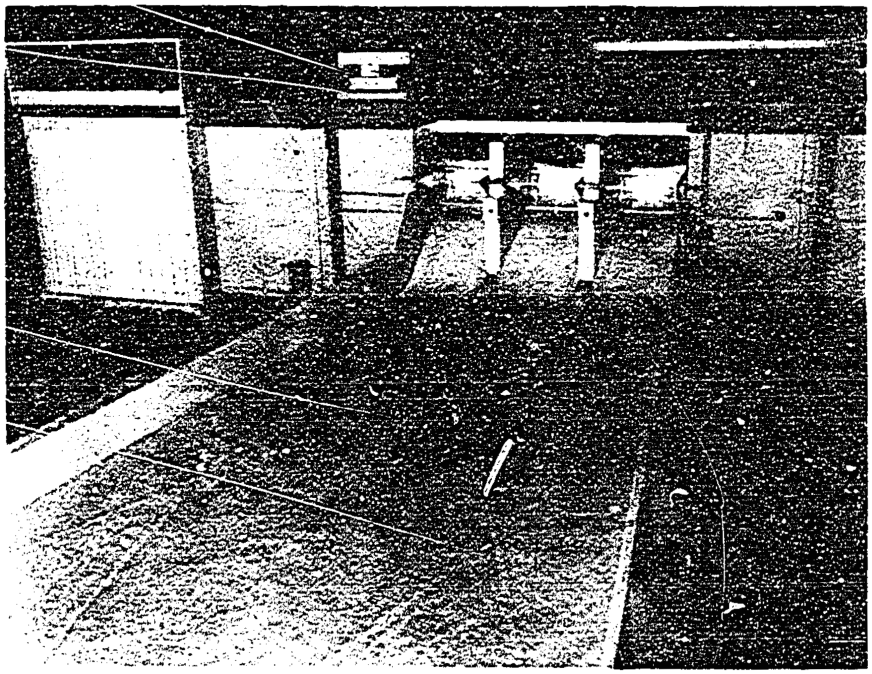


PRESSURES ON OVERFLOW SECTION OF SPILLWAY AND RIVER OUTLETS

TRENTON DAM

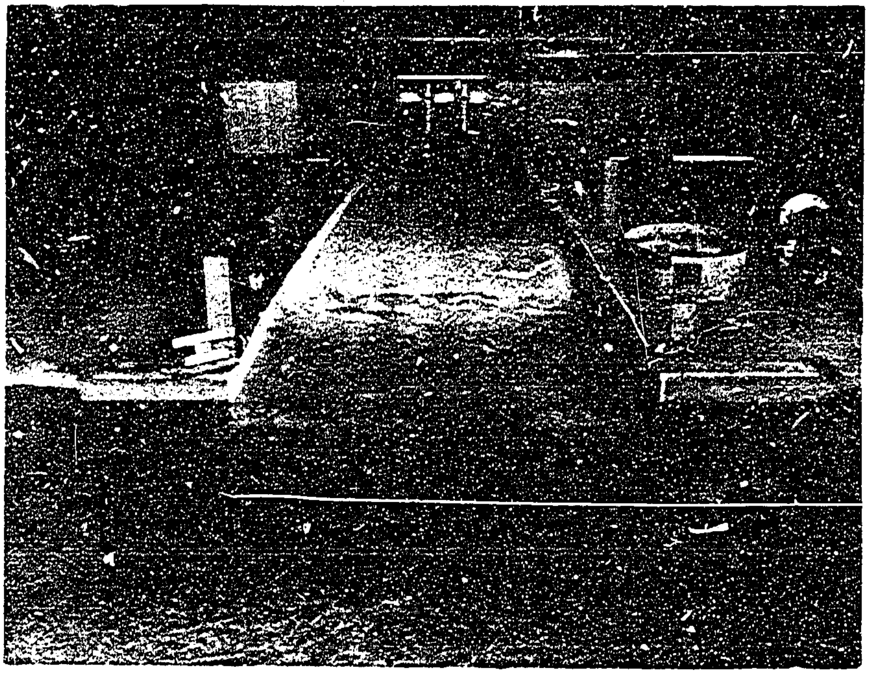


A. Flow distribution in spillway chute with river outlets discharging 875 cfs at reservoir elevation 2720

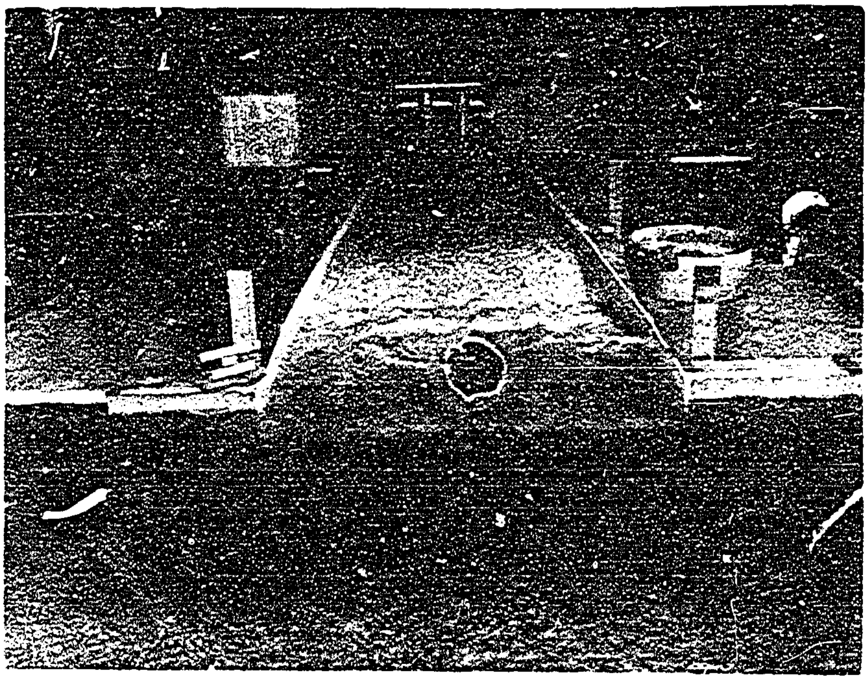


B. Flow distribution in spillway chute with river outlets discharging 3570 cfs at reservoir elevation 2752

RIVER OUTLETS DISCHARGING IN SPILLWAY CHUTE
Trenton Dam

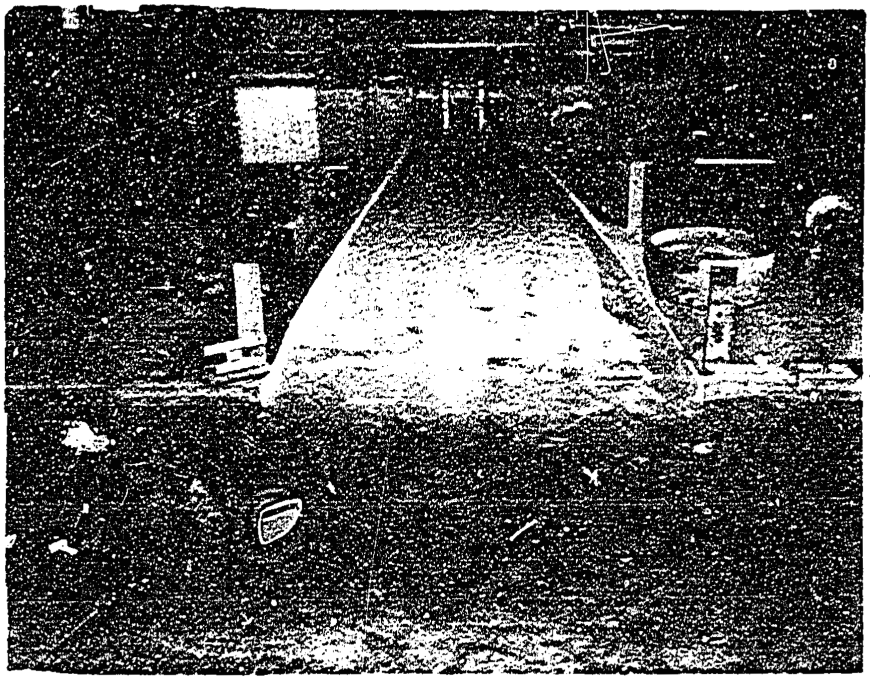


A. Flow in spillway chute and stilling basin - Discharge 25,000 cfs - Reservoir elevation 2781 - Gate opening 5.2 feet

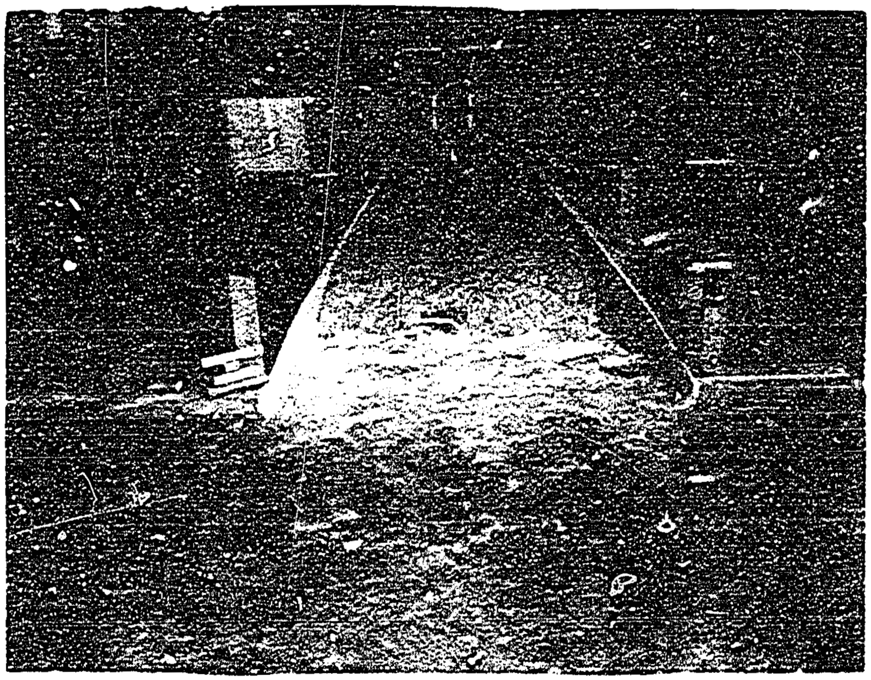


B. Flow in spillway chute and stilling basin - Discharge 50,000 cfs - Reservoir elevation 2785 - Gate opening 11.9 feet

RECOMMENDED SPILLWAY--DISCHARGE 25,000 AND 50,000 CFS
Trenton Dam

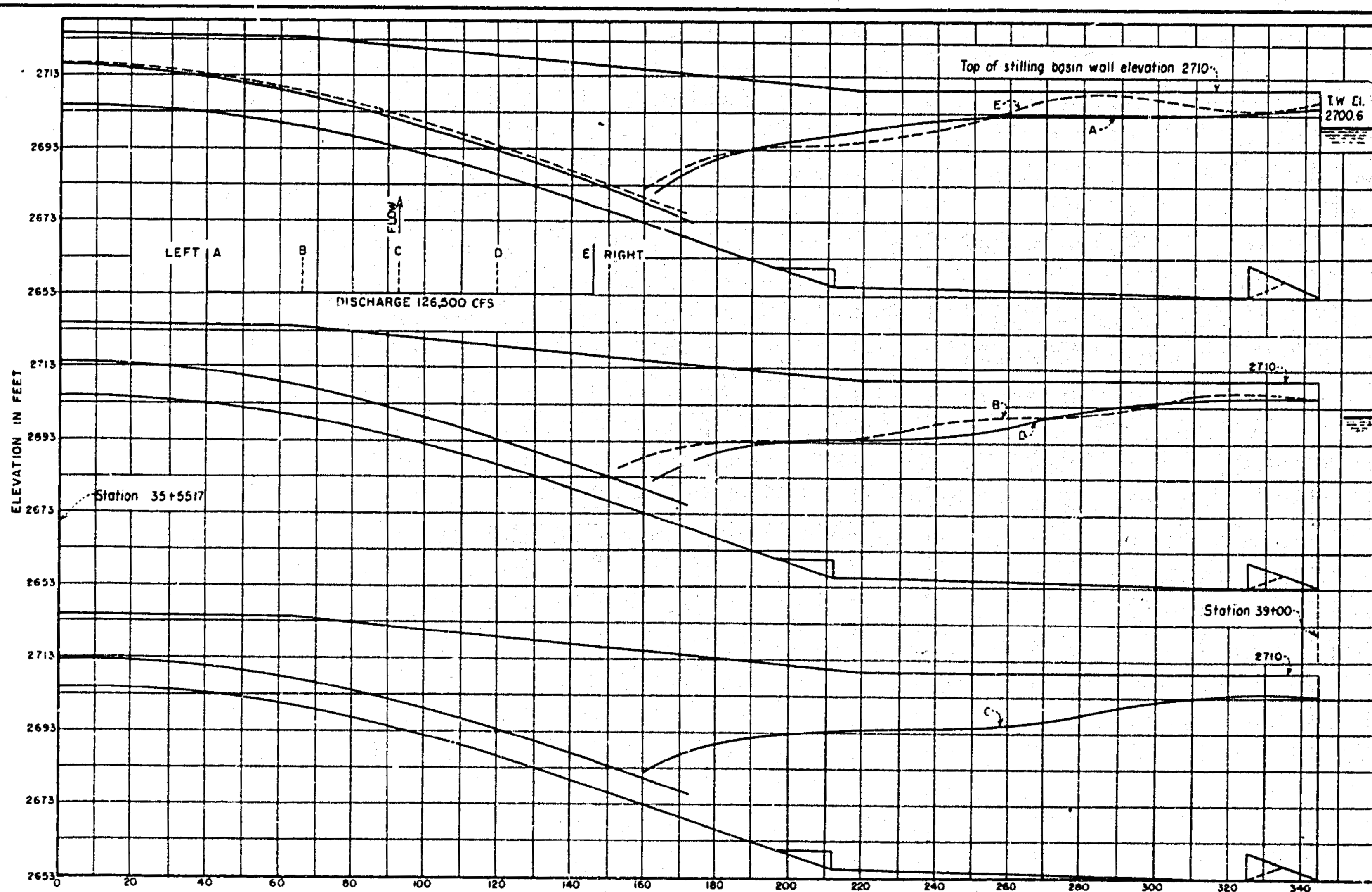


A. Flow in spillway chute and stilling basin - Discharge 100,000 cfs - Reservoir elevation 2785 - Gate opening 26.8 feet



B. Flow in spillway chute and stilling basin - Discharge 126,500 cfs - Reservoir elevation 2785 - Free Discharge

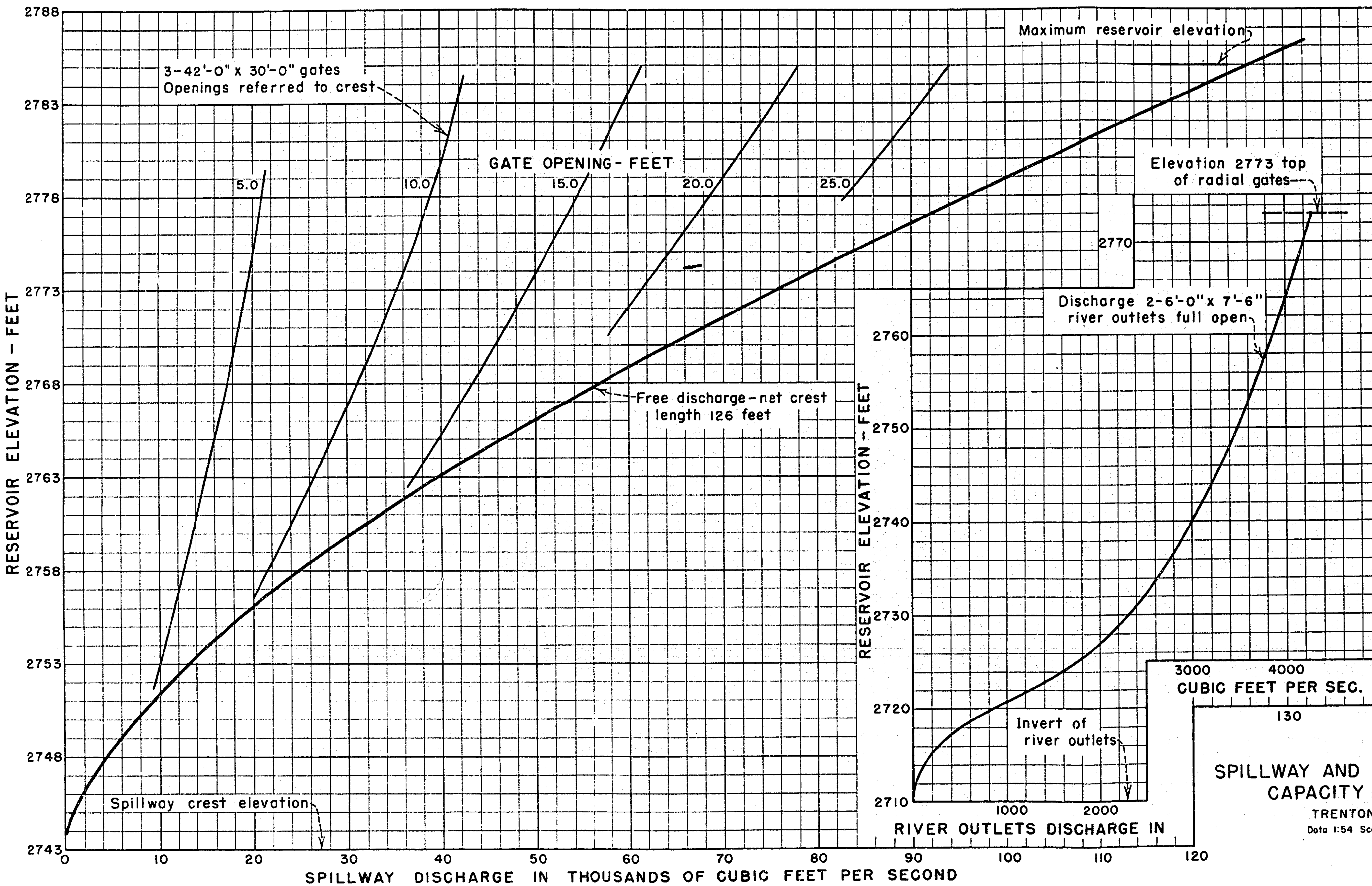
RECOMMENDED SPILLWAY--DISCHARGE 100,000 AND 126,500 CFS
Trenton Dam



WATER SURFACE PROFILES
IN SPILLWAY STILLING BASIN
TRENTON DAM

FIGURE 25
REPORT HYD. 301

FIGURE 26
REPORT HYD. 301



SPILLWAY AND RIVER OUTLET CAPACITY CURVES

TRENTON DAM

Data 1:54 Scale Model