UNITED STATES

DEPARTMENT OF THE INTERIOR

BUREAU OF RECLAMATION

MEMORANDUM TO CHIEF DESIGNING ENGINEER

SUBJECT: REPORT ON HYDRAULIC MODEL EXPERIMENTS FOR THE DESIGN OF HOOVER DAM SPILLWAYS

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by

E. W. LANE, RESEARCH ENGINEER

Under direction of

TECHNICAL MEMORANDUM NO. 263

[illegible]

Montrose, Colorado

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DEPARTMENT OF THE INTERIOR
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Memorandum to the Chief Designing Engineer.

Report on Hydraulic Model Experiments for The Design of The Hoover Dam Spillways.

E. W. Lane.

The model experiment study for the design of the Hoover Dam spillways was begun about October 1, 1930 at the hydraulic laboratory of the Colorado Agricultural College at Fort Collins and has been under way there nearly continuously since that time, a part of the time using two eight-hour shifts. Plans for the laboratory at Montrose were started in February and it was completed, together with the first model on July 18th of this year. Work has been in progress at Montrose since that time. Seven models of four different types have been experimented on, with linear scales ranging from 1:100 to 1:20, involving discharges for the design capacities from 2 to 112 sec. ft. In each model a number of major and minor changes were made during the testing.

The following is a brief summary of the work carried on to October 1st of this year. It does not include the work on the second large scale model at Montrose nor a large number of tests on the drum gate model at Fort Collins.

Glory Hole Spillway, 1:60 Scale (Model C-1)

Moving pictures and photographs.

Clear Crest Side Channel Spillway, 1:60 Scale (Model C-2)

22 Runs with various alterations to check the side channel theory.

15 Runs to determine the possibilities of vanes.

13 Runs to observe flow in various designs of horizontal bend.

Stony Gate Spillway, 1:60 Scale (Model C-3)

77 Preliminary runs to insure consistent results on gages.

40 Runs on original design to observe flow conditions and determine discharge coefficients.

17 Runs to investigate possible improvements in approach channel to Stony gate.

24 Runs to test possible improvements in the flow in the channel by means of baffles, changes in bottom and side wall.

13 Runs to observe flow from lower end of tunnel transition.

Drum Gate Spillway, 1:60 Scale (Model C-4)

15 Runs to determine discharge coefficients.

26 Runs to record water levels with transition No. 1.

97 Runs to observe conditions of flow with various types of sills, baffles, bottom shapes and weirs across end.

In addition to these 97 runs, since October 1st a large number of runs have been made with sills of various types.

14 Runs to record water levels with Transition No. 2

22 Runs to observe flow and record water levels with various forms of baffles and weir using Transition No. 3.

9 Runs to observe flow and record water levels with various forms of baffles and floor using Transition No. 4.

<u>Drum Gate Type Spillway, 1:100 Scale (Model M-2)</u>

6 Runs to observe action and record water levels with various forms of bottom and transition.

<u>Drum Gate Spillway, 1:20 Scale (Model M-1)</u>

22 Runs to record water levels in channel for various types of baffles and bottom.

20 Runs to observe flow conditions with various forms of baffles and bottom.

18 Observations of flow in vertical bend and the tunnel.

26 Observations of flow at tunnel exit.

11 Runs to determine discharge coefficients.

The following report gives all the more important facts brought out in the tests. Time has not been available to work up all the quantative data in detail. In the descriptions of the models and the tests thereon, the corresponding dimensions and quantities of the prototype have been used throughout rather than the actual model values, as the former give a much clearer idea of the conditions, and avoid the confusion which would otherwise arise from the use of several different scales for the same prototype.

Report on

HOOVER DAM SPILLWAY MODEL TESTS

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GLORY HOLE SPILLWAY EXPERIMENTS

The preliminary drawings of the Hoover Dam were made with spillways of the Glory Hole or shaft type, but this was merely a tentative design, as it was expected to make extensive model experiments and base the selection on the results obtained from them. The early designs of this type of spillway contemplated a discharge of 100,000 sec. ft. for each shaft, with the water surface in the reservoir at El. 1232.

In order to meet the condition specified for the Hoover Dam project, that the discharge from a flood as large as that in 1884 should be carried safely to the river channel below the dam, the two spillways had to limit the discharge to 62,500 sec. ft. (not including power house discharge) at El. 1229. These requirements fixed the diameter of the spillway crest at 231 ft.

The spillway designed to meet these requirements with the prototype dimensions is shown on Plate I. It consists of a circular crest of ogee cross-section below which was a morning glory shaped funnel leading to the 50 ft. diameter shaft. The purpose of the ogee crest was to produce a high discharge coefficient, in order to give a large discharge per foot of crest length, as this would enable a shorter crest to be used, resulting in a smaller, cheaper spillway.

The model of this spillway was tested in the hydraulic laboratory of the Colorado Agricultural College at Fort Collins, on a scale of 1:60 or 1" = 5 ft. The arrangement of the apparatus is shown on Plate II.

The water is drawn from a reservoir on the hill behind the laboratory through three 14" valves, with which the coarse adjustments of flow are made. The water enters the laboratory in a concrete flume 10 ft. wide. The fine adjustments of flow were made by means of a movable crest and small valve in the side of this flume, through which the excess water could be wasted.

The discharge through the model was measured on a 2 ft. Cippoletti weir which had been calibrated by volumetric measurement. The head was observed by a float gage of the Cornell type. Leakage through the forebay was collected in a measuring tank and the flow of the weir corrected by the amount to give the exact flow over the spillway crest. It is believed that the discharge results are accurate within 1 per cent.

The water was led to the spillway through a series of baffles, which produced still water upstream from the structure. The crest section of the spillway was built of paraffin. The paraffin block was first cast approximately to the shape desired and then cut accurately to dimensions by a template in the form of a knife or screed which was revolved about an axis in the center of the spillway shaft. Although paraffin made a rather soft model, easily scarred or nicked, it was easily patched and for this type of model is believed to have been as easy to build and satisfactory in its operation as could have been secured with any other material. The vertical tunnel, vertical bend and horizontal tunnel were formed of transparent pyralin, in order that the action of the water within the tunnel could be observed. The various sections were made of pyralin sheets 1/8 inch thick, formed in a hydraulic press. The straight sections were about 18" long, flanged at the ends and with a longitudinal butt joint with outside strap. The 90 degree vertical bend was made up of four $22\frac{1}{2}$ degree flanged bends, each $22\frac{1}{2}$ degree bend being divided into two similar flanged sections on the plane passing through the center line of the bend. The 90 degree bend was made in sections in order to facilitate construction and to enable experiments to be made with $22\frac{1}{2}$ degree, 45 degree, or $67\frac{1}{2}$ degree bends also.

Results

As a result of the speeding up of the Hoover Dam project as an employment relief measure, it was necessary to make a decision on the general type of spillway without the extensive tests at first contemplated. In order not to delay work on the side channel models, experiments on the glory hole type consisted only of visual observations and photographs. To make room for the model tests on the Madden Dam, the glory hole model was stored away and it is hoped later to move it back into position and make quantitive tests on it with a vertical shaft and to investigate its action with shafts inclined with the vertical.

The design of this model assumed that the water would flow over the crest and toward the spillway shaft in a radial direction, but this does not occur. To bring the cost of the spillway to a reasonable figure, it was necessary to set the structure partially in an excavation in the side of the canyon and about half of the water passing over the crest is carried to the crest in the excavated channels A and B, in which the direction of flow is not at right angles to the crest but follows the path shown in Plate I. The water approaching the crest therefore has a component in the direction of the crest, and does not flow over it radially but takes ar inclined direction as shown by the stream lines of the figure. As the water approaches the center of the morning glory shaped section, more of it tends to concentrate at C and enter the shaft on that side, causing a disturbed condition of flow at that point. This also leads to disturbed conditions in the vertical shaft and hence to considerable commotion in the bend at the bottom.

It is believed that an even distribution of the flow entering the spillway shaft could have been secured with radial piers on the crest and on the morning glory shaped section, similar to those used on the Davis Bridge spillway, but the effect of the impact of the water at the bottom of the vertical shaft was obviously so much more severe than after sliding down the inclined shaft from the side channel that with the unprecedented velocities resulting from the 600 ft. drop at the Hoover Dam it was believed to be safer to use the side channel type.

SIDE CHANNEL SPILLWAY EXPERIMENTS
Free Crest Type (Model C 2)

The first side channel plan for the Hoover Dam contemplated a plain ogee crest without gates, with the same discharge requirements as the glory hole type. The design of this spillway is shown in Plate III. The model of this structure was also erected in the Colorado Agricultural College laboratory on a 1:60 scale. The position of the apparatus is shown in Plate II. The water measurement apparatus was the same as that previously described.

The Construction of the Model

The side channel of this model was constructed of galvanized sheet iron supported on sheet metal buttresses. It was expected that several sizes of channel would be tested and the buttresses were designed so that various cross-sections of channel could be obtained by slight changes of the position of the buttresses. The ogee crest of the structure was made of paraffin. The transition from the trapezoidal section of the side channel to the circular 50 ft. diameter section of the tunnel was constructed of reinforced concrete. Three windows were cast into the top through which the flow in the transition could be observed. These windows could be closed with concrete plugs accurately cast to fit the shape of the top of the transition. The same pyralin tube which formed the model of the vertical shaft and tunnel for the glory hole model was used for the inclined and horizontal tunnels of this model. The slope of the inclined tunnel was 45 degrees with the horizontal and the bend between the two sections of tunnel was formed of two of the 22½ degree bends.

Scope of Experiments

Soon after this model was completed it was decided to increase the capacity of each spillway from 100,000 sec. ft. to 200,000 sec. ft. This necessitated a redesign of the spillway and while the studies to develop the most economic design for this size were under way, an extensive series of experiments were carried out on this model to check the theory of the side channel spillway and to investigate a number of ways of reducing its cost.

The basic theory of the side channel spillway was developed by Mr. Julian Hinds* and is based on the assumption that all the energy of the water falling over the spillway crest is dissipated as heat and the flow down the spillway trough is caused only by the water surface slope in this channel itself.

An extensive series of experiments were performed to test the reliability of this theory for various conditions of flow in the spillway. There were four series of these tests, the conditions of each being as follows:

I The side channel with trapezoidal cross-section as designed, with various discharges.

II Various discharges with submerged weirs across the downstream end of the spillway channel to change the conditions of flow in it.

III A channel with the same sides as the original design but with a flat floor at a higher elevation formed by a false bottom.

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*Side Channel Spillways -- Julian Hinds. Trans. Am. Soc. C.E., Vol. 89 (1926) P. 881

IV A channel with practically the same sides as the original design but with the bottom formed by a circular arc tangent to the sides. The bottom grade of the channel of this model was slightly higher than the original design.

Hydraulicly the side channel spillway is a very inefficient device, as so much of the energy of the water which falls over the crest is used up in heat and does not cause flow down the side channel. If part of the energy could be used in producing flow along the side channel, a greater velocity in it would result and a smaller and cheaper channel could therefore be used. To a certain extent this can be accomplished by means of guide vanes or baffles which direct the flowing water after passing over the crest into a direction more or less parallel to the center line of the channel. Different types and combinations of vanes were tried to determino how much could be accomplished in reducing the spillway cost by this means.

One of the uncertainties of spillway design is the allowance which must be made for the air which is entrained in the side channel, due to the turbulent conditions of flow existing therein. The theory as developed by Hinds considers only solid water, but if a quantity of air is entrained in this water its volume is correspondingly increased and the size of the channel necessary to carry it is therefore presumably increased also. Measurements were therefore made of the air entrained in the flow through the model.

Observations on Model of First Design

The observations to test the side channel theory consisted in measurements of the cross section of flow in the side channel for various discharges through the model. For this purpose two parallel bars were placed along the sides of the channel at about $1\frac{1}{2}$ ft. above the level of the crest. These bars were carefully leveled so that their tops were in the same horizontal plain. By means of a point gage extending down from a movable beam spanning the channel from one bar to the other, the elevation of the water surface at any point in the channel could be determined. Ordinarily cross-sections were taken at each point corresponding to an even station on the prototype or at 20-inch intervals.

In each section enough points were determined to permit the surface to be accurately plotted. The cross-sections were plotted to scale and the area of each was determined with a planimeter. A typical set of cross-sections is shown on Plate IV. This is for a discharge corresponding to 140,000 sec. ft. on the prototype.

From the cross-section area, knowing the bottom width and side slopes, the mean depth of flow was computed. In the computations this value was used for the depth of flow in the channel.

By means of piezometers, openings in the floor of the channel, connected to the glass tube monometers, observations were made of the downward pressure on the bottom at four points along the channel.

Results on Model of First Design

The results of observations on the model of the original design is shown on Plate V. The first diagram on this page shows the mean water surfaces for the various discharges observed, between 50,000 and 200,000 sec. ft. The remaining diagrams of Plate V give analyses of the individual runs. For each discharge is shown the average water surface, determined as described above; the theoretical water surface, where it could be computed; the depth of water equivalent to the downward pressure on the channel bottom, as determined by the readings of the piezometers; and the critical depth of flow for the given channel section and discharge. The level of the water surface in the reservoir above the spillway, the elevation of the spillway crest and spillway floor is also indicated.

The theoretical water surface was computed by the method given by Hinds*, which is based on his assumption that all

* Trans. Am. Soc. C. E., Vol. 89 (1926) p. 898-899

the energy of the water flowing over the crest is lost. These curves were usually assumed to start at the height of the observed water surface at Station 7+00, the lower end of the overpour section of the side channel, or the lowest station at which the cross-section was observed. The computation of these theoretical water surfaces is made by a cut and try step method very similar to that in the ordinary cut and try step method of computing a backwater curve. In fact, the operation is really the computation of a backwater curve using the laws of inelastic impact instead of the Bernoulli Theorem. In such a curve it is necessary to have a starting point, and since backwater curves can be worked upstream more effectively than downstream, the logical place to start was at the known elevation at the lower end of the channel. The conformity of the observed results with the theory would then be shown by the agreement between the theoretical and actual water surface profile.

It was not always found possible to determine the curve beginning at Station 7, since it was not possible to compute the surface curve where it passed through the critical depth. In these cases the curves were started at a station further upstream, the location of the point in each case being indicated on the diagrams.

The flow line for 50,000 sec. ft. discharge was so near the critical depth that no backwater curve could be computed. The same was true for the 75,000 sec. ft. discharge. In this case a computation was made starting at Station 4+00 and working in both directions. It departs considerably from the observed water surface below Station 4+00, indicating that where the flow is near the critical depth the water surface cannot always be computed accurately in this manner.

For discharges of 100,000, 125,000, 140,000, 150,000 and 175,000 sec. ft. there is a satisfactory agreement between the observed and the computed water levels, indicating the substantial soundness of the basic theory. For the 200,000 sec. ft. discharge the computed line falls considerably above the observed at the upper end. This may be accounted for in the fact that in the computation a uniform discharge per foot of crest was assumed, while in the actual case the crest was submerged to sufficient depth to reduce the flow at the upstream end.

The line representing the piezometric pressures on the bottom of the channel falls uniformly below the observed mean water surface. This has been assumed by some experiments to be due to the air content of the water, but measurement of air by other apparatus has shown that this is not the case. The cause of the difference is probably the fact that the depth of water over the piezometer openings, which were on the center line of the channel was less than the mean depth of flow in the channel.

The experiments with a weir across the lower end of the channel the results of which are shown on Plate VI also indicate the substantial accuracy of the basic theory. In the setup also the flow for the 50,000 sec. ft. discharge was too near the critical to enable a water surface to be computed, but for the 75,000, 100,000 and 125,000 discharges there is a satisfactory agreement between the computed and observed water levels, except at the upper end of the channel, where a wave was formed which made measurement difficult. For the 150,000 sec. ft. flow the crest at the upper end of the channel was so deeply submerged that the flow over it was less than assumed in computing the water surface, which accounts for the difference between the computed and observed water surface.

The results on the experiments on the channel made by placing a false bottom in the spillway channel, as shown on Plate VII also confirm the theory developed by Mr. Hinds. Where the water surface could be computed there was a good agreement of the computed with the observed water surface. For the 150,000 sec. ft. discharge the crest was so deeply submerged that the flow over the upper end of the crest was not as great as assumed in computing the water surface, and therefore the computed and observed water surfaces differ considerably.

The experiments on the channel formed by placing a round bottom in the original spillway also in general substantiate the fundamental theory. The results of these tests are shown on Plate VIII. For a discharge of 100,000 sec. ft. however, a peculiar wave formation occurred, the explanation of which is not apparent.

Experiments on Effect of Vanes

A number of experiments were conducted to determine to what extent it was possible to decrease the cost of the spillway by introducing vanes into the currect to deflect the water in the direction of the spillway channel, and thus increase the efficiency of the spillway. Without vanes, all the energy of the water falling over the spillway crest is dissipated and flow along the spillway channel is caused only by the slope of the water surface in the channel itself. With vanes it is possible to deflect some of the water falling over the crest in the direction of the channel, thus increasing the velocity of the water in the channel and permitting the use of a smaller and cheaper trough.

The first series of vanes tried were placed on the bottom of the spillway channel. Observations with color injected into the water and with a current direction indicator showed that the stream of water falling over the weir plunges to the bottom of the channel, thence moves across the bottom to the other side, and up that side. In the first test 8 vanes were placed on the bottom (Run No. 21) and their effect noted. More vanes, to a total of 22 were then added (Run No. 22). The results of these tests are shown on Plate IX. The average water surface for these conditions was not appreciably lowered, showing that the velocity of flow in the channel was not materially increased. It was concluded that the vanes were not sufficiently inclined downstream. Runs No. 23, 24, 25 and 26 were then made with various greater inclination of the vanes. These all showed lower water surfaces than with no vanes, indicating that the vanes were effective in increasing the velocity of flow in the channel. In order to indicate the extent to which it would be possible to lower the water surface if all the energy of the water falling over the crest were ~~was~~ used in producing velocity along the channel, the surface which the water would have if there was no loss of energy was computed. The position of this surface

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A series of vanes were also placed upon the ogee crest. The first of these were plane surfaces curved in the direction of the channel. (Runs 27 and 28). As the water passed off the side of these vanes, a set (Run No. 29) curved up at the edge was tested. This gave a better recovery, but in neither case was the saving enough to justify the expenditure necessary to construct the vanes.

Experiments were also performed with various kinds of longitudinal vanes along the bottom and rear wall of the spillway, both with the trapezoidal bottom and with the round bottom. The water impinged on these vanes and did not rise so high against the rear wall of the channel. Their effect was to level out the surface of the flow in the channel, but they caused no recovery of energy. The mean water surface for these conditions was at practically the same elevation as it would have been without the vanes.

Air Content

In measuring the air content an apparatus devised by Mr. D. C. McConaughy was used. In this apparatus the mixture of air and water was taken from the side channel through a pipe, the end of which was placed at the point where it was desired to determine the air content. The water passed into a closed tank of about 50-gallon capacity in which the air and water separated, the air coming to the top. The water was drawn off at the bottom and measured in a water meter, while the volume of air was determined from the difference in the air content of the tank at the beginning and end of the run, as indicated by a gage glass on the side, care being taken to have the air at atmospheric pressure both at the beginning and end of the run.

The air content was measured at two cross-sections, 3+50 and 7+00. (Plate X) At Station 3+50 the maximum air content was 1.41%. This occurred near the center of the channel in the region of the air "rope" which ran down the center of the channel. This "rope" seems to be due to the spiral motion of the water in the channel which causes the ~~higher~~ lighter material of the mixture, ie. the air, to move toward the center of the spiral. At Station 7+00 the greatest air content occurred near the ogee side of the channel and reached a maximum of 6.7%. At this end of the channel the fall of the water between the weir crest and the channel was greater and carried in large quantities of air.

Flow in the Tunnel Bends, Model C 2.

In order to determine what the action of the water would be in the horizontal bend in the spillway tunnel on the Nevada side of the river, several models of different degree of curvature were experimented upon. The first of these models were made with one of the curved sections of the pyralin tube. This had a central angle of $22\frac{1}{2}$ degrees and a radius (to the center line of the bend) of 150 ft. These dimensions did not exactly agree with any plan for the tunnel, but served to show what the action would be in a bend of short radius. As the water entered the bend it tended to preserve its original direction of motion, but being restrained by the wall of the pipe, was forced up on the outside of the bend. As the velocity of motion was high in comparison to the radius of the bend, the water was forced up rapidly, creating a velocity in a circumferential direction which was sufficient to produce a centrifugal force large enough to keep most of the water at the outside of the pipe for a complete revolution. The combination of this circumferential velocity and the longitudinal velocity caused the water to take a spiral path around the outside of the pipe. Near the bend nearly all the water seemed to be against the outside of the pipe, but further downstream less seemed to follow the outside and more flowed along the bottom until about 8 diameters below the bend the spiral motion around the outside of the pipe ceased. The bend shown in the contract drawings had a central angle of 32° 15' and a radius of 819 ft. on the center line. A model of this bend was constructed of 10" steel pipe. In order that the action of the water might be observed, the wall of the pipe was cut away with an acetylene torch, to as great an extent as possible without permitting the water to escape. In this bend the water also tended to pile up on the outside of the bend, but the radius of the bend was greater with respect to the velocity and the water was not forced so high nor so rapdily upward. The circumferential velocity thus produced was much less than for the 150 ft. radius bend and was not sufficient to give a centrifugal force large enough to keep the water on the outside of the pipe. The water rose to near the top and then fell toward the bottom of the pipe at the same time continuing its motion in a longitudinal direction. The water which fell had however a component of flow across the pipe which tended to make some water flow across the pipe and pile up on the inside of the bend to a certain extent, but much less than on the outside of the pipe. Figure 1 illustrates the conditions of flow just described. In order to try to improve the conditions of flow in the bend, one with a radius corresponding to 1600 ft. was built. This model was constructed of wood and concrete. Figure 1 shows this model carrying a flow corresponding to 150,000 sec. ft. The conditions with this model did not seem to be appreciably better than for the 819 ft. radius bend. Another model was constructed on a compound curve, the first portion of the curve had a radius of 1600 ft. (Prototype scale) with a central angle of 21° 27' and the second portion a radius of 2400 ft. with a 7° 09' central angle. Figure 2 shows this model with a 50,000 sec. ft. discharge. As neither of the greater radius bends seemed better than the one originally selected, the latter was adopted for test in the Montrose 1:20 scale model.

In order to determine definitely the path taken by the water in passing through the bend, the inside of the bend was coated with a thick layer of white lead paint. While the paint was still soft water was permitted to flow through the pipe for a short time. This caused the paint to form in ridges indicating the direction of flow. Figures 3 and 4 show the lines in the model of 819 ft. radius bend for discharges of 100,000 and 200,000 second feet respectively.

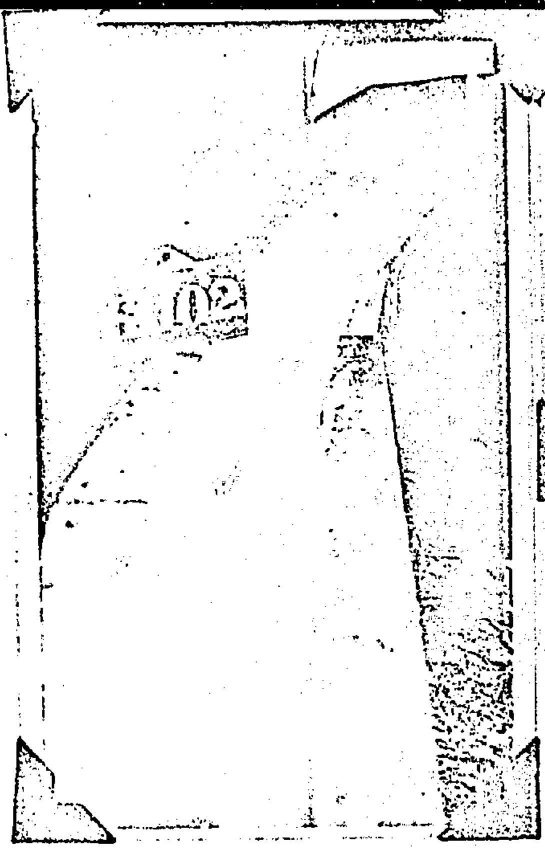


Figure 1. Flow in tunnel bend.
R = 1,600, Q = 150,000 sec. ft.

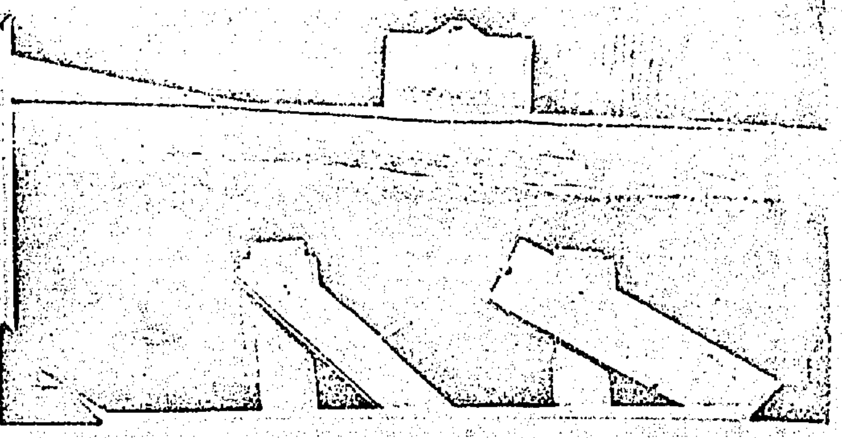


Figure 3. Lines showing direction of flow in tunnel bend. R = 819, Q = 100,000 sec. ft.

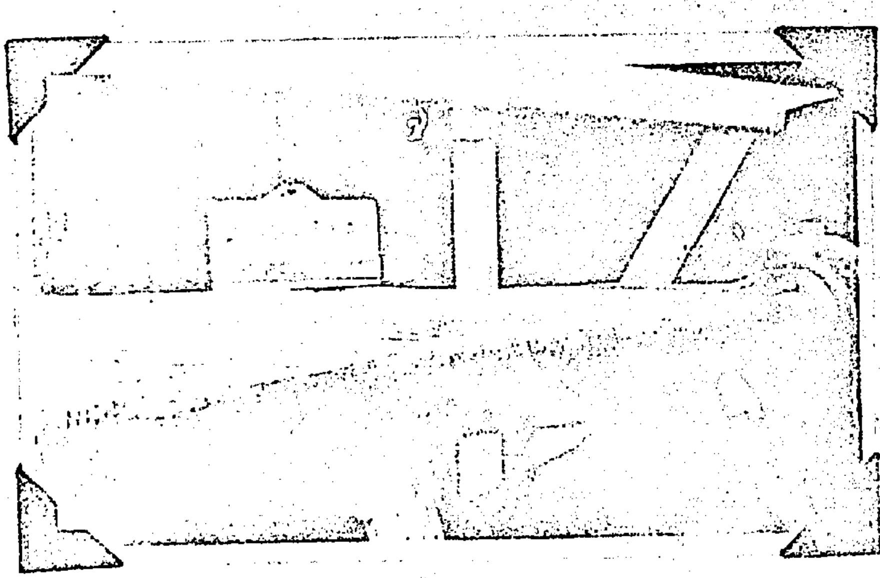


Figure 4. Lines showing direction of flow in tunnel bend. R = 819, Q = 200,000 sec. ft.

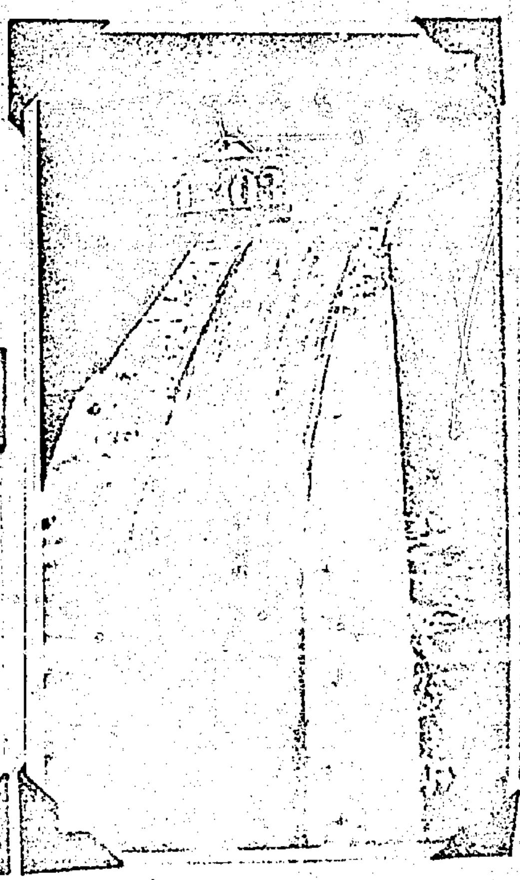


Figure 2. Flow in tunnel bend. Compound curve R = 1,600, R = 2,400, Q = 50,000 sec. ft.

STONEY GATE MODEL C-3, 1:60 SCALE

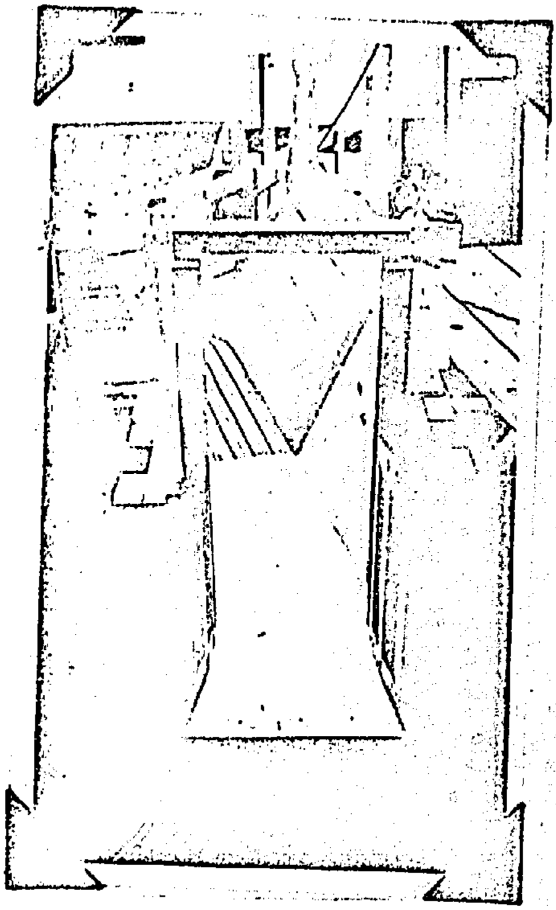
One of the contemplated designs for the Hoover Dam Spillways was a side channel spillway layout with a large Stoney Gate at the entrance to the channel, details of which are shown on Plate No. XI.

In July of this year a model of this design was built on a scale of 1:60 or 1 inch equals 5 ft., and installed in the Hydraulic Laboratory of the Colorado Agricultural College at Fort Collins, Colorado.

The scale proportions gave a model about seven feet long and one and a half feet in height, the width of the Stoney Gate being one foot. For maximum flow the depth of the water over the crest was approximately two inches and the quantity of water handled was about seven and one-half cubic feet per second.

The model was constructed of wood framing covered with galvanized iron for the most part but the accurate curve of the spillway crest was carved from a block of wood, also the warped section of the channel below the gate was shaped from boards stacked up and screwed together.

Representation of the rock topography in front of the spillway was obtained by cribbing up boards of a thickness corresponding to a four foot interval, which were cut along the contour lines.



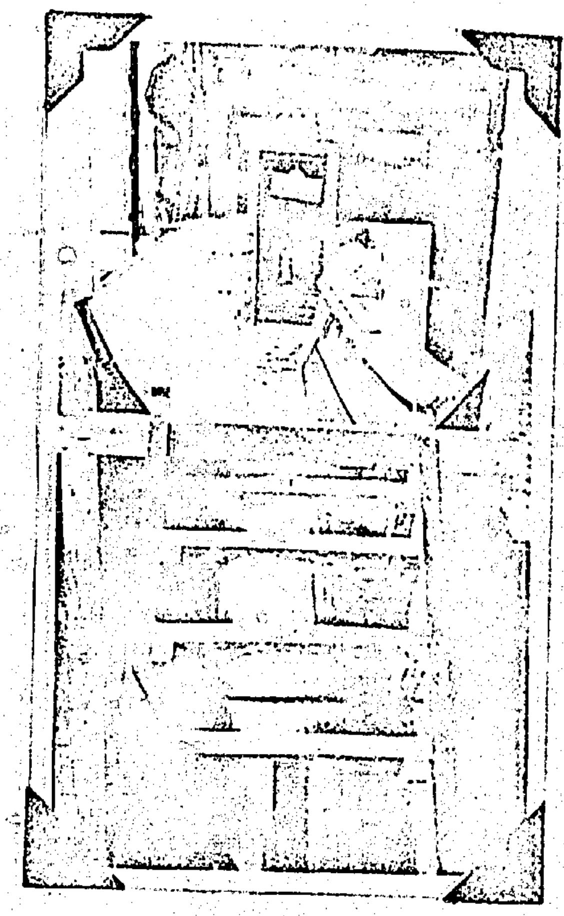


Figure 5. Looking downstream showing framing and channel transition.

Figure 6. Looking upstream showing crest block in place and tunnel transition.

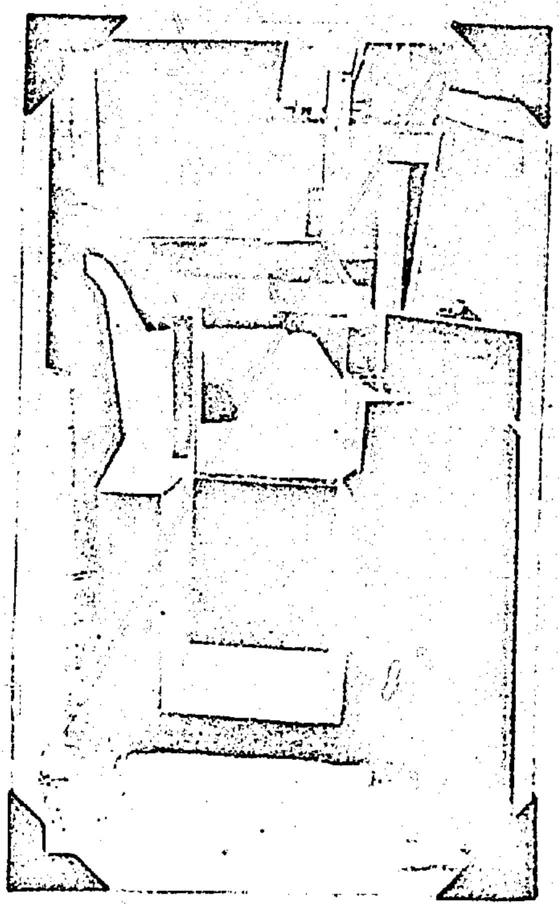


Figure 7. Completed model without contours.

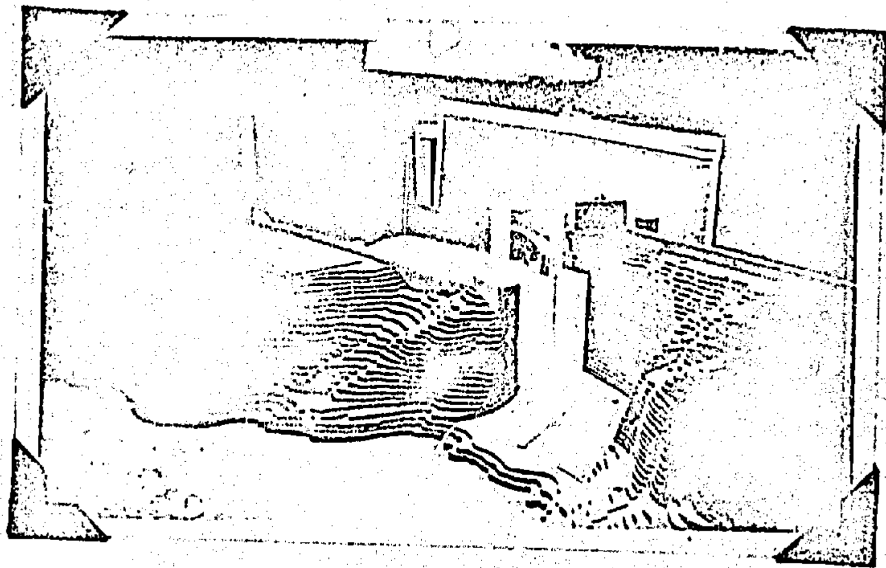


Figure 8. Completed model with contours.

9

Figure 9. Piezometer gage board.

The transition from the sloped wall side channel to the circular tunnel was shaped between collars at various intervals by wood staves which were then smoothed down and painted and the outside coated with a thick layer of paraffin to prevent swelling.

Construction of the model is indicated by Figures 5 to 8, inclusive.

The control and measurement of water for this setup was the same as the preceding ones. Water surface elevations in the forebay and pressures at various points in the channel were obtained by piezometer openings, located as shown on Plate XI, which were connected by rubber hose to the glass tubes, mounted as shown on Figure 9.

The investigations included experiments on:

I. <u>The Original Design</u>

A. To check the theory.

1. Flow over crest only.

   a. Coefficient of discharge.

2. Flow under gate only.

   a. Capacity at various openings.

3. Flow over crest and under gate during rising flood.

   a. Observation and record of operation.

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II. Alterations on the Original Design.

A. To improve flow conditions.

1. Placing dentated sill on channel floor.
2. Extending gate pier upstream.
3. Decreasing width of gate channel excavation.
4. Lengthening channel transition.
5. Extending end piers of the crest upstream.
6. False floors in channel to obtain data for further design.

Flow over Crest Only

The flow from the gate was blocked off and various discharges up to a flow corresponding to 200,000 c.f.s. were recorded. At the same time the depth of water over the crest was measured through piezometer openings located upstream therefrom. Water surface elevations were also taken in the channel by means of the point gage.

Figures 10 and 11 show the flow conditions for discharges of 55,000 and 200,000 c.f.s. respectively. It will be noted in the pictures that the water tends to pile up on the far wall of the channel and ride down the right side of the tunnel.

Flow Under Gate Only

The crest was blocked off and the gate opened at different stages with the pond kept constant at El. 1232. Quite a vortex was noted in the forebay ahead of the gate at certain discharges.

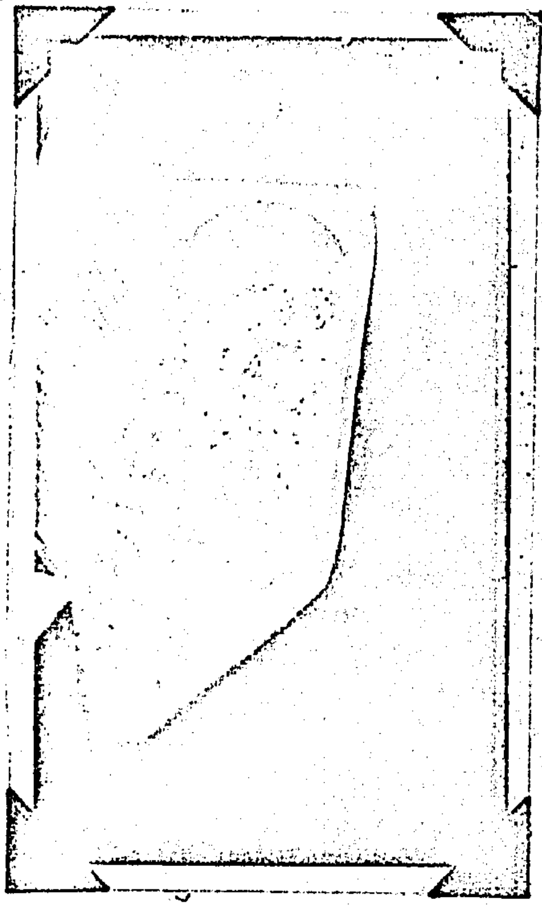


Figure 10. Flow over crest only. Q = 55,000 sec. ft. Pond El. 1232.

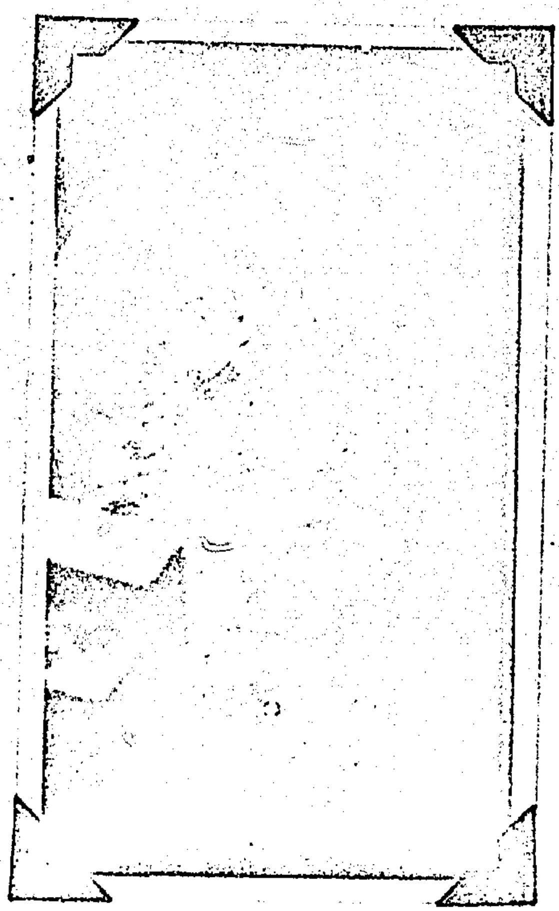


Figure 11. Flow over crest only. Q = 200,000 sec. ft. Pond El. 1247.

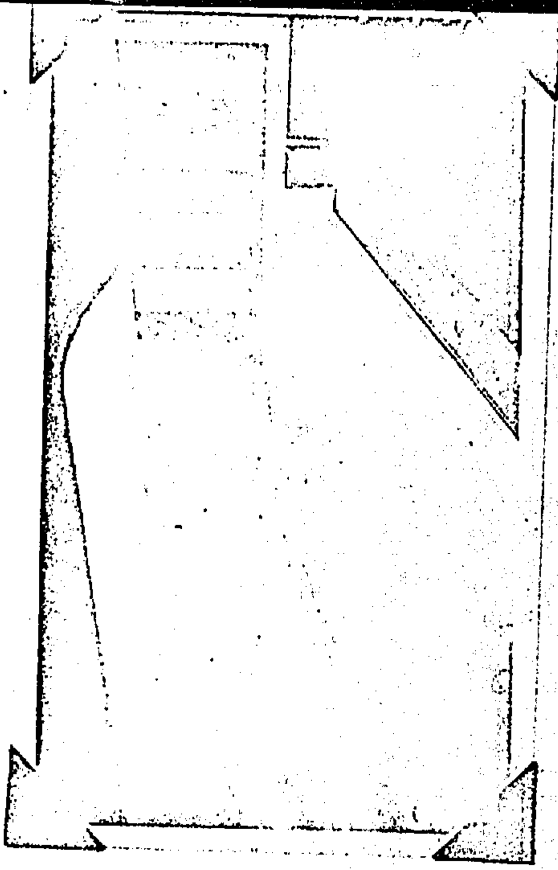


Figure 12.

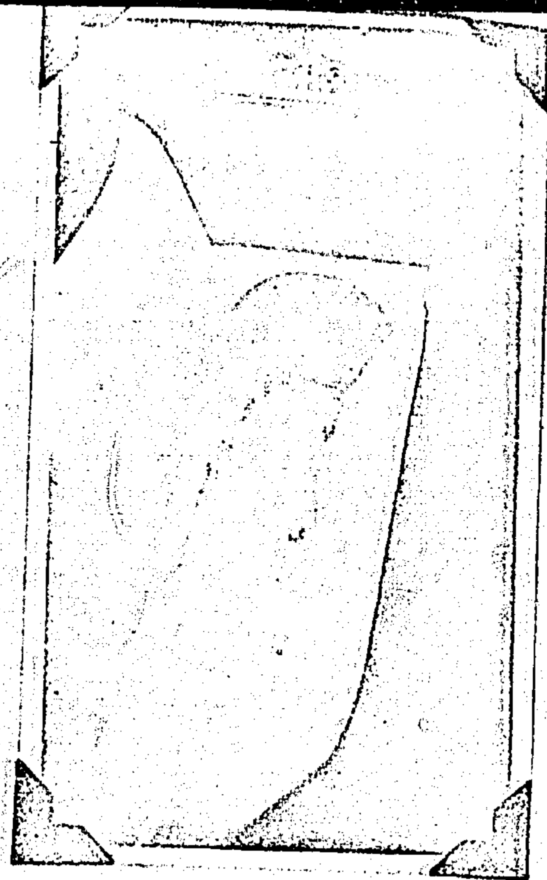


Figure 13.

Flow under gate only.
Q = 60,000 sec. ft.   Pond at El. 1232.

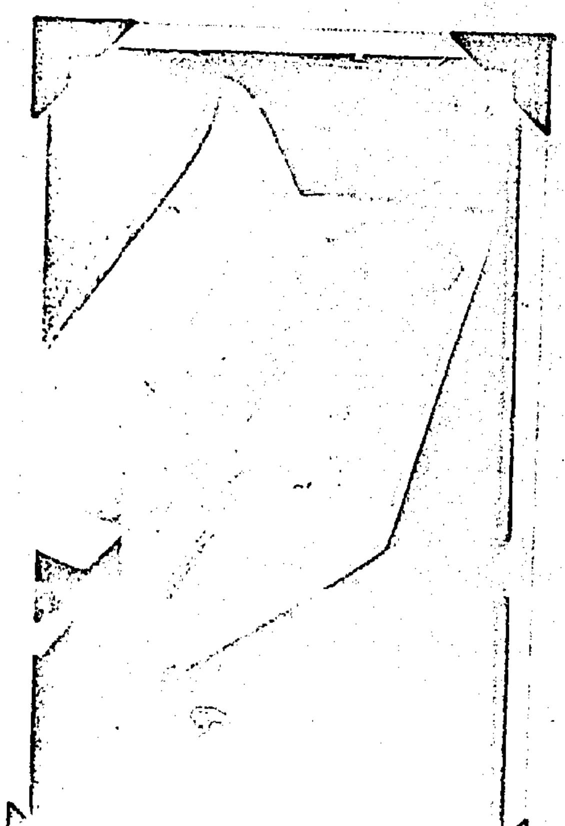


Figure 14.

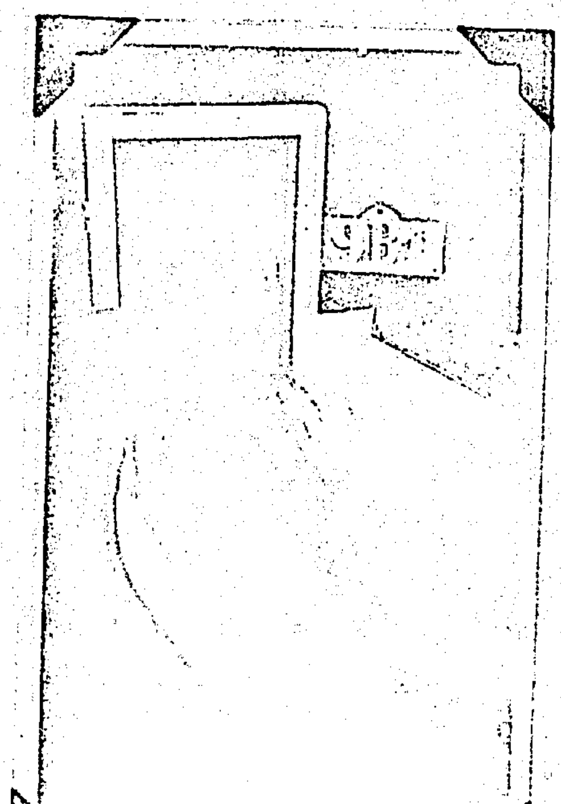


Figure 15.

Flow through gate only.
Q = 190,000 sec. ft.

It will be seen from Figures 12 to 15 that the flow from the gate only is of high velocity and straight line flow except for the component across the channel given it by the curve in the transition below the gate.

Figure 13 may be contrasted with Figure 10 and Figure 14 with Figure 11 in the foregoing section for flow conditions of approximately the same quantity of water discharging.

Flow Over Crest and Under Gate

In this test, rising flood conditions were simulated, that is, the flow was allowed to pass over the crest until the pond reached El. 1232, where it was kept constant by raising the Stoney Gate until the full discharge representing 200,000 c.f.s. was reached. Flow quantities at various forebay elevations and gate openings were recorded and observation of conditions caused by raising the gate noted. Loss in head over the horizontal excavation in front of the crest was ascertained by means of a row of piezometers upstream normal to the crest. To check any tendency of the water to leap the floor of the tunnel transition a row of piezometers were located along the center line and readings taken at the various discharges. To obtain the vacuum under the nappe over the gate and also on the crest wall near the gate where the slope was steep, piezometers were located at these points and their readings recorded.

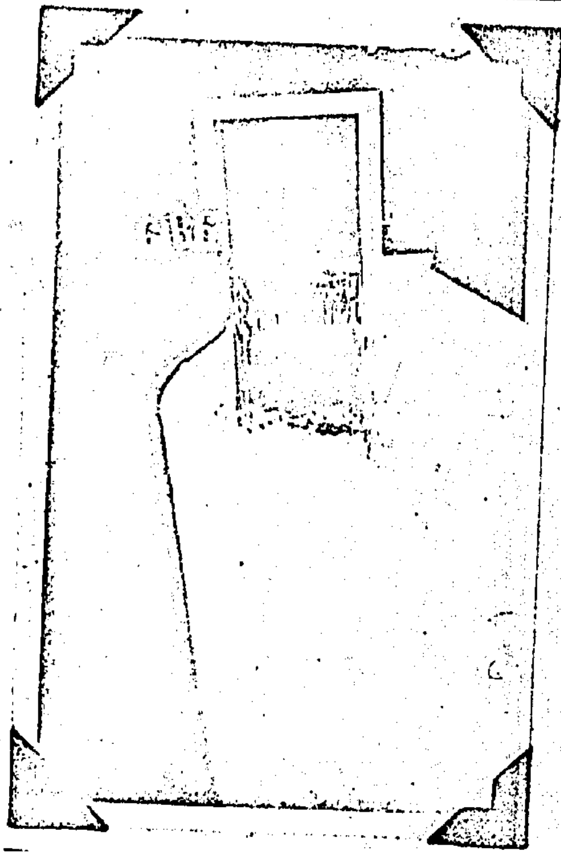


Figure 16. Flow over crest and gate.
Q = 60,000 sec. ft.

Figure 17. Flow over crest and under gate.
Q = 110,000 sec. ft. Pond at El. 1232.

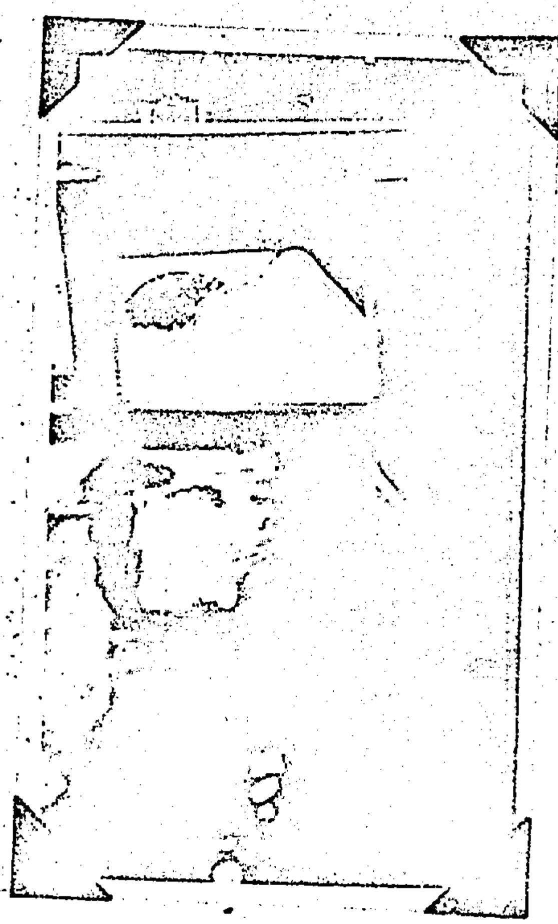


Figure 18. Flow over crest and under gate.
Q = 110,000 sec. ft. Pond at El. 1232.

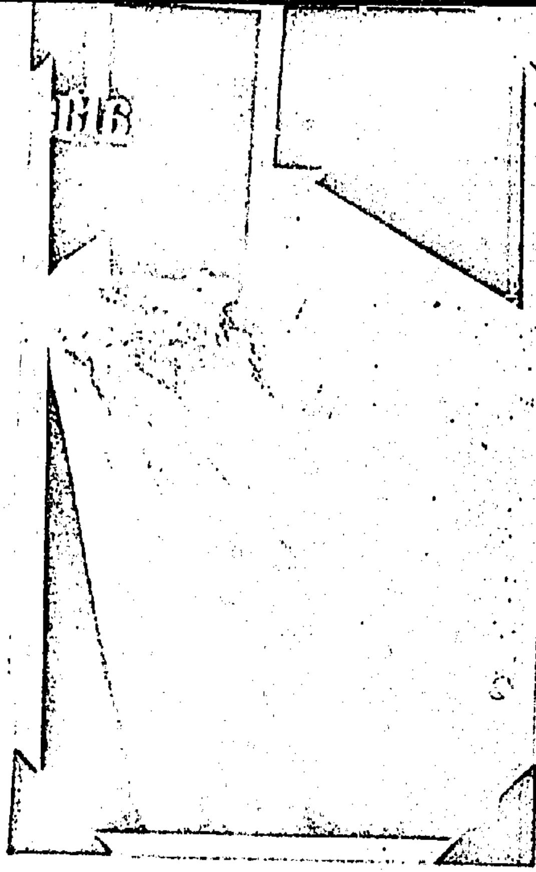


Figure 19.

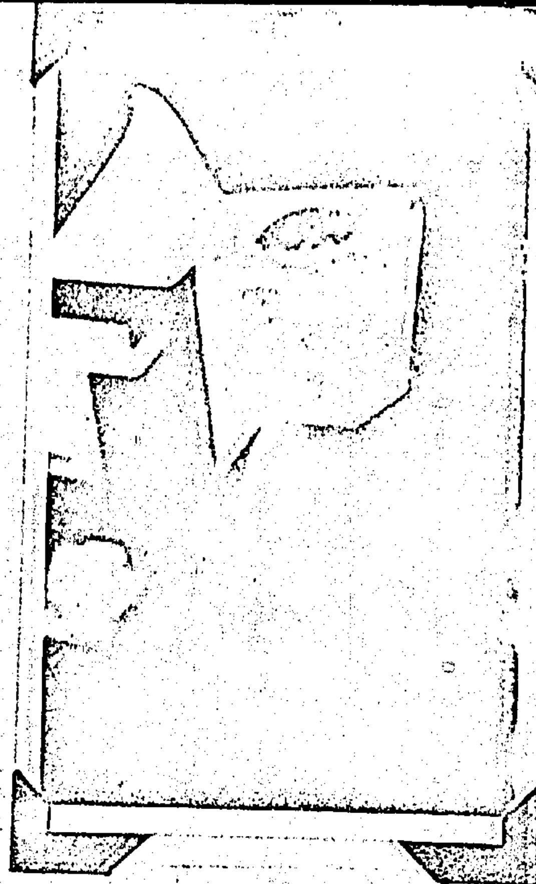


Figure 20.

Flow over crest and under gate.
Q = 160,000 sec. ft. Pond at El. 1232.

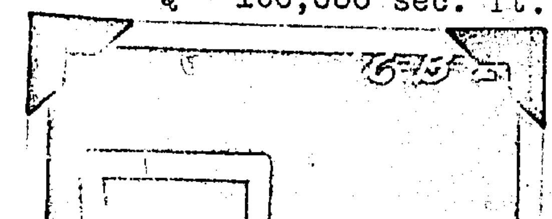


Figure 21.

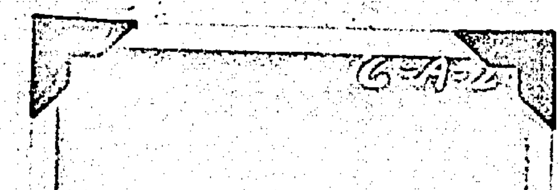


Figure 22.

Flow over crest and under gate.
Q = 200,000 sec. ft. Pond at El. 1232.

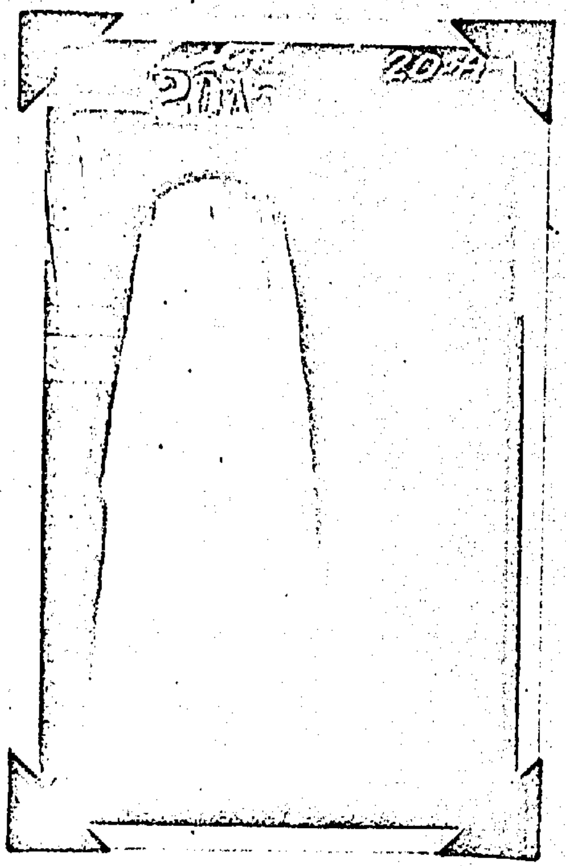


Figure 23.
Over crest and
through gate.

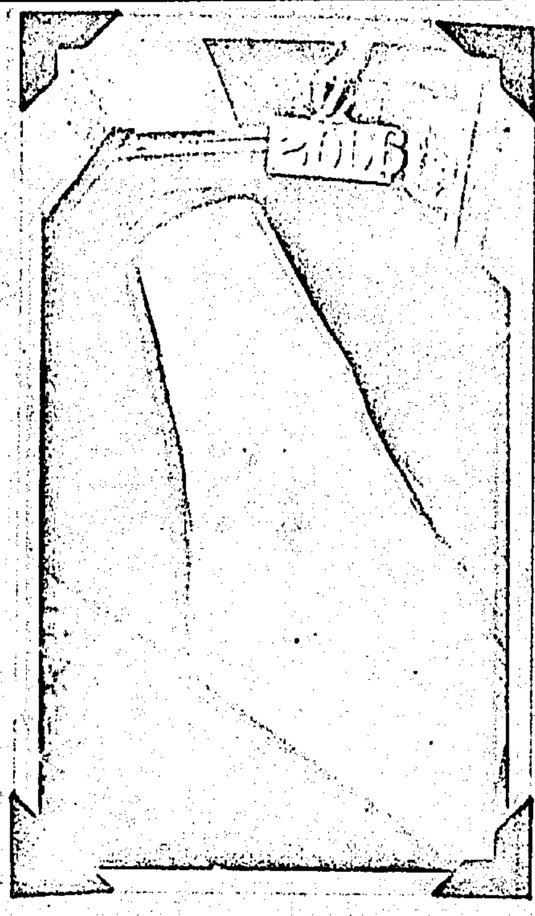


Figure 24.
Over crest only.
Gate blocked.

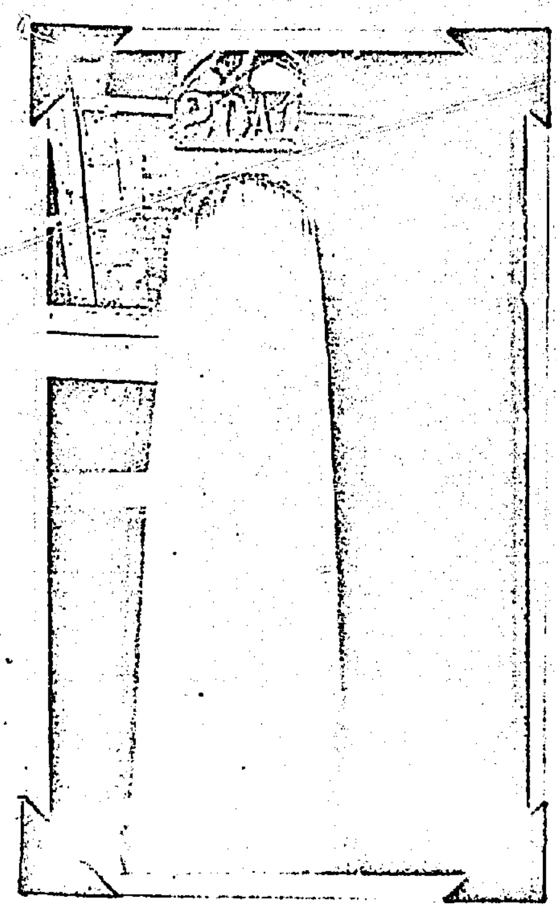


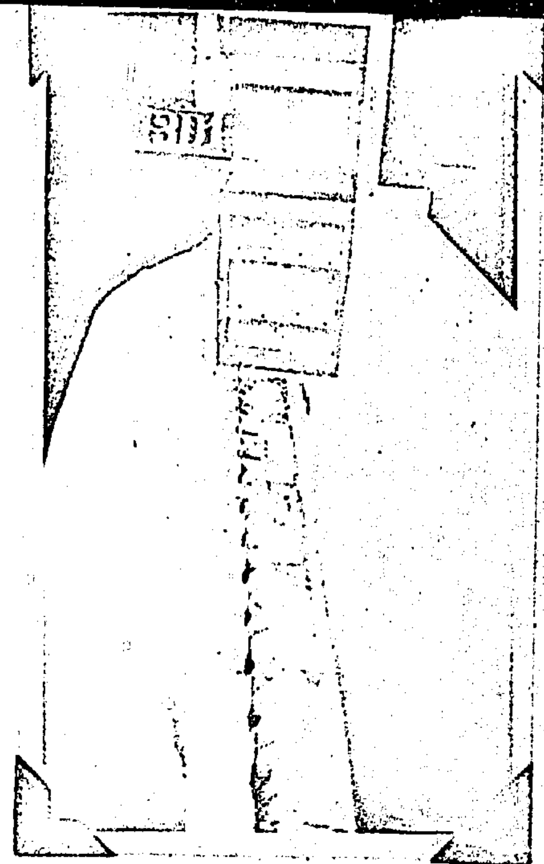
Figure 25.
Through gate only.
Crest blocked.

The conditions for a flow of 60,000 sec. ft. are shown on Figure 16. For this condition water passes over the gate, as well as over the crest.

When the gate is opened slightly, however, there is quite a disturbance due to the high velocity of the water shooting under it, as seen in Figures 17 and 18, representing a 110,000 sec. ft. discharge. As the gate is raised further, however, the flow from it tends to counteract the component across the channel, of the water from the crest, and sweep it downstream, as seen in Figures 19 and 20 (160,000 sec. ft.) and 21 and 22 (200,000 sec. ft.). It is on the smaller gate openings that the disturbance is worse.

The influence of this cross flow from the crest is best illustrated by the flow from the lower end of the transition. It will be noted in Figure 23, Photograph-20-A-5-that the flow from the crest which amounts to about 55,000 c.f.s. has imparted a twist to the jet and this is substantiated by observing the roll to the water when the entire flow is passed over the crest as seen in Figure 24.

It would seem by the pictures that the tunnel was running full, but the main bulk of the water is flowing in about the lower three quarters, the water on top being splash and spray. There is less when the flow is through the gate only as noted in Figure 25.



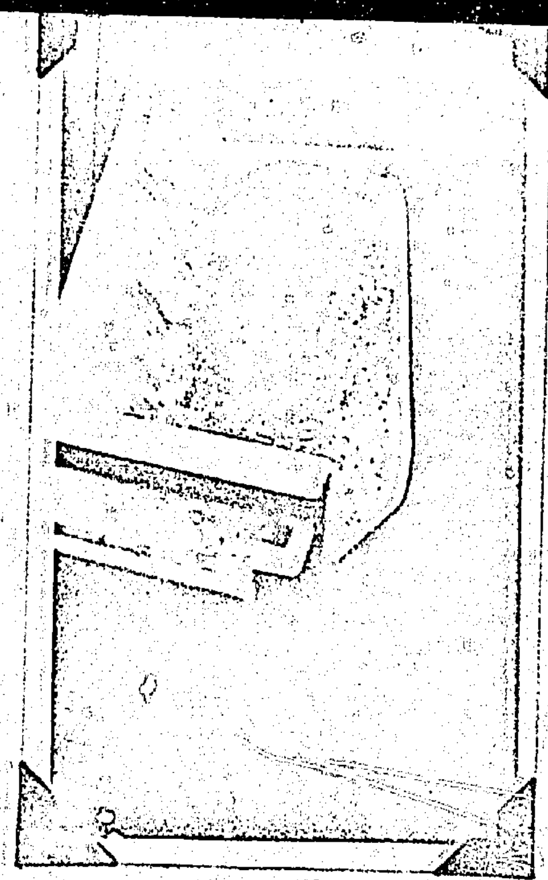


Figure 26.
Height, fifteen feet.

Curved dentates on floor.

Figure 27.
Q = 150,000 sec. ft.

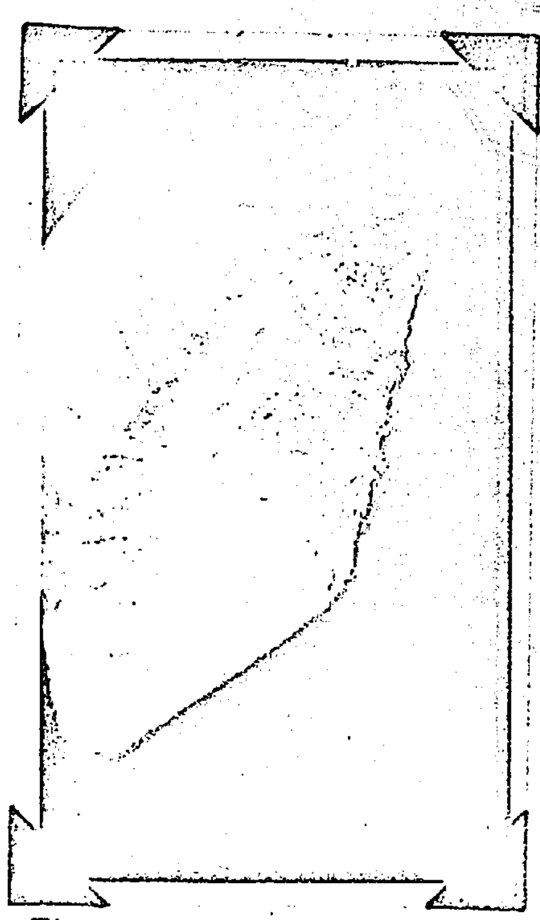


Figure 28.
No baffles. Q = 150,000 sec.ft.

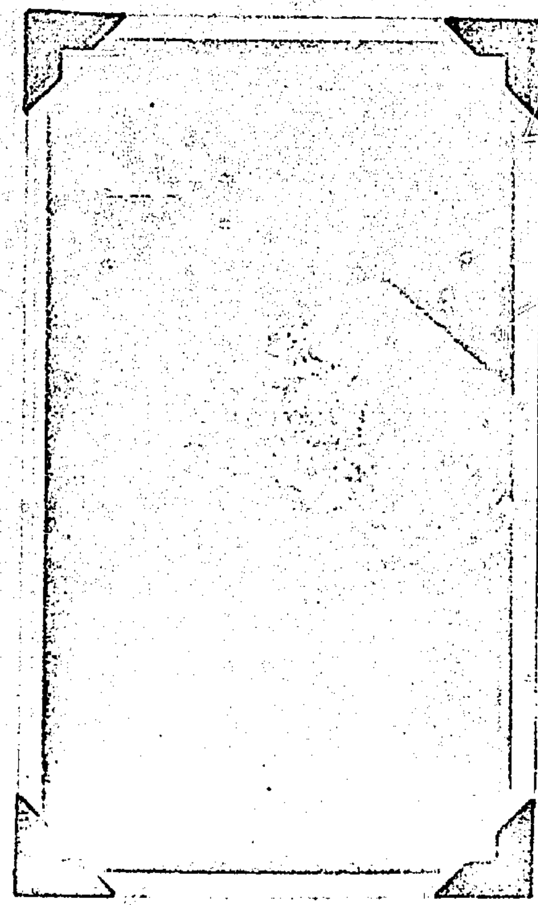


Figure 29.
Curved dentates on floor.
Q= 200,000 sec.ft.

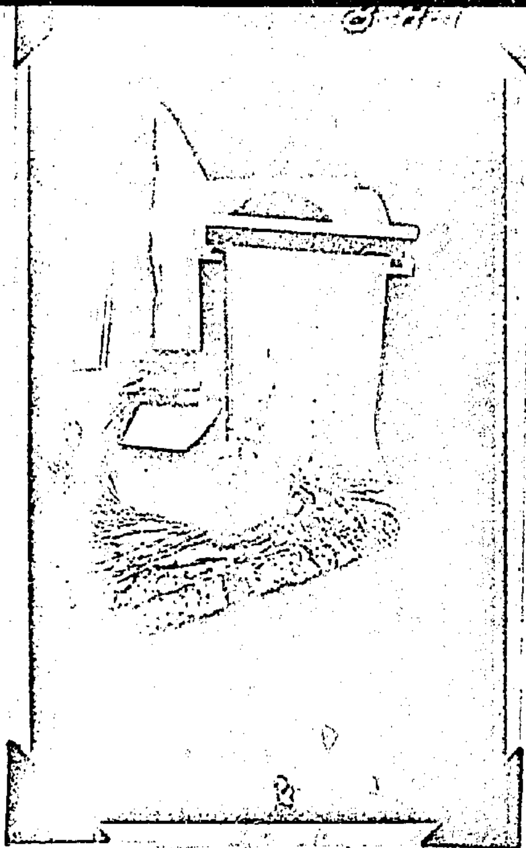


Figure 30. Gate entrance as designed. Q = 155,000 sec. ft.

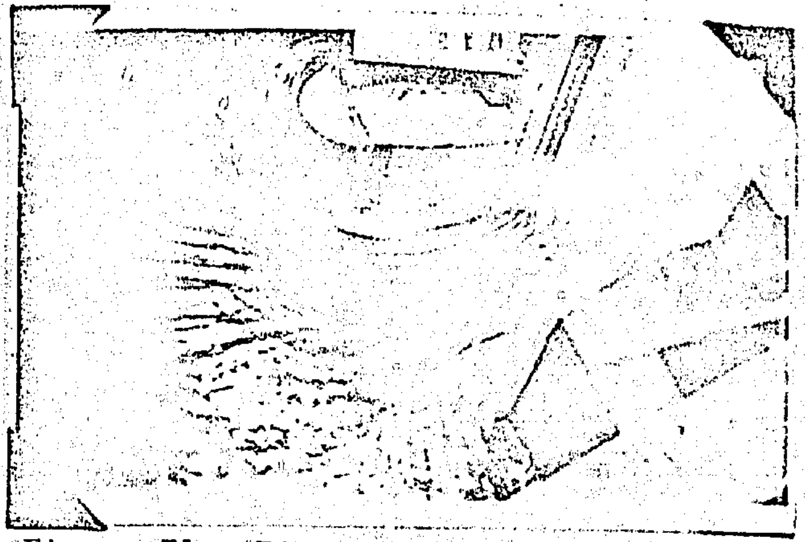


Figure 31. Pier nose rounded. Q = 170,000 sec. ft.

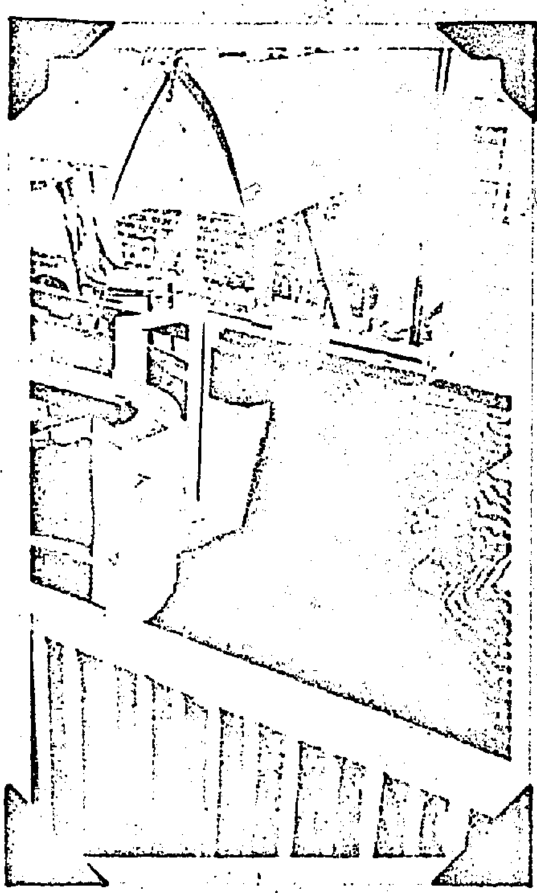


Figure 32.

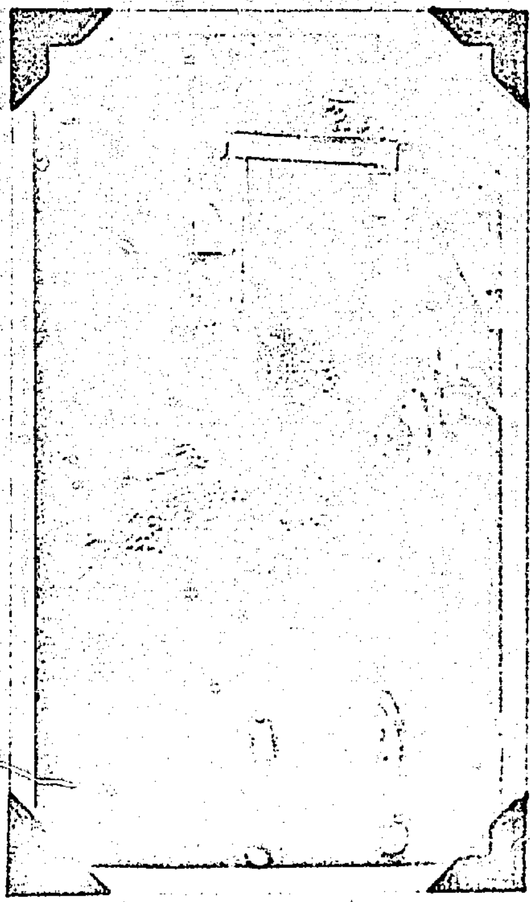


Figure 33.

Pier nose extended upstream. Q = 200,000 sec. ft.

Alterations to Improve Flow Conditions.

Dentated Sill along Channel Floor.

In an attempt to prevent the flow over the crest from piling up along the right wall, two sets of curved dentates were tried along the channel floor, the larger representing fifteen feet height is shown in Figure 26.

The action of the dentated buckets in leveling out the water surface was very satisfactory as shown by comparison of the Figures 27 and 28, showing the same flow (150,000 sec. ft.) with and without the sill. At the maximum discharge, however, the effect is almost lost due to the deep submergence of the baffles as shown by a comparison of Figures 11 and 28.

Gate Pier Extension

At the Stoney Gate entrance there is quite a swirl and eddy loss around the outside pier due to the gate draw down, as shown on Figure 30. This was not alleviated by rounding the nose (See Figure 31) but was overcome by extending the curve of the pier upstream. If the extension was taken to the entrance of the intake excavation, the difficulty is entirely overcome. (Figure 32 and 33) It is believed that this extension might be shortened considerably and still obtain good results, but of course the further upstream the extension is made the more the flow is improved.

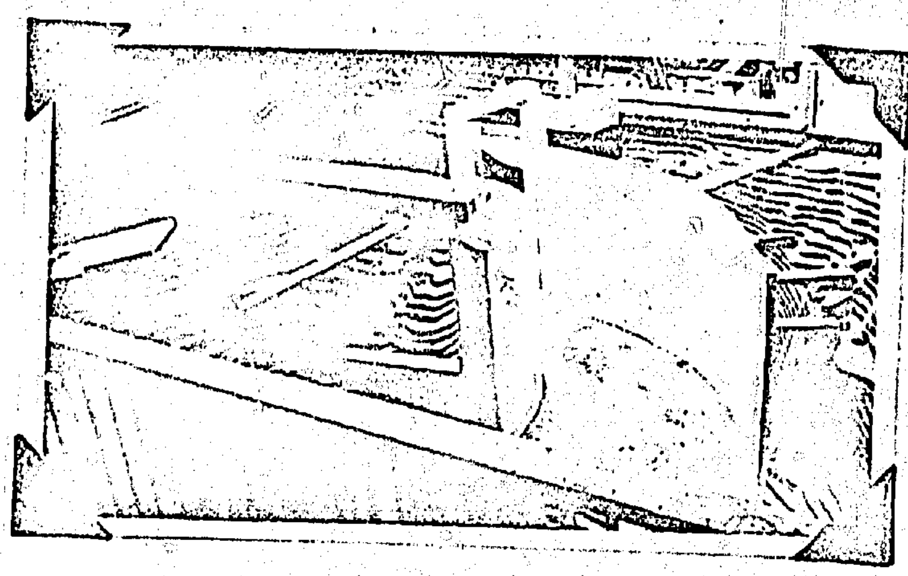


Figure 34. False wall in Gate Excavation. Left Pier Extension.

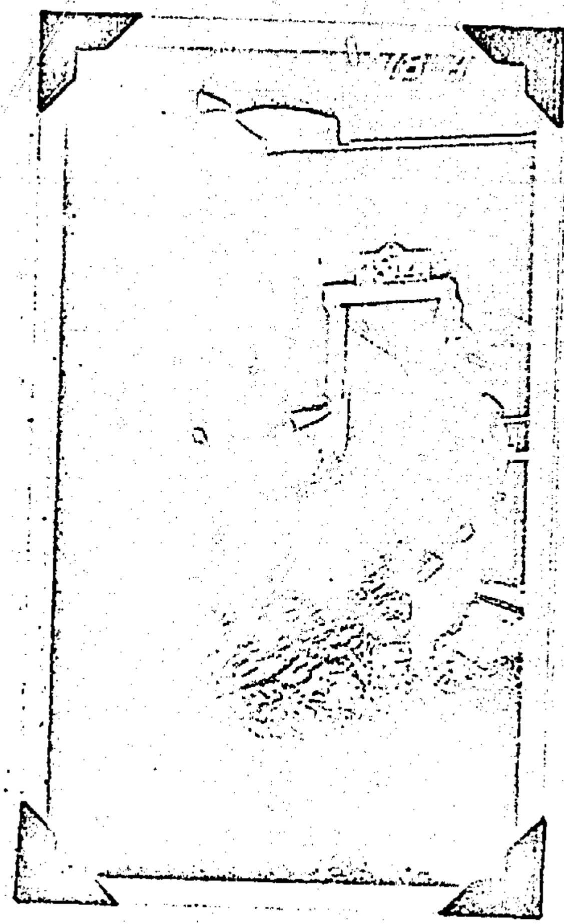


Figure 35. False wall in Gate Excavation. Left Pier Extension. Q = 200,000 sec. ft.

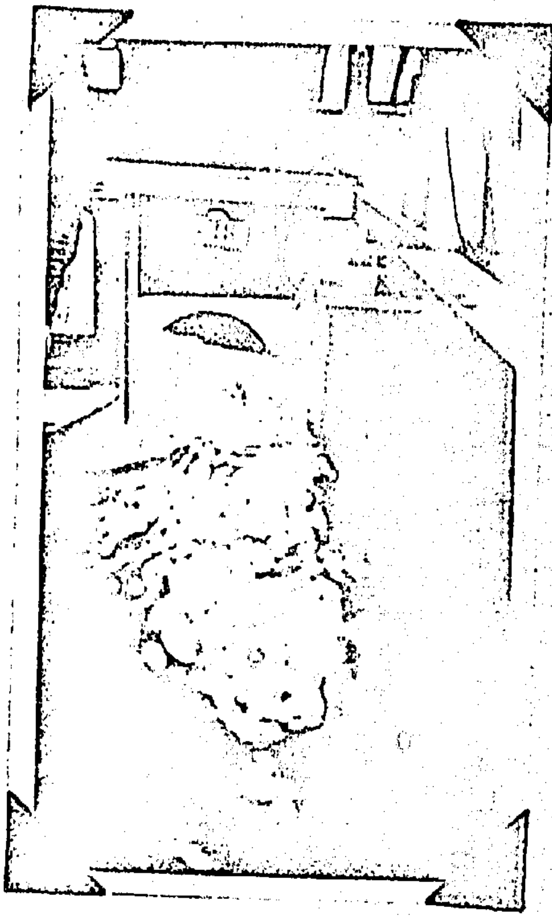


Figure 36. Looking downstream. False side in channel transition. Q = 170,000 sec. ft.

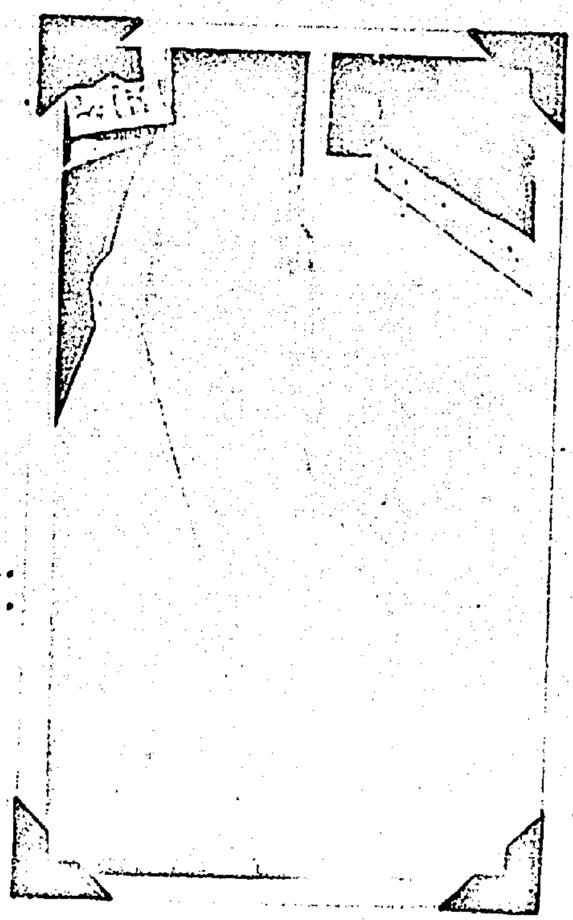


Figure 37.
Looking upstream. False side in channel transition. Q = 170,000 sec.ft. approx.

Gate Intake Excavation

The experiments showed that the excavation for the Stoney Gate Intake was too large as there was dead water along the right bank. Four different settings were tried, gradually drawing the right side wall in, and it was found that the velocity of the beginning could be doubled, without any detriment, but with improvement in the flow lines. With the extension on the left pier, as illustrated in Figures 34 and 35, the flow into the gate was balanced and smooth.

Transition - Gate to Channel

The cause of the turbulent conditions of flow in the spillway channel did not seem to lie in the turning of the water in front of the gate, where the velocities were comparatively low, but below the gate in the side channel proper. The radius of the transition seemed too sharp causing the water to pile up on the right wall, thence it was deflected across the channel to the crest side causing irregular flow down the tunnel as shown on Figure 14.

This was improved to some extent by lengthening the transition section, making the radius of the turn flatter, which was effected by the false wall. The result of this change can be seen by comparing Figures 36 and 37 with Figures 14 and 15.

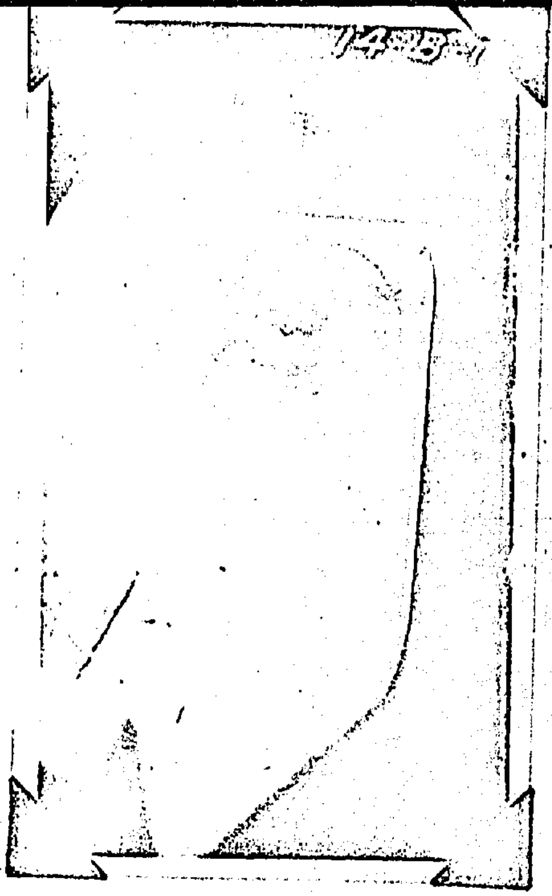


Figure 38.

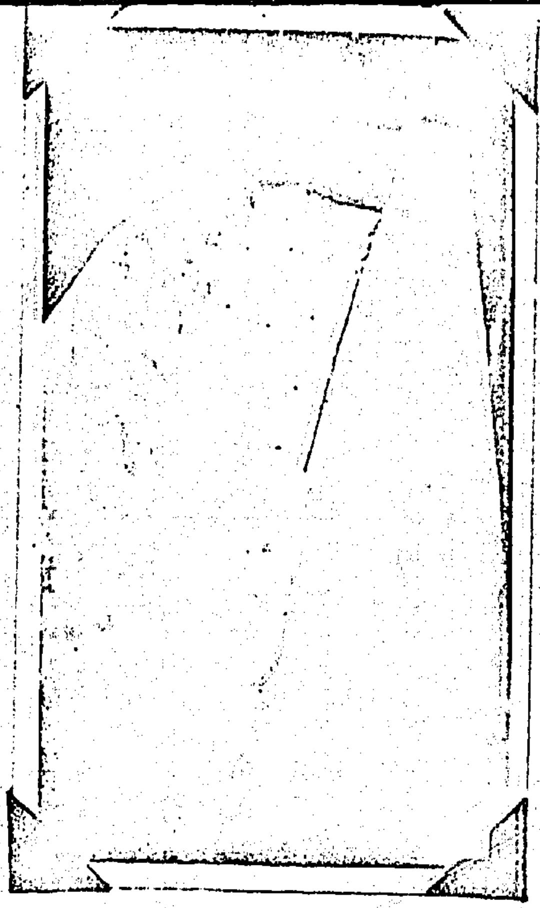


Figure 39.
Floor raised 45 feet at upper end.
Q = 55,000 sec. ft. approx.

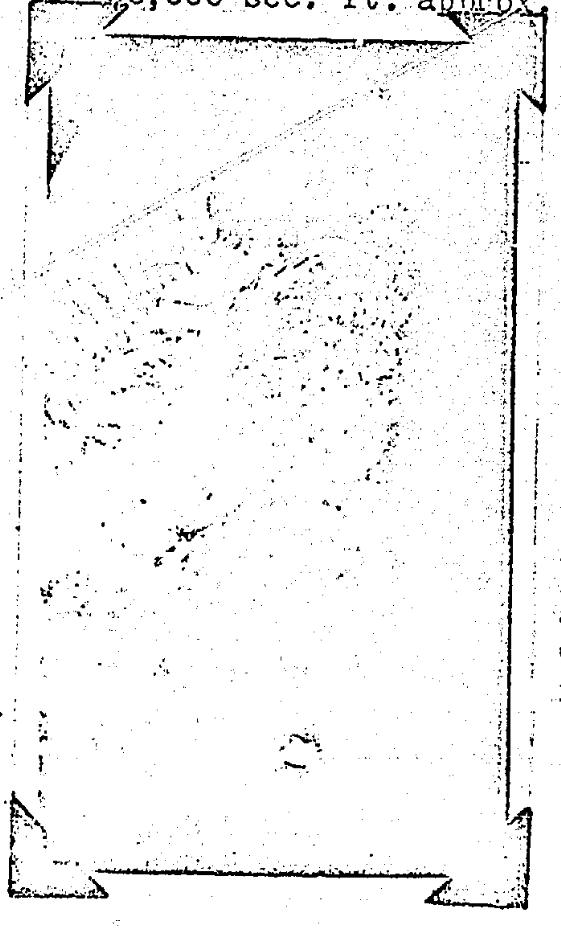


Figure 40. Floor raised 45 feet at upper end.
Q = 200,000 sec. ft. approx.

Upstream Forebay Walls at Ends of Crest

Extension of the end piers upstream decreased the loss in head due to the flow around the corners at those points and increased the discharge slightly. This extension can be easily made on the downstream end as a facing wall along the excavation cut.

False Floors in Channel

To note the flow conditions if the channel floor was steepened, the Stoney Gate was blocked off entirely, and three false bottoms placed in, varying in height 15, 30, and 45 feet above the original floor. At the maximum flow over the crest, for which it was designed, in all set ups, the flow was handled in a satisfactory manner, although the bottom was swept clean at the upper end. See Figures 38 and 39. For a flow of 200,000 c.f.s., however, it would seem that the slope was too steep, resulting in very bad flow conditions down the tunnel. Figure 40 may be compared with Figure 11, where the floor was as originally designed. From a hydraulic standpoint it would seem that the flatter the floor slope, the better.

Stoney Gate Type Abandoned

Although several changes indicated that improvement was possible in the original design of this model, no great improvement was obtained. It is believed that by making the spillway channel straight and throwing all

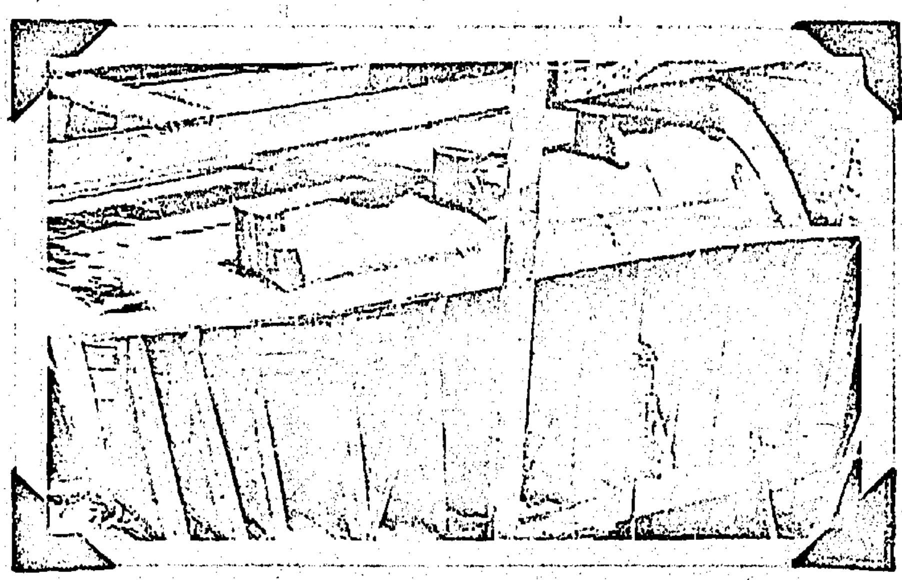


Figure 41. Side view Drumgate model as designed (M-1).

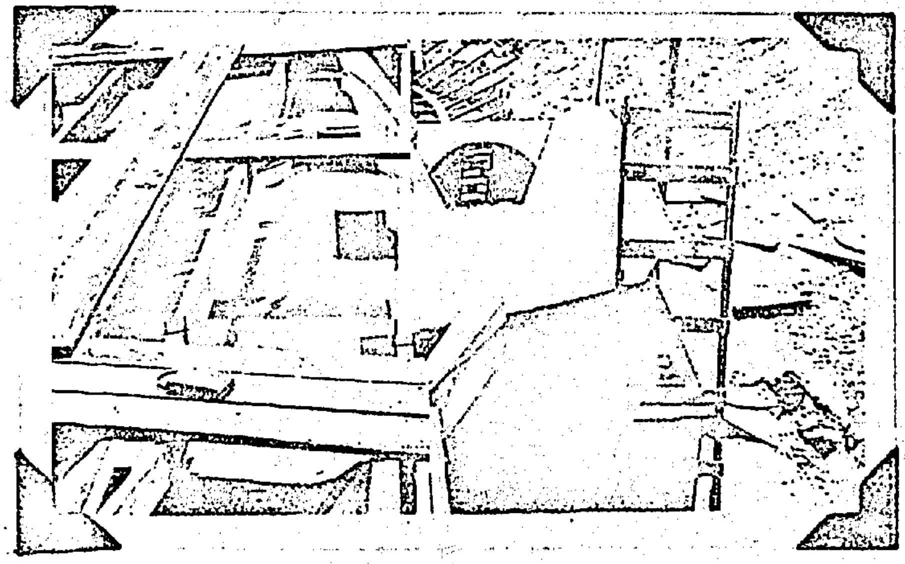


Figure 42. Looking downstream. Drumgate model as designed.

gate structure the water entered a wooden flume, where it turned at right angles and flowed parallel to the canal to the measuring weir. The flume was 12 ft. wide. At the upper end were three sets of baffles which were constructed as a result of considerable experimentation, in such a manner that the turbulence of the water entering the flume would be as far as possible damped out and the water approach the weir with a uniform velocity of approach throughout the channel cross section.

The weir was located 62 ft. below the baffles. It was mounted in a concrete weir box founded on hard shale to prevent settlement. The weir was a duplicate of Francis' suppressed weir except that the length was 12 ft. instead of 10 ft. The height of the crest above the floor was 4.6 ft. and the head was measured at a point 6 ft. upstream from the weir. The crest was formed of a steel angle planed to a flat crest $\frac{1}{4}$" wide. Great care was taken to keep a smooth face on the upstream side of the weir angle and a sharp edge between this face and the top face, in order to insure accurate measurements. The head on the weir was read by means of float gages at each end. These were of a modified Cornell type.

After passing over the weir the water turned at right angles and passed through two sets of baffles into the forebay. This was a wooden channel 8 ft. deep and 39 ft. long expanding in width from 14 ft. to 40 ft. Across the end of the forebay was a bulkhead built to form a model of the topography of the spillway site. In this topography was located the crest of the spillway model. After passing over the crest, the water fell into the spillway channel, along which it flowed into a transition section and thence into the model of the inclined tunnel. The transition was constructed of wood and the inclined tunnel of wood stave pipe. The vertical bend which connects this with the model of the horizontal portion of the tunnel was cast in concrete. The horizontal tunnel, including the horizontal bend of 819 ft. radius, was a 30" wood stave pipe. This pipe discharged into a 4'-0" x 4'-0" box flume which carried the water back to the South Canal. In order to observe the flow in the vertical bend and the wood stave "tunnel", windows were placed in them at intervals.

EXPERIMENTS ON MODEL M-1, ORIGINAL DESIGN

The first drum gate model experimented upon at the Montrose laboratory (designated as M-1) was a 1:20 scale ratio model of the spillway shown in Figures 41 and 42.

The crest was composed of four drum gates 100 ft. long with 10 ft. piers between. The channel had a bottom width varying from 26.5 to 52.4 ft. and a 0.375:1 slope on the side opposite the weir. The bottom sloped steeply, having a fall of 104 ft. in the 403 ft. length to the beginning of the transition section. The dimensions were computed by the methods developed by Mr. Hinds.

This model did not function satisfactorily. Because of the steep slope the velocity of flow along the channel was great, and the cross section of the flowing water was relatively small. The depth of flow was therefore less than in the models previously investigated. The average fall of the water passing over the crest, between the reservoir level and the channel level was also somewhat greater. This greater fall tended to make the water pile up on the side of the channel opposite the crest to a higher elevation. The smaller depth of flow in the channel also offered less resistance to this tendency, since the greater the depth of water the greater is the tendency to dissipate the energy of the overflowing stream. The combination of greater fall and smaller depth of flow resulted in a high wave on the back side of the channel, which caused considerable impact on the end wall at the portal of the tunnel for discharges of 100,000 sec. ft. and over. The conditions in the channel for

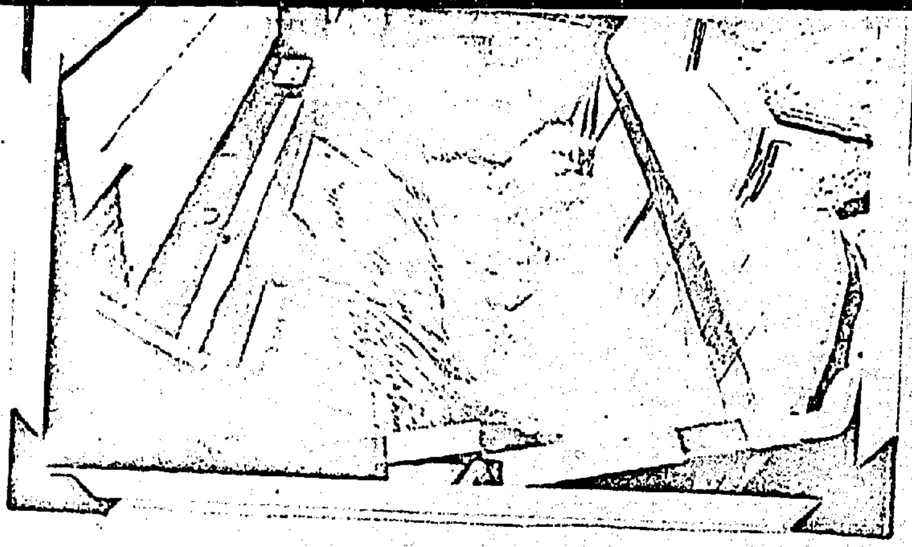


Figure 43.
Original design.
Q = 50,000 sec. ft.

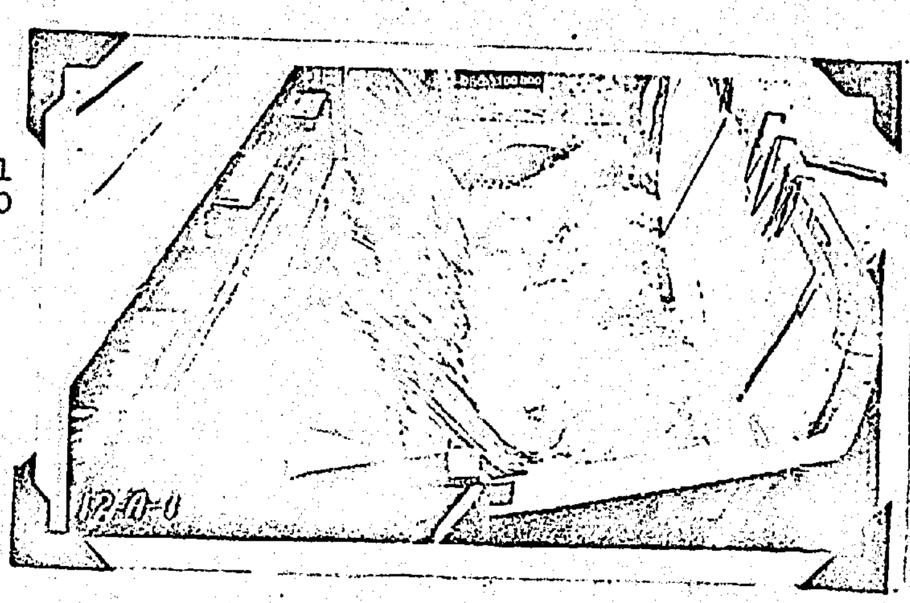


Figure 44. Original design. Q = 100,000 sec. ft.

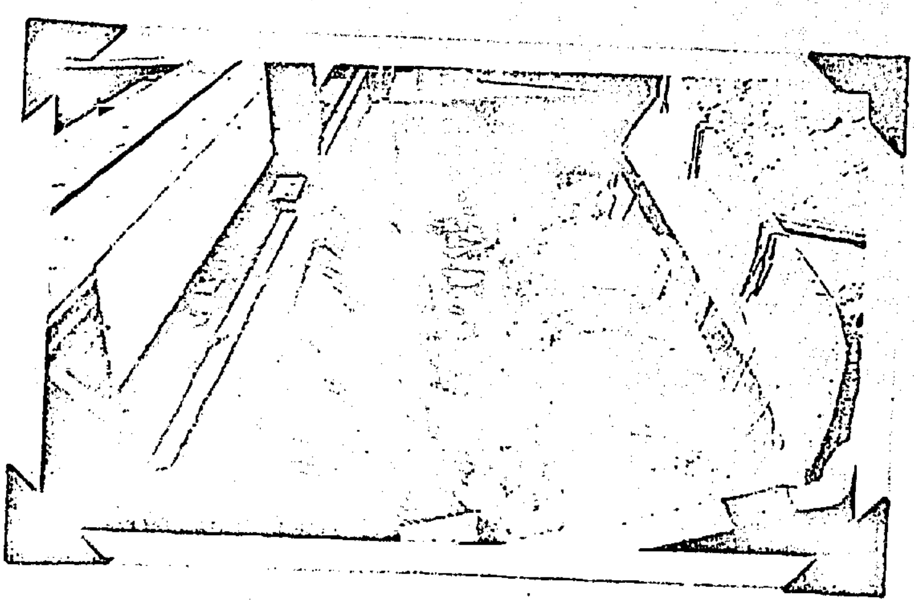


Figure 45. Original design. Q = 150,000 sec. ft.

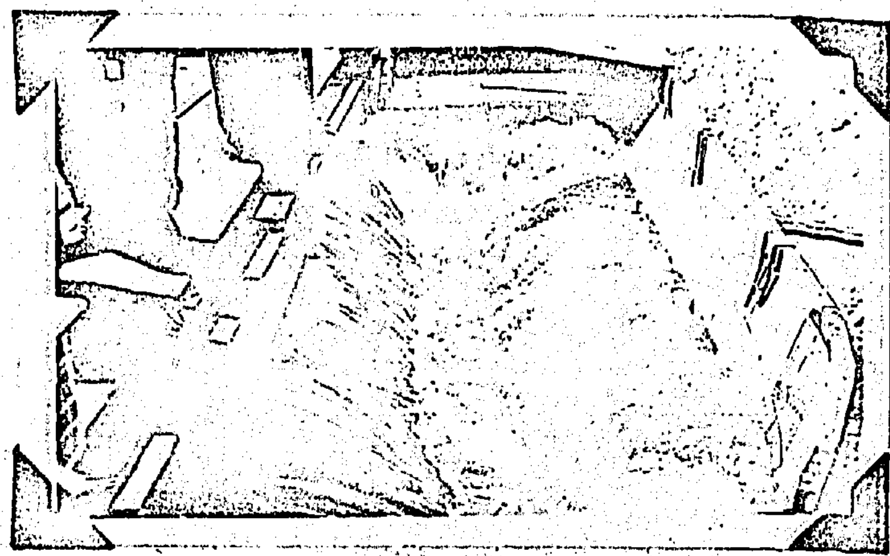


Figure 46. Original design.
Q = 200,000 sec. ft.

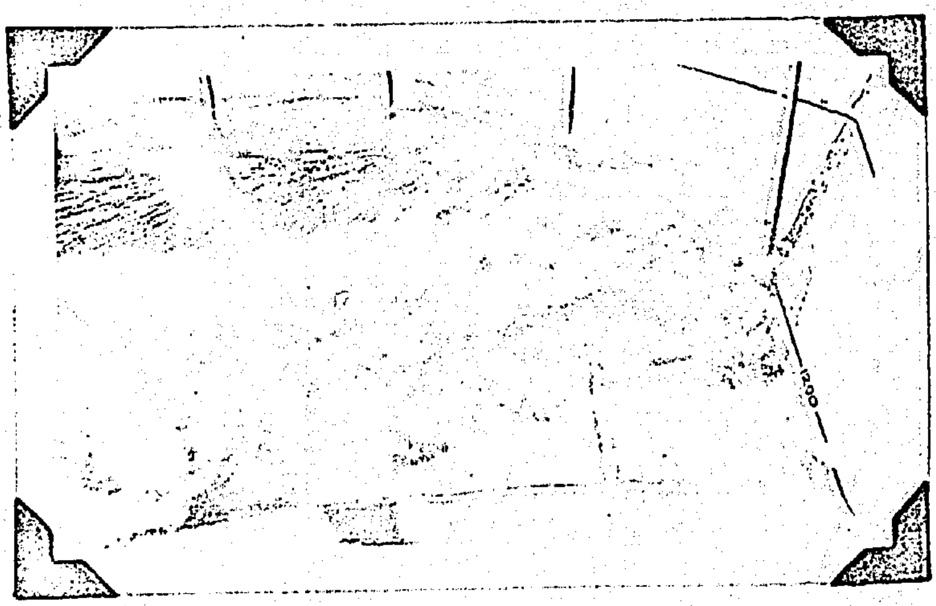


Figure 47. Original design showing wave
at upper end. Q = 100,000 sec. ft.

discharges of 50,000, 100,000, 150,000 and 200,000 sec. ft. are shown in Figures 43 to 46, inclusive. At the upper end of the channel at discharges up to 150,000 sec. ft., the water flowing over the crest swept the floor clear and formed on the back side of the channel a ridge resembling a wave on an ocean beach just as it breaks, shown in Figure 47. The high fall and slight depth of water also caused a decided spiral motion in the flow down the channel, which set up considerable commotion in the transition section at the top of the inclined tunnel. They also caused severe splashing at the vertical bend at the bottom of the inclined tunnel. Water began to splash out of the lower of the two windows in the bend at a discharge of 110,000 and severe splashing occurred at 120,000 sec. ft. With a discharge of 200,000 sec. ft. there was a very distrubed condition of flow at this point and the splashing, together with the air brought down by the water exerted considerable pressure on the windows in the top of the bend. The flow through the pipe, as observed through the windows, was very much disturbed and near the end of the tunnel gave rise to severe vibration.

The cause of this vibration or thumping is not certain but seems to be related to the well known fact that resistance to flow of water in pipes decreases as the depth of flow increases until a depth of about 0.95 of the diameter is reached, beyond which the resistance rapidly increases. In an inclined pipe into which water is emitted in gradually increasing quantity, when a depth of 0.87 of the diameter is reached, if the water splashes to the top it suddenly fills the whole pipe and in this condition the pipe will not carry as great a discharge as when flowing 0.87 full. As the water flows down the tunnel its velocity is reduced by friction and the depth of flow increases. Near the downstream end, for discharges of near 200,000 sec. ft. the depth approaches 0.87 of the diameter of the pipe. When a surge or splash occurs the flow suddenly jumps to the full condition, instantly increasing the resistance to flow and causing a hammer or thump in the pipe. The flow does not continue at the full depth, however, but quickly drops down to part depth flow. This may be due to air brought in at the upper end of the tunnel forcing itself out at the lower end. Another possible explanation is that the sudden increase in resistance tends to form temporarily a hydraulic jump, which cannot maintain itself because the friction loss to the end of the pipe is not sufficient to create the required back pressure. These changes between full and partially full flow give rise to a series of thumps or blows of varying intensity at irregular intervals, which are felt only near the downstream end of the pipe. For the original design

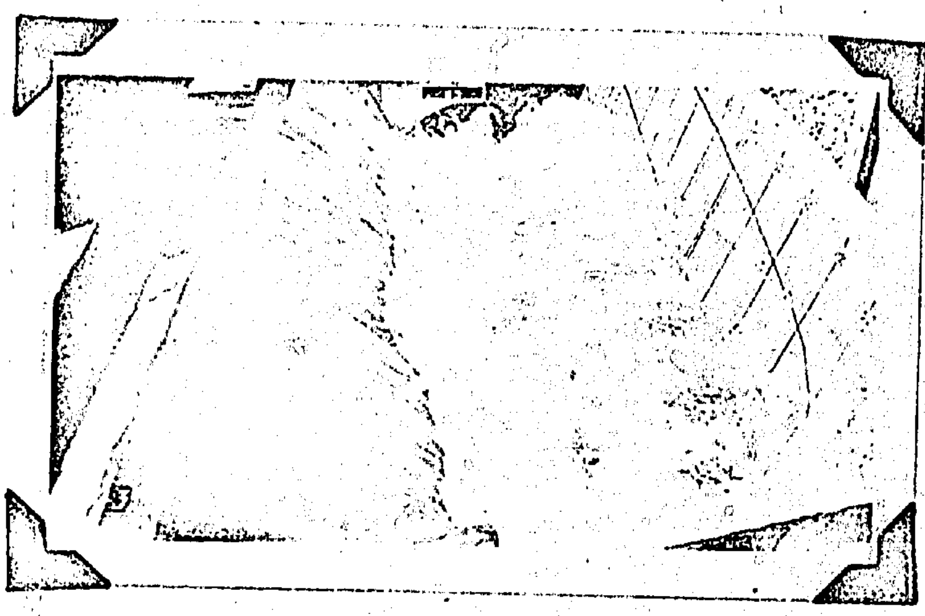


Figure 48.. Raised Floor in Channel.
Q = 90,000 sec. ft.

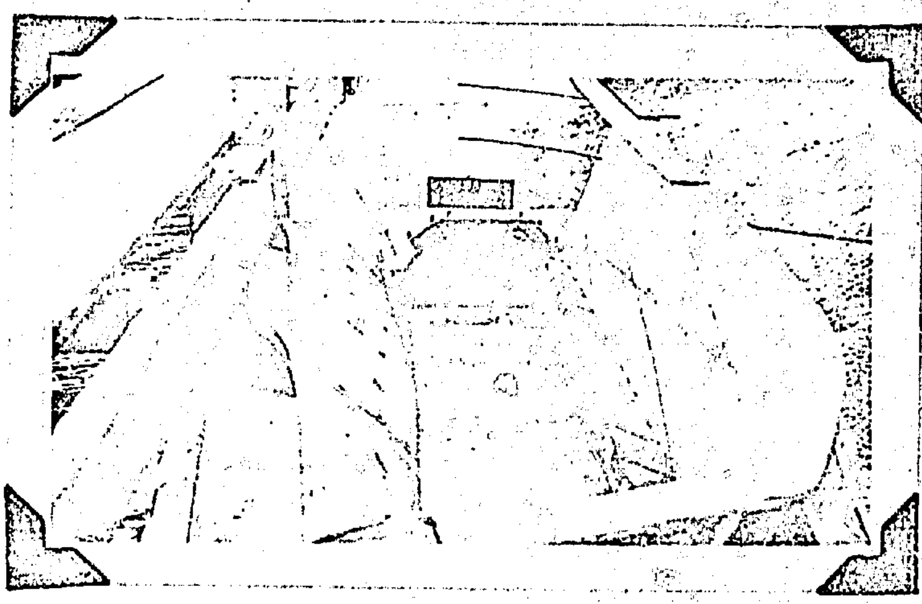


Figure 49. Coping on Back Wall, 20 ft. Flat Top in Transition, #1 Bell Mouth and Raised Bottom.

a severe blow came about once every ten seconds. In the prototype these disturbances would no doubt set up a perceptible jar in the rock of the canyon wall, which would be undesirable.

Raised Floor in M-1 Model

In order to reduce the height of the wave on the back wall of the channel, the floor of the channel was raised, a maximum of 19 ft. at Station 3+00 and sloped from this point to the original bottom levels at the upper end of the channel and down to the beginning of the transition. The grade of this channel is shown on Plate XIII. This decreased the slope of the upper end of the channel, at the same time decreasing the height of the fall of the water over the crest. The conditions of flow in the channel were improved but there was still considerable splashing at the portal of the tunnel. A discharge of 90,000 sec. ft. is shown on Figure 48. The water still rose to an undesirable height against the back wall.

Although the original design was computed to have a cross section just sufficient to carry the 200,000 sec. ft. discharge without causing sufficient submergence of the ogee crest at the upstream end of the channel to reduce the flow over it, it was found that the floor could be raised and the flow over the crest at the upper end still maintained. This is probably due to the fact that the water level determined in the computations is the mean water level in the channel cross section, while the level which determines the submergence effect is the level on the side of the channel near the weir, which is considerably lower than the mean level.

Coping on the Back Wall

In order to reduce the height to which the water rose against the back wall a curved faced baffle in the form of a coping was placed along this wall with the top of the curved face about 20 ft. above the floor. The position of this baffle is shown in Figure 49. This tended to throw the water out toward the center of the stream and reduced the height of the wave against the wall. It considerably improved the conditions at the entrance to the tunnel and reduced the disturbances throughout the tunnel. The conditions of flow with this baffle for discharges of ~~50,000~~, 100,000, and 150,000 ~~and 200,000~~ sec. ft. are shown on Figures, ~~50~~, ~~51~~, 50 ~~52~~ and 51 ~~53~~, respectively.

Flat Top Contraction in the Transition

The original design contemplated on free surface for the water in flowing in the spillway and tunnel.

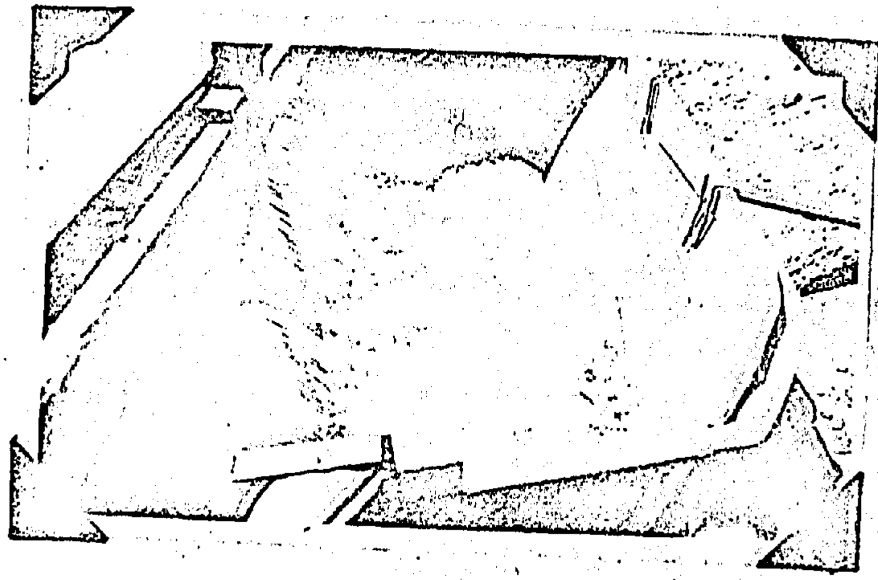
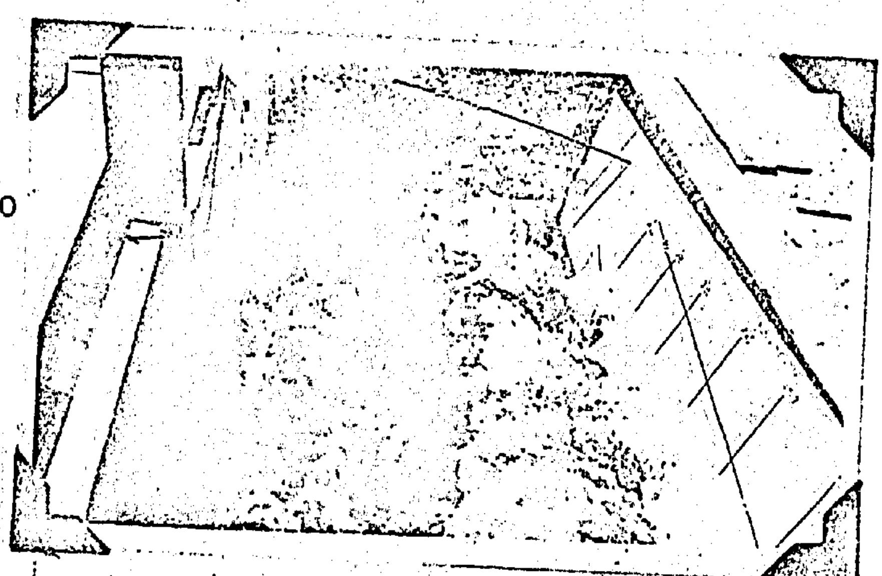


Figure 50. Coping on Back Wall. Q = 100,000 sec. ft.

Figure 51. Coping on Back Wall. Q = 150,000 sec. ft.



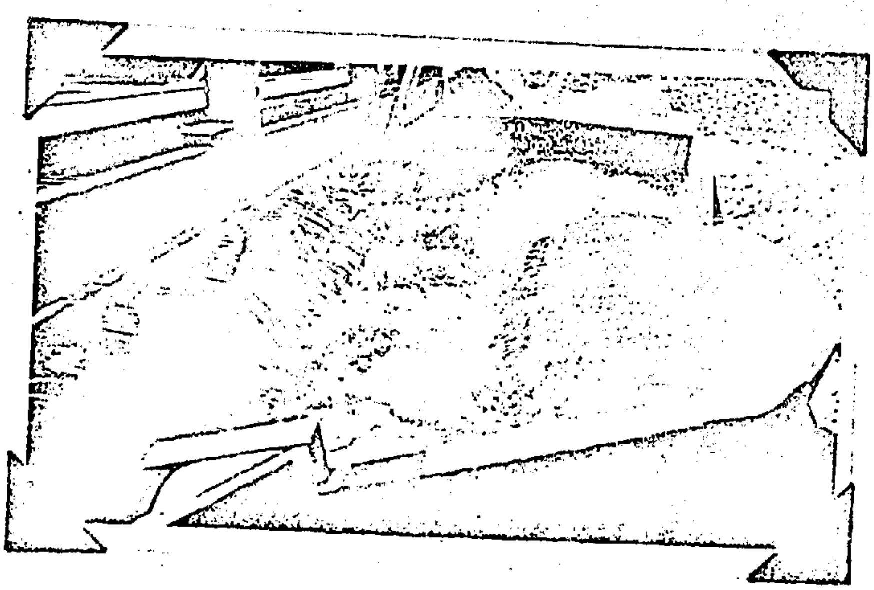


Figure 52. Flat Top in Transition. Q = 200,000 sec.ft.

On account of the turbulent conditions of flow this surface was very rough and produced a great deal of splashing and impact. In order to smooth out the water surface a contraction was placed in the transition section as shown in Figure ~~51~~ 49 and on Plate XIII. This consisted of a flat top in the transition gradually contracting the waterway from 0 at the upper end of the transition to a distance 20 ft. down from the top at the lower end. A vent was built at the lower end to admit air above the water in the inclined tunnel downstream from the contraction. For discharges over about 175,000 sec. ft. this formed an orifice with a horizontal top edge at the upper end of the inclined tunnel and caused the water to start down the tunnel with a flat upper surface, this largely reducing the splashing in the pipe. When this contraction acted as an orifice it raised the surface levels in the side channel, reducing the drop in the water falling over the crest and increasing the depth in the channel, both of which effects tended to improve the flow conditions. Figure ~~55~~ 52 shows the conditions of flow in the channel with the raised bottom but without the coping on the back wall. Although the contraction considerably raised the water level in the channel at the lower end, it did not back up enough at the upper end to reduce the flow over the spillway crest.

The conditions of flow in the pipe were considerably improved, both in regard to the splashing at the vertical bend and the thumping at the end of the pipe. A still further improvement was secured by using the coping on the back wall in addition to the flat top transition in the contraction. The splashing from the lower window in the vertical bend started at 130,000 sec. ft. and the thumping in the pipe was still further reduced.

A contraction in the transition which restricted the opening less than that previously described was also tested. The flat top in this case extended 15 ft. down from the top of the transition section as compared with 20 ft. in the previous case. The conditions of flow in the channel, vertical bend and at the end of the tunnel were all somewhat less desirable than with the greater contraction. In all the following experiments therefore the greater contraction was used.

Vane in Inclined Pipe

The water flowing over the ogee crest imparted to the water in the side channel a spiral motion, which persisted for a long distance down the tunnel, and caused at least a portion of the disturbance in the flow. In order to reduce this spiral motion, and hence the disturbance, vanes in various positions in the transition and inclined tunnel were experimented upon. The first vane was about 6 ft. high and extended down the center of the flat top contraction in the transition. No beneficial effect from this was noticed. A vane 6 ft. high was then placed along the bottom of the inclined tunnel from the end of the transition to the beginning of the vertical bend. This resulted in a marked improvement in the splashing at the vertical bend. With the flat top contraction in the transition and the coping on the rear wall, this vane increased the discharge at which splashing began at the lower window from 130,000 sec. ft. to 162,000 sec. ft. Severe splashing did not occur till the discharge reached 190,000 sec. ft.

As the splashing out at the window always occurred on the left side, it was at first thought that by inclining the vane toward the right side of the pipe, more water could be forced to that side and the splashing relieved. The direction of the vane was therefore changed so that it extended from the middle of the bottom of the pipe at the upper end to 5 ft. to the right of the middle at the lower end. This change however increased the splashing, reducing the discharge at which splashing began from 162,000 sec. ft. to 155,000 sec. ft. and the discharge for severe splashing from 190,000 sec. ft. to 170,000 sec. ft. The vane was then moved so that the lower end was 5 ft. to the left of the center of the bottom. In this position the splashing from the lower window began at about the same discharge as with the vane in the center, but no severe splashing occurred for discharges up to 200,000 sec. ft.

The explanation of this unexpected action seems to be that the water approached the vertical bend with a greater depth on the right side. In passing around the vertical bend it was acted upon by a centripetal force, and since the mass of water on the right side was greater than on the left, the force was greater on the right side. This greater force caused a greater pressure in the water at the bottom of the right side than at the bottom of the left side and the water moved from the position of greatest pressure toward that of less pressure; that is, from the right side to the left side. The greater mass at the beginning of the bend was on the right side and in passing around the bend moved across the bottom toward the left side, up the left side and across the top of the bend toward the right, having moved with a spiral clockwise motion which made it cross the face of the window, which is in the upper side of the bend, in a direction inclined toward the right. The vane with its end toward the left reduced this motion because it forced more water in the inclined tunnel toward the left side of the pipe and thus made the centripetal force on the two sides of the pipe more nearly equal. Moving the vane toward the right side at the lower end increased the mass on the right side, making the two sides more unequal, and therefore increasing the cross motion which caused the splashing.

Bell Mouth Portals

To improve the conditions at the entrance to the transition, a bell mouth was placed on the portal. This was built in two stages, the first shape being as shown in Figure ~~54~~ 49. This somewhat improved the entrance conditions, but was not large enough and a larger one was built as shown in Figure ~~50~~ 53. Since the tunnel portal as originally built was practically opposite the end of the ogee crest, these bell mouths extended beyond the end of the crest and it was necessary to leave a space on the crest side of them through which the flow from the end of the crest could pass. This would not be a practical form of construction for the prototype, but the set up served to indicate what results might be secured with a bell mouth entrance and if it was installed in the prototype the necessary adjustments could be made. The larger bell mouth improved entrance conditions considerably and it was used throughout the remainder of the tests.

Cylindrically Faced Baffles

In order to break up the wave which piled up on the side wall of the spillway, and to more equally distribute

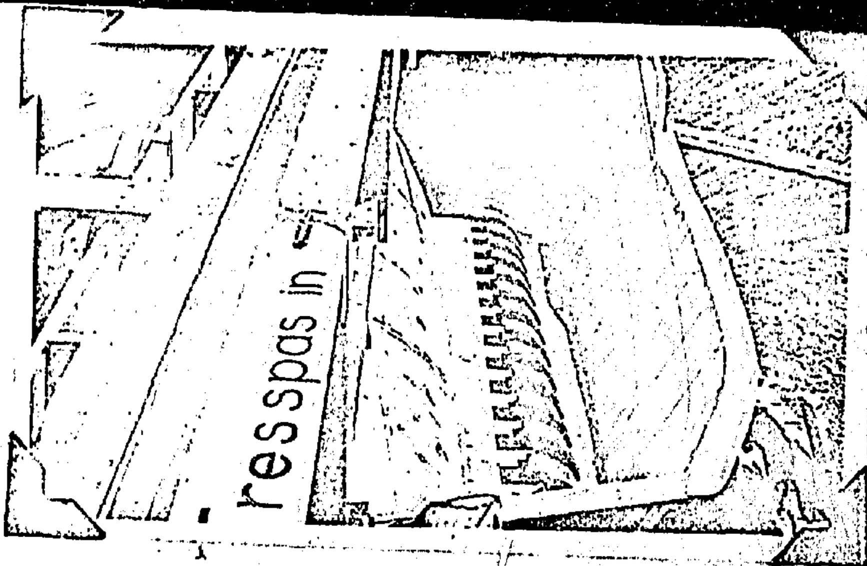
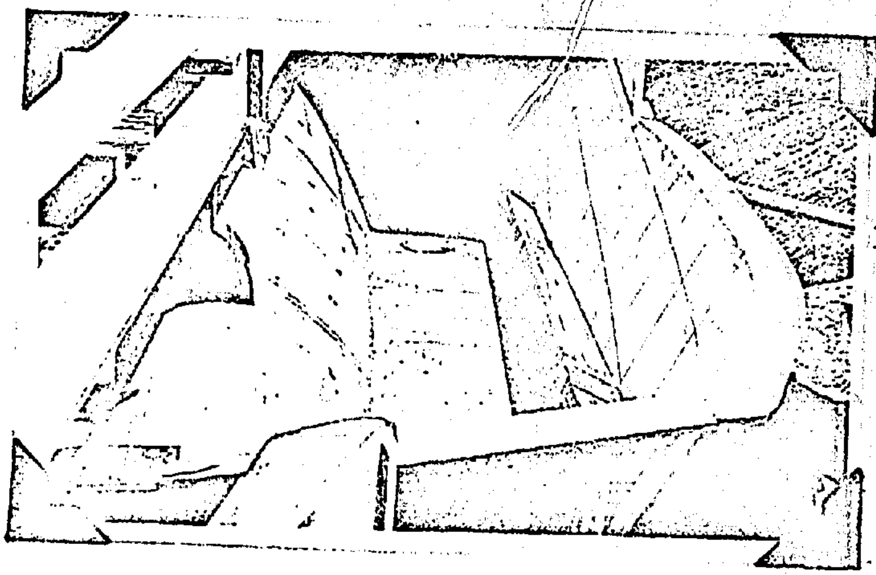


Figure 54. Cylindrical-faced Baffles on Center Line. Radius 6 ft.



Bell Mouth Portal #2
Figure 53

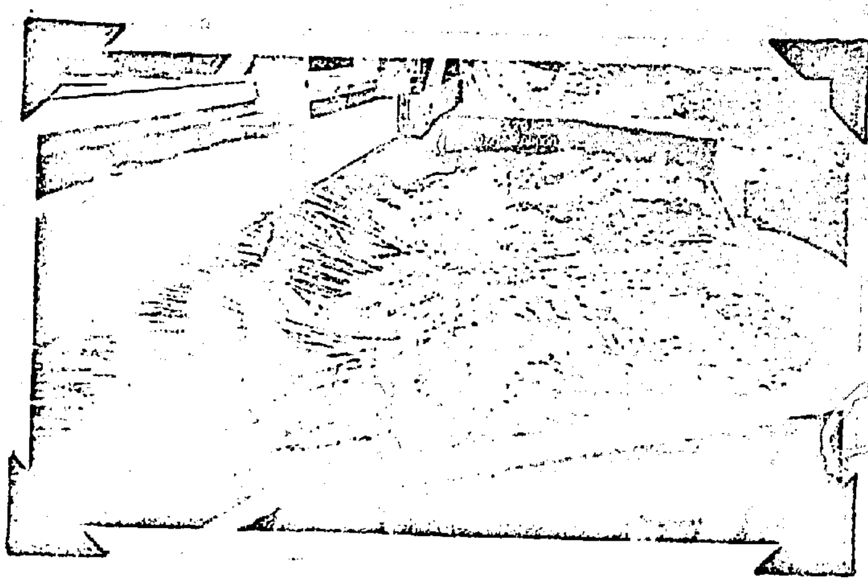


Figure 55. Cylindrical-faced Baffles on Center Line, Radius 6 ft. Q = 200,000 sec.ft.

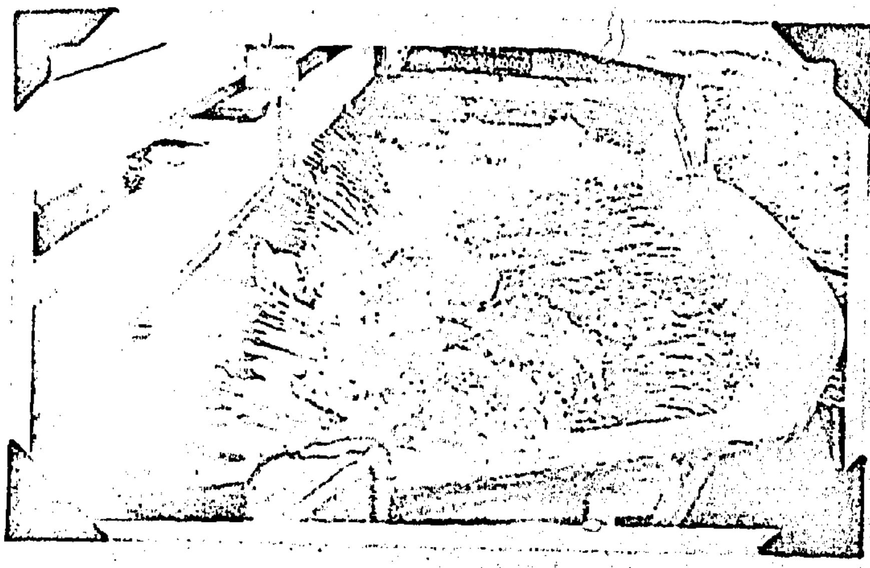


Figure 56. Cylindrical-faced Baffles on Center Line, Radius 6 ft. Q = 180,000 sec. ft.

Figure 57. Cylindrical-faced Baffles on Center Line, Radius 6 Ft. Q = 13,000 sec. ft.

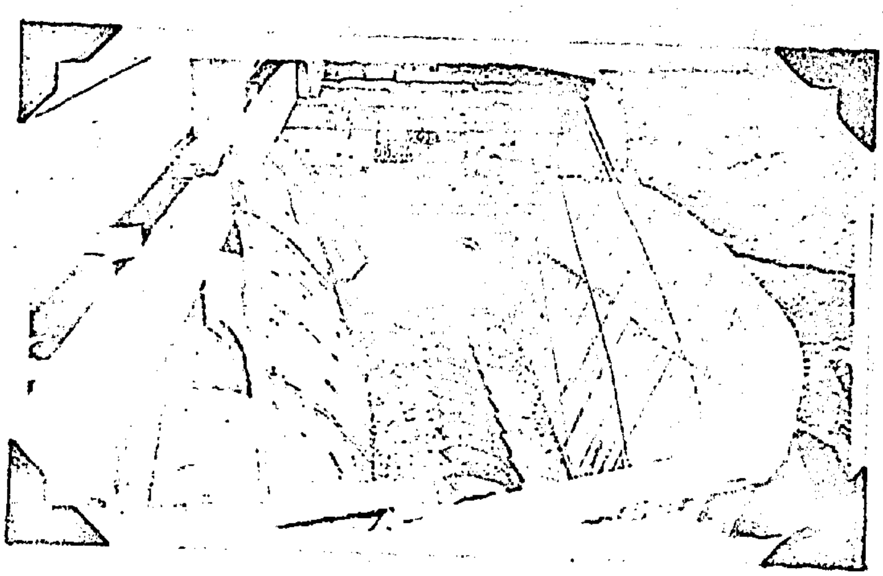


Figure 58. Cylindrical-faced Baffles at 15° with Center Line.

the flowing water in the cross section of the channel, various combinations of baffles with cylindrical faces were investigated. The first set up of these baffles is shown in Figure ~~53~~ 54. It consisted of sections of baffle 10 ft. long with 10 ft. spaces between, set on the center line of the channel. The cylindrical face of the baffle had a radius of 6 ft., giving it a height of 12 ft. and the baffle had an over-all height of 14 ft. These baffles improved the flow in the channel and tunnel somewhat, but not as much as the baffle on the rear wall. The splashing began at the lower window in the vertical bend with a discharge of 130,000 sec. ft. The conditions of flow are shown on Figures ~~58~~ 55 and ~~59~~ 56. The baffles considerably obstructed the flow in the channel however, and submerged the weir at the upper end to sufficient extent to obstruct the flow over it and require a higher headwater level than contemplated in the design to produce the 200,000 sec. ft. discharge.

The baffles located on the center line of the channel, with their faces parallel to it, were not exactly at right angles to the direction of flow, and a considerable portion of the water was deflected downstream and impinged on the next baffle below, as shown in Figure ~~60~~ 57. To remedy this condition the baffles were placed with their faces inclined upstream 15^{o} with the center line of the channel, the row of baffles still extending down the channel center line as shown in Figure 62. The flow conditions with this set up were a slight improvement over those with the previous one, both in the channel and in the tunnel, but the baffles still so obstructed the flow in the channel that a water level above El. 1232 was necessary in the forebay to produce a discharge of 200,000 sec. ft. The location of the baffles was then changed from the center line to a line half way between the center line and the bottom of the crest, the individual baffles being inclined 15° with the center line of the channel as before. This set up was slightly better than the preceding one, but the obstruction caused by the baffles was still too great. As the two upstream baffles seemed to obstruct the flow without improving conditions, they were removed, but the result was not a material improvement.

The cylindrically faced baffles were next placed in a continuous line down the channel, beginning $6\frac{1}{2}$ ft. to the right of the center line at Station 1+00 and 15 ft. to the left of the center line at Station 3+50. The submergence effect was not as great as for the preceding test, but was still too great, and severe thumping occurred at the downstream end of the pipe.

In order to reduce the submergence, smaller sized baffles were tested. These had a cylinder radius of $4\frac{1}{2}$ ft. and an over-all height of 11 ft. They extended in a line

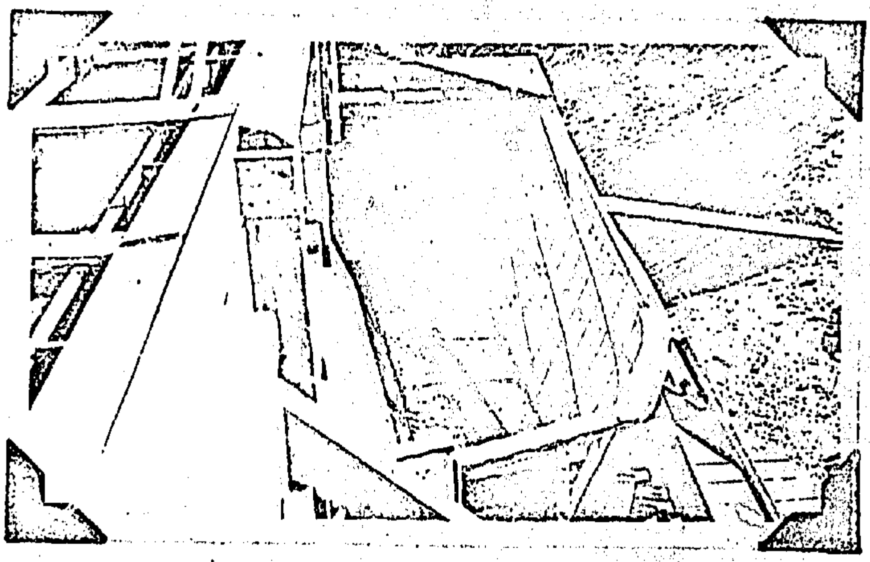


Figure 59. Cylindrical Baffles End to End at Angle with Center Line, Radius 4½ ft.

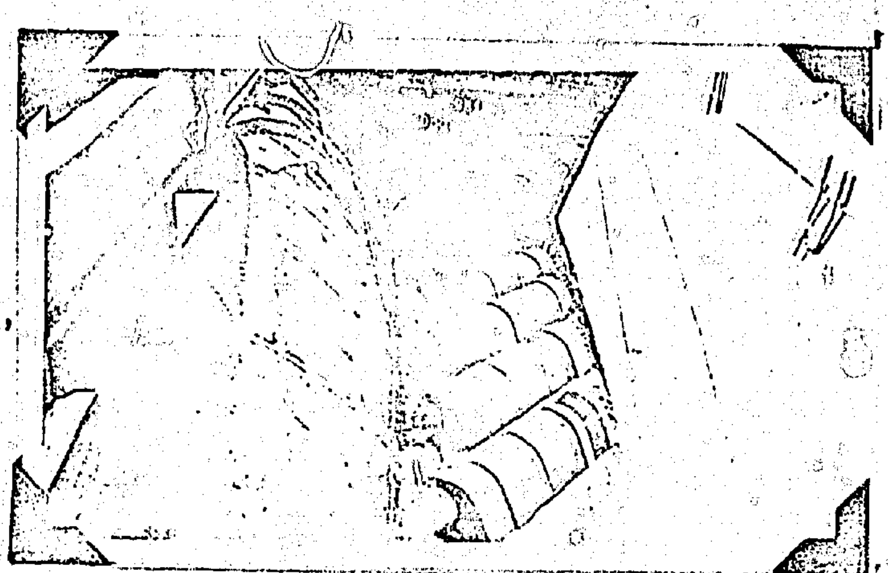


Figure 60. Cylindrical Baffles, Faced Diagonally Downstream, R = 4½.

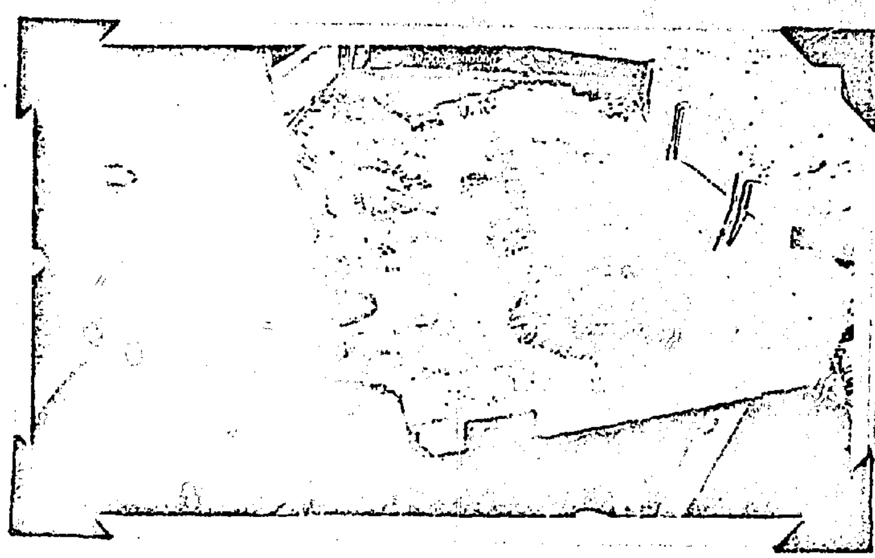


Figure 61. Cylindrical Baffles, Faced Diagonally Downstream, R = 4½. Q = 200,000 sec. ft.

from 8½ ft. to the right of the center line at Station 0+65 to 20 ft. to the right of the center line at Station 3+75. (Figure ~~58~~ 59). With this baffle the submergence did not obstruct the flow over the crest. The results of this set up seemed to be somewhat of an improvement over the previous ones and the baffle was extended downstream to Station 4+50. The extension, however, resulted in bad splashing at the vertical bend. The splashing began at 132,000 sec. ft., and was severe at 150,000 sec. ft. The shocks at the lower end of the tunnel were not large and occurred on an average, once in 5 seconds.

The continuous baffle of the previous run was made up of blocks 10 ft. long. A test was made with every other block removed. This set up was similar to the first ones run with the cylindrically faced baffles but the baffles in this case were smaller. The conditions of flow in the channel were fairly good and excessive submergence at the upper end did not occur. The splashing from the lower window began at 155,000 sec. ft. and became severe at 178,000 sec. ft. The conditions at the end of the tunnel were very desirable, shocks occurring at average intervals of 40 seconds.

Cylindrically faced baffles were also placed as shown in Figure ~~63~~ 60. The water flowing over the crest, upon striking the face of these baffles was deflected

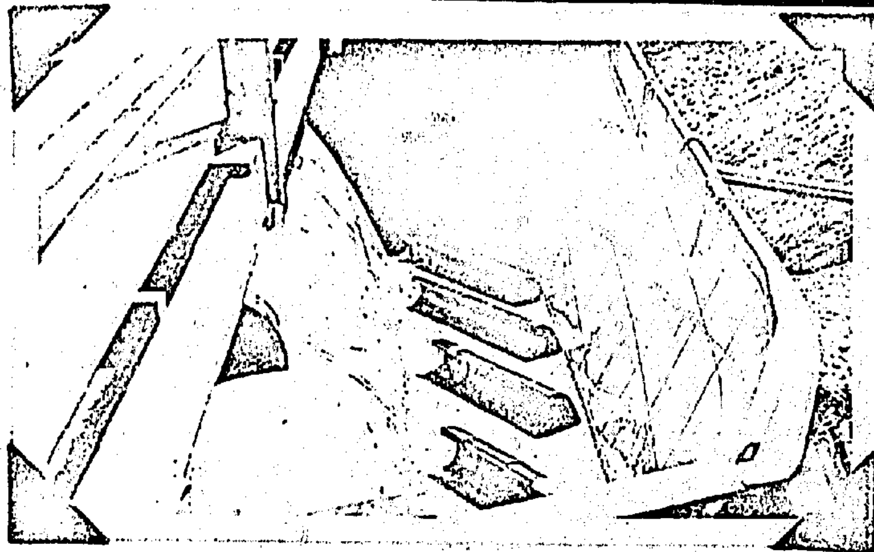
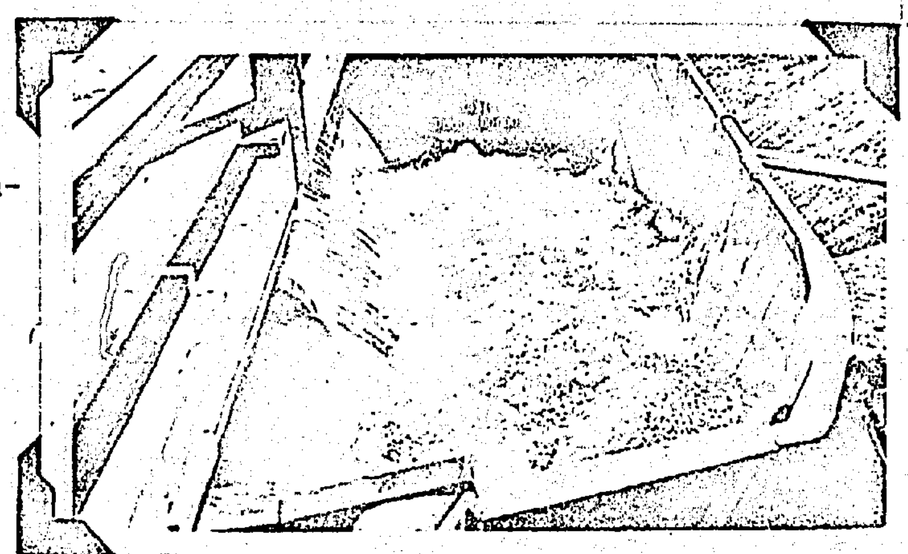


Figure 62. Cylindrical Baffles Faced Diagonally Upstream, R = 4½ f

Figure 63. Cylindrical Baffles, Faced Diagonally Upstream, R = 4½ ft. Q = 100,000 sec. ft.



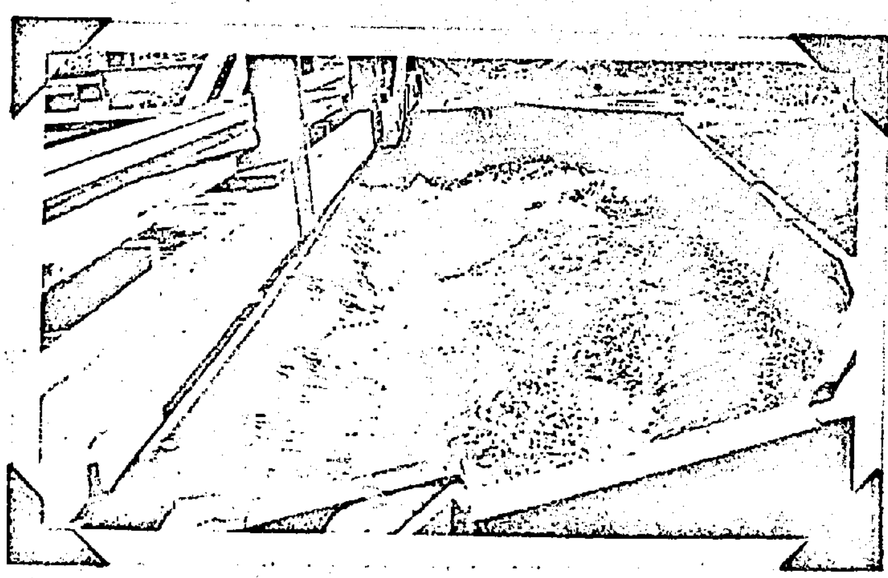


Figure 64. Cylindrical Baffles, Faced Diagonally Upstream, R = 4½ ft. Q = 200,000 sec. ft.

downstream, thus tending to increase the velocity in the channel and reduce the cross section necessary and consequently the cost of the spillway. At small discharges the action of the baffle in deflecting the flow from the crest downstream could be observed. This action probably continued for higher flows also, but the obstruction offered by the baffles offset the advantage gained and at the 200,000 sec. ft. discharge (Figure 61) the conditions of flow were not appreciably improved over the conditions for no baffles.

Tests were also made with the cylindrically faced baffles placed as shown on Figure 62. With baffles in this position it was believed that the water would be forced toward the ogee side, making the flow on the two sides of the channel at more nearly the same height. Figure 63 shows that at a 100,000 sec. ft. discharge this result was accomplished. The obstruction of the baffles and the effect of the upstream deflection of the water so impeded the flow in the channel that at a 200,000 sec. ft. discharge (Figure 64) submergence at the upper end of the channel became excessive and a water level of more than El. 1232 was required to produce a 200,000 sec. ft. discharge.

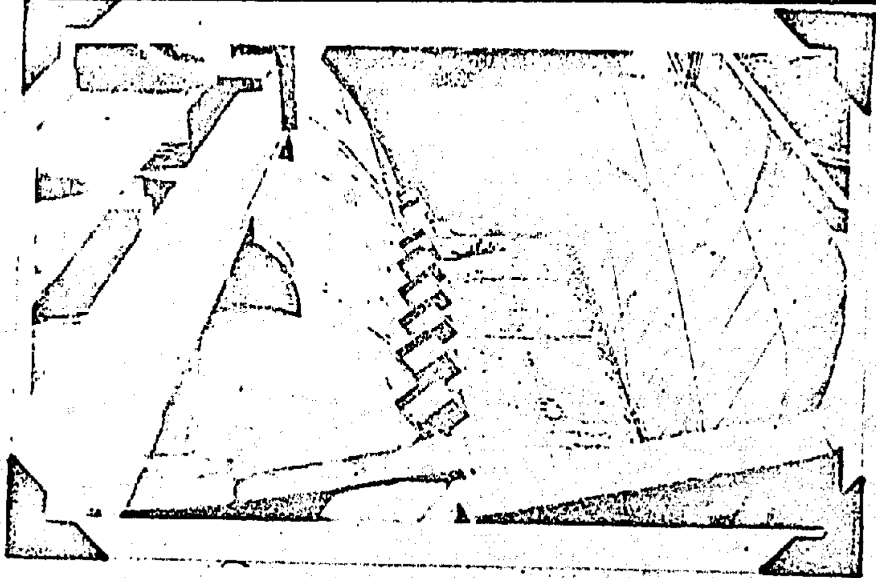


Figure 65. Vanes on Ogee Crest.

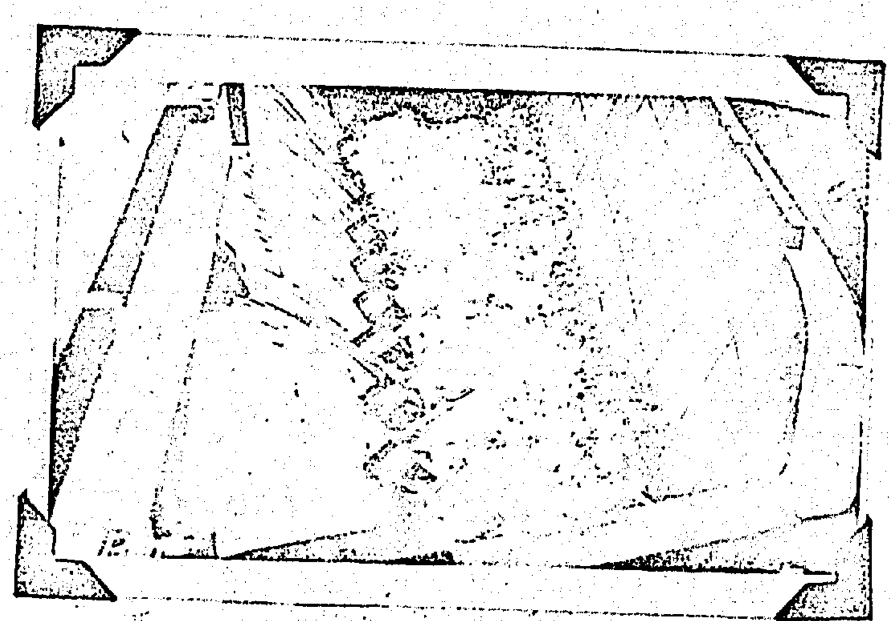


Figure 66. Vanes on Ogee Crest. Low flow.

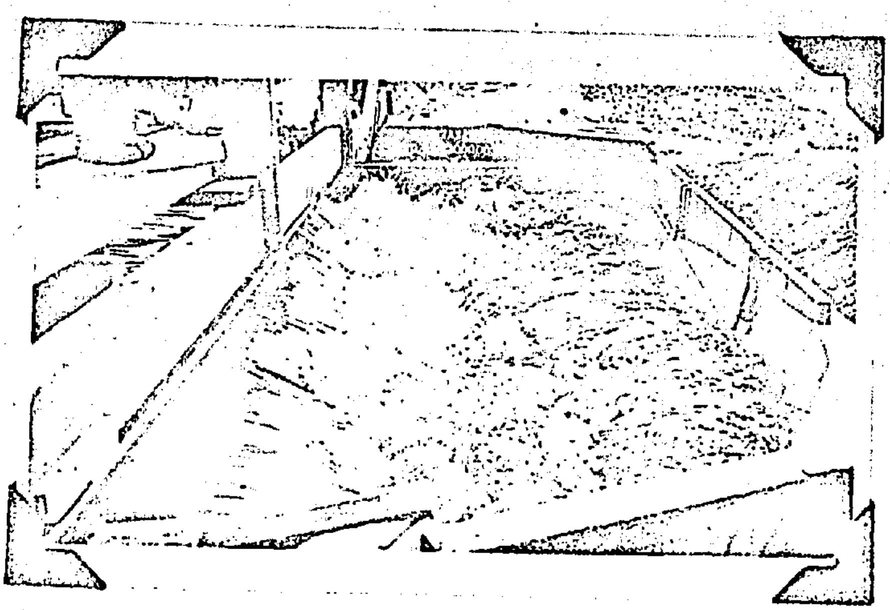


Figure 67. Vanes on Ogee Crest. Q - 200,000 sec. ft.

Vanes on the Ogee Crest

Vanes on the ogee crest were also tried to direct the overfalling water downstream and permit the use of a smaller channel. The first set up is shown in Figure 65. The water falling over the weir was not deflected directly downstream but somewhat out to the side, with a considerable downstream component, as shown in Figure 66. At the larger discharges these vanes obstructed the flow and caused excessive submergence at the upper end of the crest. The conditions in the channel for 200,000 sec. ft. discharge are shown on Figure 67. Considerable vibration also resulted at the downstream end of the tunnel, the average period for severe shocks being 4 seconds. The upstream four vanes were removed, as they appeared to obstruct the flow. This slightly improved conditions in the channel but excessive submergence was still present and the frequency of severe shocks at the tunnel exit was increased to a 3 second average. The downstream four vanes were raised to a position where they would not so greatly obstruct the flow. This eliminated the vibration at the end of the tunnel, but produced an unstable condition in the channel.

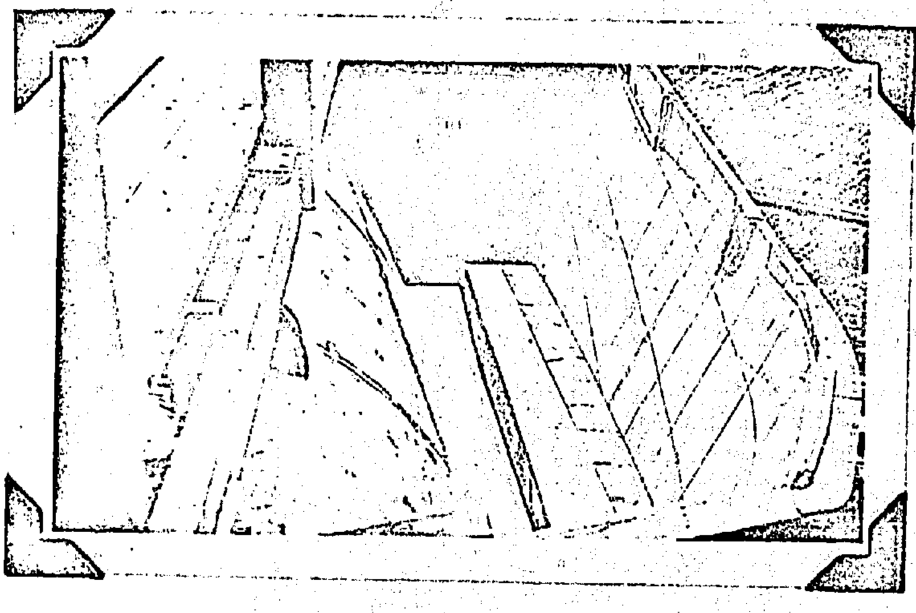


Figure 68. Trough on Crest side.

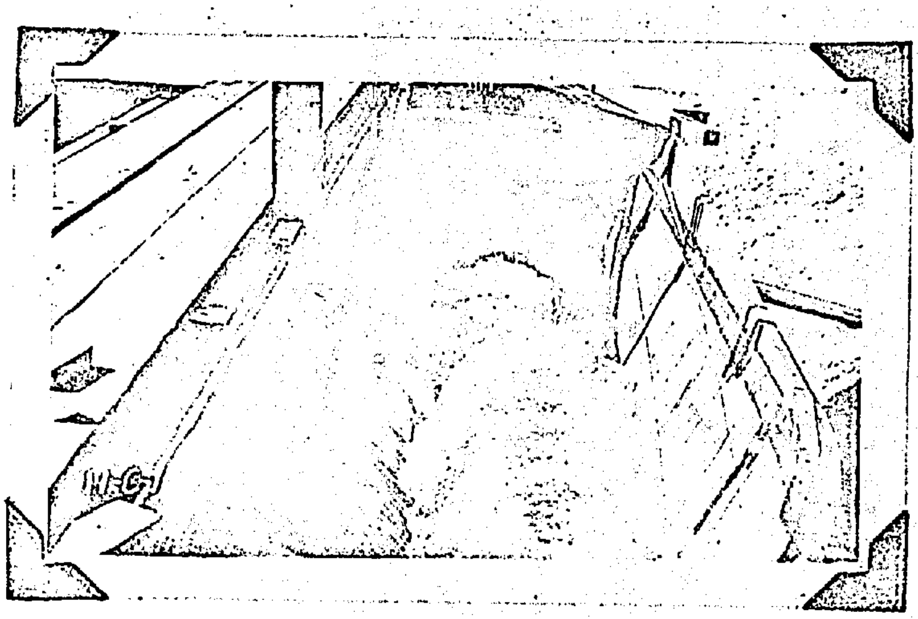


Figure 69. Trough on Crest Side.
Q = 100,000 sec. ft.

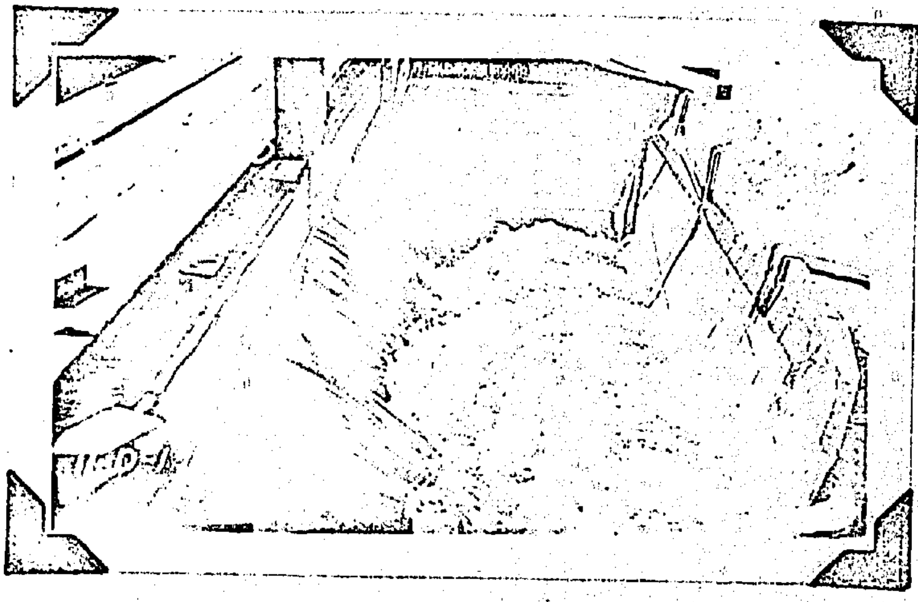


Figure 70. Trough on Crest Side.
Q = 150,000 sec. ft.

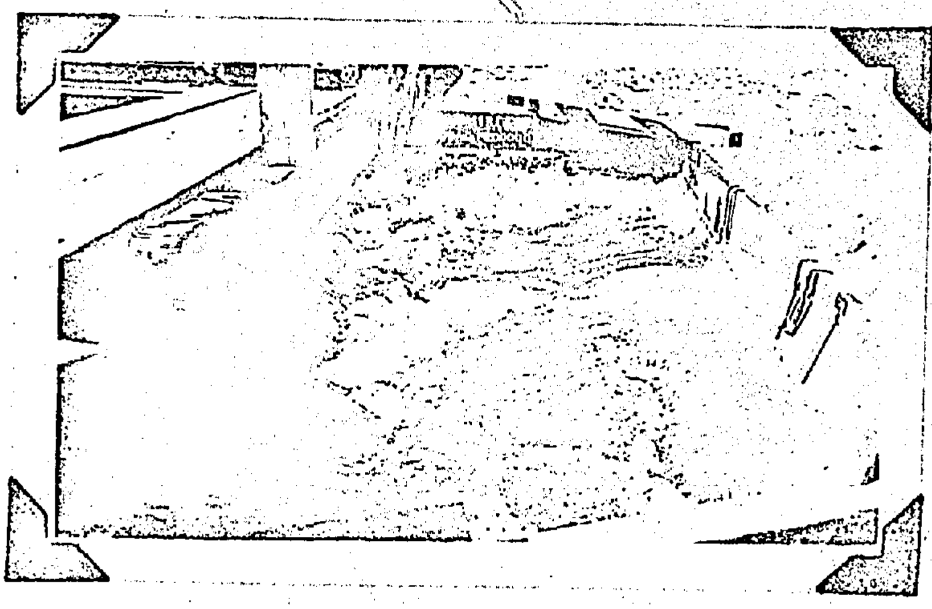


Figure 71. Trough on Crest Side.
Q - 200,000 sec. ft.

At times the channel at the lower end was full to El. 1225 and at others it dropped to a level 20 ft. lower. No reason for this was apparent.

Trough on Crest Side of Floor

This model (Figure 68) was made by removing the false floor and building a raised section on the wall side. For flows up to 100,000 sec. ft. the raised section caused a very disturbed condition in the channel, with a stream of water rising up in a fountain effect along the center line of the channel. The condition for a 100,000 sec. ft. discharge is shown in Figure 69. Distrubed conditions still existed at 150,000 sec. ft. (Figure 70). At 200,000 sec. ft. the conditions in the channel were reasonably good (Figure 71). The severe shocks at the end of the tunnel occurred at intervals of about 15 seconds.

A dentated sill 6 ft. high was placed at the left edge of the raised portion of the channel floor. This tended to hold the water to the right side of the channel and at low flows caused the trough at the upper end to be swept out and produce very turbulent conditions. A weir was also tried across the lower portion of the channel bottom at the downstream end, but this did not noticeably improve the flow.

Plain Sill and Dentated Sill on Channel Floor

Experiments were made with a plain sill with

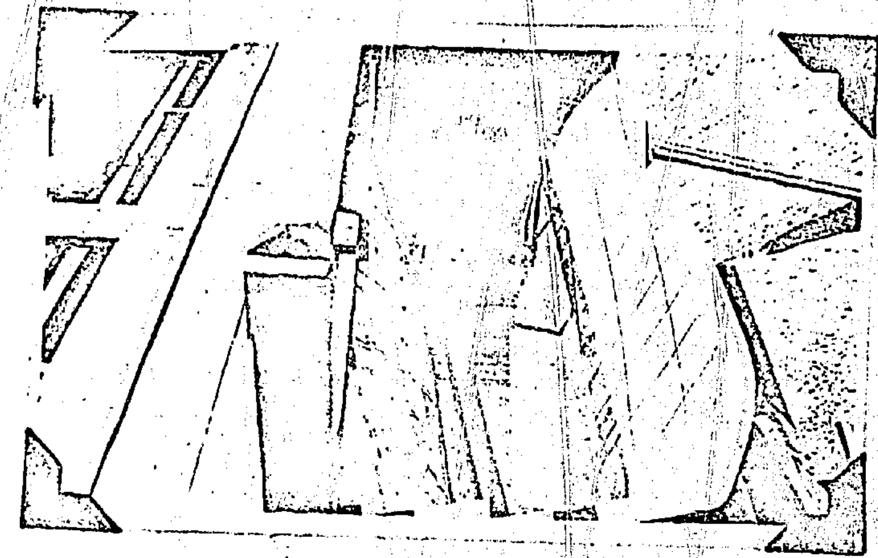
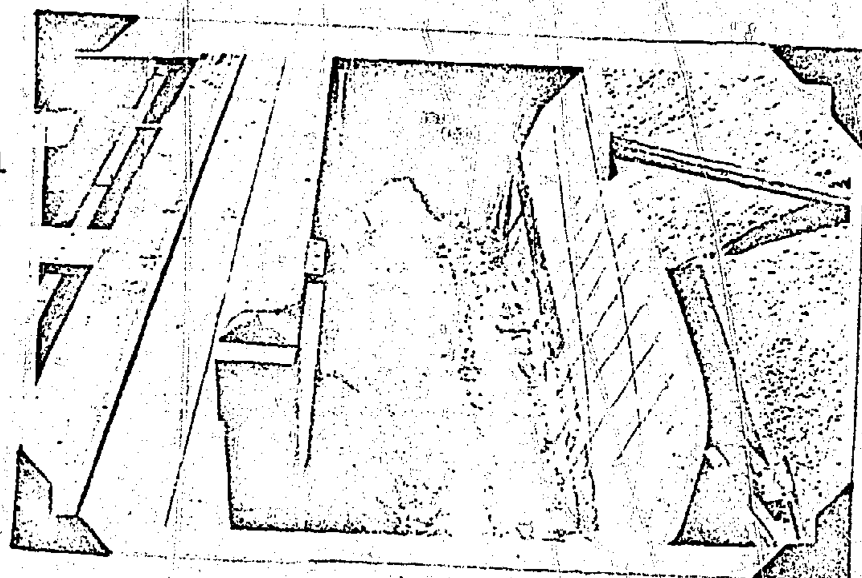


Figure 72. Plain Sill on Floor.

Figure 73. Plain sill on Floor. Q = 15,000 sec. ft.



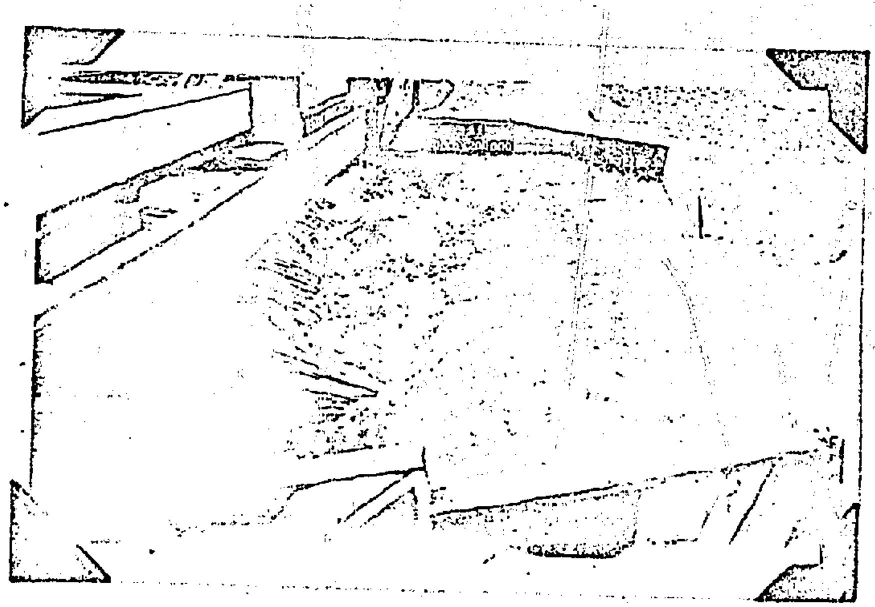


Figure 74. Plain Sill on Floor. Q = 200,000 sec.ft.

vertical face 6 ft. high extending from 20 ft. to left of the center line at Station 3+00 to 4 ft. to left of the center line at Station 0+50. At high flows this produced very good conditions in the channel and at the end of the tunnel, small shocks occurring only about once in 15 seconds. At low flows however the conditions in the channel were undesirable. At a discharge of 15,000 sec. ft. the stream was thrown vertically into the air in a fountain formation. At a somewhat higher flow a pressure area was formed upstream from the baffle, which forced the stream upward so that it passed over the baffle without impinging on it, but instead rose high above the bottom and dashed against the rear wall. Views of this set up for several conditions are given on Figures 72, 73 and 74.

The plain sill was replaced by a dentated sill 6 ft. high extending from 23 ft. to left of the center line at Station 3+00 to a position 14 ft. to left of the center line at station 0+50. This improved the conditions of flow in the channel at low discharges. The conditions at the end of the pipe however were worse, as severe shocks occurred at intervals averaging 1.3 seconds.

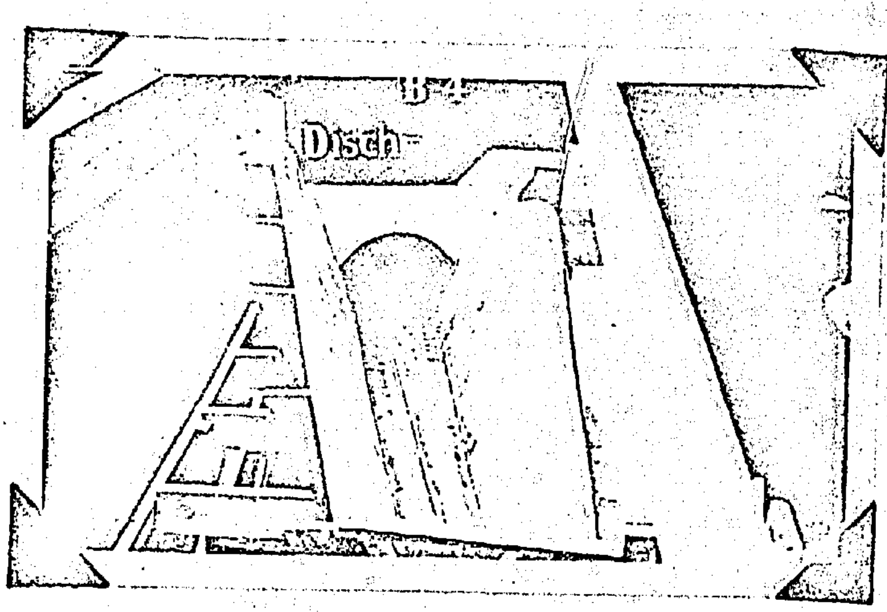


Figure 75. Trough in Floor Along Rear Wall. Model (M-2), Scale 1:100.

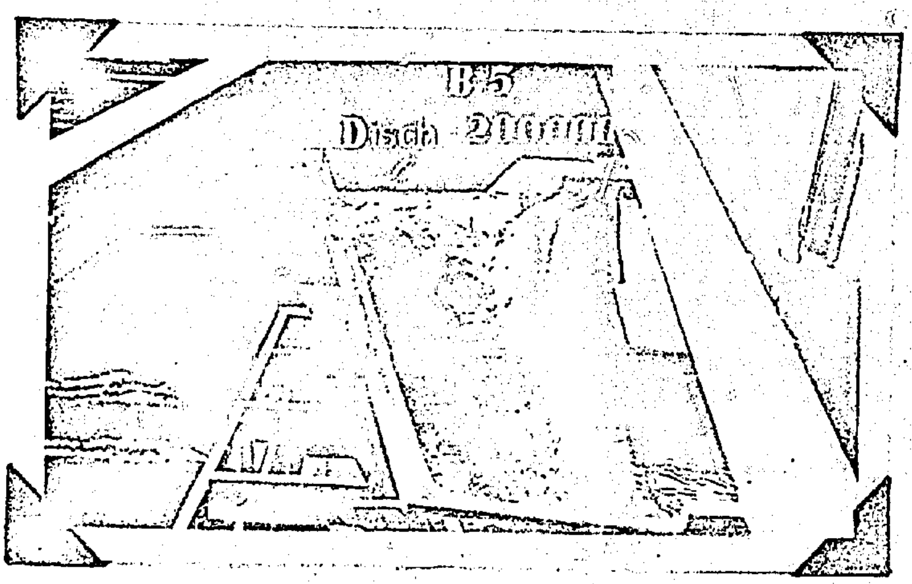


Figure 76. Trough in floor Along Rear Wall. Q = 200,000 sec. ft.

DRUM GATE TYPE- 1:100 (MODEL M-2)

A small model of the drum gate type of spillway was constructed by Mr. W. H. Price on a 1:100 scale to test out the possibility of using a trough in the floor of the channel along the bottom of the rear wall. It was expected that this would reduce the height of the wave along the real wall. The channel of this model was approximately the same design as that of the M-1 model. The transition was slightly larger in proportion and of less abrupt curvature than the M-1 model. In order to simplify construction it was built without gates or the intermediate piers. Figure 75 shows the spillway with no flow in it. The trough on the wall side of the floor proved ineffective in reducing the height of the wave. The stream from the weir seemed to jump off the edge of the trough and flow in an inclined direction downward to the bottom of the trough, thence across the bottom and up the rear wall, with practically as much energy as with no trough. The condition of flow in this model for a 200,000 sec. ft. discharge is shown in Figure 76.

The trough was then moved to the ogee side with better results. The action in this case was very similar to that for the M-1 model. The conditions

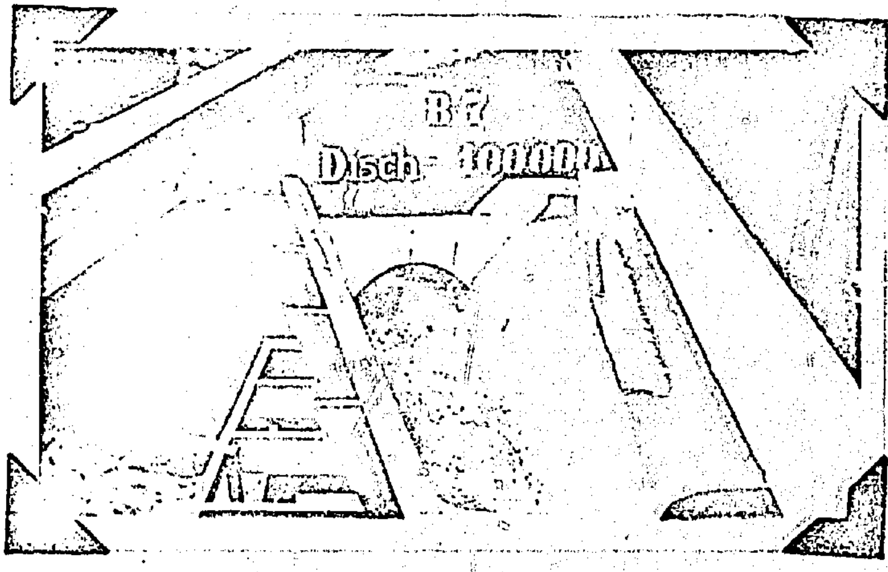


Figure 77. Trough on Ogee Side.
Q = 100,000 sec. ft.

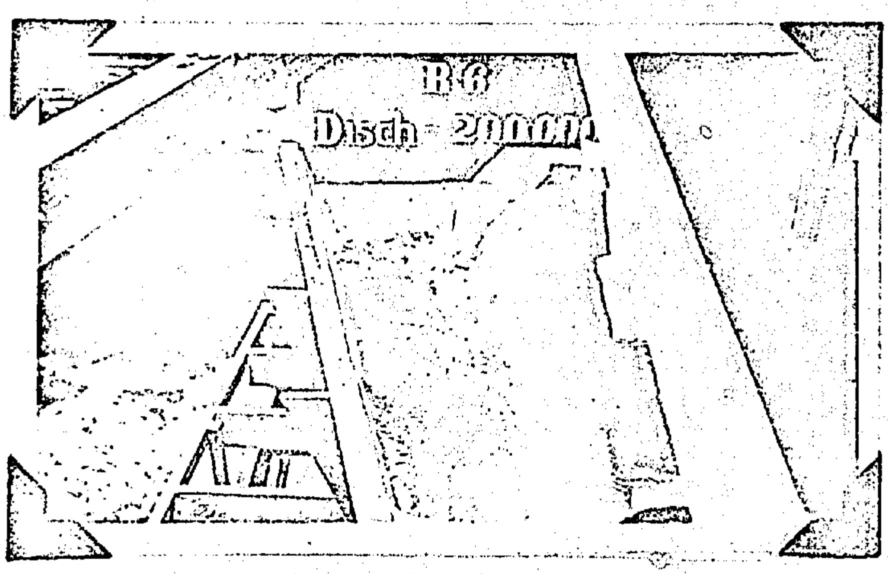


Figure 78. Trough on Ogee Side.
Q = 200,000 sec. ft.

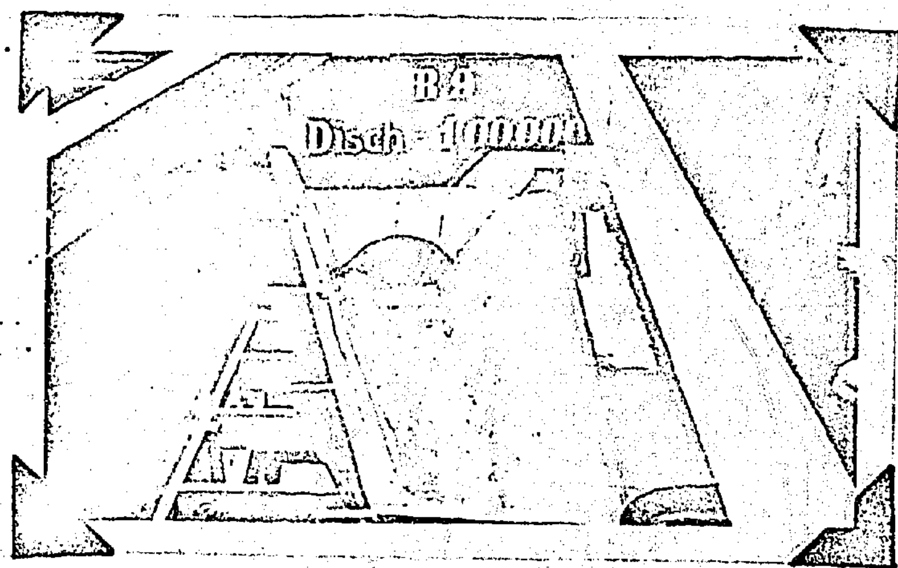


Figure 79. Raised Floor. Q = 100,000 sec.ft.

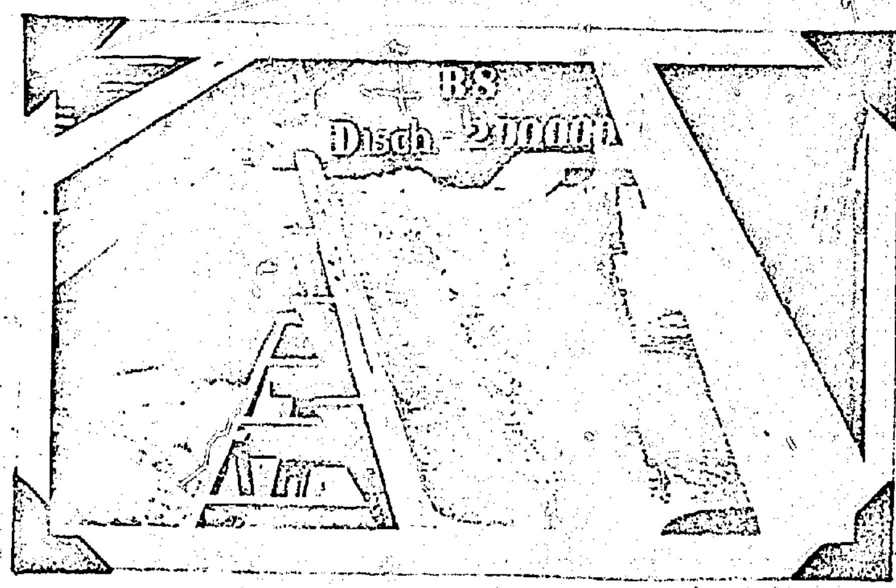


Figure 80. Raised Floor. Q = 200,000 sec.ft.

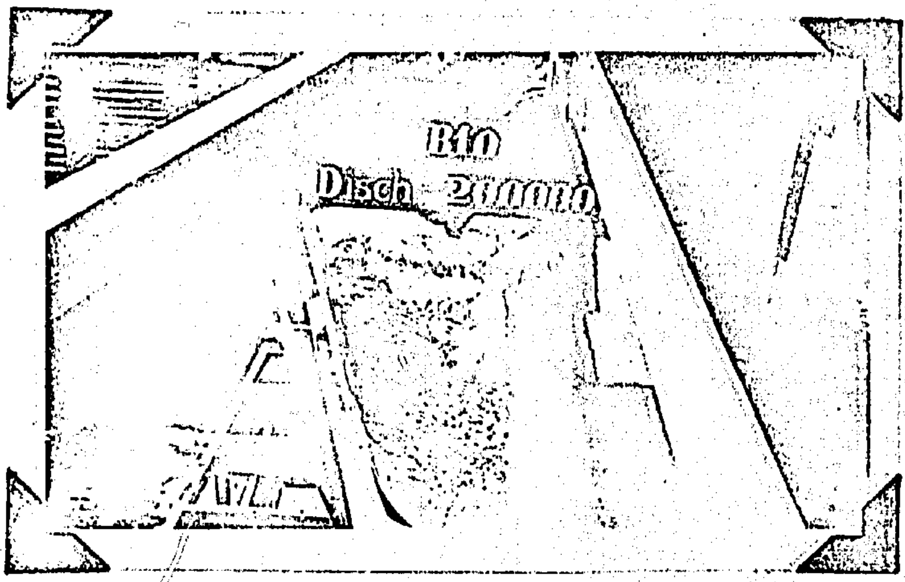


Figure 81. Altered Transition.
Q = 200,000 sec. ft.

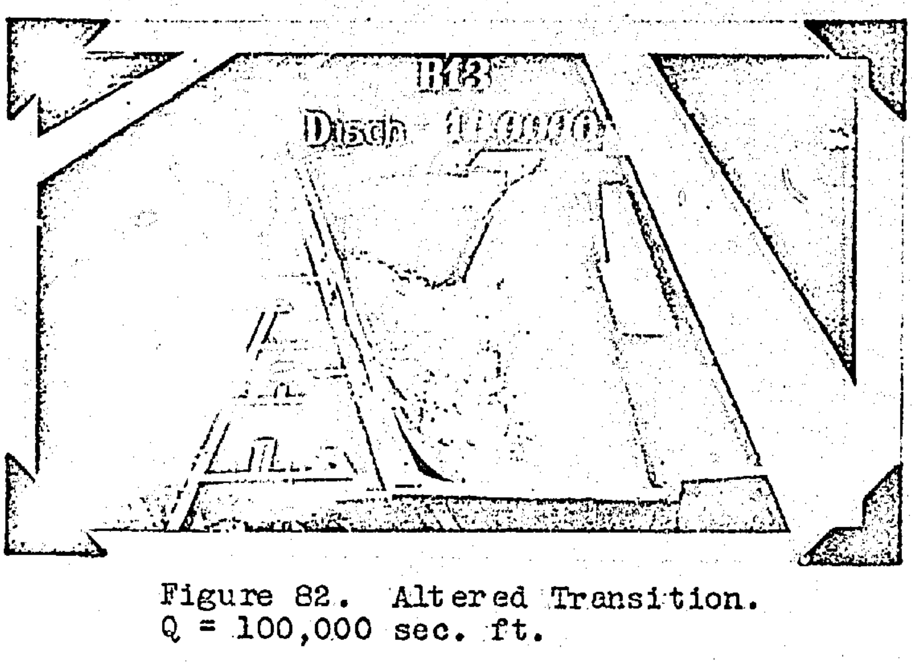


Figure 82. Altered Transition.
Q = 100,000 sec. ft.

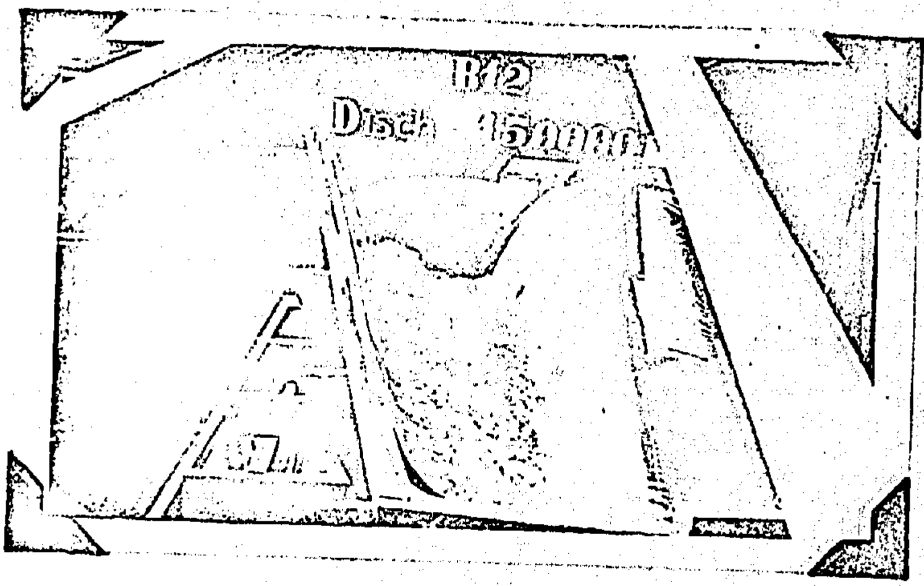


Figure 83. Altered Transition.
Q = 150,000 sec. ft.

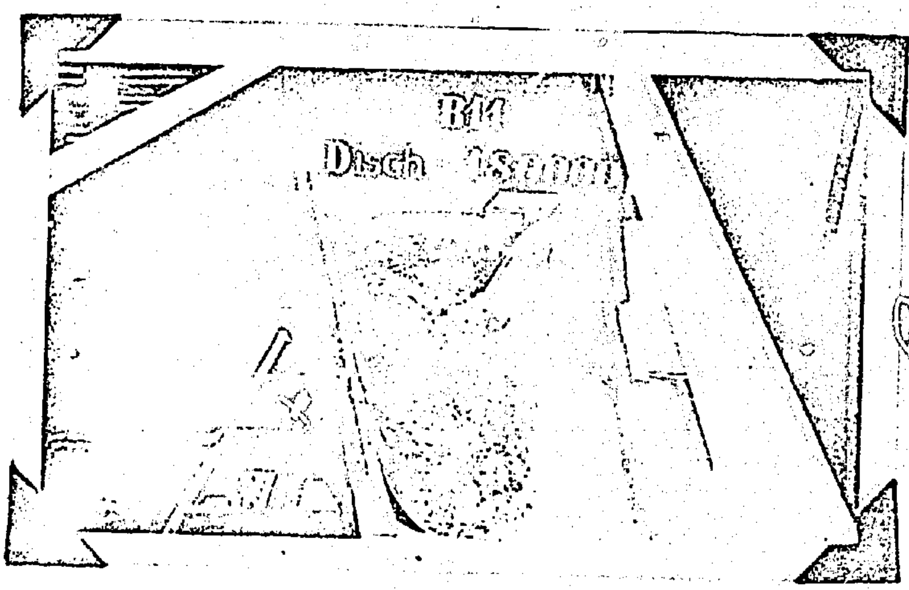


Figure 84. Altered Transition.
Q = 200,000 sec. ft.

of flow for 100,000 and 200,000 second feet are shown on Figures 77 and 78, respectively. A comparison of Figure 77 with Figure 69 shows a very close similarity for 100,000 sec. ft. flows on the M-2 and M-1 models, respectively. The pictures of the two models for a 200,000 sec. ft. flow are not comparable, as the flat top in the transition of the M-1 was not built in the M-2 model. At the 100,000 sec. ft. discharge this did not come into action.

This small model was also tested with a flat floor similar to the raised floor in the M-1 model. The conditions for discharges of 100,000 and 200,000 sec. ft. are shown on Figures 79 and 80, respectively.

It was believed that considerable improvement in the flow conditions at the lower end of the spillway could be made by altering the transition between the channel and the inclined tunnel pipe. The top of the transition was raised at the upper end and made to incline more steeply downward. The results however, were unsatisfactory; at a 200,000 sec. ft. discharge, the water from the ridge against the rear wall impinged on the top of the transition and formed an undesirable wave, as shown in Figure 81. The conditions for flows of 100,000, 150,000 and 180,000 are shown on Figures 82 to 84, respectively.

FORT COLLINS 1:60 SCALE
DRUM GATE SIDE CHANNEL DESIGN

The characteristics of this design may be seen on Plate XIV which shows the first model set up. In all the tests the crest and side channel relation remained the same but several different designs of the tunnel transition were tried.

The model was built on a 1:60 ratio or a scale of 1" on the model representing 5' on the prototype. This gave four drum gate openings of 20 inches each and a side channel trough approximately 9 inches deep at the upper end and 19 inches at the lower end. The crest and side channel were constructed of wood frames covered with galvanized iron. The transition to the tunnel was made by running laths, between collars at various sections, and plastering over them with a mixture of quick setting cement, lime and sand. The circular tunnel section below was made of transparent pyralin tubing 10 inches in diameter, which, after making a vertical bend of 50 degrees, ran into a section of cast iron pipe in which the horizontal bend of the tunnel was accomplished. The top of this pipe was cut away as much as possible to obtain a view of the water flow around the bend.

The head of water over the crest was approximately 5 inches at the maximum flow and was measured by means of a float gage in a well connected to a pipe that ran out in front of the crest about 30 inches upstream therefrom.

The maximum flow over the model was about 7-1/2 cubic feet per second and was measured over the weir as described on page 6.

The weir was submerged for all but the lowest flows as the forebay was backed up over it, due to placing the model crest too high in the tanks. This introduced a probability of a small error in the flow measurement, for although the weir had been calibrated when submerged, the measurements were not made at exactly the range used in the model tests, and the quantities had to be taken from exterpolated calibration curves. The difficulty could have been overcome by lowering the model crest but this meant practically reconstruction of the model and it was decided not to take the time for the alteration.

The main object of the experiments was to develop a design that would carry the water down the tunnel, without completely filling it, in as smooth and straight line flow as possible.

The principal experimental methods followed to accomplish this were:

I. By blocking off transverse flow from crest at the transition entrance.

A. By blocking off the last 60 feet of the crest and placing a 60 ft. gate on the center line of the channel at the upstream end, to maintain the capacity.

Test No. 13. Compare with Test No. 28 to show similarity of action to B.

B. By moving the Transition Entrance 60 ft. downstream.

Test No. 16. Compare with Test No. 12 which was made on the original design.

II. By placing a weir across the lower end of the channel.

A. To create a dissipation pool and over-flow crest that would level off the water surface at the transition entrance.

Test No. 24 shows the effect of the different height weirs.

Test No. 28. Compare with Test No. 16 to show the effect of the weir on the channel water surface.

III. By use of weir and offset in tunnel entrance.

A. Offset to prevent high velocity along the crest face and fill in water surface depression there.

1. With a 70 ft. width weir and a 10 ft. offset.

Test 38. Compare with Test 28 to show the effect of offset and weir.

2. With a 40 ft. width weir and a 14 ft. offset.

Tests 34 and 35. Compare with Tests 43 and 44 which have about the same water surface elevations.

IV. By giving a flatter slope to the transition floor.

A. To check high velocity fall and level off surface.

Tests 43 and 44. Compare with Tests 41 and 42 which were made on similar transition that had a steeper floor slope.

Also tests 34 and 35. Compare with Tests 52 and 53.

V. By placing baffles along channel floor or on the crest face.

A. By impact and reversing the flow, to dissipate the energy of the overflow from the crest.

Test 44. Compare with Test 43 which was made without baffles.

B. By creating two opposing rolls to dissipate the energy of overflow.

Test 54. Compare with Test 52 which was made without baffles. Compare with Test 44 to check flow conditions in the channel.

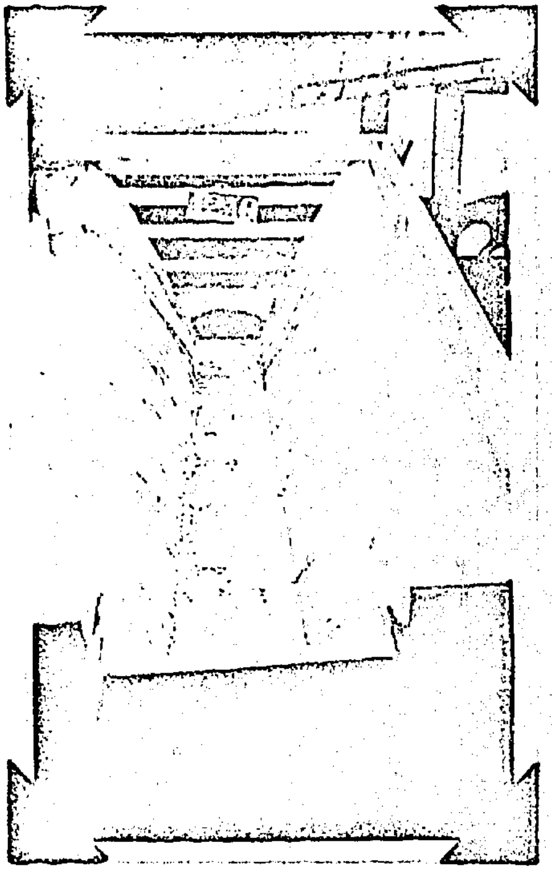


Figure No. 85.
No discharge.

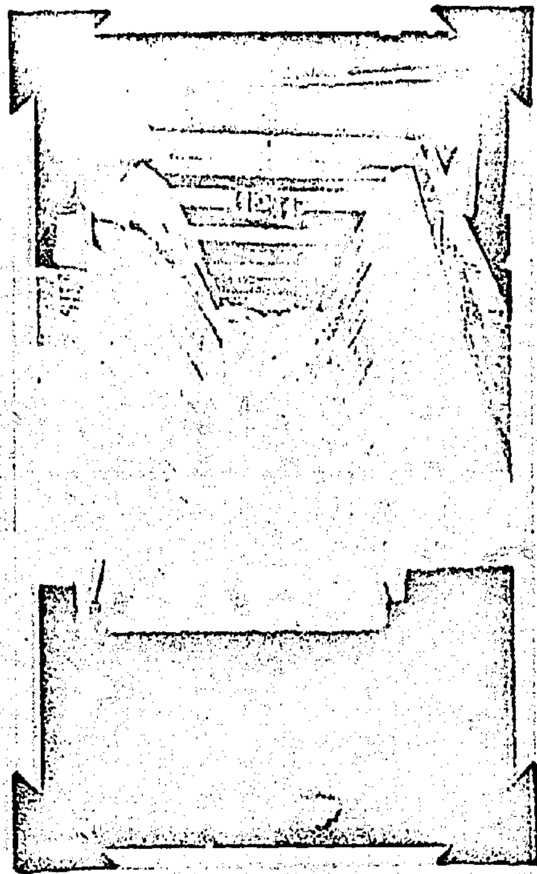


Figure No. 86.
Q = 40,000 sec. ft.

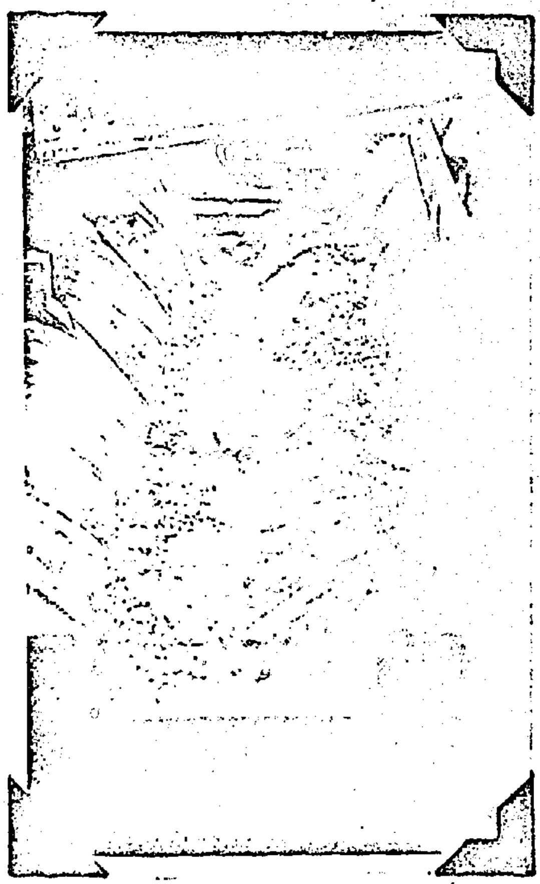


Figure No. 87.
Q = 120,000 sec. ft.

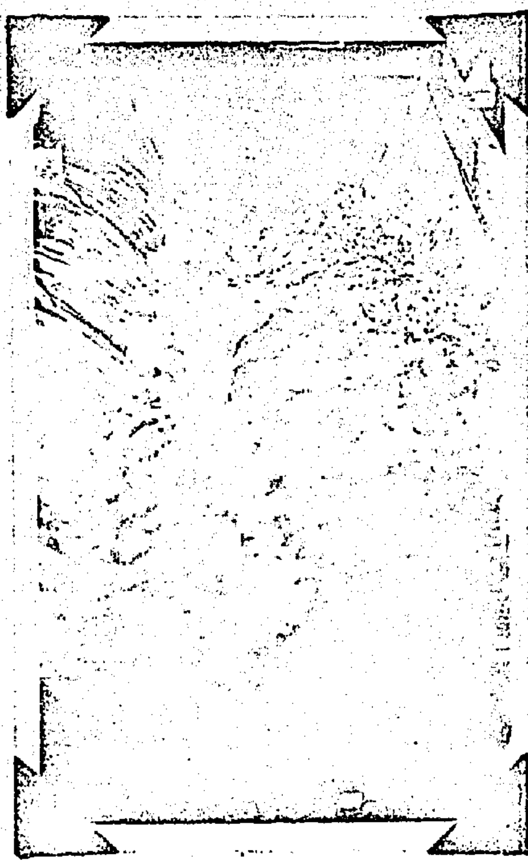


Figure No. 88.
Q = 200,000 sec. ft.

ORIGINAL DESIGN

MODEL C-4

The layout of the original design is shown on Plate XIV. It will be noted that the roof of the transition on the model is left off for a distance down so that observations of the flow could be made. The action of the flow down the tunnel, as observed here, whether it was symmetrical or not, and the extent to which it filled the tunnel was the main criterion for judgment of the worth of the set up being tested. Unfortunately no satisfactory pictures could be obtained of views down the tunnel, due to interference of the framing cross pieces and the lack of light. Figures 85 to 88 inclusive, show the conditions existing in the channel and only indicate in a general way what is happening down the tunnel. As a general rule, the more level the water surface at the lower end of the channel, the better the flow down the incline and the comparison in this way can be made of many of the photographs. In the pictures of Model C-4 (Figures 85 to 88 inclusive) the piling up of the water against the right side wall will be noted, giving an unbalanced water surface which produces a whirl and tends to clog the flow down the tunnel.

It was thought that if a length of channel was allowed below the crest, the water surface would have more chance to level off before going down the tunnel. This extension would also eliminate the transverse flow kick just at the tunnel entrance.

LOWER 60 FT. OF CREST BLOCKED OFF, GATE IN WALL AT UPPER END

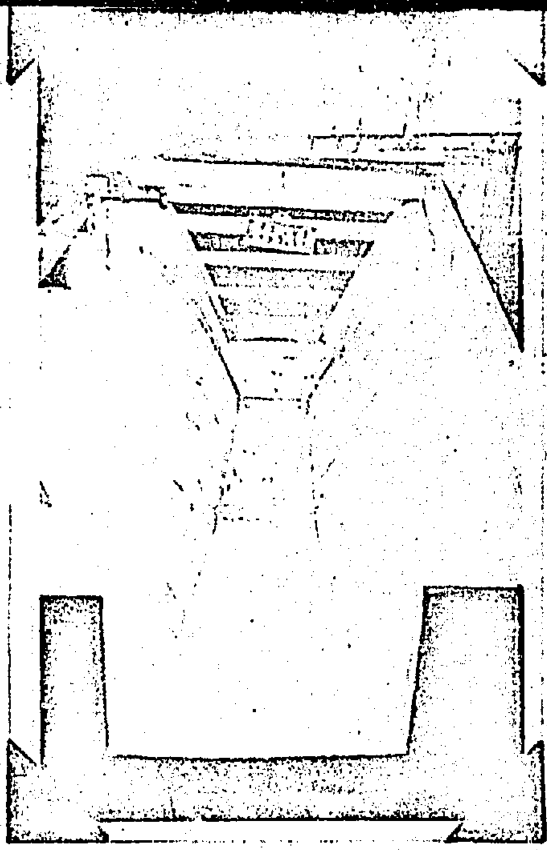


Figure 89. No Discharge.
30 ft. Weir Across Lower End of Channel.

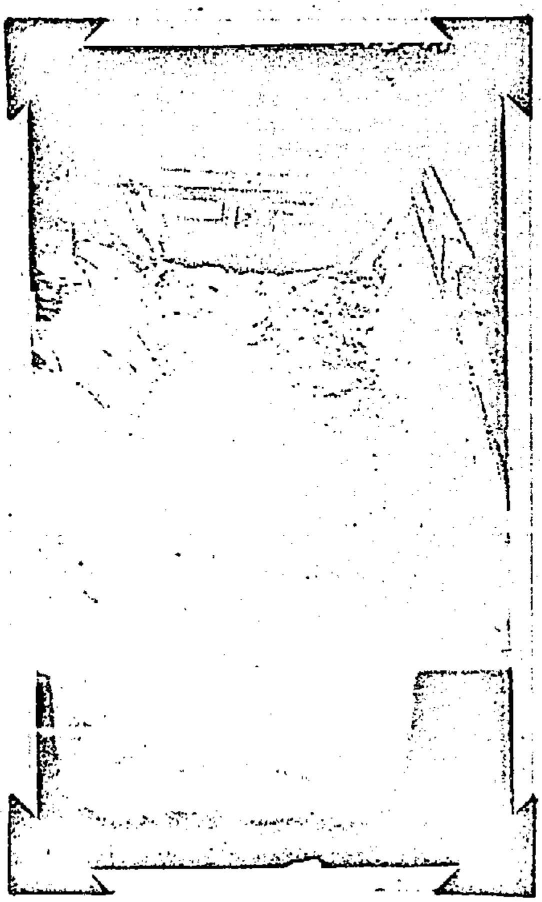


Figure 90.
Q = 120,000 sec. ft.

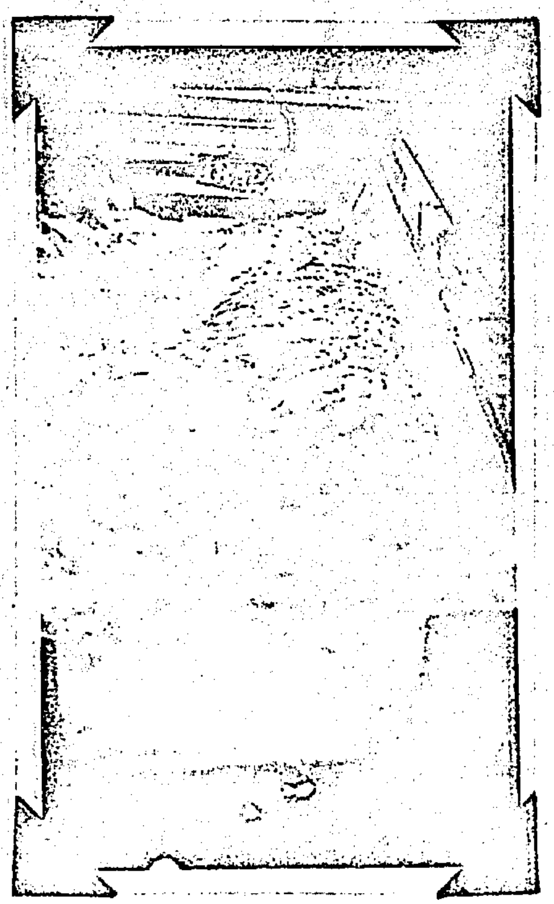


Figure 91.
Q = 200,000 sec. ft.

To test the idea, the last sixty feet of the crest were blocked off and in order to maintain the same capacity, a gate sixty feet wide was cut in the upstream end of the channel as shown in Figure 89. This materially improved the flow as was anticipated. To apply the idea to the original design the tunnel transition was moved sixty feet further downstream from the last gate as shown on Plate XV.

Figures 90 and 91 illustrate the flow in the set up with the end gate and the last sixty feet of the crest blocked off, but in this instance there was also a 30 ft. weir installed across the channel at the lower end (discussed later). This test may be likened to Test 28, in which the same height of weir was used in the channel after the transition had been moved sixty feet downstream.

MODEL C-4a

To obtain the same effect as blocking off the flow from the last sixty feet of the crest, the tunnel transition entrance was moved sixty feet downstream as shown on Plate XV. The photographs of the flow in the channel for this set up showed so little variation from those of Test 12, Page 5? that they will not be included here. It was hardly possible to discern a difference in the pictures and to tell what was actually happening to the flow down the shaft. It is more readily seen, from

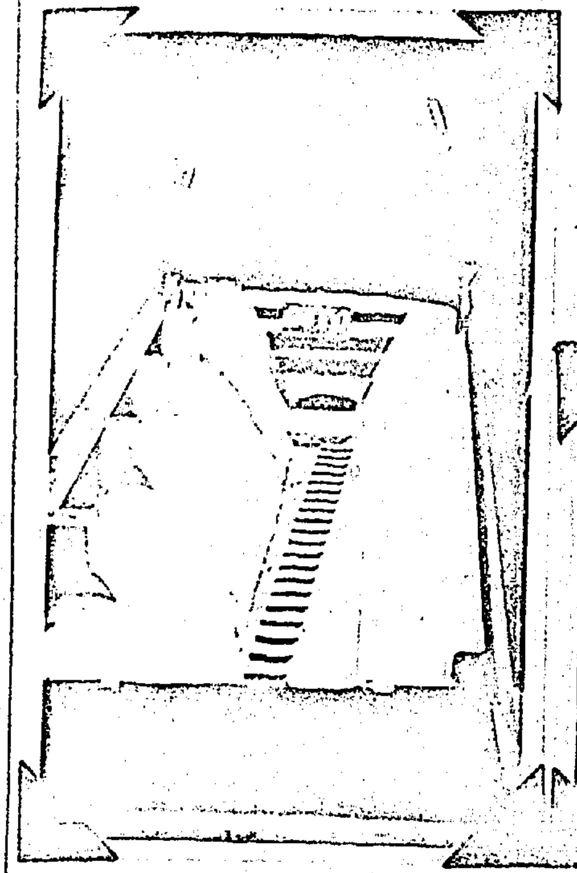


Figure 92. Set up.

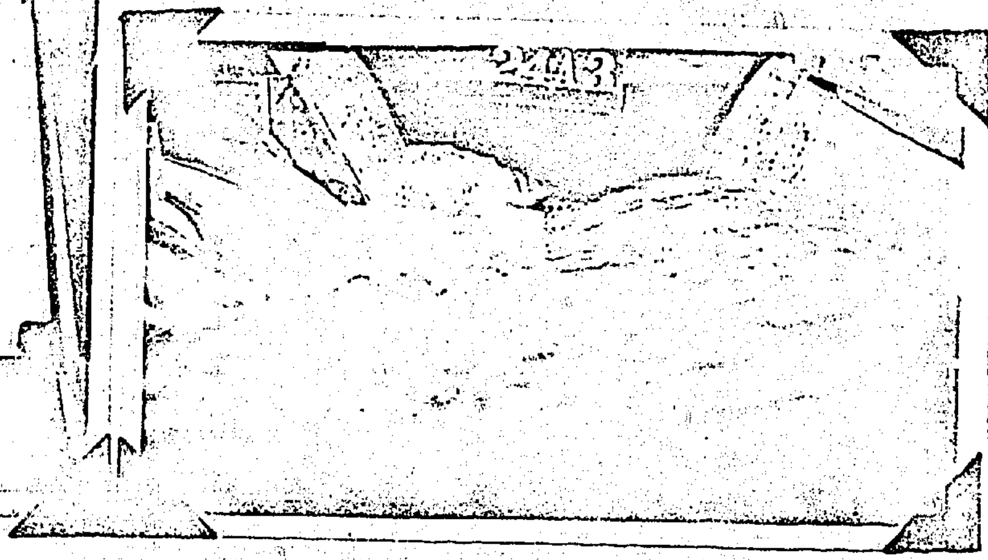


Figure 93. 15 Foot Weir.

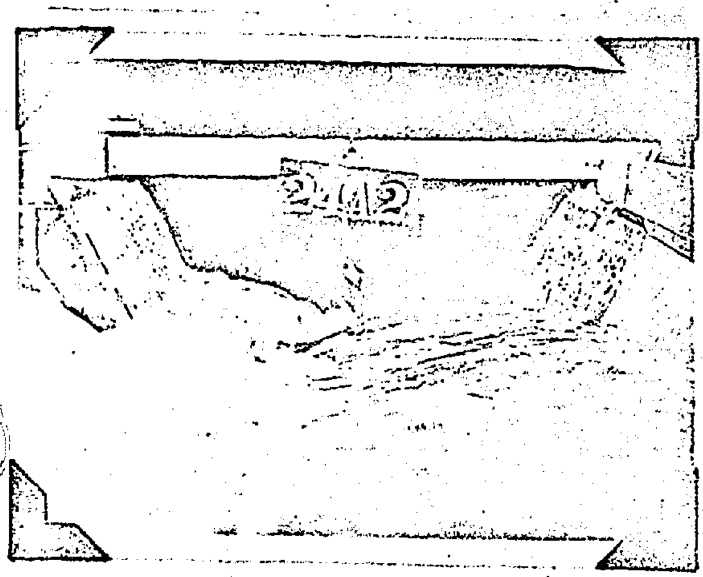


Figure 94. 20 Foot Weir.

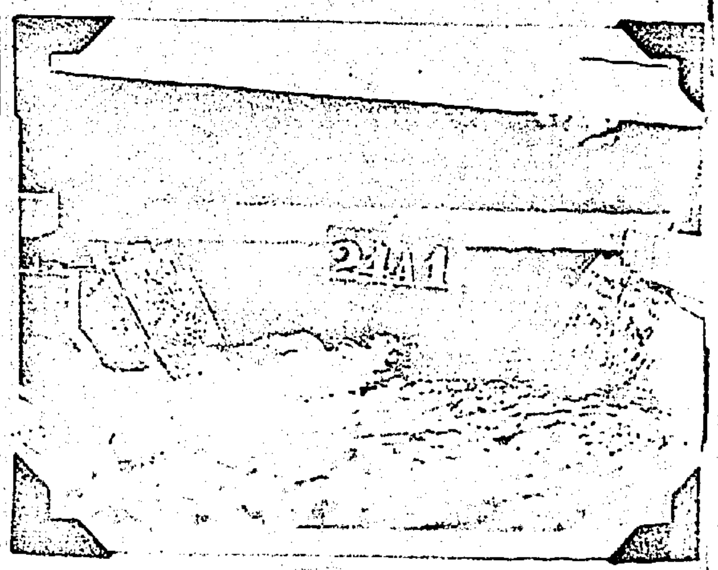


Figure 95. 30 Foot Weir.

DENTATED SILL, VARIOUS HEIGHT WEIR ACROSS LOWER END OF CHANNEL. Q=200,000 sec. ft.

contrasting the water surface curves (not included), that there is a leveling off and greater depth of the water surface at the lower end of the channel, which consequently resulted in a smoother flow down the transition. This greater depth of water in the channel for the same quantity of water flowing was only at the lower end and did not reduce the coefficient of discharge by creating a submergence of the crest.

The extension of the channel below the spillway crest was considered extremely beneficial, if not absolutely necessary, and was therefore incorporated in most of the set ups following.

Also, as the increased depth of the water in the channel seemed advantageous, the idea was carried further by trying weirs of various height across the channel at the lower end. This created a pool in which some of the energy of the spillway overflow was dissipated and the weir afforded a barrier which tended to smooth out the overflow from it.

The effect of the various height weirs may be seen in Figures 92 to 95 inclusive. Although this set up had a dentated offset in the channel floor (discussed later), the smoother flow, in comparison, at the tunnel entrance is caused by the use of a higher weir. Compare to Figure 88.

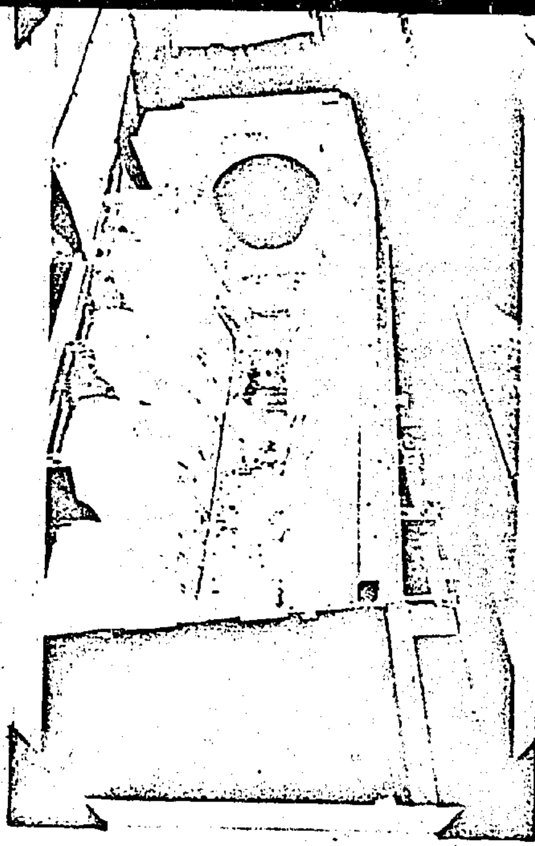


Figure 96. Set up.

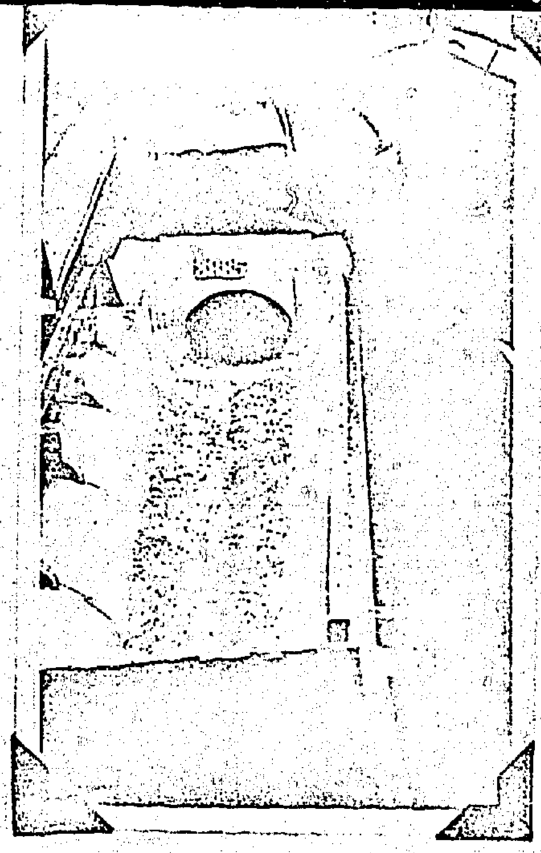


Figure 97. Q = 40,000 sec.ft

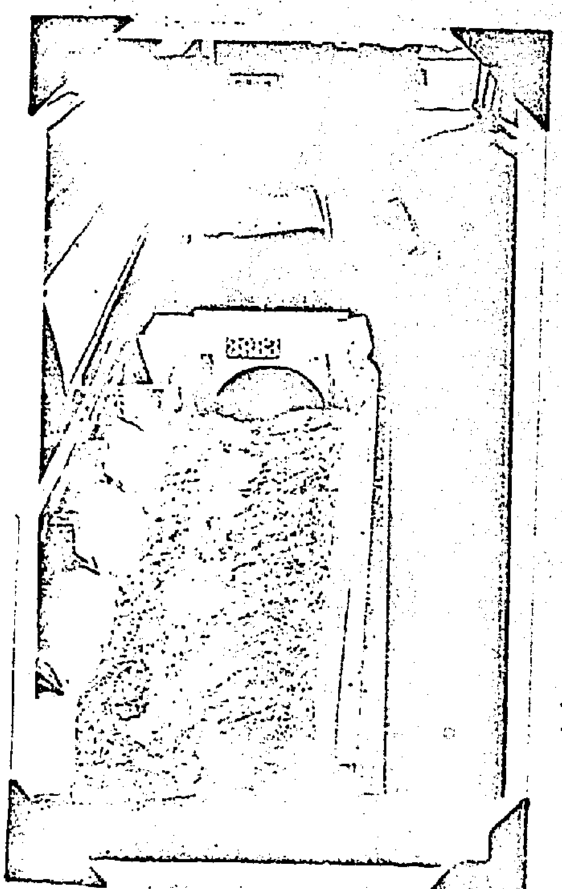


Figure 98. Q = 120,000 sec.ft.

35 FOOT WEIR ACROSS LOWER END OF CHANNEL, 10 FOOT OFFSET AT LOWER END OF CREST

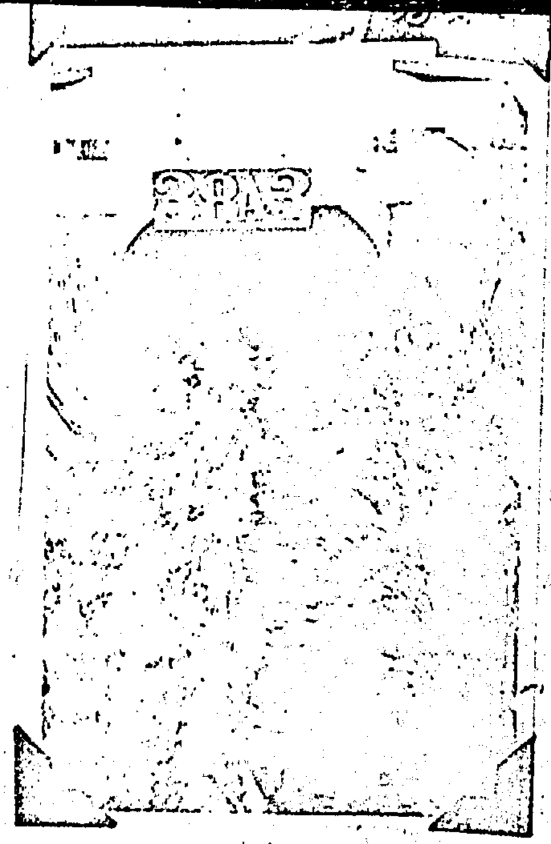


Figure 99. Q = 120,000 sec.ft.

Figure 101. Q = 100,000 sec.f

Tunnel Entrance.

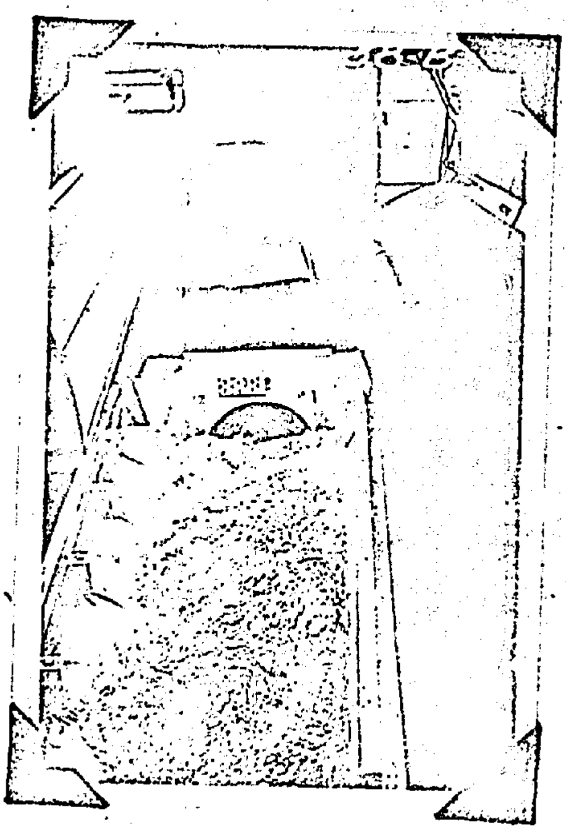


Figure 100. Q = 200,000 sec. ft.

35 FOOT WEIR ACROSS LOWER END OF CHANNEL, 10 FOOT OFFSET AT LOWER END OF CREST

The increased depth in the channel caused a submergence of the crest at the upper end that would necessitate a rise of 0.5 ft. in the forebay level to get the maximum of 200,000 c.f.s. over the crest.

Although naturally the water surface is higher over the 30 ft. weir than the others at the entrance, further down the tunnel the water surface from the higher weir is below those of the lower weirs, showing the tendency of the flow over the higher weirs to flatten out down the incline.

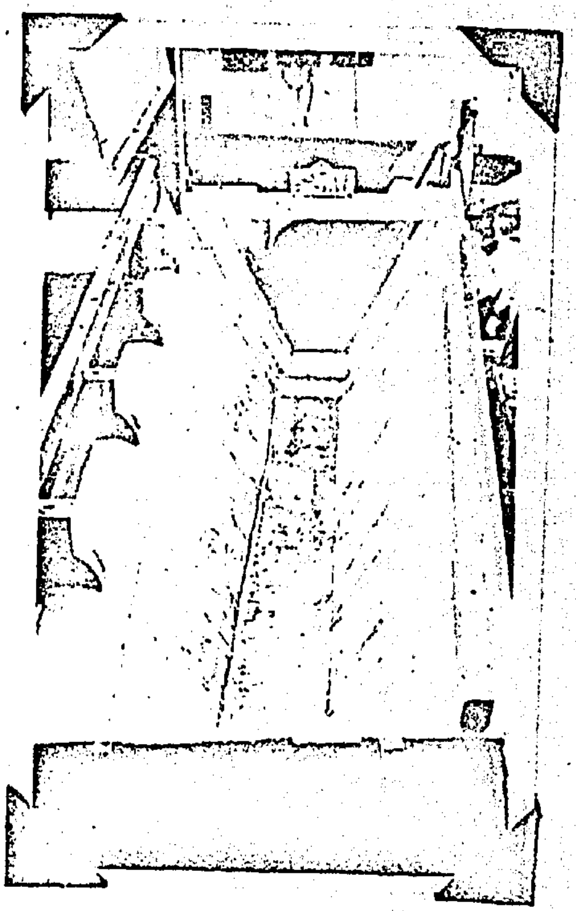
MODEL C-4b

It will be noted from the photographs (Figures 93 to 95 inclusive) that there is a depression in the water surface on the crest side and that the bulk of the flow seems to be down the right. This suggested an offset in the center line of the tunnel which would project the end wall from the crest face. The projecting wall tends to check the velocity along the crest face, fill in the depression and raise the water surface at this point. A layout with the combination of a 35 ft. weir located 60 ft. downstream from the end of the crest and a 10 ft. projecting end wall as shown on Plate XVI was built and tested. Figures 96 to 101 inclusive illustrate flow conditions in the channel.

Note the exceptionally smooth flow into the tunnel at the maximum discharge as seen in Figure 101. Down the incline the flow was well on the bottom leaving the roof clear, with only an occasional splash upwards. This set up gave better flow conditions down the tunnel than any yet tested but backed the water up in the channel causing a submergence of the crest that reduced its capacity. It would require a pond 0.60 ft. above the normal Elevation 1232 to pass the 200,000 c.f.s. Consequently, in the final design, that is, the best design developed to date, the tunnel transition was dropped 4.8 ft. making the weir at the entrance at Elevation 1140. See model C-48. The use of the lower weir decreased the submergence on the crest and increased its capacity which, however, was not yet quite up to the maximum required. This was later obtained by an alteration of the crest that increased the coefficient of discharge.

It would seem that the higher the weir the smoother the flow over it. It is believed that the elevation of the channel floor at the upper end could be decreased, even dropped to make the floor level, to cut down the submergence on the crest and a higher weir used which would result in improved flow conditions.

A slight rise in water surface especially against the crest face was noted. This however did not decrease



SET UP
MODEL C-4c

Figure No. 103.

Figure 104.

Q = 100,000 sec. ft.

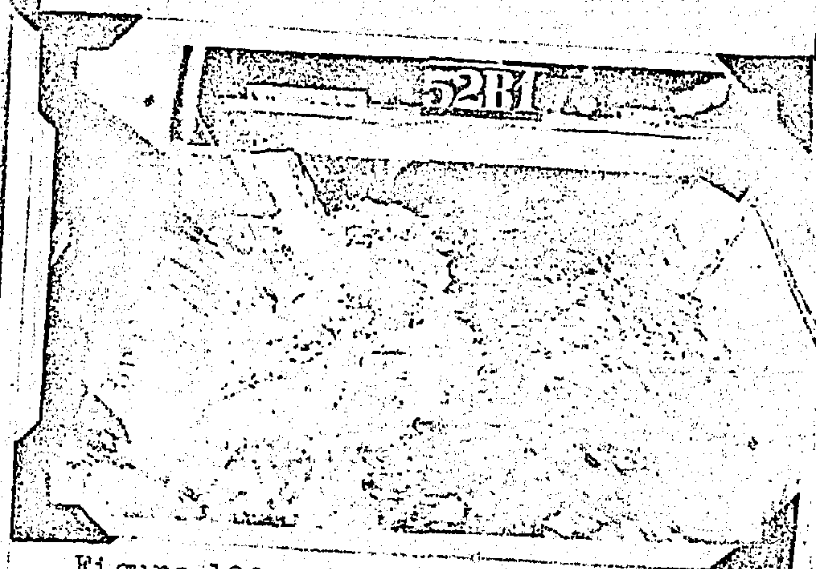


Figure 106. Q = 200,000 sec.ft.

Figure 105. Q = 200,000 sec. ft.

MODEL C-4c

MODEL C-4c, WITH BAFFLES

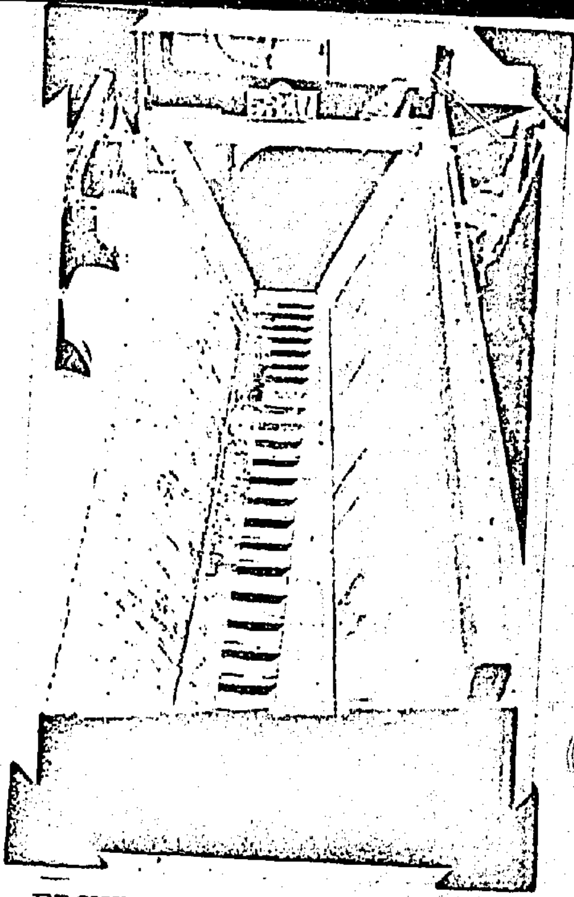


FIGURE 107. Set up.

Figure 108.

Figure 109.

Q = 100,000 sec. ft.

Figure 110.

Figure 111.

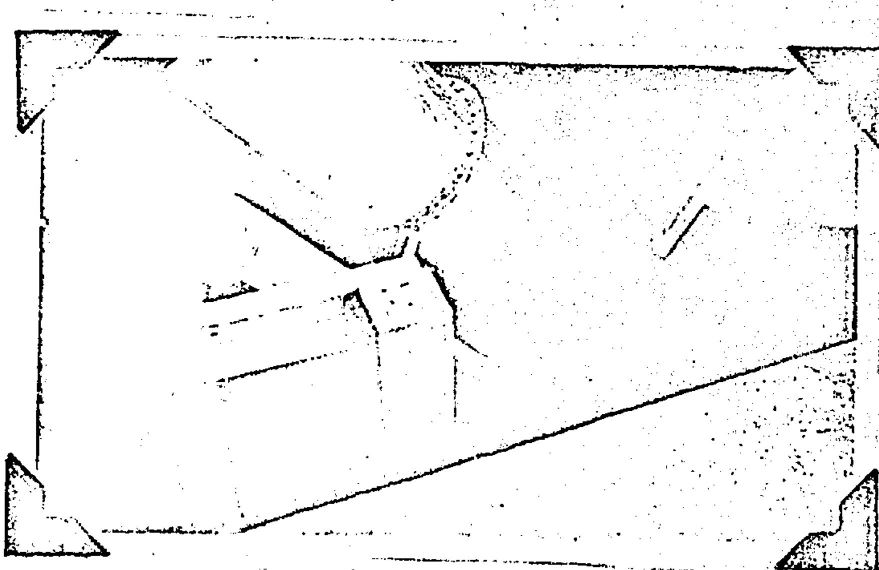


Figure 112. In Tunnel at End of Transition.

MODEL C-4c, WITH BAFFLES.
Q = 200,000 SEC. FT.

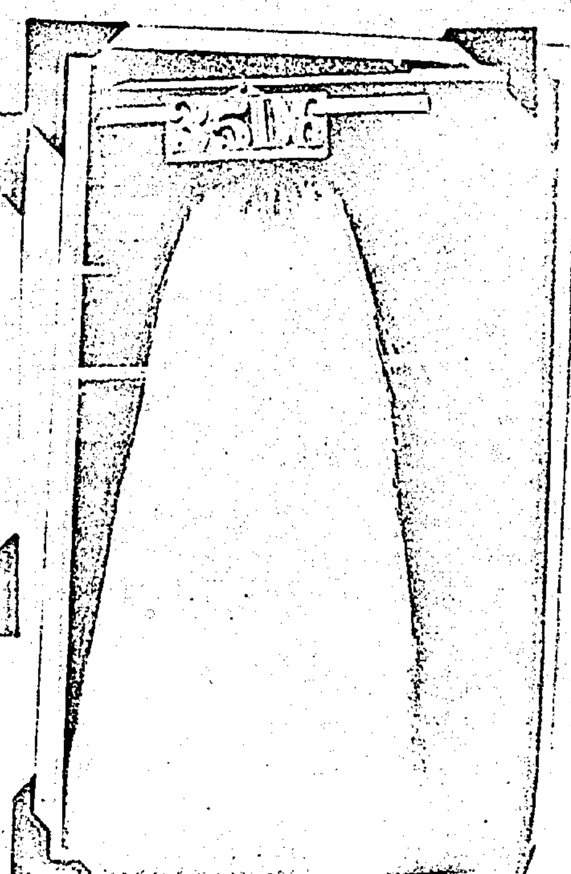


Figure 113. End of Transition Tunnel, Pipe Removed.

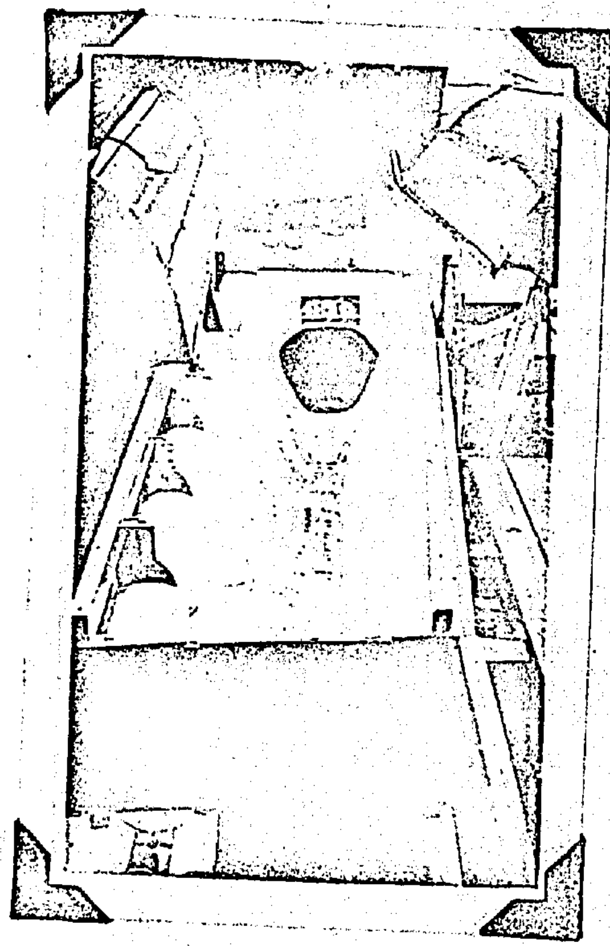
the capacity of the spillway but the rise on the left side tended to level off the surface which was beneficial.

Comparison with Test 16 may be made to show the benefits obtained from the use of a weir across the lower end of the channel. Compare Figures 100 and 88.

MODEL C-4c

It will be noted that in the set up just discussed above, see Plate XVI, the weir was raised across the end of the channel, making a length of seventy feet along its crest and the transition narrowed into the tunnel from this width.

It was seen that considerable saving in the cost of construction could be made if the original forty foot width of bottom could be held. In the set up for Model C-4c, see Plate XVII, the original design of the transition, with respect to width, was slid up the right wall to give a weir and offset as large as possible without backing the water over the crest and cutting down the capacity. Photographs of the flow, some of which were taken when a dentated baffle (discussed later) was installed in the channel are shown on Figures 103 to 113, inclusive. The improvement made by the use of the baffles was very marked in this set up as seen by comparison of the photographs.



SET UP
MODEL C-4g

Figure 115. Q = 40,000 sec.ft.

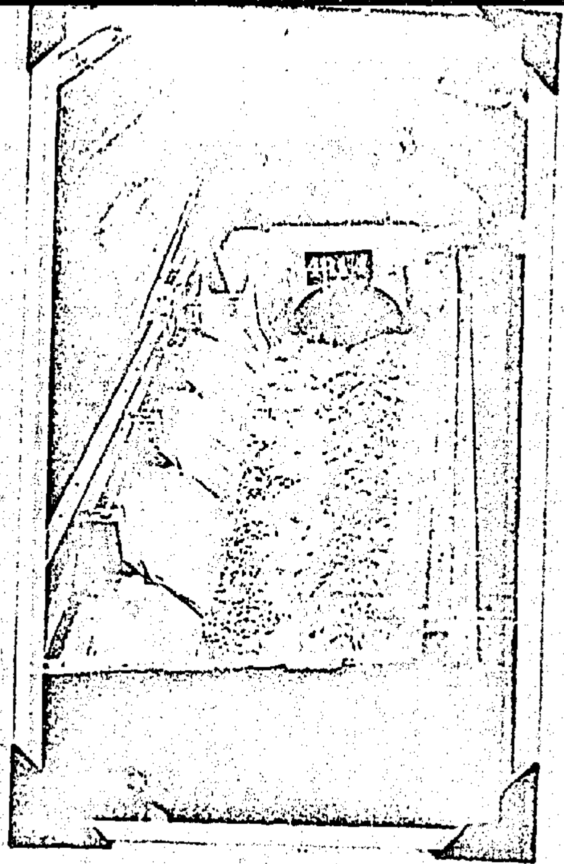


Figure 116. Q = 80,000 sec.ft

Figure 117.

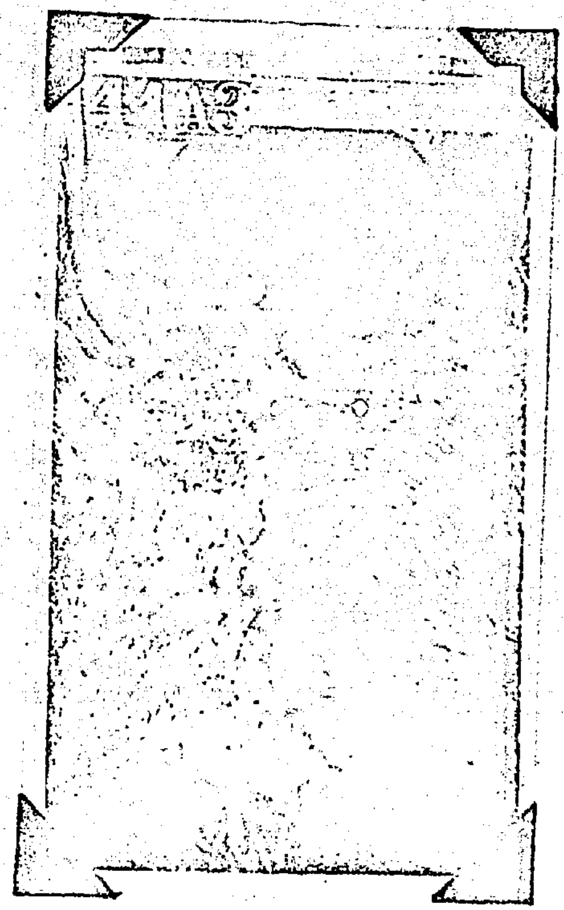


Figure 118.

Q = 120,000 sec.ft.

MODEL C-4g

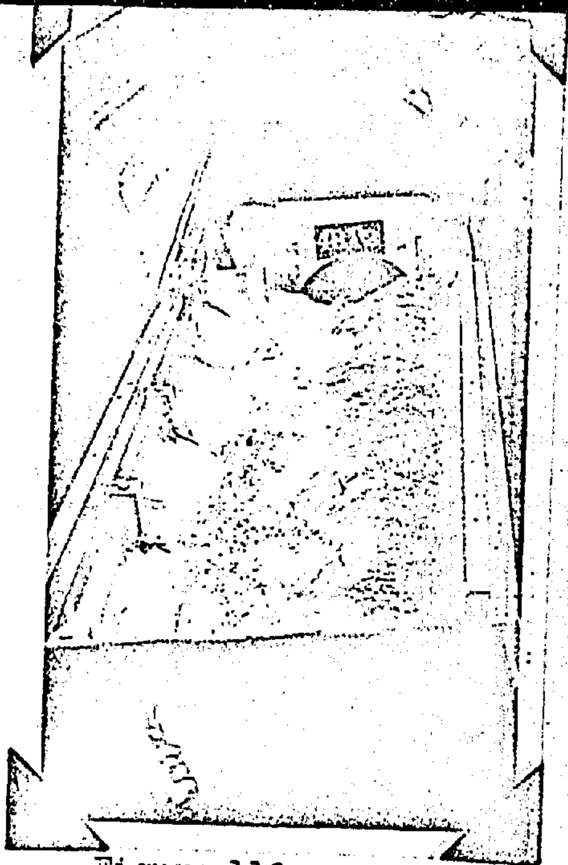


Figure 119.

Figure 120.

Q = 160,000 sec.ft.

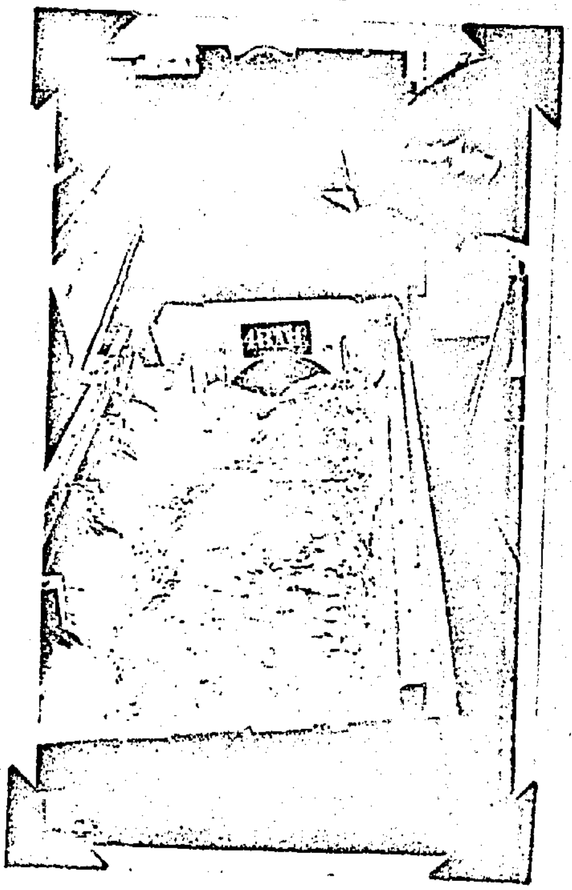


Figure 121.

Figure 122.

Q = 200,000 sec. ft.

MODEL C-4g.

MODEL C-4g, WITH BAFFLES.

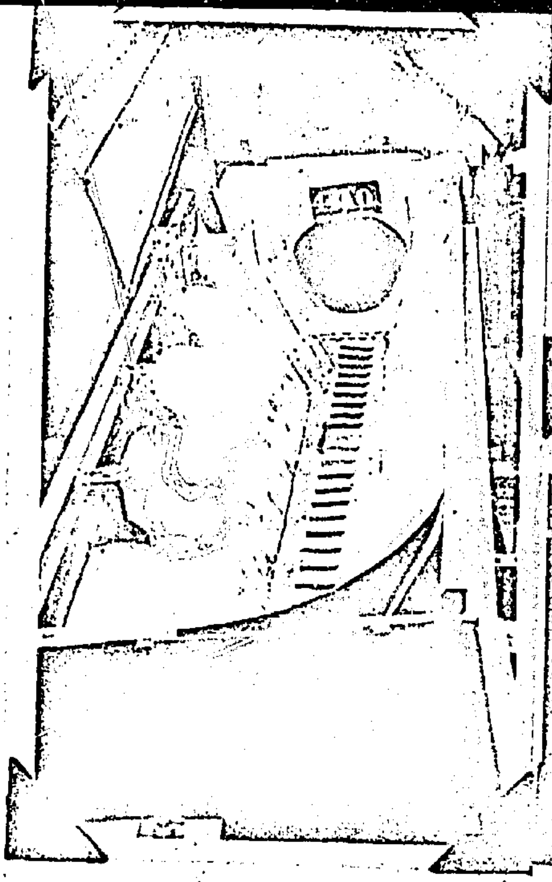


Figure 123. Set up.

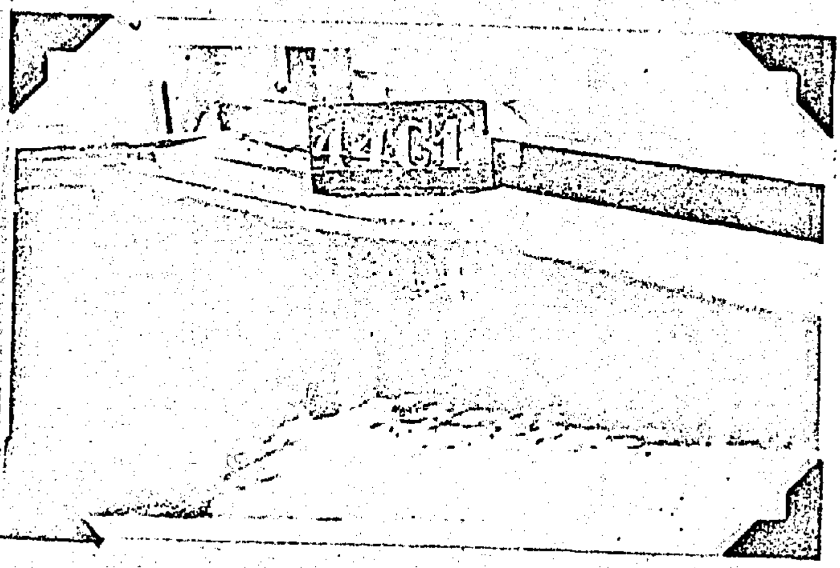


Figure 124. End Wing Wall, Q = 200,000 sec. ft.

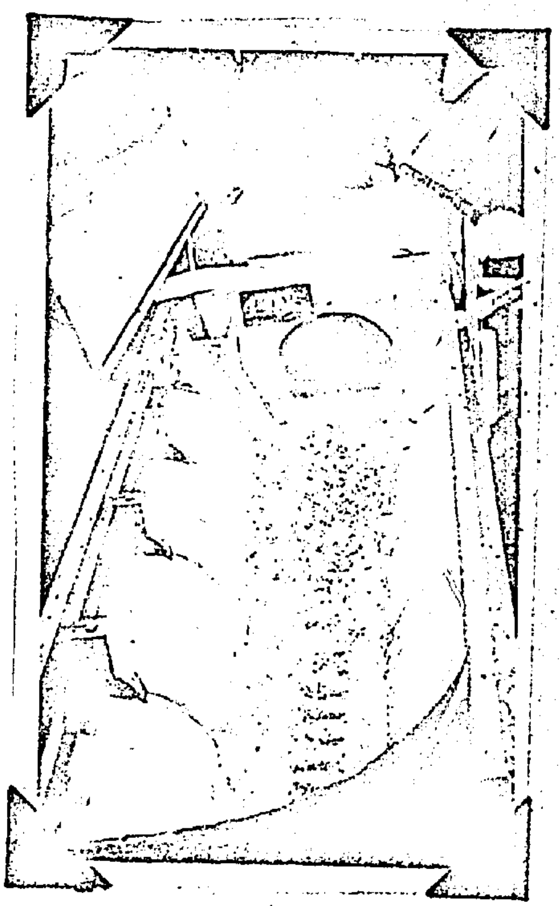


Figure 125. Q = 40,000 sec.ft.

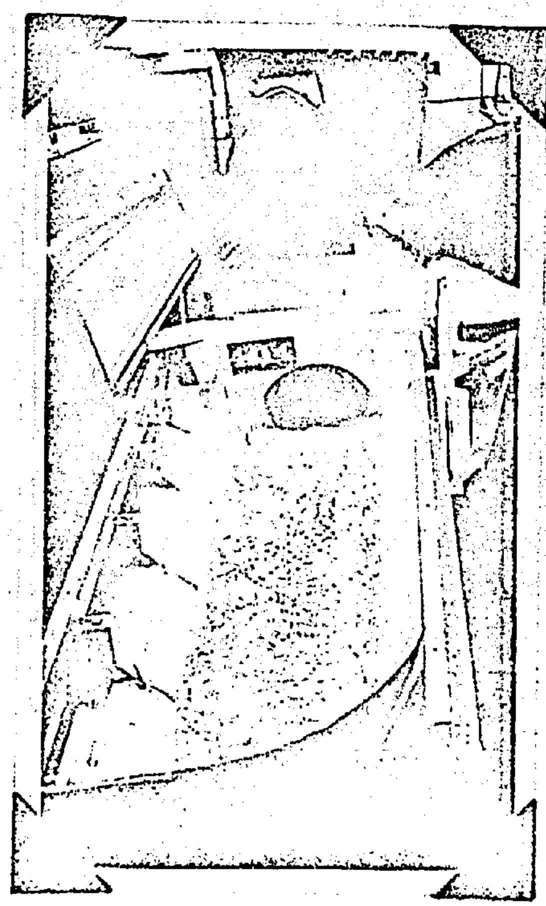


Figure 126. Q =80,000 sec.ft.

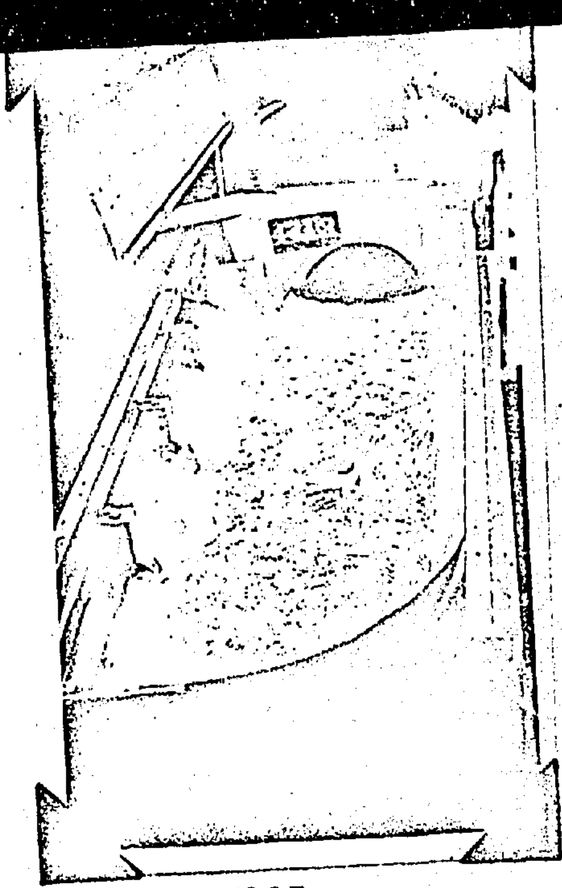


Figure 127.

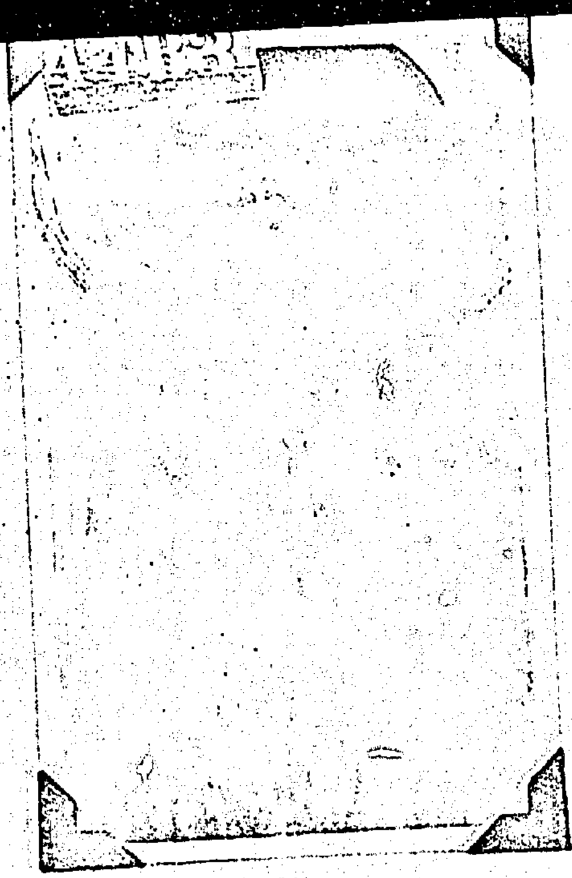


Figure 128.

Q - 120,000 sec.ft.

Figure 129.

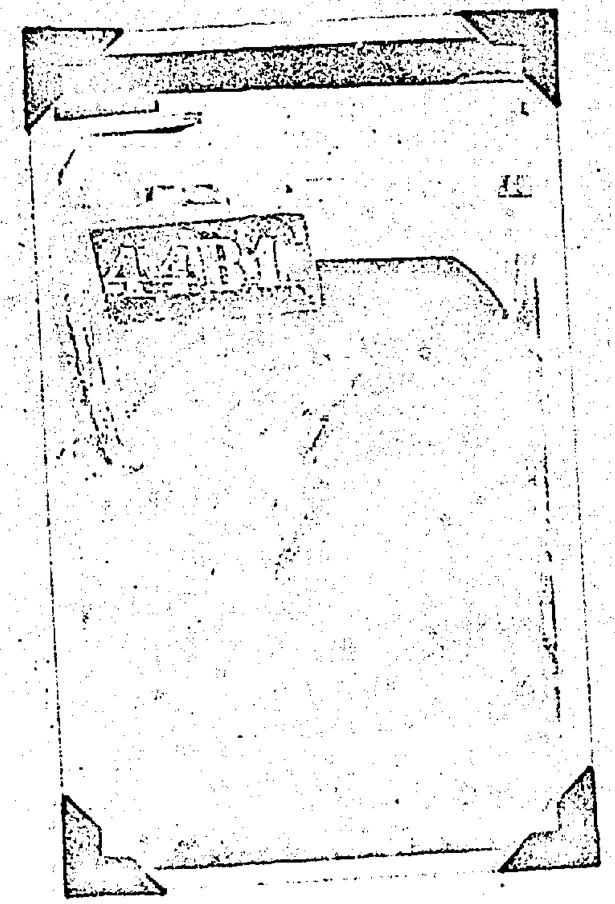


Figure 130.

Q = 200,000 sec.ft.

MODEL C-48, WITH BAFFLES

The flow down the tunnel with this set up was a shade less desirable, filling the shaft more than in the final design (Model C-4g), but appeared to be satisfactory. With the dentated baffles set in along the floor, conditions of flow were about as good as any we obtained with any set up, although they caused a higher water surface in the channel that cut the coefficient of discharge down from 3.64 (required) to 3.57. The alteration made to the crest later, however, would allow this set up to pass the required quantity.

Comparison of these tests can be made with Tests 43 and 44, Model C-4g, to check flow action in the designs using different weir lengths. From the photographs there seems to be little choice according to the behavior in the channel, although for Model C-4g the flow down the tunnel transition was a bit more satisfactory.

MODEL C-4g

This set up produced the best flow conditions of any yet tested, that is, without cutting down the spillway capacity by excessive submergence of the crest, and incorporates the most beneficial features of design developed by the investigations, i.e., the weir located downstream from the end of the spillway and the offset in the tunnel entrance, as well as the flatter transition floor parabola discussed later.

With the baffles in along the floor of the channel, an improvement in the flow down the tunnel was noted, especially at the lower flows.

Compare these pictures with those of Test 38, Pages 62a and 62b in which the same set up was used except that the weir was 4.8 ft. higher, See Plate XVI. There is seen to be a marked improvement in flow conditions in the channel, according to the photographs, in the set up with the weir only slightly higher, but the apparent decided difference was not manifested in greatly improved flow conditions down the tunnel transition, although there was some improvement noted. The use of the higher weir is in the right direction but as previously stated, the set up for Model C-4b necessitated a forebay elevation of about 1232.6 to pass the required 200,000 c.f.s. With the crest alteration made later however, it was found that the capacity would be amply carried, if no baffles were used along the floor, and would just pass the required flow with them installed.

MODEL C-4d

The set up for Model C-4d is shown on Plate XVIII and differs from Model C-4g, Plate XVI, only in the slope of the transition floor, which was steeper for this set up. The pictures of the flow in the channel for this model do not differ from those of Tests 43 and 44, shown in Figures 119 to 130 inclusive, ~~to be included here~~.

The flow down the tunnel for this set up was not as satisfactory as in Model C-4g, tending to splash more and fill the shaft to a greater depth. It was indicated that if the jet could be supported to a greater extent, that is, if the inclination of the shaft could be made flatter, better flow conditions down it would result.

MODEL C-4f

Model C-4f, Plate XIX, differs from C-4c, Plate XVII, in that the floor slope of the transition was made flatter to check the result of the tendencies indicated in the tests of Models C-4g and C-4d --that to lessen the abruptness of drop into the tunnel would be beneficial. The result was somewhat disappointing as very little improvement in the flow down the tunnel over the Model, C-4d, set up was apparent, and the conditions of flow were still not as satisfactory as those of Model C-4g.

It would seem that the high weir does the work, the higher the better, which feature is the outstanding difference between this model and Model C-4g.

Photographs of flow in this set up differ little from those of tests No. 34 and 35 for Model C-4c, Figures 110 to 113 inclusive. Photographs of the maximum flow will be found on Figures 110 and 111.

BAFFLES

Numerous types of baffles were tried in the channel to check the velocity of the spillway overflow and prevent it from piling up along the right wall, producing unbalanced flow down the tunnel.

Tests on some of the set ups will be discussed briefly later and only the two types that gave the best results will be considered here.

Figures 102 and 123 show dentated baffle that gave good results. This type of baffle is merely an offset ledge left in the bottom excavation, with overhanging blocks built projecting out from it, and is quite feasible structurally and economically.

In operation, the overflow jet is reversed back toward the spillway face, causing greater depth of flow along that side, which tends to level off the water surface, with consequent smoother and more symmetrical flow down into the tunnel. The dentates allow an infiltration of flow on the right side that has very little transverse velocity to cause piling up on the wall.

The face of the baffle or ledge was set diagonally out across the channel to give a deflecting surface downstream, tending to cause more flow along the crest side. This also eliminates a tendency of the water in the channel to more back and forth in a regular recurring roll that was noticed when the baffle face was set on the center line. See Test No. 17.

The diagonal baffle, at the lower end of the channel, draws away from the weir face leaving a barrier across the full width of the channel over which the flow is symmetrical.

Operation of the baffle at the different stages of flow can best be seen from Figures 123 to 130 inclusive, and Figures 107 to 113 inclusive. At the low flow of 40,000 c.f.s. the baffles are barely covered, but at 80,000 and 120,000 c.f.s. the beneficial flow results are beginning to be seen. Note the leveling off of the channel water surface and the comparatively smooth flow. Contrast Figures 127 and 117, which is the same set up without baffles. Note in Figure 128 the straight line transparent flow at the tunnel entrance as compared to the turbulence seen in Figure 118.

At the maximum flow of 200,000 c.f.s. the water in the channel backs up on the crest to such an extent as to deflect the overflowing sheet of water over the baffles so that they do not operate as effectively as at the lower discharges. Their beneficial action at this high flow, however, may be readily seen by contracting Figures 121 and 129.

Another type of baffle, or means of breaking the energy of the overflow that gave very good results, is pictured in Figure 131. It consists of a false bucket located rather high up on the crest with dentated openings one-third the width of the block.

The theory of operation is to create two opposing rolls that break the energy of the overflow. The horizontal bucket blocks turn some of the overflow out across the channel where it strikes the back wall and is deflected downward meeting the upward flow that passed through the dentates which had traveled to the bottom and up the back wall.

The blocks must be set high on the crest in order to allow room for the reverse roll to form under them. This is somewhat objectionable at low flows as the jet sprays off them to the channel floor. For about half the length of the crest, however, there is a pool, formed by the weir below, to receive the impact of the overflow sheet. The pool rapidly becomes deeper as the flow increases and it is only at exceptionally low flows that the bottom is exposed. It does not seem that the impact on the floor, even if exposed, would be greater than the impact on the back wall in the operation without the baffles.

The following photographs may be contrasted to check flow conditions without and with the two types of baffles discussed above. Flow down the tunnel was materially improved by the use of either baffle, conditions being more favorable with the baffle in the floor. See Figures 110 and 102 for

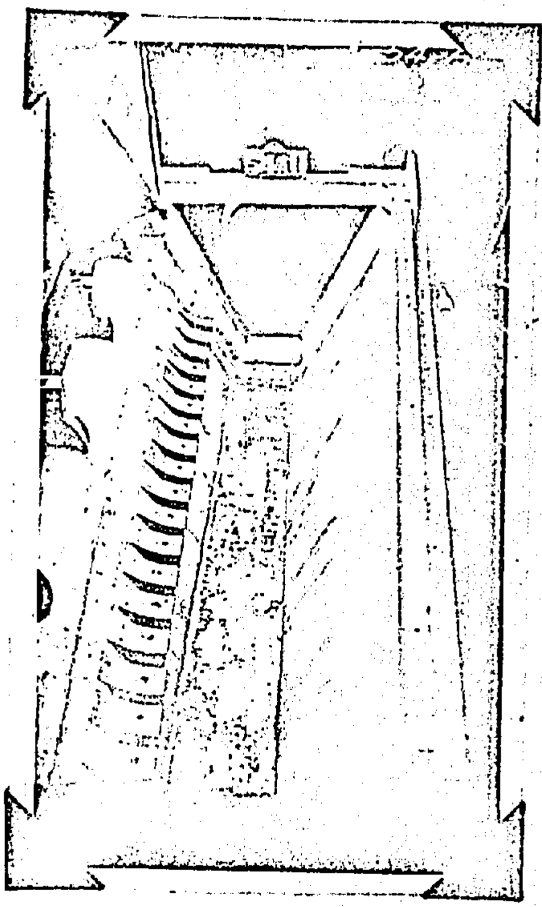


Figure 131. Set up, Crest Baffles.

Figure 132. Q = 200,000 sec. ft.

MODEL C-4f.

flow without baffles. See Figures 107 and 110 for flow with the floor baffles set in.

CREST ALTERATION

The spillway crest as first built gave a coefficient of discharge slightly less than that required to allow a flow of 200,000 c.f.s. at a head of 26.6 feet over the crest with the set up as shown in Model C-4g. This was due to some extent to the submergence on the upper end of the crest but as this submergence seemed beneficial in smoothing out the flow in the channel, an attempt was made to increase the coefficient by an alteration to the crest. This consisted of slicing away the upstream nose of the crest on a 45 degree slope. An increase of the coefficient from 3.60 to 3.76 was obtained from tests on the Model C-4g set up. A slight loss was indicated when the drum gates were installed on the crest.

The use of the baffles along the channel bottom in Model C-4g cause an increased depth of water along the crest side, cutting the coefficient down to about 3.67. With this coefficient the capacity is obtained, however, as the "C" required is 3.64.

With the dentated buckets along the crest face, the coefficient is not decreased perceptibly.

If the baffles in the bottom are not going to be used in the final design, the indications are that a slightly higher weir could be used at the transition entrance, which would aid flow conditions down the incline. A test made with a false addition to the top of the weir in Model C-4g, bringing it up to El. 1144.8, indicated a coefficient of 3.70 which is ample to allow the maximum discharge to pass.

Perhaps, the right back wall could be moved in at the upper end raising the water surface, to improve flow conditions but not enough so that the submergence causes a decrease in the maximum capacity required.

Tests up to Test 46 were made with the original crest, from Test 46 to Test 56 with the altered crest, and from Test 57 on, with drum gates installed in the crest.

The whole of the above discussion treats of the main alterations only, made during the investigations, and comments on each of the seven transition models tested, but there were numerous experiments performed on each of these models; alterations in the form of false weirs, bottoms and sides and varied designs of baffles that have not as yet been touched upon.

Some of these experiments were good and gave vital leads to the development of the final design; some, naturally were in the wrong direction.

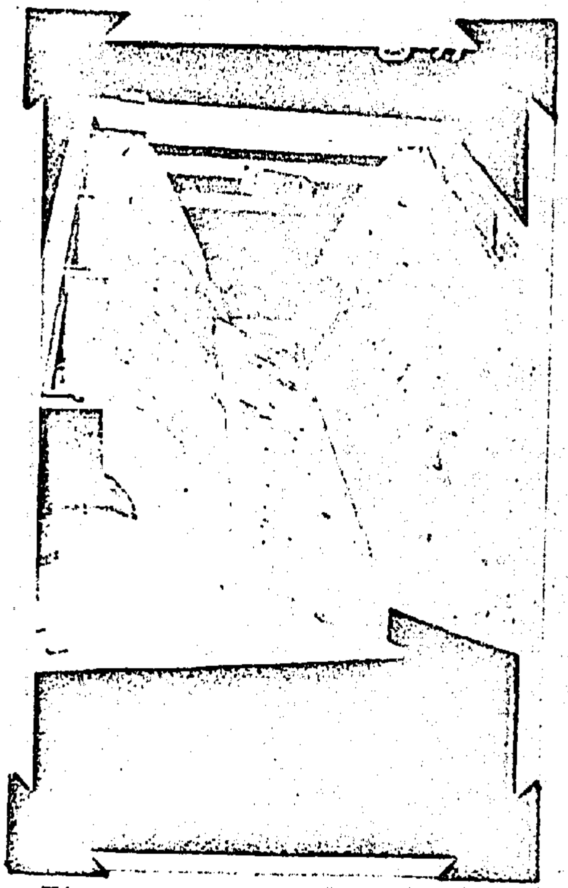


Figure 133. Set up.

Figure 134. Q=200,000 sec.ft.

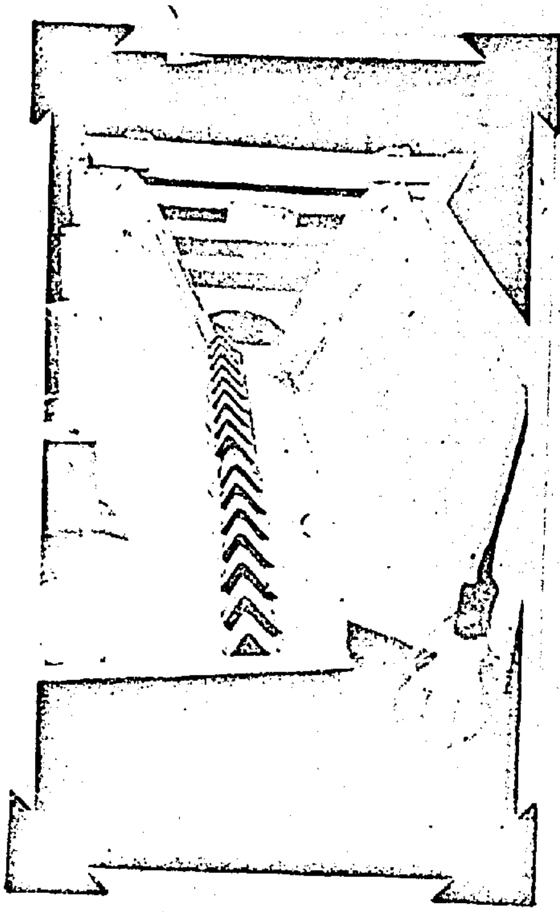


Figure 135. Set up.

Figure 136. Q-200,000 sec.ft.

With Dentated Baffle on Crest side.

MODEL C-4, FALSE BOTTOM

MODEL C-4

Figure 133 shows a false floor set perpendicular to the back side. The theory was that this would raise the water surface on the crest side as well as direct the overflow jet against a normal surface on which the tendency to ride up it would be less than if the wall was at an obtuse angle.

The photograph of the maximum flow shows that there was little improvement in the conditions. Compare with Figure 88.

A solid sill in place of the dentates was tried, but the deflected jet created too much turmoil for satisfactory performance.

A plain offset in the channel floor on the right side as pictured below, smoothed out the surface flow considerably. With a dentated sill set along the front edge, conditions of flow were even better. This led to cutting the dentates in the offset ledge itself and projecting the blocks out over the channel as seen in Figures 123 and 107 with marked improvement.

With the shelf on the left or crest side there was no improvement in the flow discernible.

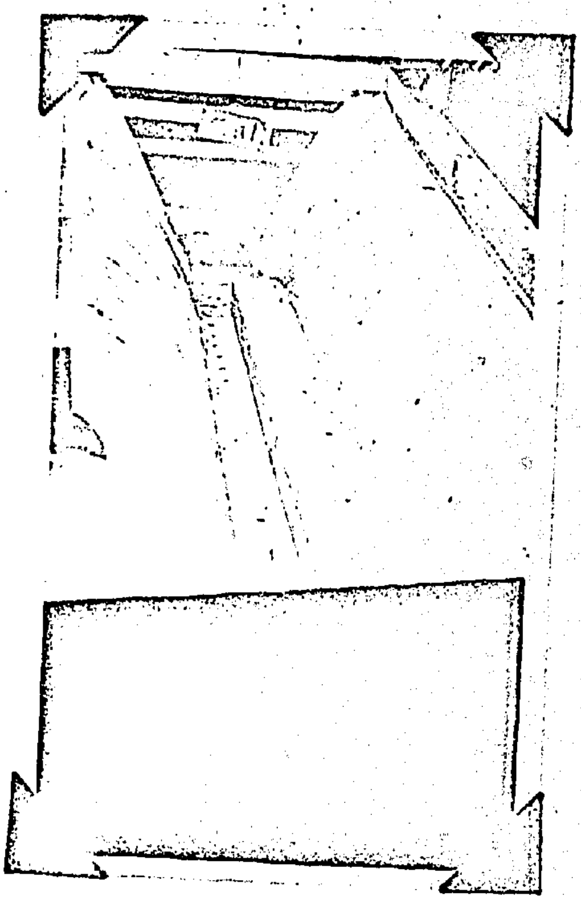


Figure 137. Set up.

Figure 138. Q = 200,000 sec. ft.

MODEL C-4
TROUGH ON CREST SIDE

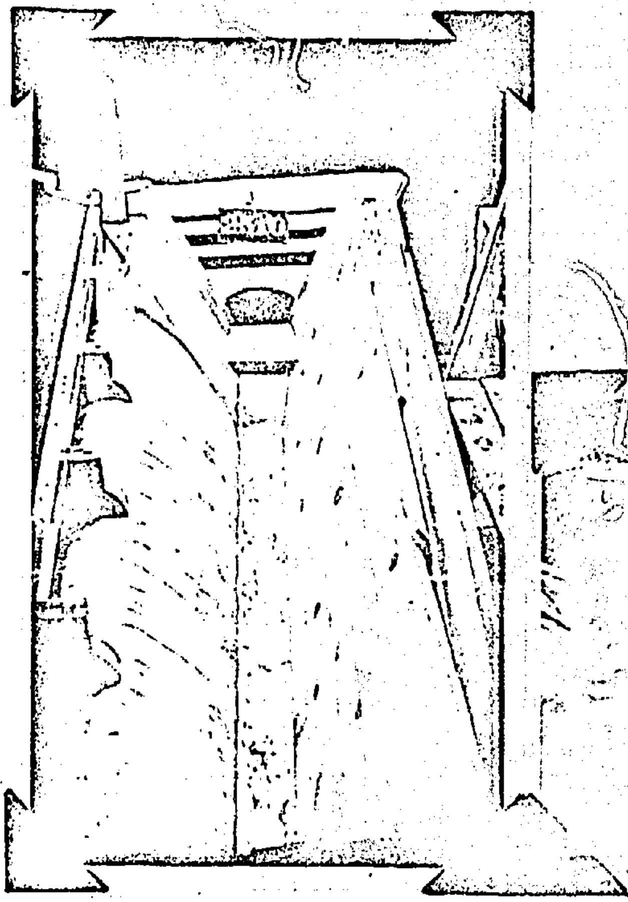


Figure 139. Set up.

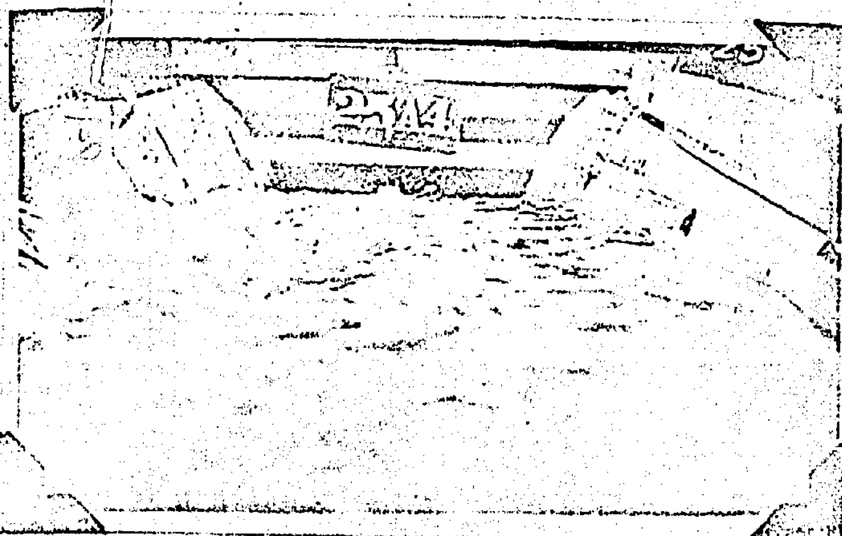


Figure 140. Q = 200,000 sec.ft.

MODEL C-4a.
FALSE WALL, WEIR AT EL. 1145

MODEL C-4a

A false right wall was placed in the channel, on the center at the upper end, and coming tangent to the original wall at the lower end. This, of course, decreased the capacity of the trough and raised the water surface which improved conditions of flow down the tunnel. However, in this instance the channel was narrowed too much and the submergence of the crest was great enough to choke the capacity discharge.

Indications were that the channel should be made narrow at the upper end with a flatter bottom slope than the original design, or in other words leave the back wall where it is and flatten the floor slope.