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HYDRAULIC MODEL STUDY of the KREMASTA DAM SPILLWAY

KREMASTA HYDROELECTRIC PROJECT ACHELOOS RIVER DEVELOPMENT GREECE



COLORADO STATE UNIVERSITY RESEARCH FOUNDATION
CIVIL ENGINEERING SECTION
HYDRAULICS LABORATORY
FORT COLLINS, COLORADO

KINGDOM OF GREECE
PUBLIC POWER CORPORATION
ATHENS, GREECE

FINAL REPORT ON A MODEL STUDY

OF THE

KREMASTA DAM SPILLWAY

KREMASTA HYDROELECTRIC PROJECT
ACHELOOS RIVER DEVELOPMENT
GREECE

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SUMMARY

A model study of the spillway for Kremasta Dam was conducted at the Hydraulics Laboratory of Colorado State University, Fort Collins, Colorado. The study was conducted for the consulting engineering firm of Engineering Consultants Inc., Denver, Colorado. Several modifications and improvements to the preliminary design resulted from this study. A functional flip bucket design was developed and information was obtained relative to potential scour and fluctuations of the power plant tailrace water levels.

Improvements are recommended to the spillway approach channel to provide better hydraulic flow conditions to the crest. Additional excavation is desirable at the right entrance channel bank and at the entrance on the left bank. Minor modifications to the entrance wing walls are recommended and a concrete protection apron should be provided at the foot of the right wing wall.

The preliminary shape of the pier nose in the gate structure was satisfactory, but a possible improvement is suggested. The gate pins should be raised to elevation 277.04 m. to be above the water surface at all discharges. A center guide wall as an

extension to the pier should be included in the transition section to reduce the standing waves and prevent overtopping of the chute walls. The importance of this extension to the pier as indicated by the model study cannot be overemphasized--the standing waves were 2 - 3 meters higher without the pier extension.

A functionally satisfactory flip bucket was developed as shown in Fig. 51 and is recommended for construction. The jet developed from the flip bucket can cause considerable scour of the river bed, depending upon actual river conditions; principally depth of alluvial material and location of bed rock. The tail water levels at the power plant, assuming calculated river stages to be correct, were not critical with respect to the tops of the walls and if the river bed between the power plant and the spillway jet impact area remains at the present level, the minimum tailrace level should be above elevation 139.0 meters.

It is recommended that the control gates on the spillway be raised and lowered equally and simultaneously to prevent severe standing wave patterns in the chute which could cause overtopping and erosion of the hillside above the power plant. Spillway rating curves with and without gated control are provided.

TERMINOLOGY

Right and Left -- As used in this report, these terms refer to the observer's right and left looking downstream.

Scale Ratio -- or scale when stated as 1:40 means model size with respect to prototype size.

Undistorted Scale -- means that the vertical and horizontal model scales were the same.

INTRODUCTION

General Description of Project

Kremasta Dam, to be constructed by the Public Power Corporation of Greece, will be an earth fill dam located in a narrow gorge of the Acheloos River approximately 55 kilometers from the town of Agrinion in Greece. The crest of the dam will be at elevation 287.0 m., approximately 150 meters above the floor of the canyon. The crest length of the dam is approximately 440 meters. The upstream face of the dam is sloped at 2.5:1 and the downstream batter is 2:1. The general layout of the dam and appurtenant works is shown in Fig. 4.

A chute spillway will be located at the left abutment, controlled by an ogee spillway crest and two radial gates each 14.7 meters high. The width of each bay is 11 meters. The maximum design spillway capacity is 3000 cubic meters per second (cms). There will be a curved approach channel to the spillway with a level channel bed at elevation 263.0 m. The elevation of the crest is 267.60 m. The chute is reduced in width from 26 m. near the crest to 18 m. in a distance of approximately 110 meters. The chute will terminate at a flip bucket with a lip elevation near 183.0 m. and the kinetic energy of the spillway flow will be dissipated in the river channel. Figure 2 shows the layout and general arrangement of the preliminary spillway.

Power will be generated initially from four turbine-generator units, each rated at 98,000 kw at a turbine discharge of approximately 100 cms. A fifth unit is planned for future installation. A semi-

outdoor power plant will house the units and is located near the toe of the dam on the left bank of the river. The power station switchyard will be situated near the top of the canyon approximately 225 meters above the power plant.

Scope of the Model Study

A model was constructed in the laboratory at Colorado State University to study hydraulics of the spillway flow and effect of spillway discharge on the power plant tail water levels. For the purpose of this study the model included a portion of the reservoir and dam, all of the spillway, and approximately 720 meters of the downstream river channel including the power plant. Specifically, the objectives of the study were to:

1. Improve the flow in the approach channel to the spillway.
2. Determine and improve the adequacy of the spillway to safely convey the maximum design discharge.
3. Determine and improve the adequacy of the transition in the spillway chute.
4. Develop a flip bucket at the end of the chute to direct the spillway flow into the river channel.
5. Determine the effect of the spillway flow on scour in the river in the area of jet impact and on the water level in the power plant tailrace.

THE MODELS

Two separate models were constructed for the performance of this study. A general model including all the pertinent project features was constructed to an undistorted scale of 1:80. A second model of only the spillway chute was constructed to a scale of 1:40. The larger scale model of the chute was used to study:

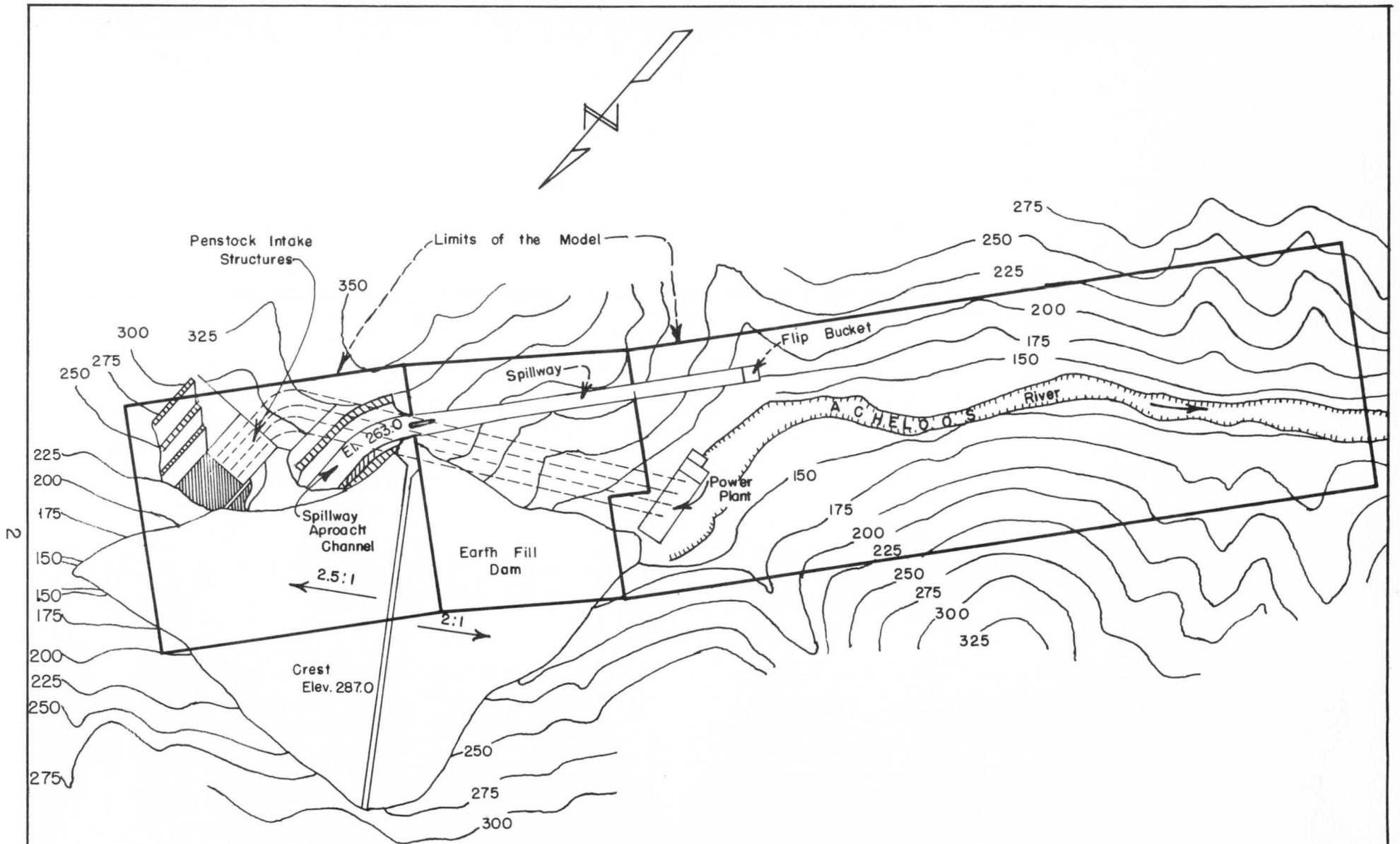
1. Crest pressures along the vertically curved boundary.
2. Effect of the pier on standing waves in the chute and development of means to reduce the wave heights.
3. Suitability of the transition with regard to standing waves.
4. Determination of the adequacy of the chute width and height of the walls at design capacity.
5. Pressures along the floor of the vertical

curve of the chute.

Schematic drawings of both the chute model and the general model are shown in Fig. 3.

The general model was used to study:

1. Adequacy of the spillway approach channel at large discharges to provide safe reservoir water levels and hydraulically smooth flow over the spillway crest.
2. Establish spillway rating curves at free flow and with partial gate openings and to determine the hydraulic problems created with non-symmetrical gate openings.
3. Develop a flip bucket to direct the spillway flow into the river channel at all discharges up to maximum capacity.



PLAN

Scale: 1:5000

Figure 1. GENERAL PLAN

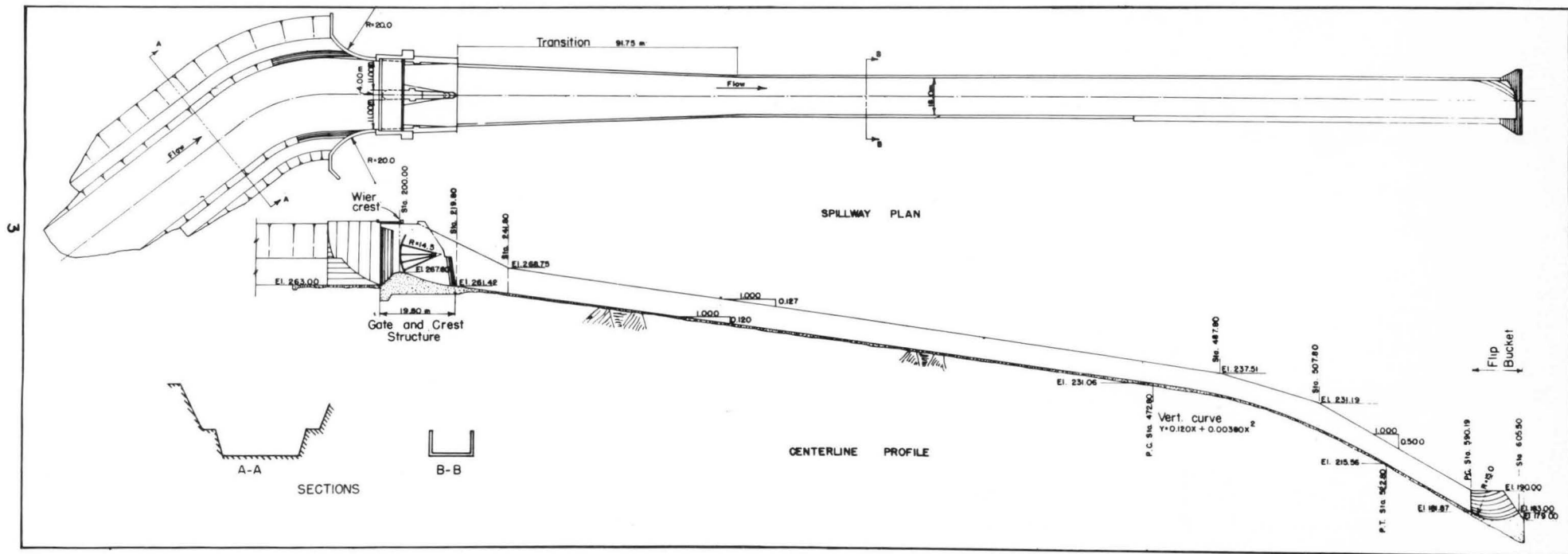


Figure 2 PRELIMINARY SPILLWAY

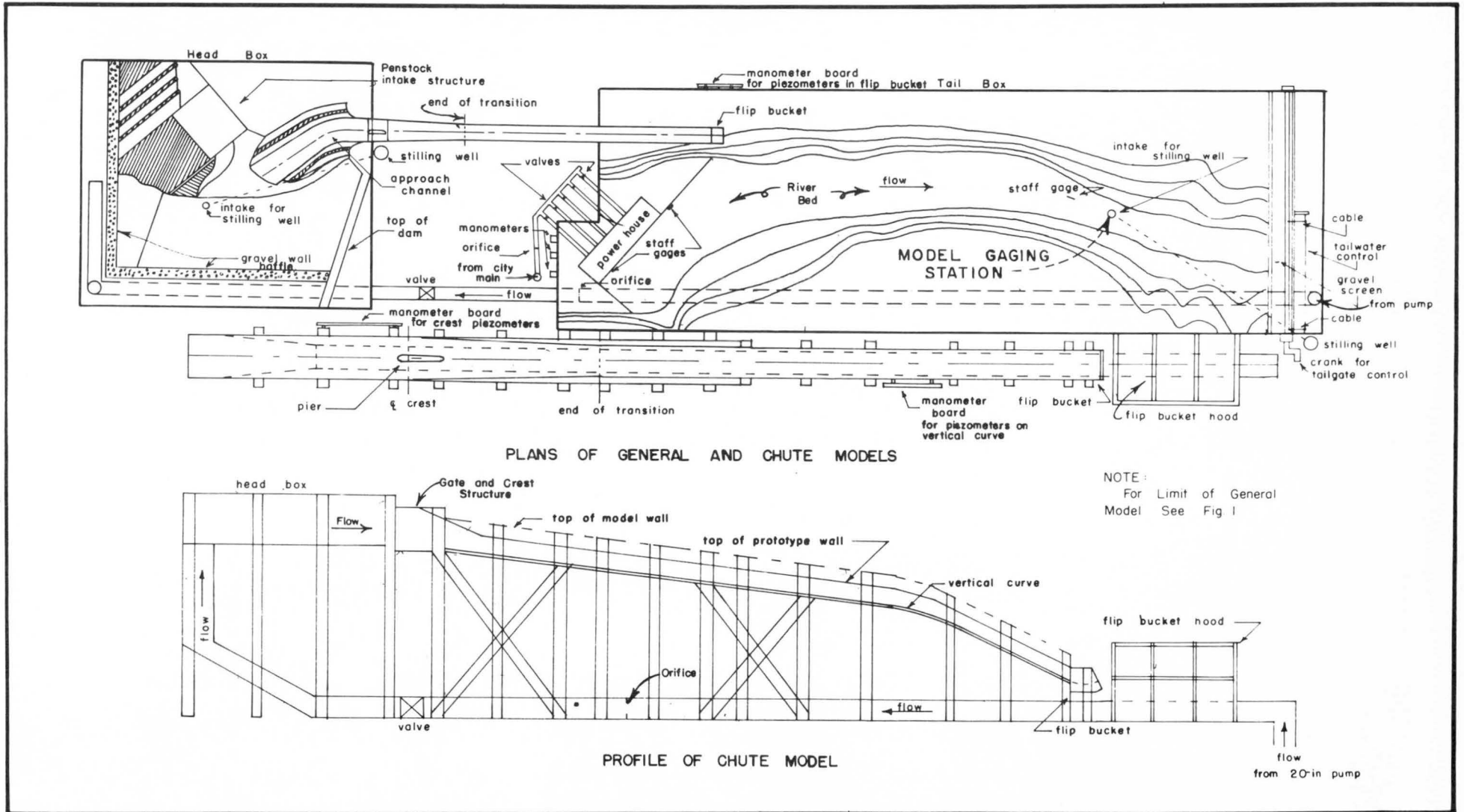


Figure 3 SCHEMATIC LAYOUT OF THE MODELS

4. Provide qualitative observations on scour in the river caused by the spillway flow and determine the resultant effects on the tailrace level of the power plant.

The model to prototype relationships of significant items for chute and general model are given in the table below.

Table of Model-Prototype Relationships

Item	Prototype	Chute Model	General Model
		Scale 1:40	Scale 1:80
Length	1 meter	0.082 ft	0.041 ft
Depth	1 meter	0.082 ft	0.041 ft
Discharge	3000 cms	10.47 cfs	1.85 cfs
	2000 cms	6.97 cfs	1.233 cfs
	1000 cms	3.49 cfs	0.616 cfs
Velocity	1 m/sec	0.519 ft/sec	0.367 ft/sec
Time	1 day	3 hrs 48 min	2 hrs 41 min
Roughness (Manning's n)			
Concrete	.014	.007	.00625
Rock cut	.025	.014	.012
River	.035	.019	.017

Chute Model

The chute model, constructed to an undistorted scale of 1:40, consisted of a head box and rectangular approach section 10 feet long, gate and crest structure, the chute transition, the downstream chute, and the flip bucket. The model is shown schematically in Fig. 3. Water was supplied to the model through a 14-inch pipeline connected to a 20-inch turbine pump. The flow was measured by a calibrated sharp edge stainless steel orifice in the pipeline and regulated by a gate valve downstream of the orifice.

The crest was constructed in the manner depicted in Fig. 4. Templates forming the crest were cut with a milling machine and covered with a 24-gauge sheet metal shell as shown in Figs. 5 and 6. Piezometers were installed near the centerline of the left bay at the locations shown in Fig. 7. These piezometers were connected to a manometer board, to measure pressure heads. Piezometers were also installed along the centerline of the vertical curve at the locations shown in Fig. 8. An overall view of the completed model is shown in Fig. 9.



Figure 4. Construction and assembly of the crest structure for the chute model.

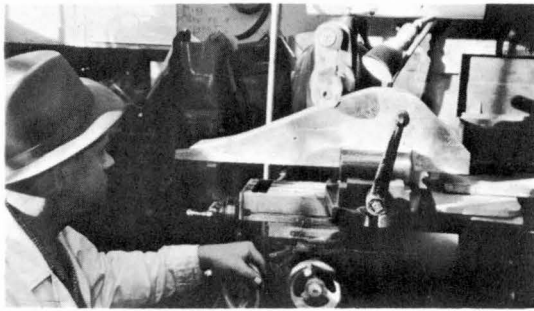


Figure 5. Shaping crest templates.

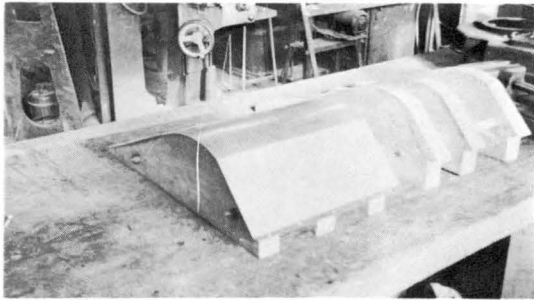
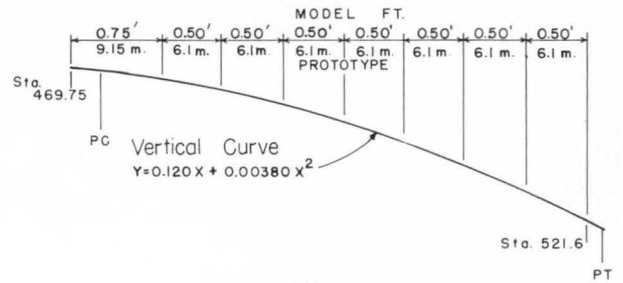
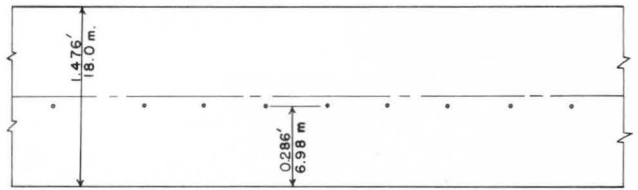


Figure 6. Crest templates formed the foundation for the sheet metal shell which formed the surface of the crest. Lacquer was sprayed on the surface and sanded smooth.

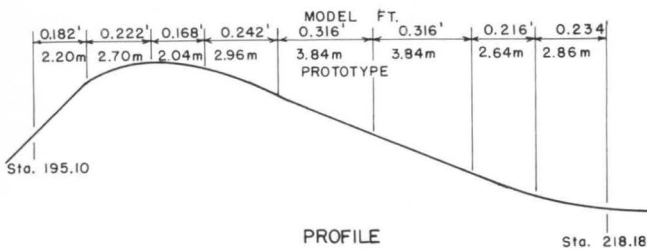


PROFILE



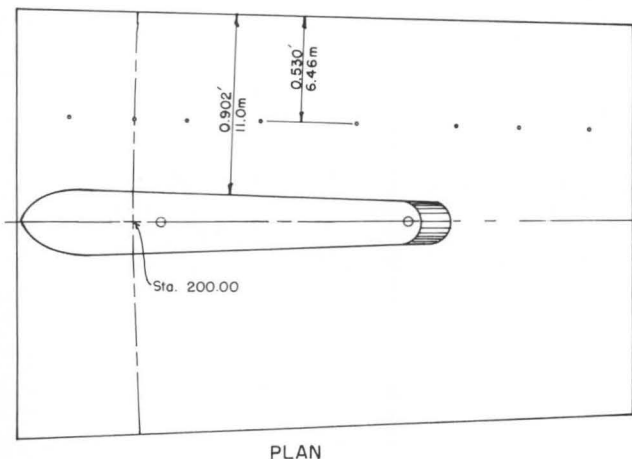
PLAN

FIGURE 8. LOCATION OF PIEZOMETERS IN VERTICAL CURVE



PROFILE

Sta. 218.18



PLAN

FIGURE 7 LOCATION OF PIEZOMETERS IN THE CREST



Figure 9. Completed chute model as viewed from the downstream end of the chute. The lateral lines are 10-meter markers used to aid laboratory measurements.

General Model

The limits of the general model are outlined in the plan of Fig. 1 and drawn schematically in Fig. 3. The model was built in two parts, a reservoir section and a downstream river section, which were connected by the spillway chute. The reservoir section included a portion of the left abutment of the dam, all of the spillway approach channel, the penstock intake cut area and a sufficient distance into the reservoir area to prevent unnatural model flow effects. The inside dimensions of the reservoir or head box were 12 feet wide by 14 feet long representing a prototype coverage of 293 by 342 meters respectively. A photograph of the completed model reservoir section is shown in Fig. 10. The spillway approach channel was constructed in removable sections in anticipation of need to change the alignment but a change was not required.

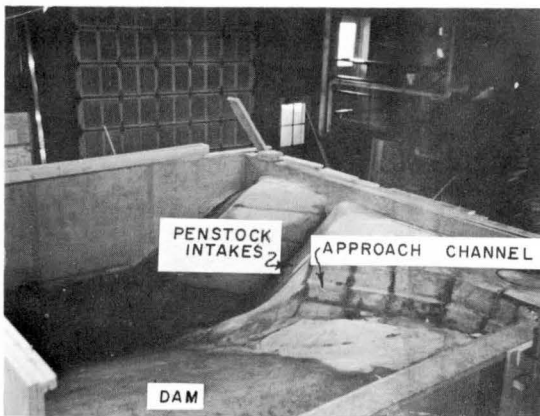


Figure 10. Completed topography in reservoir head box.

Flow entered the head box from a corner of the reservoir section through an L-shaped manifold. A rock-fill baffle was installed along two sides of the box. The purpose of the baffle was to establish uniform flow towards the spillway approach channel. Topography in the reservoir was constructed with concrete having a brushed finish and the surface of

the concrete in the approach channel was trowel-finished in order to approximate the rock-cut roughness in the prototype.

Topography of the downstream river section was constructed according to ECI Drawing AX-T-7. The completed model is shown in Fig. 11. The concrete river banks were extended vertically to the floor of the model since the depth from the prototype river bed to rock was unknown. A depth of 1 ft was provided in the model to allow for this unknown. As will be described elsewhere in this report, three river bed conditions were studied to cover a sufficiently wide range of conditions.

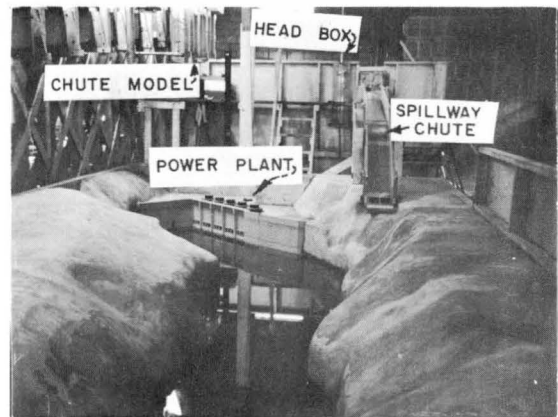


Figure 11. Completed general model.

Water was supplied to the general model by an 8-inch turbine pump and flow was measured with a calibrated pipe orifice upstream of the control valve. Flow to the power plant was provided from the city water line and controlled by separate valves to each turbine unit. Figure 12 shows the construction of the draft tube units in the model and Fig. 13, the completed power plant. The draft tubes were constructed to provide independent controlled flow through each unit. The river level was controlled by a flap gate at the end of the model.

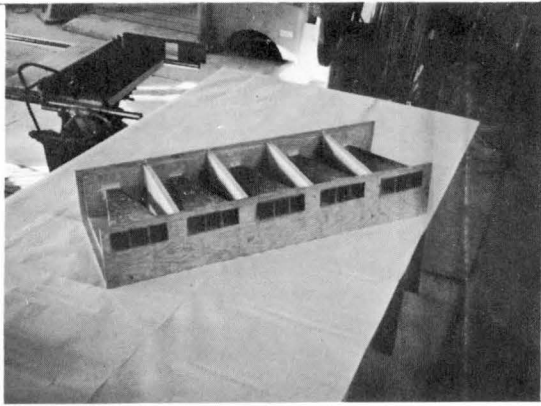


Figure 12. Model draft tubes. Dimensions of the draft tubes were scaled to size



Figure 13. Completed power plant in the general model.

MODEL TESTS AND RESULTS

Chute Model

Spillway Crest -- Pressure head measurements on the spillway crest in meters of water (prototype) were made at various discharges with and without gated control. The data are given in the

Appendix of this report and the significant data are shown graphically in Figs. 14 and 15. As the data and figures show, the pressures along the crest were positive for all flows and gate openings.

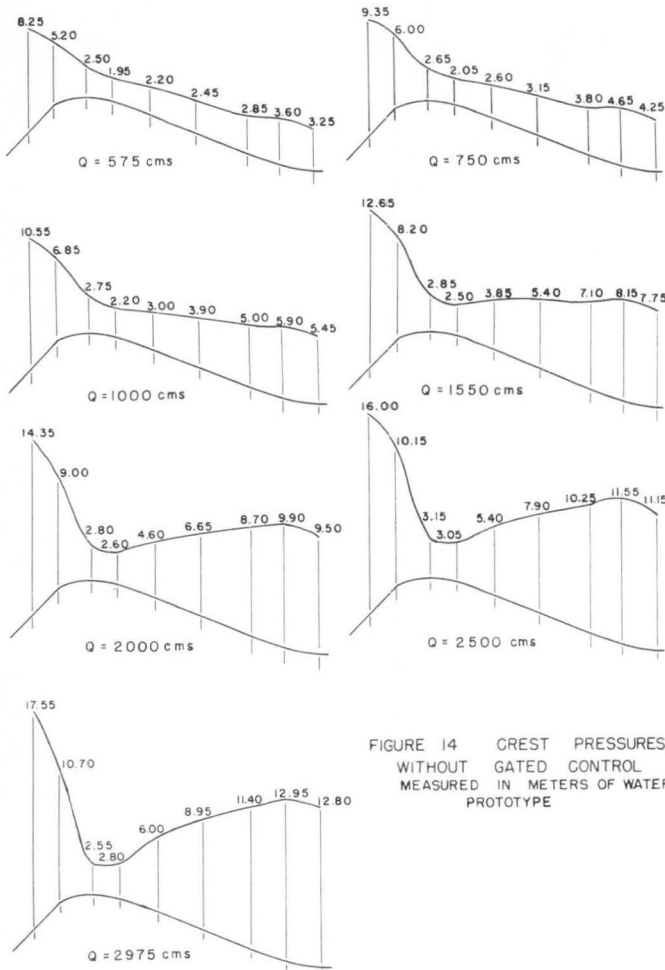
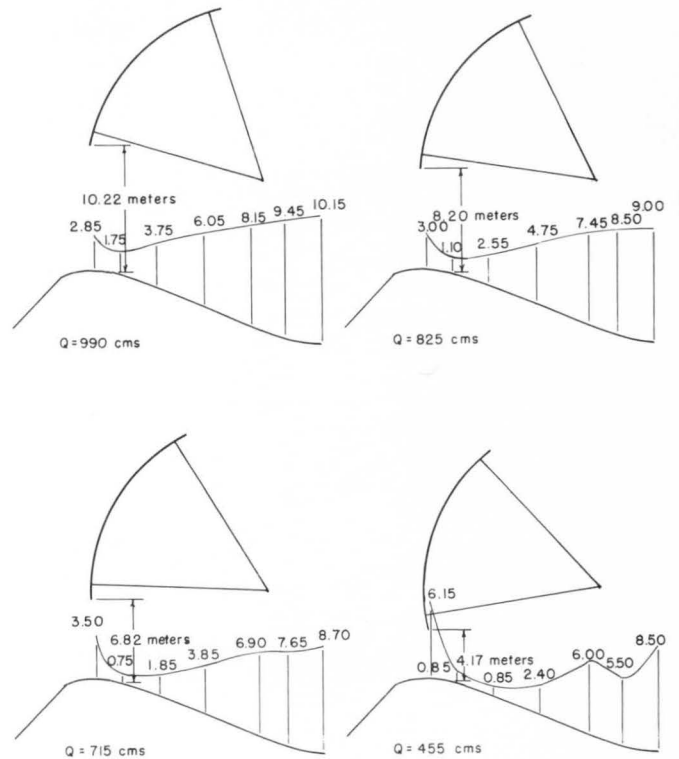


FIGURE 14 CREST PRESSURES WITHOUT GATED CONTROL MEASURED IN METERS OF WATER PROTOTYPE



Note: Water depth above crest = 14.08 meters
 FIGURE 15 CREST PRESSURE WITH GATED CONTROL MEASURED IN METERS OF WATER PROTOTYPE

Flow conditions over the spillway crest are illustrated by the series of photographs in Fig. 16. There was some pile-up of water at the pier nose and because of the acceleration of the flow around the pier on both sides, there was draw-down of the water surface near the pier. The amount of draw-down at the crest is shown by the lateral profiles in Fig. 17 together with longitudinal water surface profiles along the crest structure wall and the pier for discharges of 1000, 2000, and 3000 cms. The preliminary position of the gate pins was too low, for at the maximum discharge of 3000 cms, the gate pins were periodically underwater as indicated in the photograph of Fig. 16 (d)

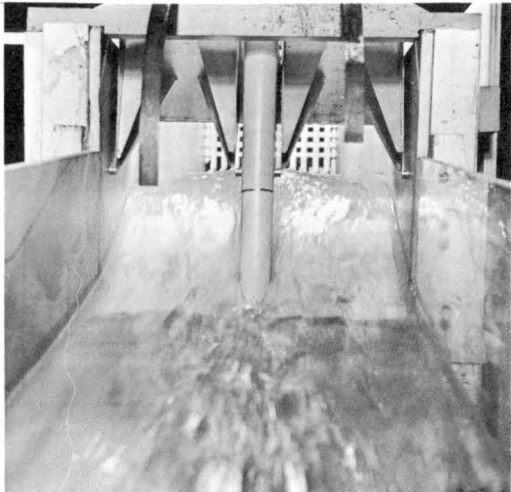


Figure 16 (a). Flow through gate structure looking upstream. $Q = 1000$ cms.

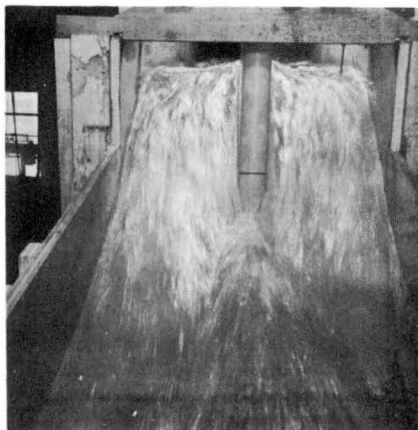


Figure 16 (b). Looking upstream at gate structure. $Q = 3000$ cms.



Figure 16 (c). Flow profile through gate structure. $Q = 2000$ cms.

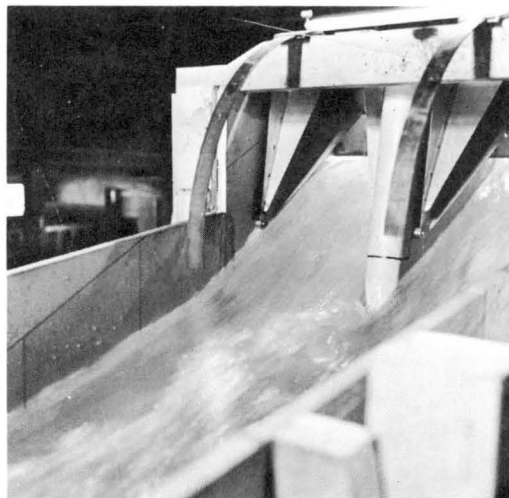


Figure 16 (d). At maximum spillway capacity the gate pin in the wall was at the water surface. The pin was subsequently elevated by tilting the gate slightly forward.

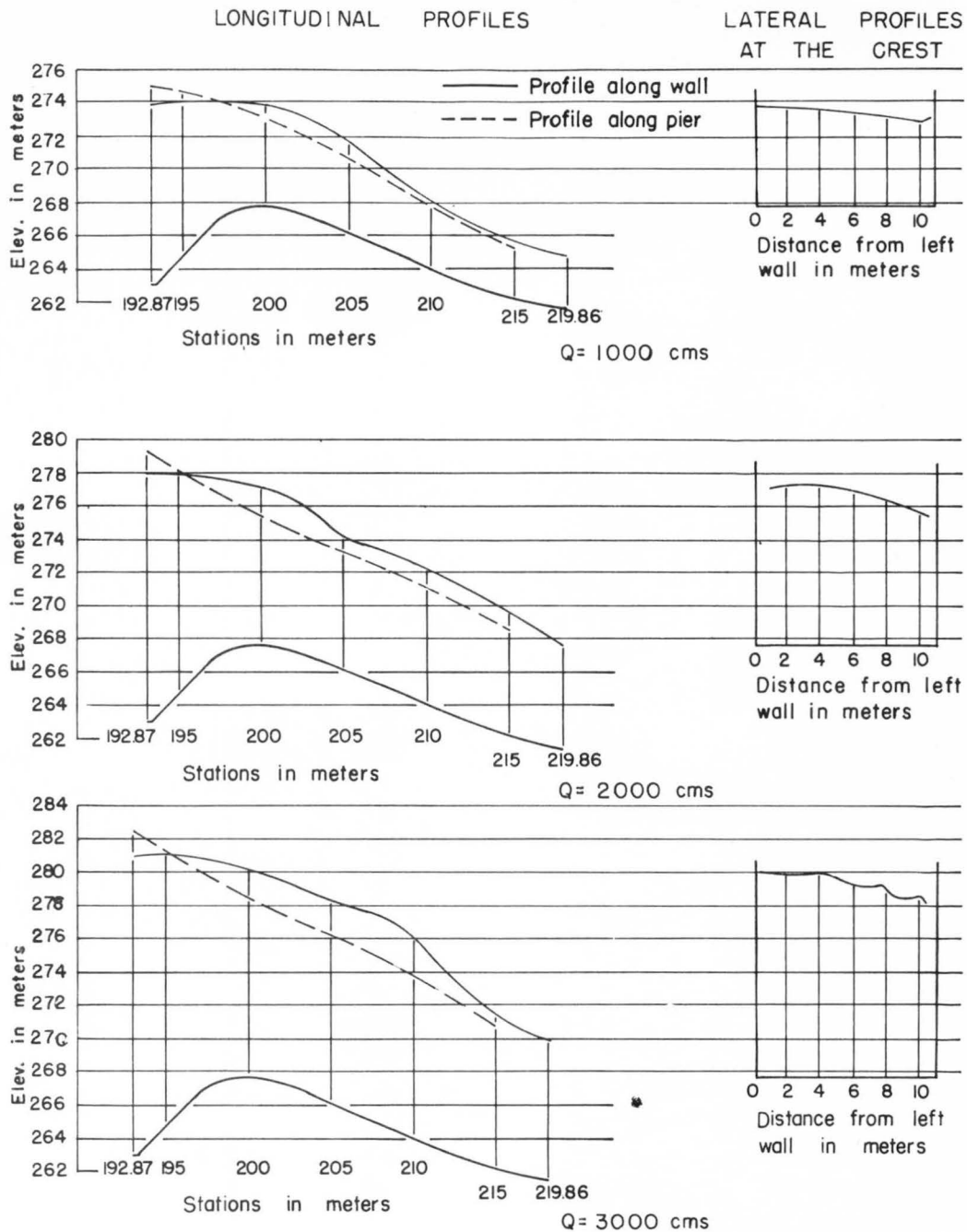


FIGURE 17 WATER SURFACE PROFILES IN THE
PRELIMINARY CREST STRUCTURE

Spillway Chute -- A standing wave (fin) was created at the downstream end of the pier because of the confluence of the flows from the two spillway bays at a slight angle. The fin (see Fig. 18) was the origin of a pronounced standing wave pattern which existed along the entire length of the chute. At several locations the waves overtopped the chute wall as is shown in Fig. 19. Although the chute walls were

overtopped only at discharges near 3000 cms in the model, in the prototype structure, overtopping may occur at a slightly less discharge because of bulking due to air entrainment. Model measurements of water surface profiles at the left wall and along the centerline of the chute with the preliminary pier were taken as shown in Figs. 20 through 22 for discharges of 1000, 2000 and 3000 cms.

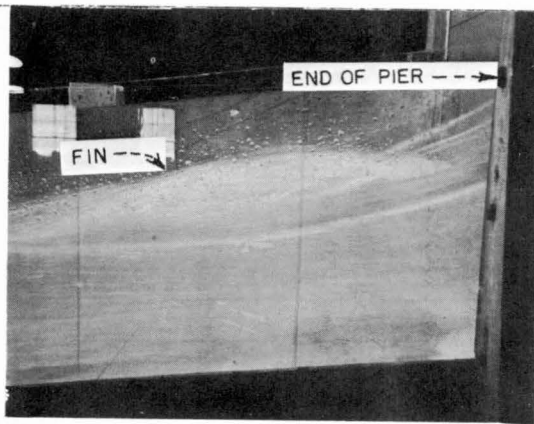


Figure 18. Fin (light area with convex top surface) at the downstream end of the pier. $Q = 3000$ cms.

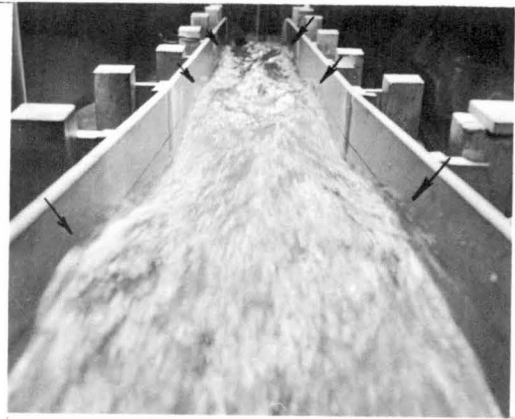
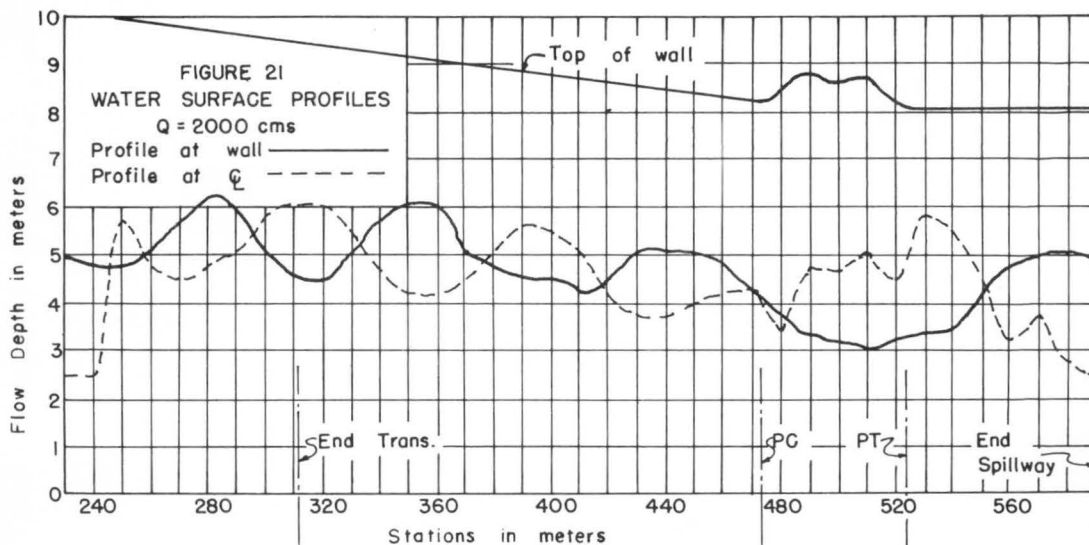
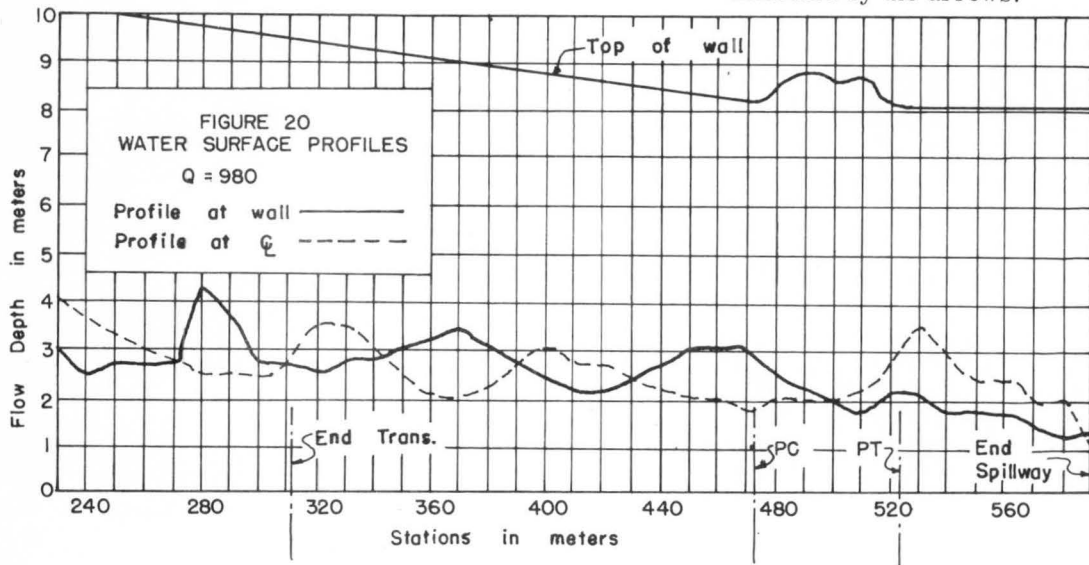
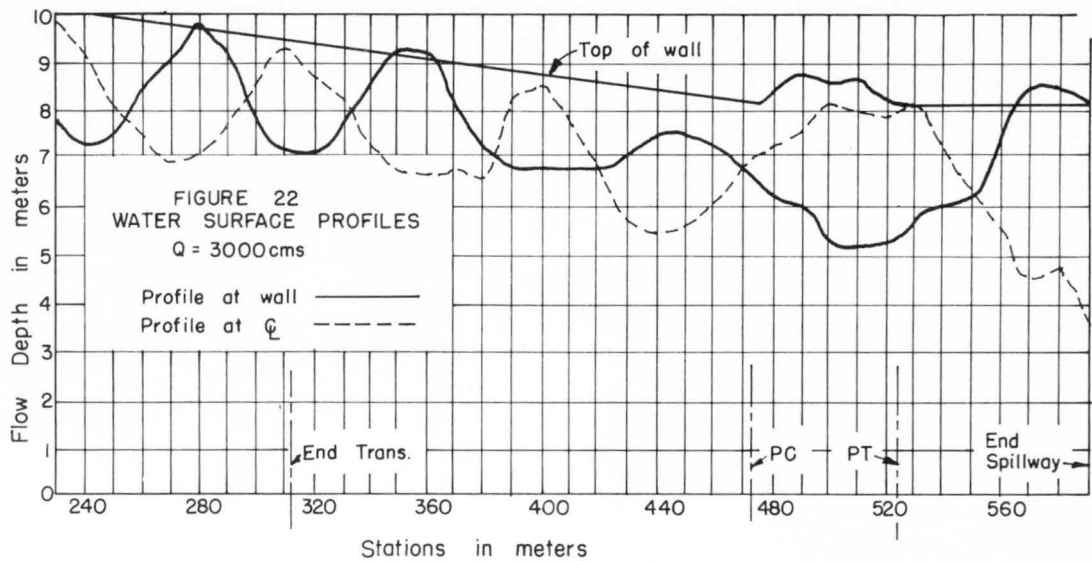


Figure 19. Standing waves in the chute at $Q = 3000$ cms looking downstream. The horizontal lines painted on the model walls represent the tops of the prototype walls. Note waves overtopped walls at the points indicated by the arrows.





The thickness of the pier at the downstream end was the primary contributor to the formation of the fin. In an attempt to eliminate the fin, the pier was undercut below the gate pins in the manner shown in Fig. 23. The undercut did not successfully reduce the standing waves, however, and because structural requirements of the pier prevented further undercutting, pier extensions were tested as a means of eliminating the standing waves.

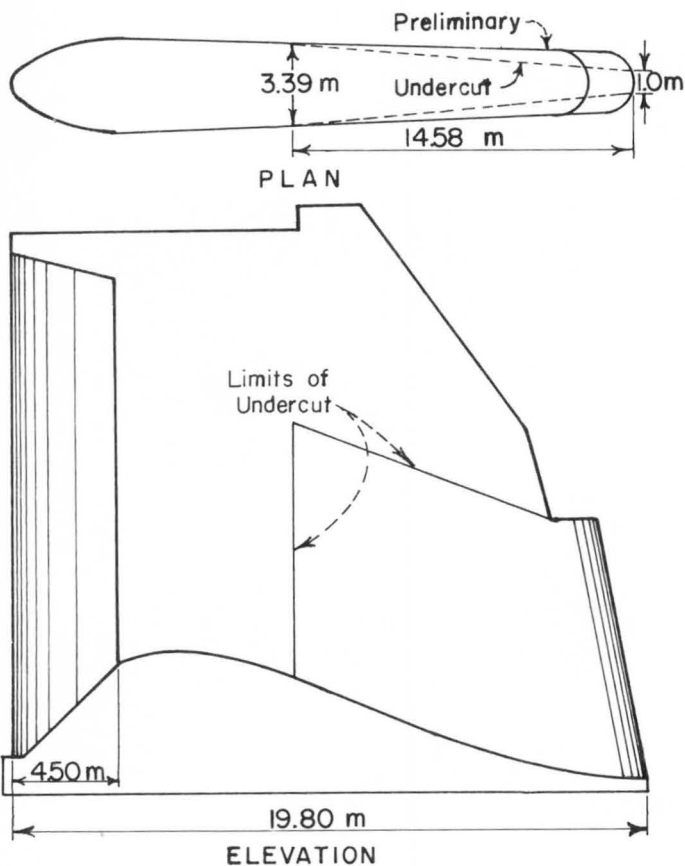


FIGURE 23 PIER UNDERCUT

Various lengths of extensions from 5 to 30 meters, as shown in Fig. 24, were tested. The results of the tests are shown in Figs. 25 (a) to (d). The water surface profiles along the wall were measured and are shown in Figs. 26 to 31 inclusively. These results show that the extension wall should be at least 20 meters long to effectively reduce the wave heights. Wave heights with pier extensions longer than 20 meters were not substantially more effective. A profile view of the flow in the chute transition with a 20-meter pier extension at 3000 cms is shown in Fig. 32. Reduction in wave height at the beginning of the flip bucket effected by the pier extension of 20 meters is comparatively shown in Figs. 33 (a) and 33 (b). The standing waves originating from the change in wall alignment at the end of the transition were not serious enough to cause concern, see Fig. 25 (d). After discussion of this observation with the engineers of ECI, it was decided that a change in transition length was not necessary and the preliminary transition length would be adopted for construction.

Vertical Curve -- Pressures on the vertical curve were measured at various discharges at the points along the curve indicated in Fig. 8. No negative pressures existed at any of the points of measurement. The data are tabulated in Appendix B.

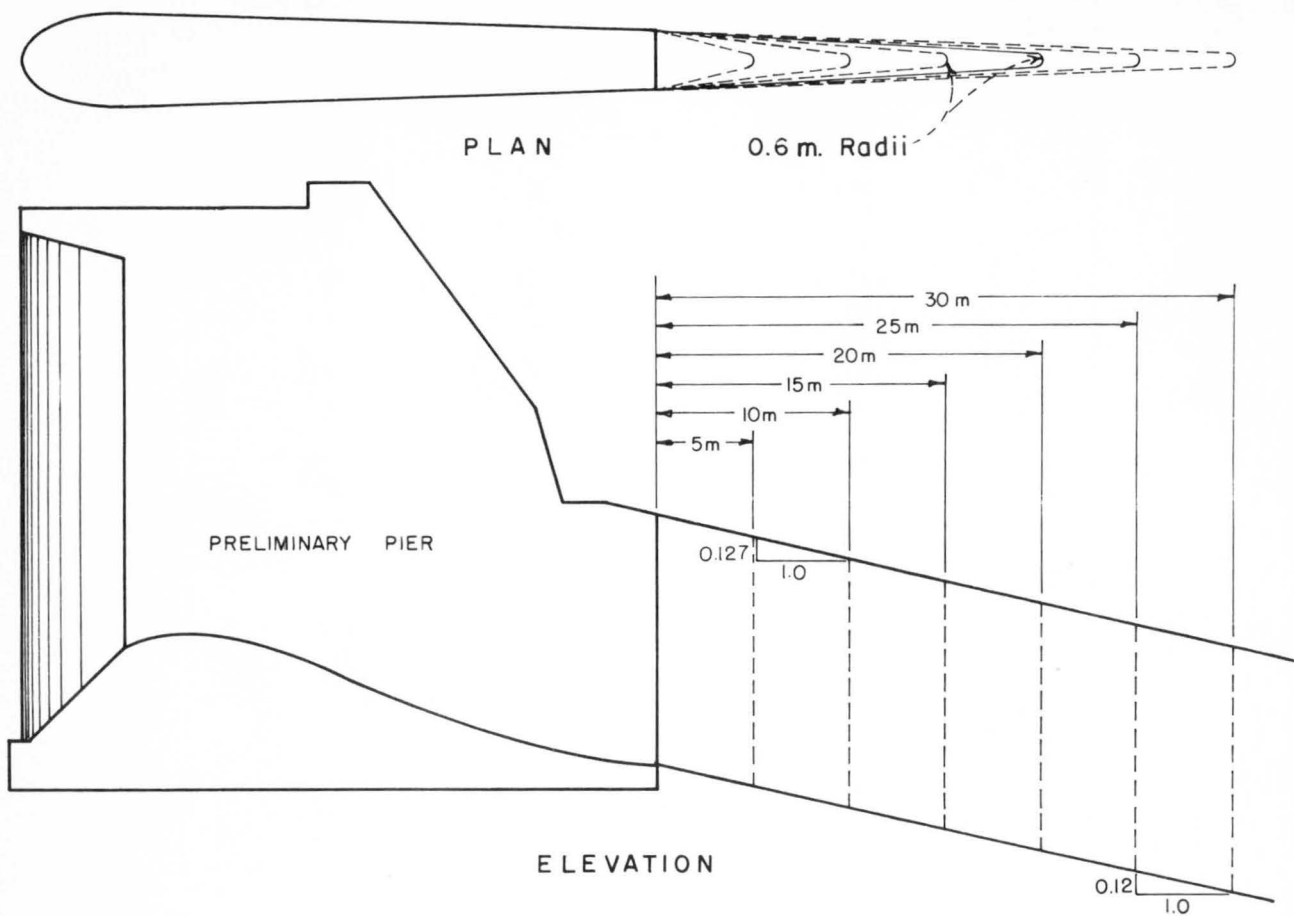


FIGURE 24 PIER EXTENSION WALLS



Figure 25 (a). Pier extension of 5 meters (looking downstream). Wave height remained substantially the same as that for the original pier. The painted horizontal line on the model wall indicates the top of the prototype wall.

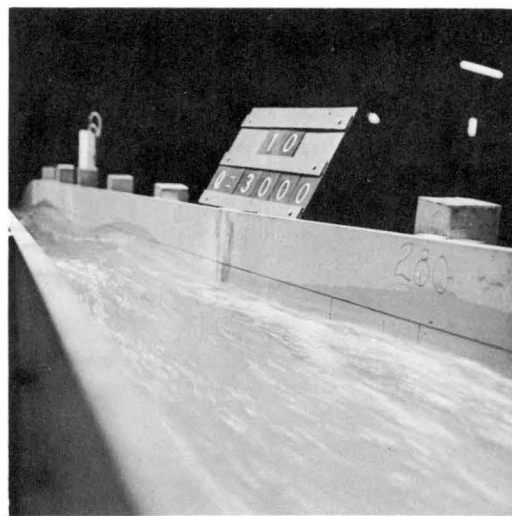


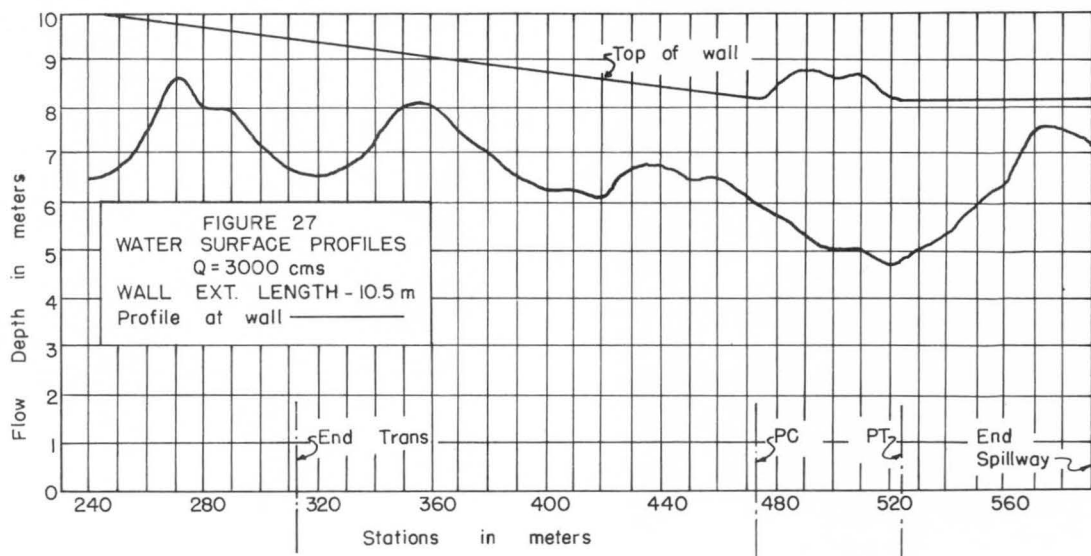
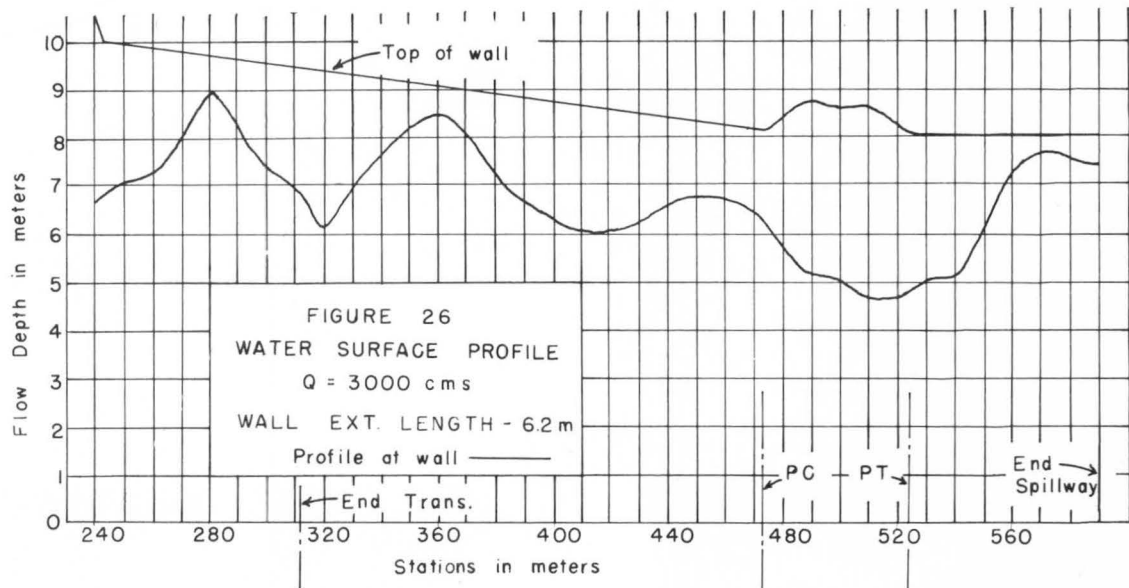
Figure 25 (b). Pier extension of 10 meters. No marked reduction in height of standing waves.

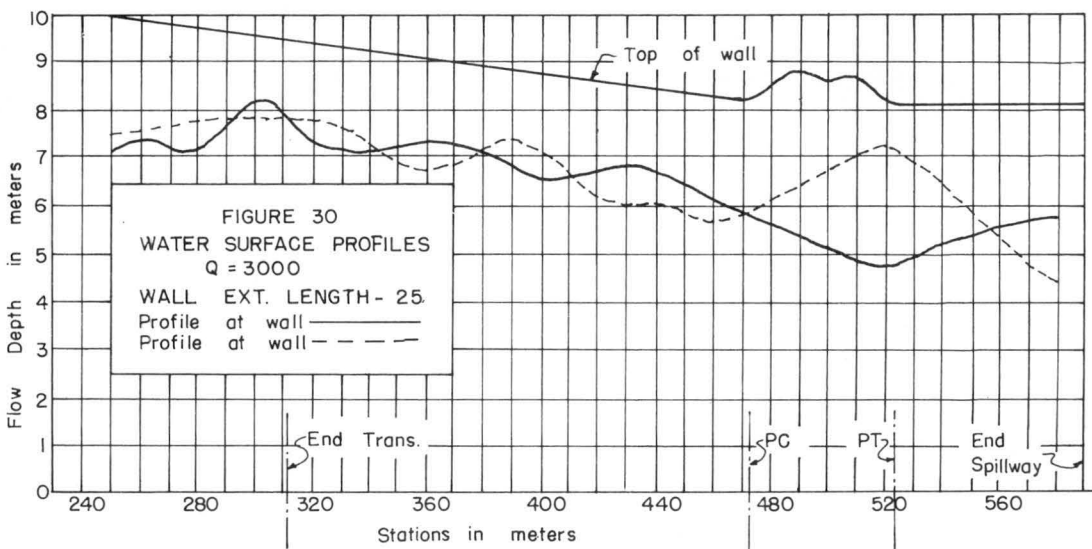
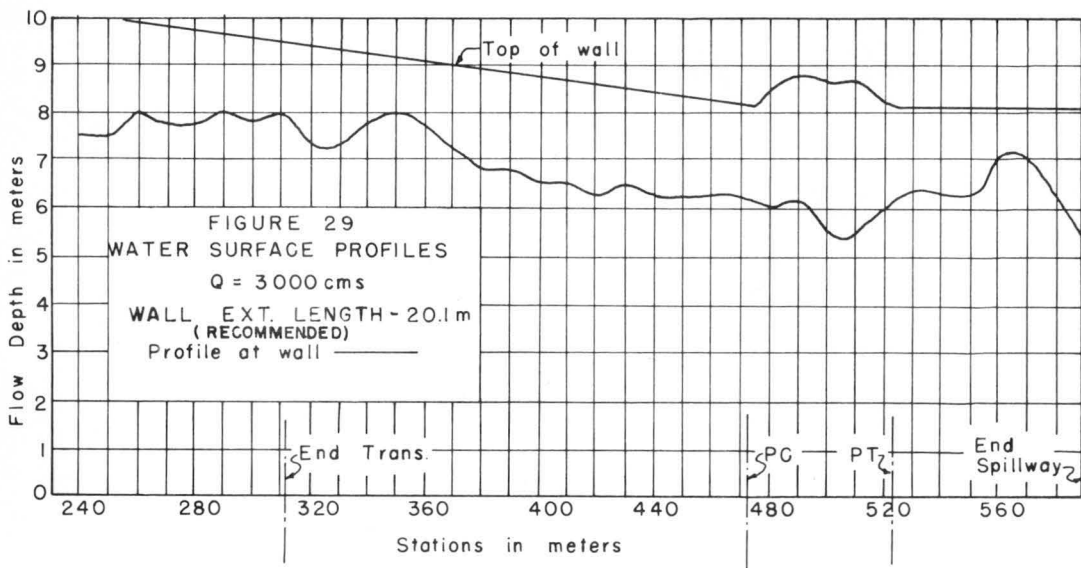
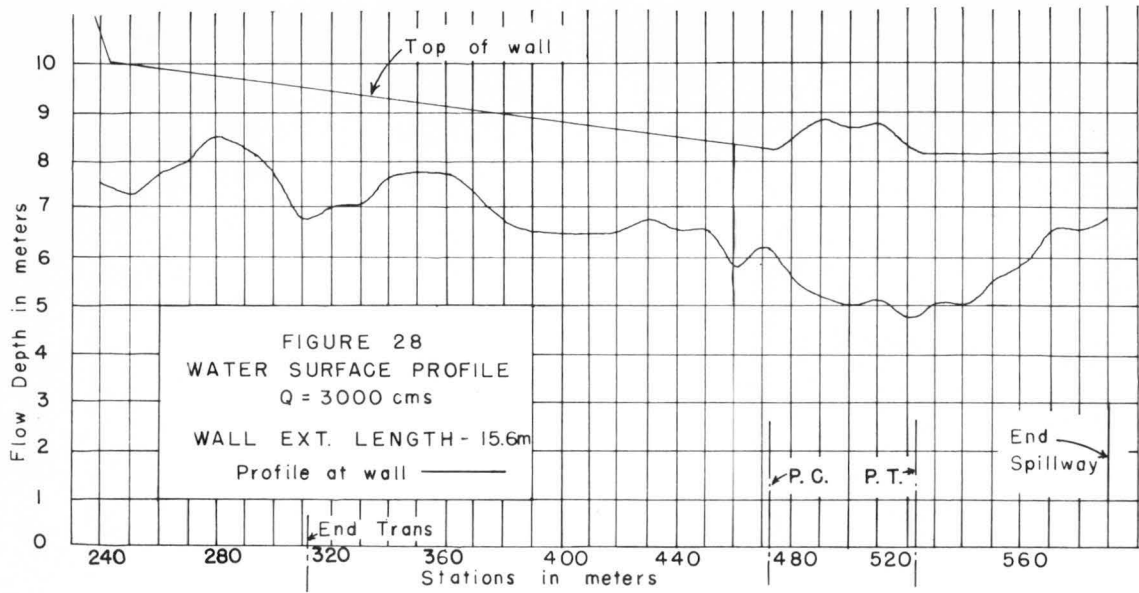


Figure 25 (c). Pier extension of 15 meters (looking downstream). Some reduction in wave height from the preliminary pier is evident.



Figure 25 (d). Pier extension of 20 meters. Definite reduction in height of standing waves. The wet line seen on the wall is from previous tests.





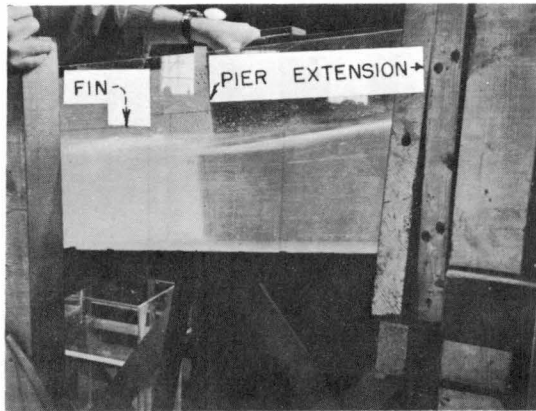
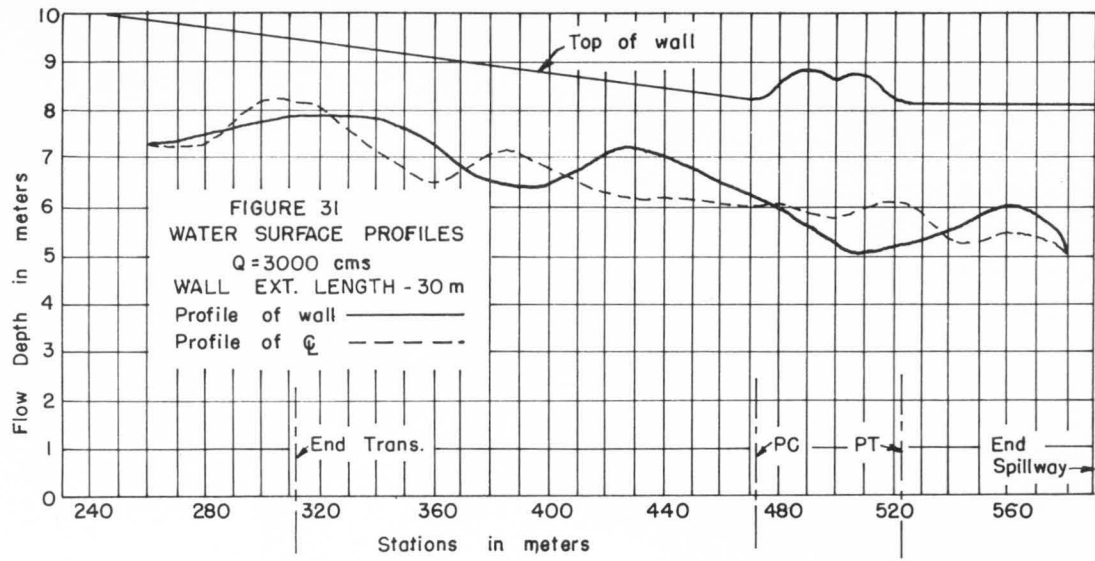


Figure 32

Water surface profile along a 20-meter long pier extension. Note the reduced fin height in comparison to Fig. 18. $Q = 3000 \text{ cms}$.

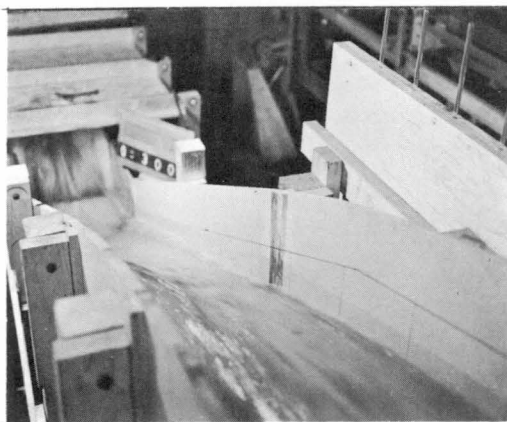


Figure 33 (a)

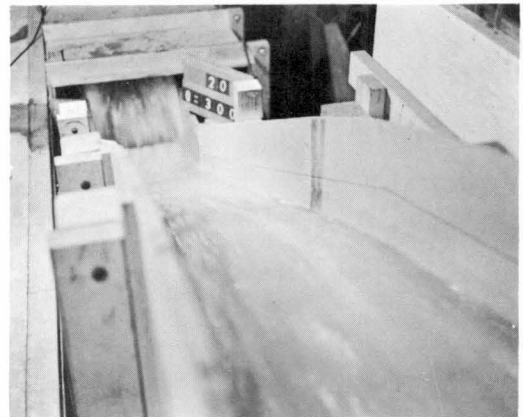


Figure 33 (b)

Comparative views (looking downstream) of the wave height reduction at the end of the chute. The left photo is without pier extension; the right is with a 20-meter pier extension wall.

The General Model

Spillway Approach Channel -- The approach channel to the spillway was generally satisfactory. There were two areas which were improved by modifications. First, the right bank of the channel was

excavated to the berm elevation of 273.0 meters to the extent shown in Fig. 34, and second, the radial wing walls were extended as a continuous arc. In the case of the left wing wall, the arc was extended to intersect the cut at elevation 287.0 meters; the right wing wall was continued to a quarter circle.

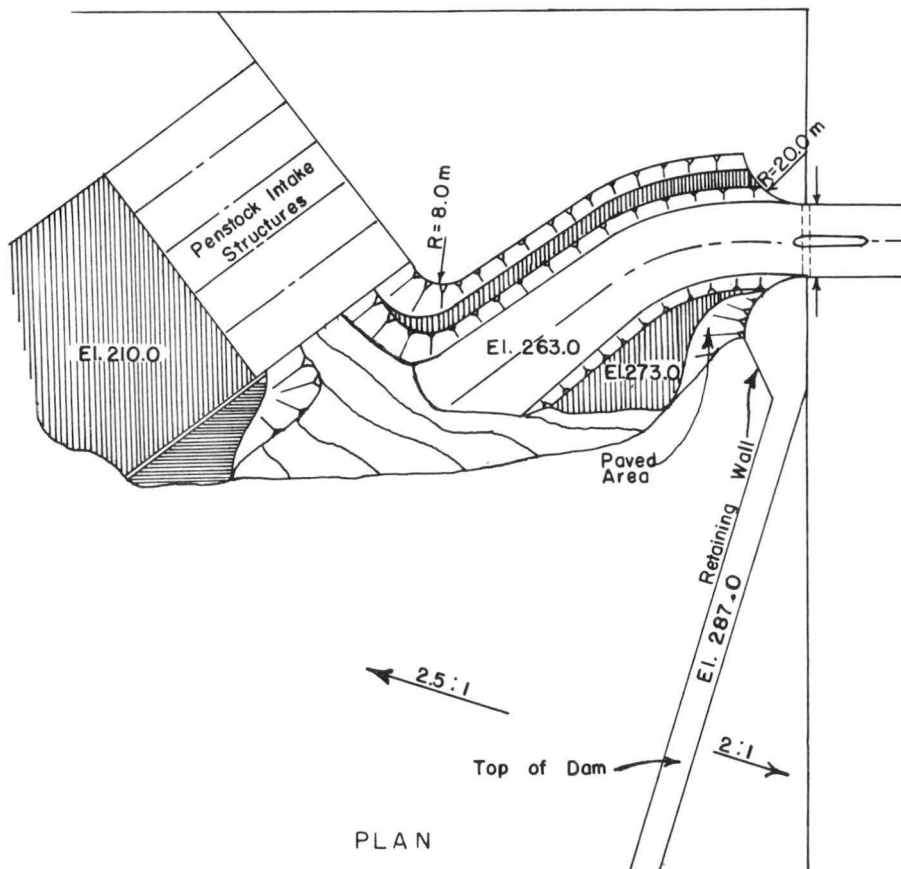


FIGURE 34 SPILLWAY APPROACH CHANNEL MODIFICATIONS (RECOMMENDED)

The excavation of the right bank appeared necessary because at discharges near 2000 cms the flow over the bank from the reservoir attained locally high velocities, causing severe turbulence along the bank. The additional frictional resistance and loss of energy resulted in slightly higher reservoir levels, although this was not serious at the discharges concerned. With high velocities at the right bank, the bank could erode and the material could be washed into the approach channel and swept over the spillway

by the relatively high velocities (near 3.5 mps) in the approach channel. If deterioration of the right bank occurs in such a manner as to cause sizable material to be swept into the spillway chute and over the flip bucket, the abrasion resulting on the concrete surface of the channel could induce potential sources of cavitation and considerable structural damage could result. The flow conditions in the channel at discharges of 1000, 2000 and 3000 cms are illustrated in Fig. 35 (a) through (c).

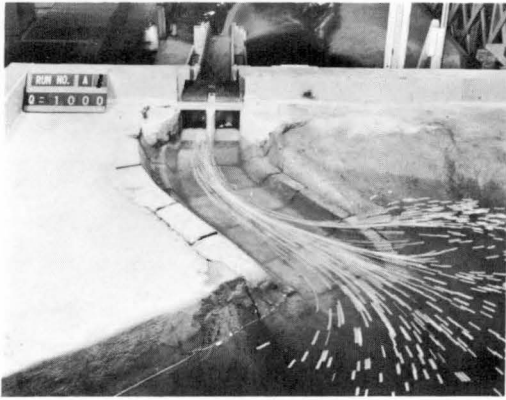


Figure 35 (a). Preliminary channel cut showing flow lines from the reservoir into the approach channel.



Figure 35 (b). Preliminary channel cut. Notice the rough water surface along the right bank where velocity over the cut is high and a horizontal vortex along the bank is created.

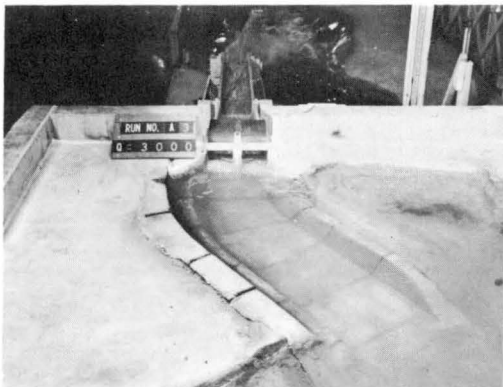


Figure 35 (c). Preliminary channel cut. The problem is not so acute at this discharge because of the higher reservoir level and greater flow depth resulting in reduced velocities over the bank.

It is difficult to visualize the flow pattern by still photographs as shown in these figures. The flow pattern can be better seen in the motion picture film which accompany this report to the ECI firm. The improved flow over the modified cut of the right bank are illustrated by the photographs in Fig. 36.

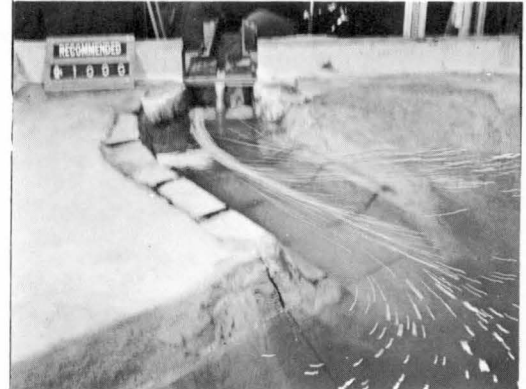


Figure 36 (a). $Q = 1000$ cms.



Figure 36 (b). $Q = 2000$ cms.



Figure 36 (c). $Q = 3000$ cms.

Modification to the left wing wall was minor and after continuation of the arc to the limit of the channel cut no further modification was required. By continuing the wing wall as an arc, the separation point between the straight wall and the arc was eliminated thus improving the flow pattern and reducing head losses some minor amount.

Because of turbulent flow condition at the right wing wall, several modifications were tried with the aim of eliminating the turbulence. The modifications are shown dimensionally in plan in Fig. 37, varying from a quarter circle to a quarter ellipse with a length along the major axis of 35 meters. For convenience of reference, they are numbered R-1 through R-4 respectively. The flows at the wing wall with 3000 cms discharge are shown in Fig. 38 (a) through (d).

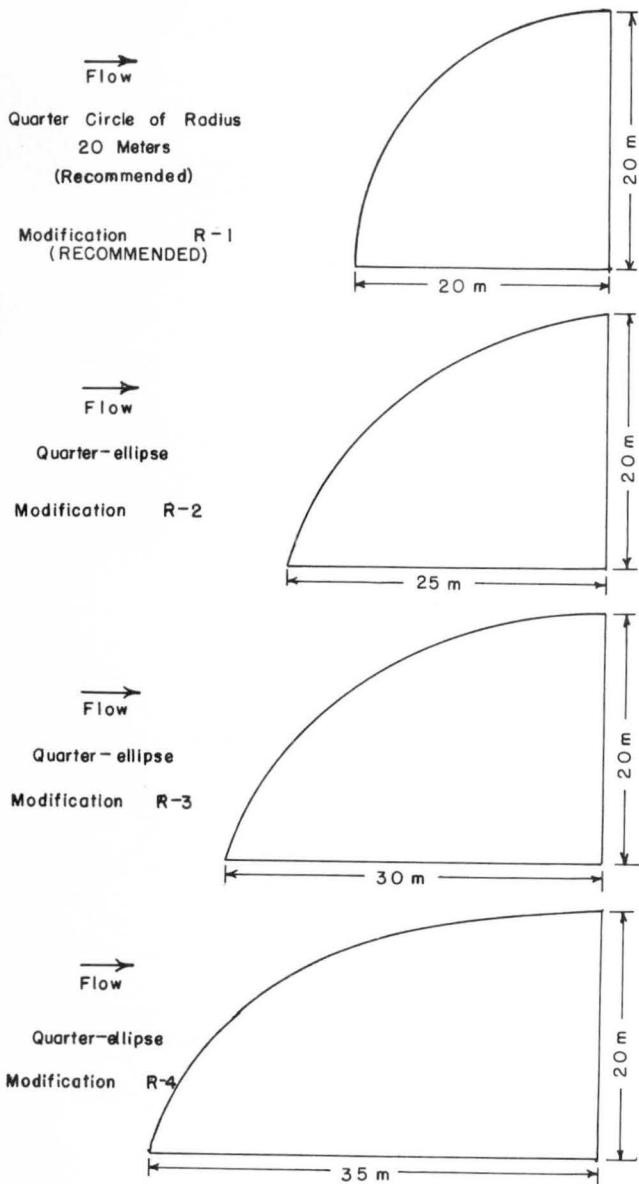


Figure 37 RIGHT WING WALL MODIFICATIONS



Figure 38 (a). Modification R-1 (recommended). Notice the flow over the cut bank adjacent to the wing wall.



Figure 38 (b). Modification R-2. The turbulence not materially reduced. Little improvement in flow condition was noticed.



Figure 38 (c). Modification R-3. Some minor reduction in turbulence and draw-down was noticed.



Figure 38 (d). Modification R-4. Turbulence was reduced, but draw-down at the beginning of the wing wall remained about the same as previous modifications.

Although turbulence along the wall was reduced with each successive increase in wing wall length, the draw-down of the water surface at the point of the wing wall was not materially reduced. The improvement of flow in comparison to the added cost of the increased size of structure provides insufficient justification to recommend a larger structure. It was found that the abrupt draw-down could be materially reduced in the case of modification R-4 by extension of the structure into the reservoir as illustrated by the temporary arrangement shown in Fig. 39 (a). Similar improvement was indicated in Fig. 39 (b) by an extension of the modified wall R-3. In neither case, however, does it appear justified to impose the added cost of construction for relatively minor hydraulic improvement. Instead of adding length to an already sizable wing wall it is suggested that a concrete paving slab over the face of the cut slope, adjacent to the recommended wing wall shown in Fig. 34, would be sufficient to provide adequate protection to the structure foundation from erosion due to the high velocity adjacent to the wing wall.



Figure 39 (a). Extension of the wing wall of modification R-4 into the reservoir. The turbulence is further diminished by a smooth transition.



Figure 39 (b). Extension of modification R-3. Flow is smoother along the wall.

A minor modification was also made to the cut slope on the left bank of the approach channel at the entrance. As shown in Fig. 34, the entrance was slightly rounded to eliminate separation at that corner. This modification did not alter the reservoir level by a measurable amount. The decision of whether or not to incorporate this modification will be left to the engineers of ECI since it is a matter of economics.

Gate Structure - Pier Nose -- In view of the water surface draw-down around the pier nose resulting from tests of the chute model (Fig. 17), extended pier noses were tested as shown in Fig. 40, but with no noticeable improvement. Pier noses were studied in the general model since it was necessary to include the approach channel flow effects, and such effects could not easily be duplicated in the chute model.

The water surfaces at modified pier B, as shown in Figs 41 (a), (b), and (c), indicate flow disturbances around the pier at large discharges. Since the velocity of flow in the approach channel are high, elimination of draw-down around the pier does not seem possible short of a complex shape not much different from a ship hull. Nor does it seem necessary to develop a completely smooth water surface in view of achievement of satisfactory spillway discharge coefficients. The preliminary pier nose shape was hydraulically satisfactory. If an increased length of pier is structurally necessary upstream of the crest because of the forward tilting of the gates to raise the gate pins above the water surface, an elliptical pier nose similar to modification A would be hydraulically satisfactory. In fact, anticipating this structural need, the spillway rating curve was developed with modified pier nose A.

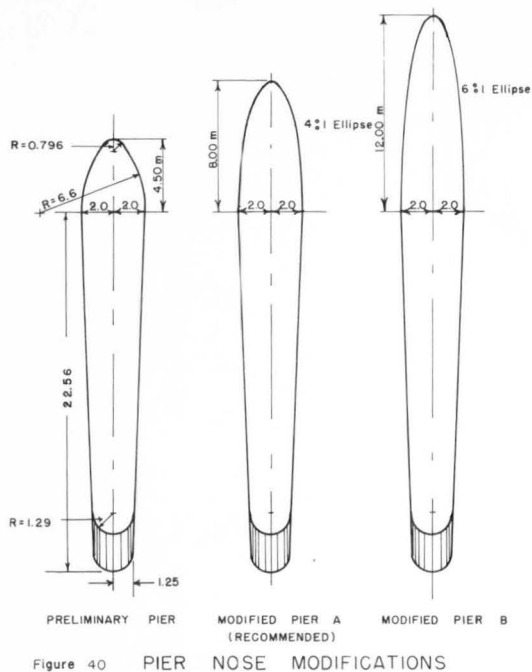


Figure 41 (a). Pier nose modification B.
Q = 1000 cms.



Figure 41 (b). Pier nose modification B.
Q = 2000 cms.



Figure 41 (c). Pier nose modification B.
Q = 3000 cms.

Spillway Rating Curve -- A spillway rating curve was developed with the general model, which included the recommended modifications to the approach channel and the 4:1 elliptical pier nose. The rating curve is shown in Fig. 42, and was developed for free flow, that is, without gated control, and also with gated control. For the curves with gated control both radial gates were equally opened, and indicated openings are the vertical distances between the bottom of the gate and the point of contact on the gate seat. The spillway discharge coefficients for free flow are also shown in the figure.

Unequal gate openings caused undesirable standing wave patterns. It is recommended that the gates be operated uniformly and simultaneously. In the unlikely event that failure in the hoist system of one gate should occur, discharges up to 1000 cms could be contained within the chute. Larger discharges through only one gate would cause overtopping of the chute walls.

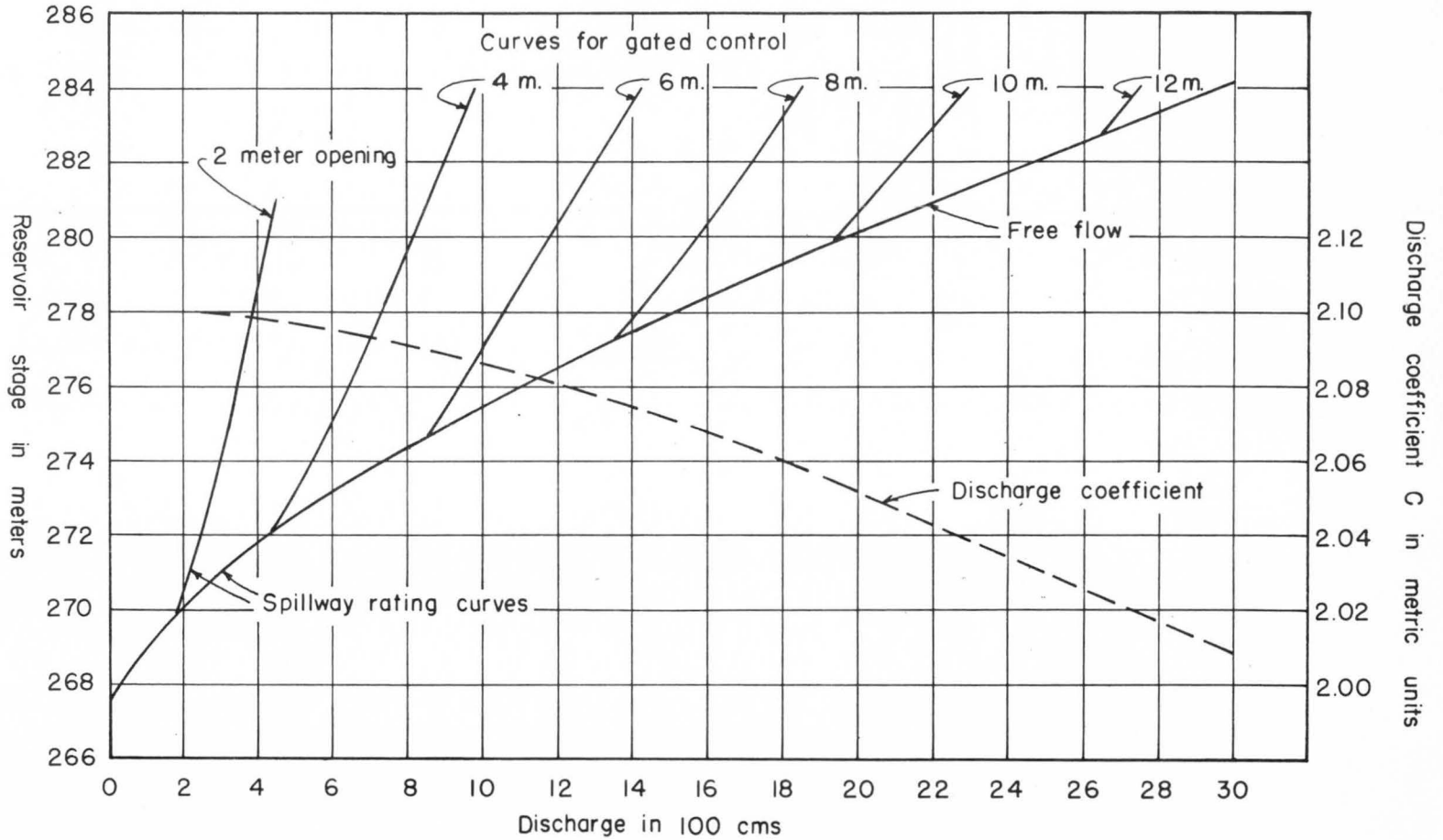


FIGURE 42 SPILLWAY RATING CURVES RECOMMENDED

Flip Bucket -- The alignment of the spillway chute with respect to the downstream river channel was such that the spillway flows had to be deflected approximately 15 degrees to the right (looking downstream) for the flip bucket jet to impinge in the river rather than on the river bank. In order to effect the change in flow direction the preliminary design indicated a curved fillet on the left side of the radial bucket. This design was tested at various discharges in the range of expected spillway flows and was found to be inadequate. While the fundamental concept of a fillet on the left side is sound, the size of the fillet was insufficient as the photographs of Figs. 43 (a) and (b) show. The jet impinged high on the left bank of the river. In the prototype this would undoubtedly erode the bank very seriously.



Figure 43 (a). Preliminary design. The jet impinged high on the left river bank.



Figure 43 (b). Preliminary design. $Q = 3000$ cms. The jet deflection was inadequate.

Many modifications were tested, some of them recorded and other trials visually inspected and rejected. Initial modifications involved larger fillets to deflect the flow. The results of some of the nearly successful modifications are shown in Figs. 44 and 45. In Fig. 44, in addition to a larger fillet, the floor of the chute downstream of the vertical curve was superelevated along the left side in an attempt to initiate general movement of the entire flow to the right. The superelevation was effected gradually to 2 meters in height, measured normal to the floor surface, across the entire chute width. Little beneficial effect from the superelevation was noted. The superelevation was increased to 4 meters but developed no notable result. The right wall of the chute was then warped outward beginning from the end of the vertical curve in combination with the superelevated floor, but again no improvement was indicated. The result of the latter modification is shown in Fig. 45.

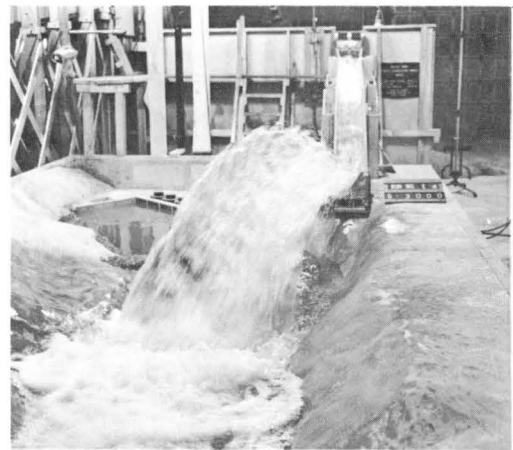


Figure 44. Results of a modification involving a larger fillet and a superelevated floor. $Q = 3000$ cms.

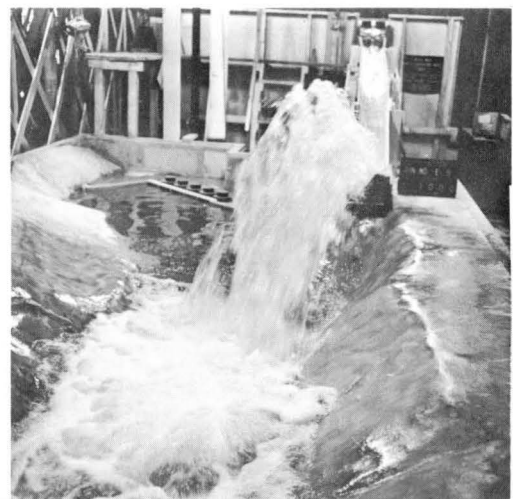


Figure 45. The right wall of the chute (when one looks downstream) was warped outward in combination with a superelevated chute floor and large fillet on the left side of the bucket. $Q = 3000$ cms.

Later modifications included a longer bucket by about 4 meters and a return to the original spillway chute geometry involving no superelevation or warped walls. A photographically recorded result from this change is shown in Fig. 46 with a spillway discharge of 3000 cms.

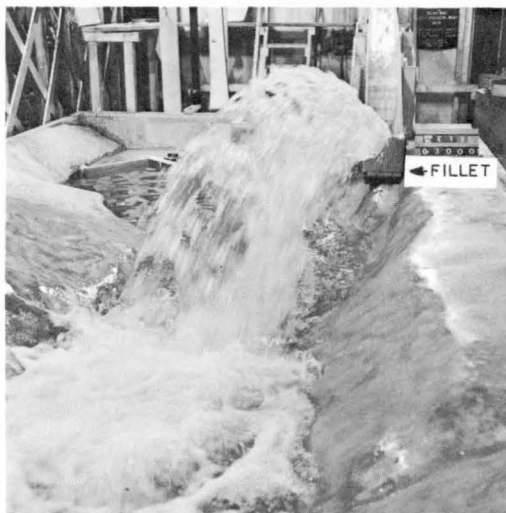


Figure 46. The modification involved a 4.2-meter longer flip bucket and larger fillet on the left side.

In all of the preceding modifications the bucket and deflector were comprised of curved surfaces. Definition of such surfaces were complex and construction would be even more difficult. Because of this, modifications to the flip bucket geometry were thereafter made with a view to developing more simple geometry, involving if possible plane surfaces with a minimum of curved fillets. The results of one such attempt is shown in Fig. 47 with a view of the model bucket in Fig. 48.

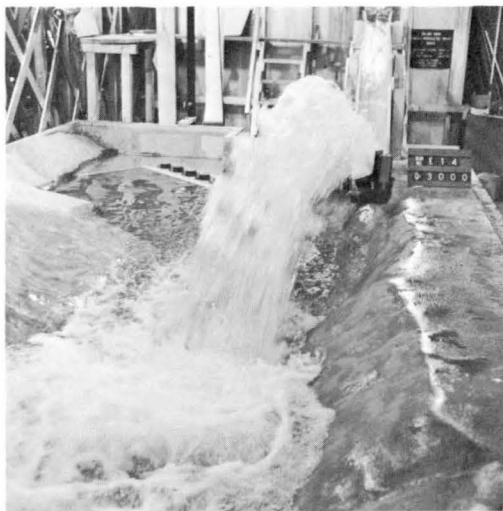


Figure 47. Modified flip bucket using plane surface fillets. The model bucket giving the above result is shown in Fig. 48.

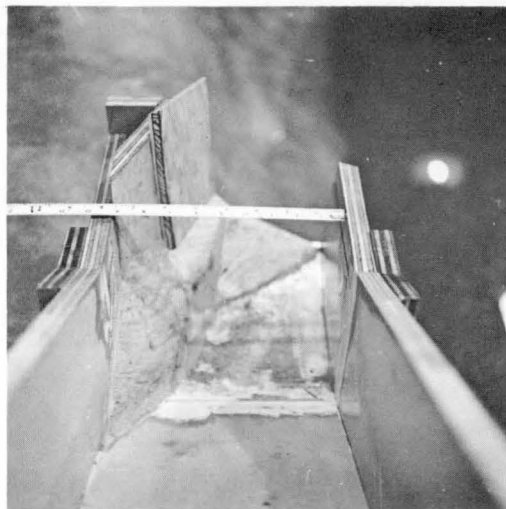


Figure 48. Modified flip bucket including radially curved floor in the vertical plane and plane surface deflectors.

The recommended flip bucket included only plane surfaces. The model structure of the recommended bucket is shown in Fig. 49 and flow conditions resulting therefrom are shown in Fig. 50 (a) through (j).

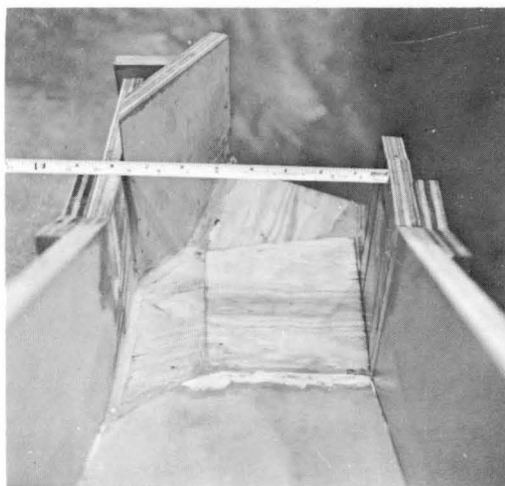


Figure 49. Recommended flip bucket. The dimensions of this bucket are given in Fig. 51.



Figure 50 (a)



Figure 50 (d)



Figure 50 (b)



Figure 50 (e)

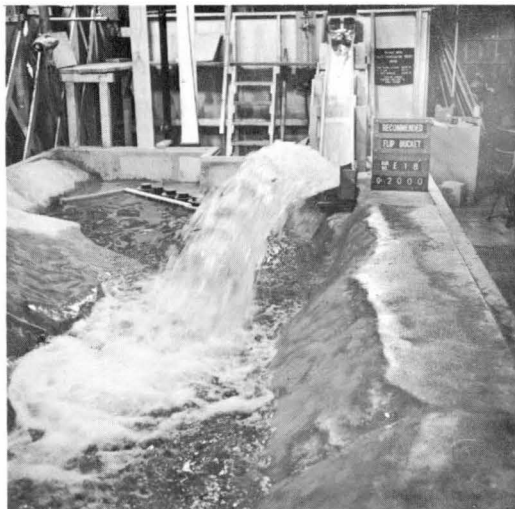


Figure 50 (c)



Figure 50 (f)



Figure 50 (g)

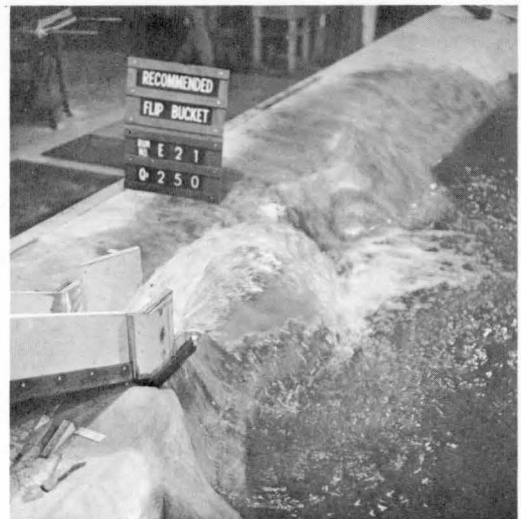


Figure 50 (j)



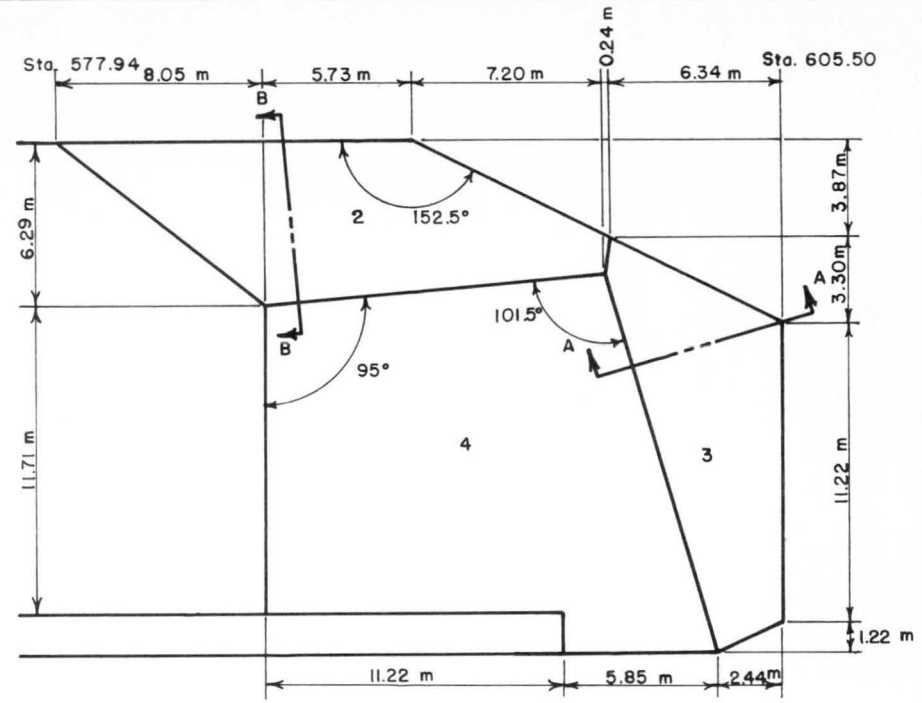
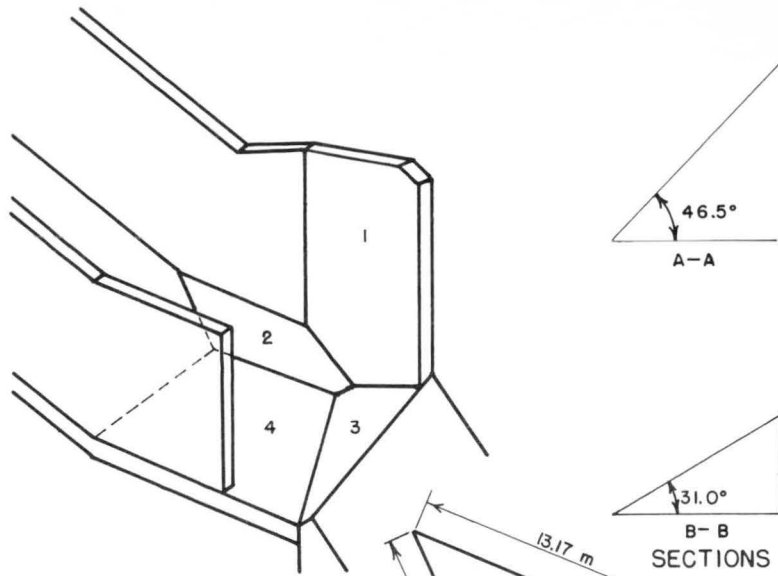
Figure 50 (h)



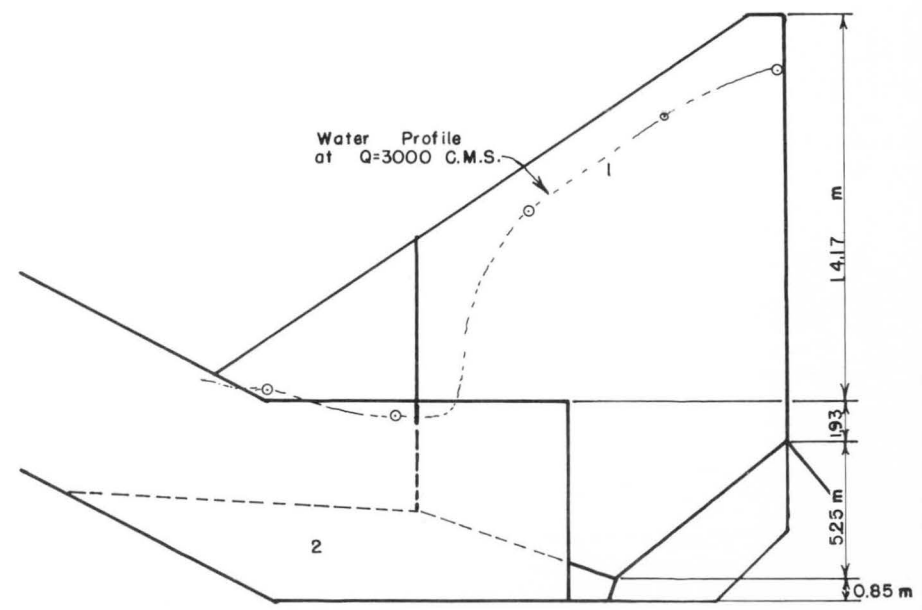
Figure 50 (i)

A drawing of the bucket giving pertinent dimensions is shown in Fig. 51. The jet from the flip bucket appeared satisfactory within the flow range from 100 to 3000 cms. The jet did not impinge on the river banks. At less than 100 cms a hydraulic jump was formed in the bucket and the flow fell to the right side of the bucket and cascaded down the hillside. In order to protect the structure from possible foundation failure due to these small discharges, it is suggested that the hillside be paved or a chute be constructed down to the river level to contain these flows. If the foundation rock is sufficiently erosion-resistant to withstand the small flows or if the gates can be operated in such manner as to minimize the small flows, no extra protection will be necessary.

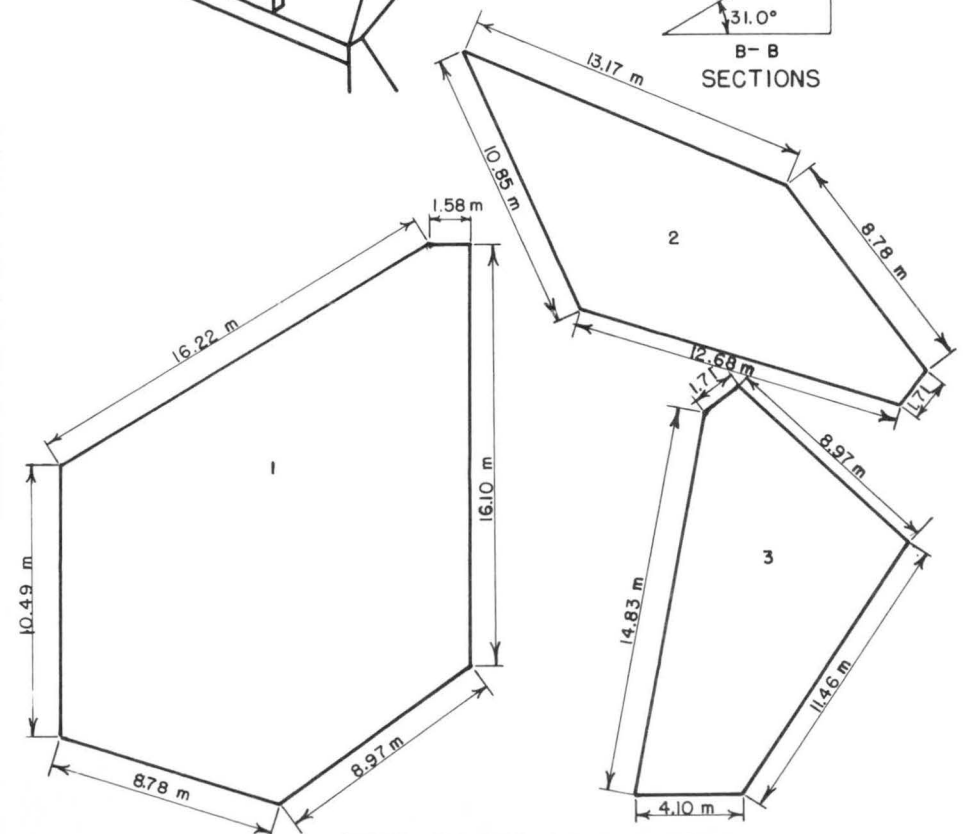
Dynamic pressure heads at selected points were measured for discharges of 1000, 2000, and 3000 cms. The locations of the pressure points in the flip bucket are shown in Fig. 52 along with plots of pressure heads at 3000 cms. The data for all three discharges are given in Appendix C. No negative pressures were recorded. Because of the increase in flip bucket length of 4.20 meters, it would be advisable to shorten the spillway chute downstream from the vertical curve in a horizontal distance by a like amount to prevent an excessively large foundation structure for the flip bucket. The vertical curve should be continued with the same equation for curvature as the preliminary design and should meet the revised chute slope tangentially. This change was incorporated in the model and tests showed that the hydraulic performance of the flip bucket would not be affected.



PLAN OF FLIP BUCKET



PROFILE OF FLIP BUCKET



TRUE SHAPES OF PLANES
Fig. 51 RECOMMENDED FLIP BUCKET GEOMETRY

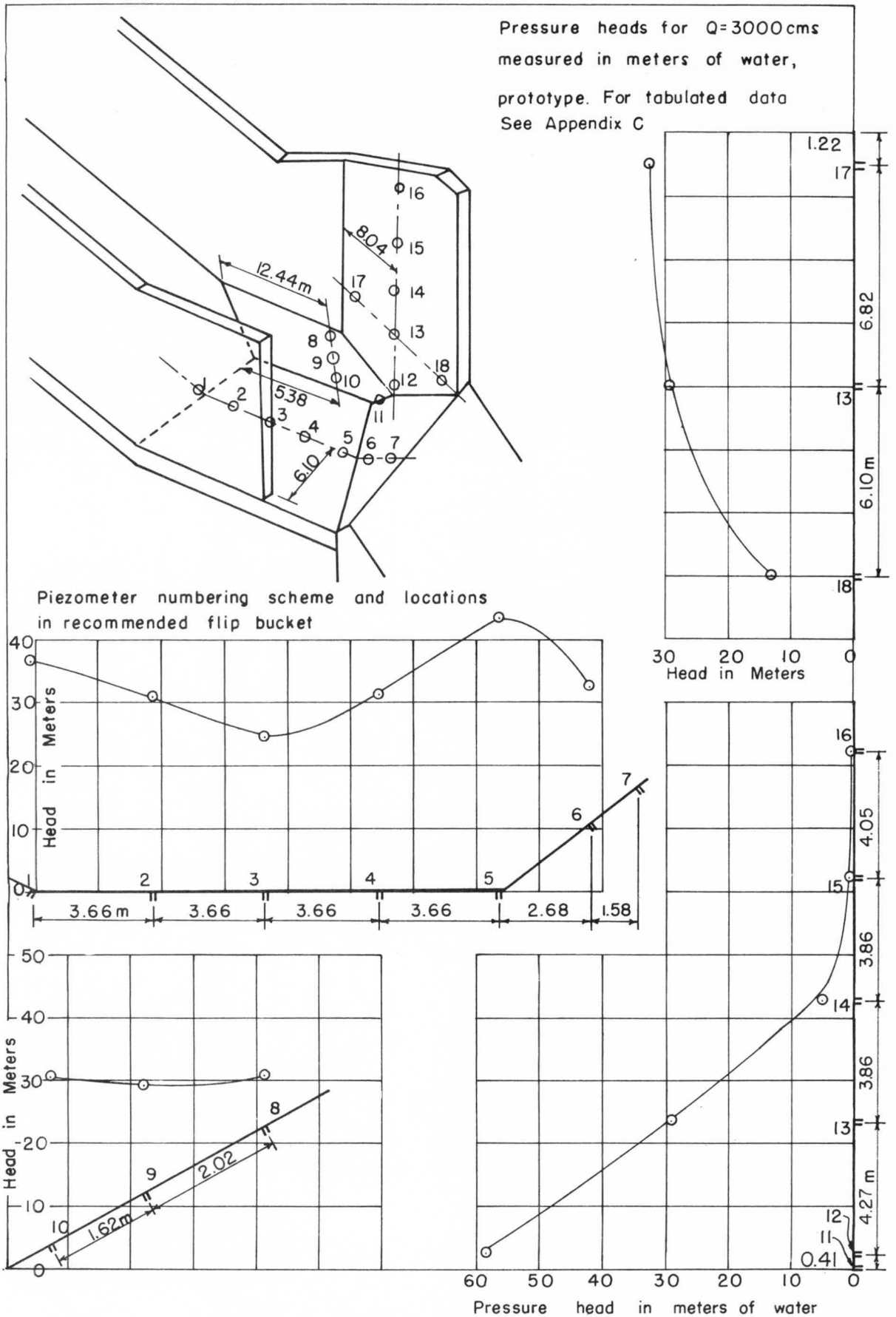


FIGURE 52 MEASURED PRESSURE HEADS IN FLIP BUCKET

River Bed Erosion and Power Plant Tail Water Levels -- Theory governing models of movable bed river channels is not yet (1962) sufficiently established to provide reliable quantitative model results with respect to scour depth, quantity of scoured sediment moved in the river channel, and time scale for scour. The results of scour tests in this study are therefore qualitative and are intended only to indicate possible scour patterns in the prototype.

In a report on geology and materials at the Kremasta Project site prepared by ECI, (Ref. 5) it was stated that the river bed consisted of homogeneous deposits of well-graded gravel, ranging in sizes from 18 inches in diameter to fine sand, with a median sieve diameter of about 1-1/2 inches. It was

also stated in the report that from one to two percent of the total material was estimated to range between 10 to 18 inches in diameter, and in size analysis curves it was shown that approximately 20 percent passed the U. S. Standard sieve no. 4 (sand sizes). No information was given in the report about the depth of alluvial material in the river channel in the vicinity of the jet impact area.

The depth of flow in the river was controlled by the tail gate at the end of the model as shown in the schematic drawing of Fig. 3. The river stage-discharge curves developed by Wright Engineers of Denver, Colorado, for ECI were used in the study. The stage-discharge curves at the location of the model gaging station shown in the schematic diagram (Fig. 3) are reproduced as Fig. 53.

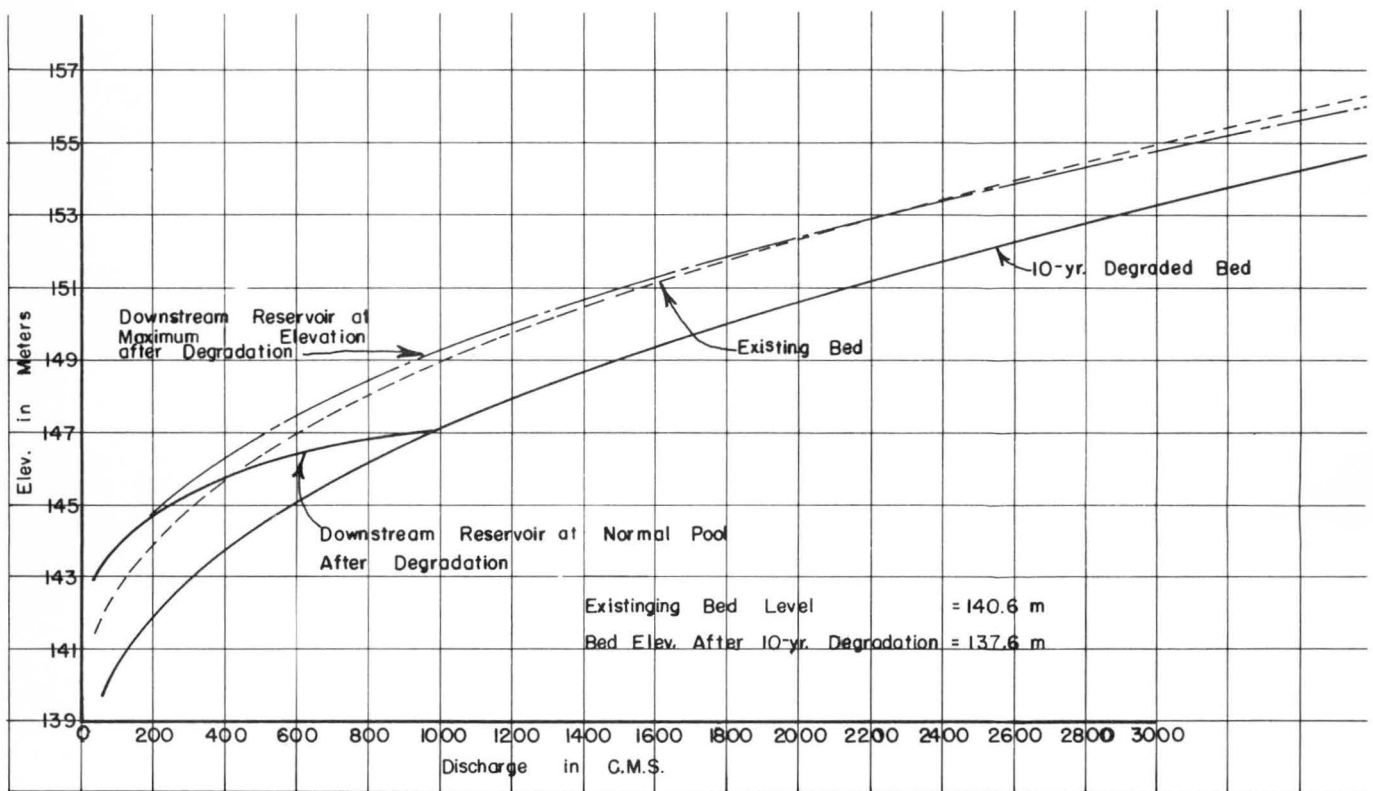


Figure 53 STAGE DISCHARGE RATING CURVES AT MODEL GAGING STATION

The singularly important phenomena, aside from scour is the effect of the gravel bar which would form downstream from the scour area on the tail-water elevation of the power plant. The higher the bar formation, the greater would be the possibility of inundating the power plant. To provide a range of possible conditions of the river bed in the prototype, the model tests were arranged to include two different compositions of river bed materials and the elevation of "bed rock" was studied at two levels, at 115.0 meters and 139.0 meters. The initial material used in the river bed consisted of commercially available 3/8-inch washed gravel ranging in sizes from 1/8 to 3/8 inch. This material represented the larger fraction of river bed material reported for the prototype from approximately 10 to 30 inches in diameter. While admittedly the size used in the model was much larger than that reported for the

prototype, by using the larger sizes a higher model gravel bar would be formed because less material would be moved downstream and critical power plant tail water levels (if any) could be observed in the model. The second river bed material tested in the model consisted of even larger size gravel, nominally 3/4 inch but ranging in sizes from 1/2 to 1 inch.

The development of scour in the river bed with the 3/8-inch gravel and spillway discharge of 1000 cms is shown in Fig. 54 (a). The tail water levels at the power plant are shown in Fig. 54 (b) with progressive stages of scour beginning with the existing river bed level at elevation 140.6 m. The white lines in the photograph are contour lines. The minimum elevation in the scour hole was 126.0 m. Test results for discharges of 2000 and 3000 cms are shown in Figs. 55 and 56 respectively. The power plant wall was not overtopped in any of the tests.



Figure 54 (a). Scour pattern with 3/8-inch gravel
 Spillway $Q = 2000$ cms
 Power Plant $Q = 0$
 Model Time = 45 minutes
 Maximum scour depth = 126.0 m
 Beginning river level = 140.6 m

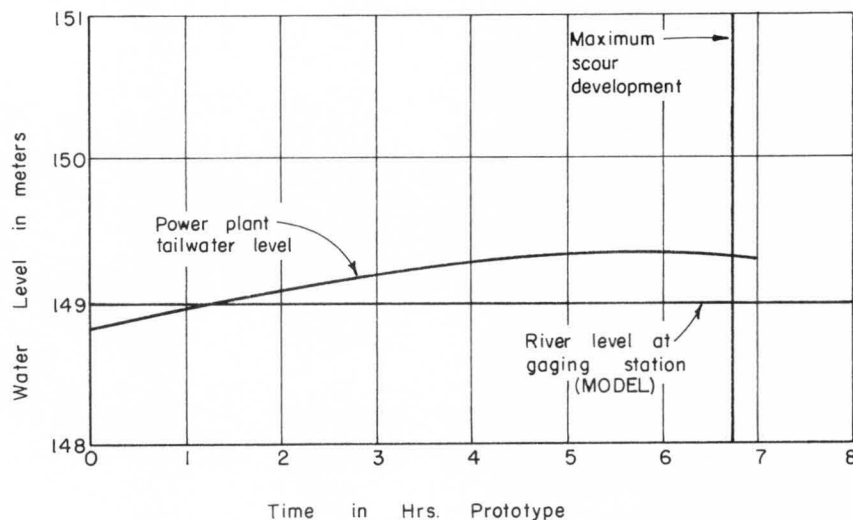


FIGURE 54 (b)



Figure 55 (a). Scour pattern with 3/8-inch gravel
 Spillway $Q = 2000$ cms
 Power Plant $Q = 400$ cms
 Model Time = 38 minutes
 Maximum scour depth = 124.0 m
 Beginning river level = 140.6 m

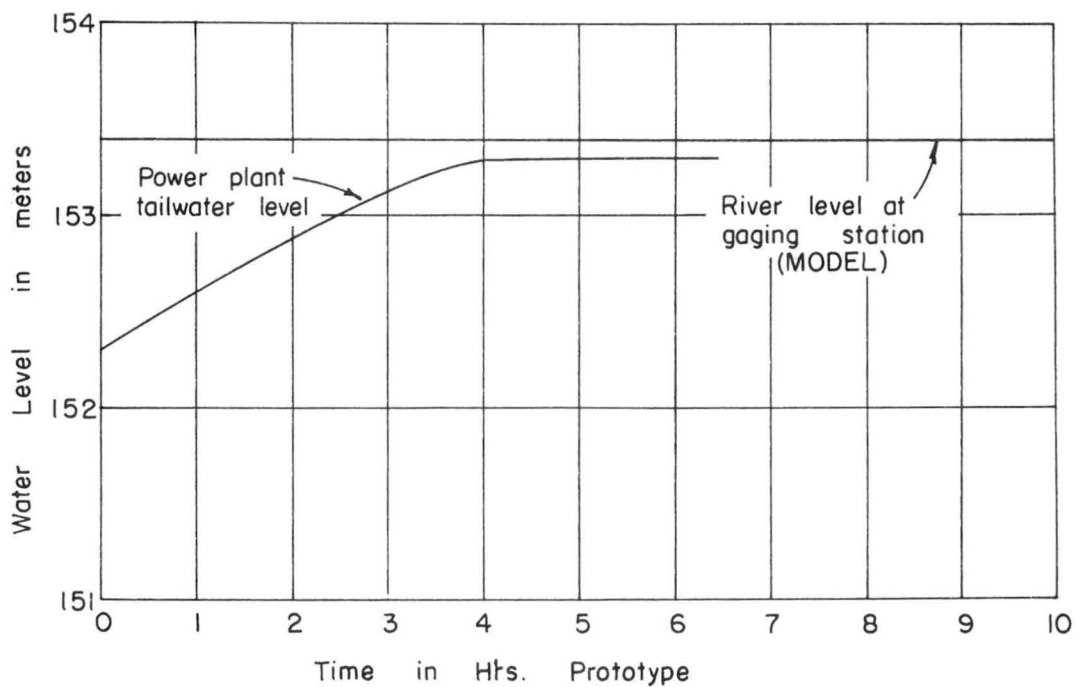


FIGURE 55 (b)

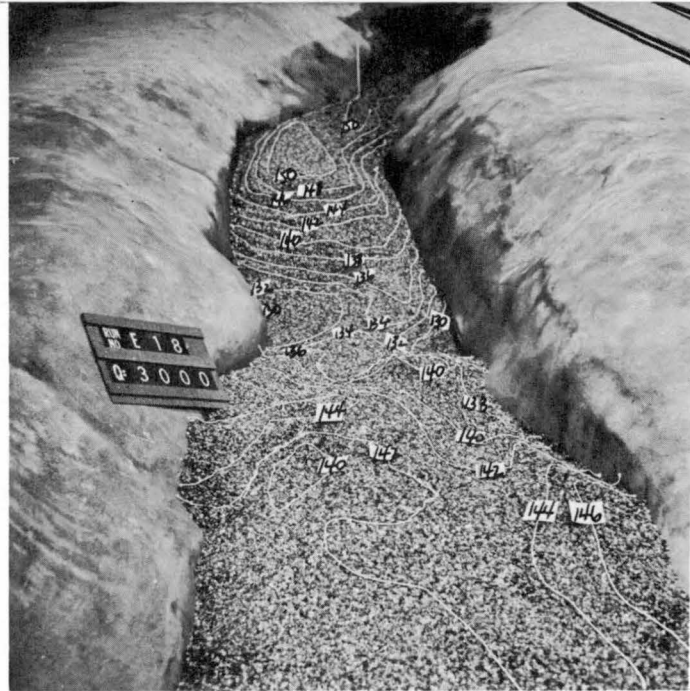


Figure 56 (a). Scour pattern with 3/8-inch gravel
 Spillway $Q = 3000$ cms
 Power Plant $Q = 400$ cms
 Model time = 26 minutes
 Maximum scour depth = 128.0 m
 Beginning river level = 140.6 m

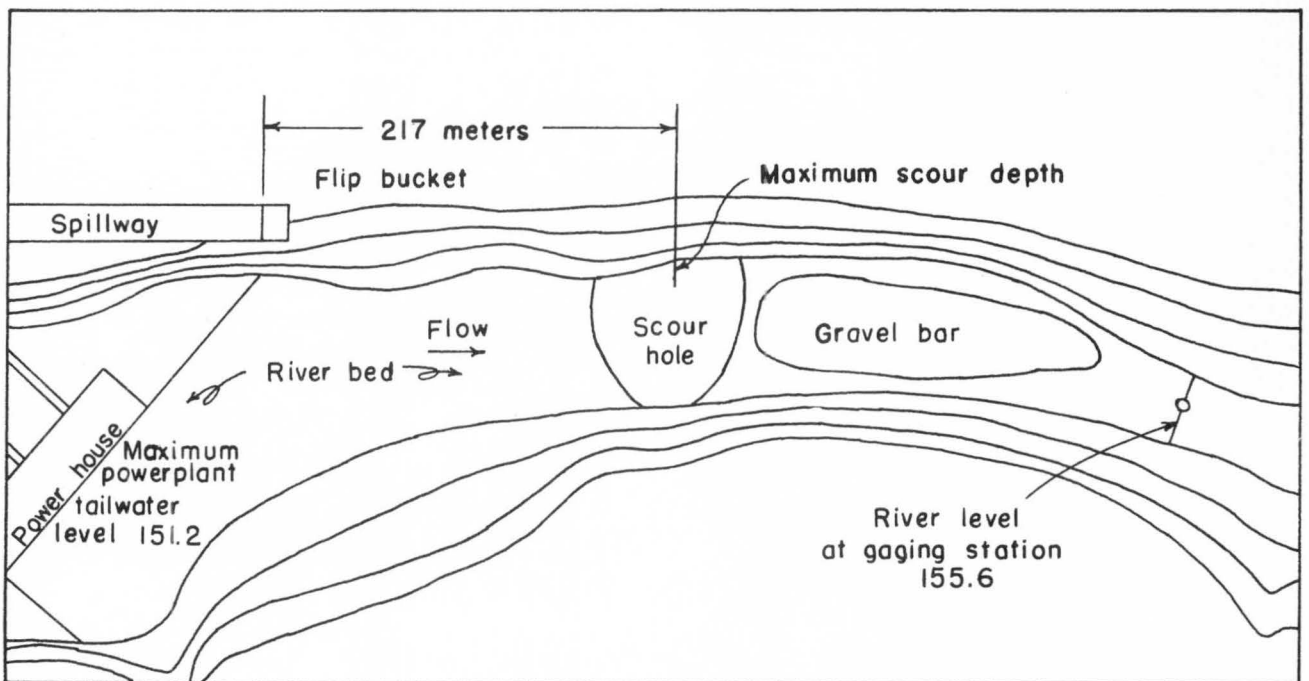


FIGURE 56 (b)

Scour development at varying spillway discharges would differ from uniform discharges over the spillway. To evaluate the scour pattern and to determine the effect on the tail water level at the

power plant, a uniform step-hydrograph was discharged over the spillway. The hydrograph is shown in Fig. 57. The resulting scour pattern is shown in Fig. 58 (a).

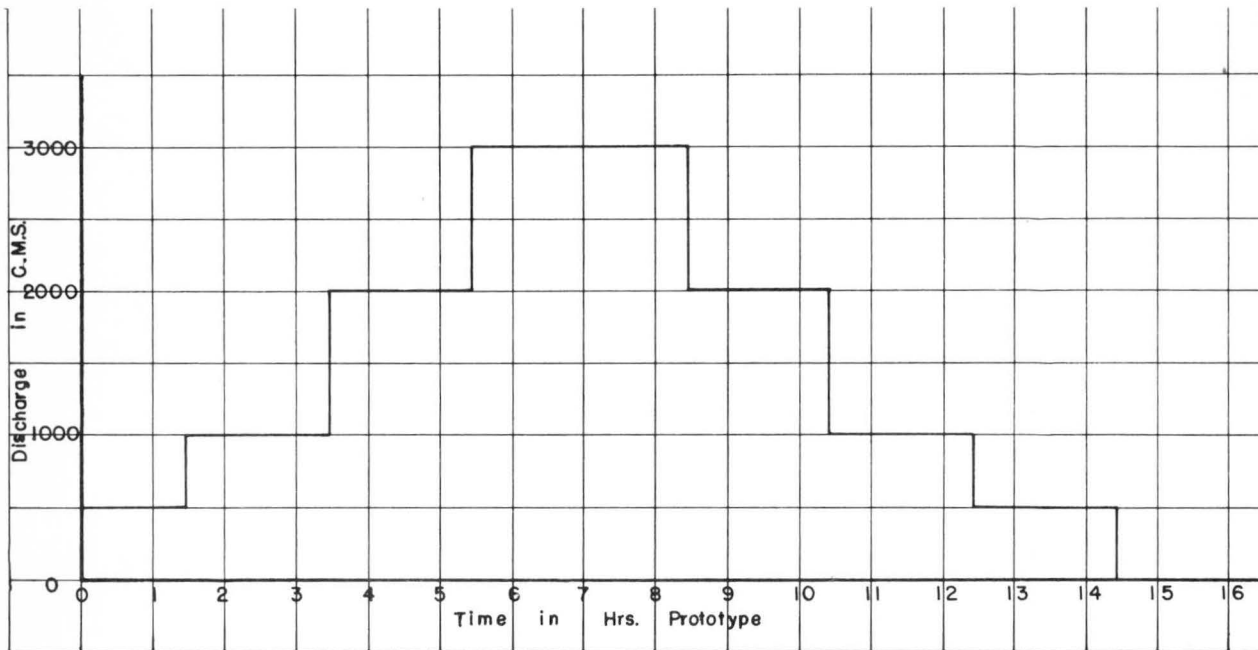


Figure 57

STEP HYDROGRAPH



Figure 58 (a). Scour pattern with hydrograph of Fig. 57.

The scour area was more extensive and a greater amount of river bed material was transported downstream. Variation of tail water levels with discharge are shown in Fig. 58 (b). The maximum tail water level was 153.3 providing 2.7 meters of freeboard to the top of the wall. On the recession cycle of the hydrograph the power plant was shut down and tail water levels were measured with only the spillway flows. The result is shown in Fig. 58 (c).

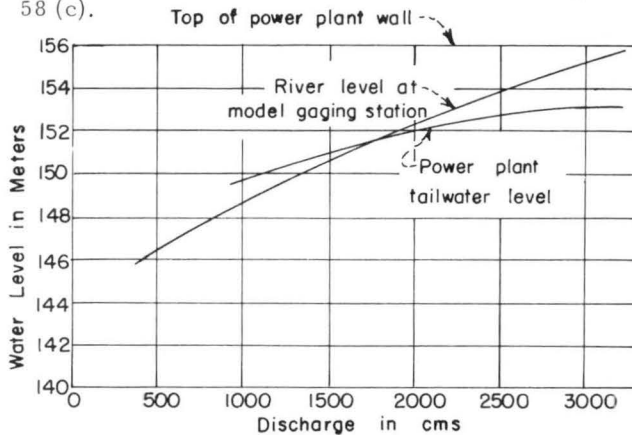


FIGURE 58 (b) POWER PLANT TAILWATER LEVEL
Power Plant Q = 400 cms

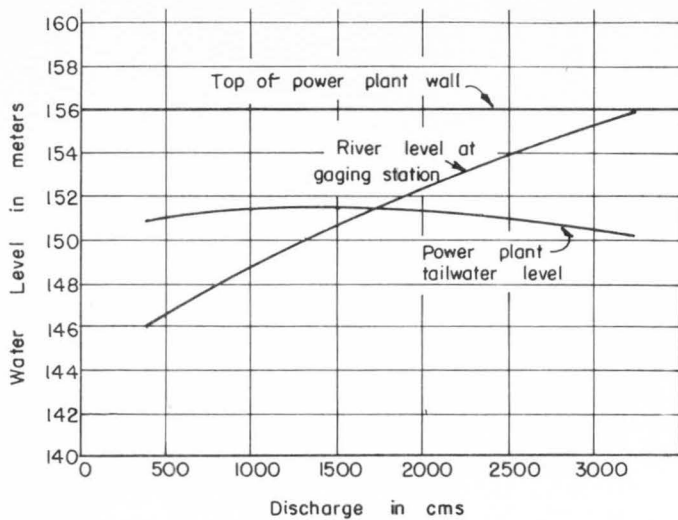


Figure 58 (c). Power plant tailwater level. No power plant discharge.

Because of the gravel bar formed downstream of the scour area, the tail water levels at the power plant were higher than at corresponding discharges during the ascending cycle of the hydrograph. There was sufficient freeboard to the top of the power plant wall at all discharges.

The preceding results were predicated on the supposition that the estimated river stage curves shown in Fig. 53 were correct. If these curves were underestimated, the model observations of tail water levels at the power plant would in turn be too low. To determine at what river stages the power plant wall would be overtopped, tests were run at several discharges with imposed high river levels to cause overtopping. The tests were made with maximum discharges through four turbine units. Results are shown in Fig. 59. There was at least 5 meters of allowance in depth between the calculated stage and that which would cause overtopping.

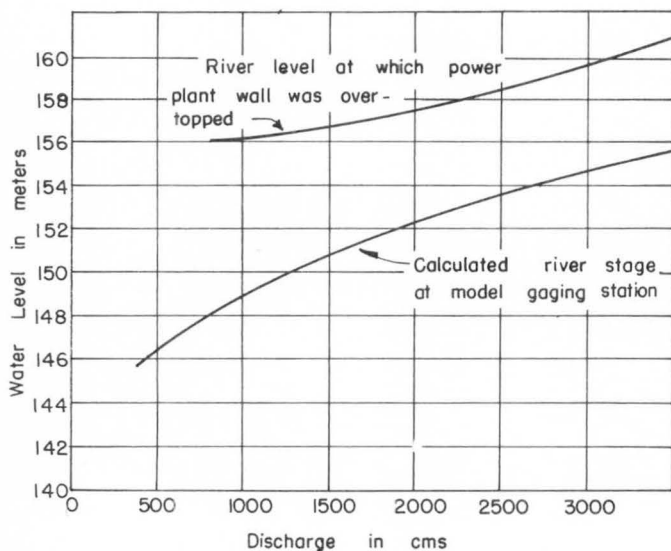


Figure 59. River Stage Studies.

The results of tests with 3/4-inch gravel in the model river bed are shown in Fig. 60 (a) through (c) for discharges of 1000, 2000, and 3000 cms. The power plant tail water levels were the same as tests with the 3/8-inch gravel bed.



Figure 60 (a). Scour patterns with 3/4-inch gravel. $Q = 1000$ cms.



Figure 60 (b). Scour patterns with 3/4-inch gravel. $Q = 2000$ cms.



Figure 60 (c). Scour patterns with 3/4-inch gravel. $Q = 3000$ cms.

Test results with river bed rock located at elevation 139.0 are shown in Fig. 61 (a), (b), and (c) for comparable discharges to Fig. 60. Measured tail water levels at the power plant are shown in Fig. 62. The tail water at the power plant was drawn down with increasing spillway discharges with the bed rock level at 139.0. This is reasonable since greater downstream horizontal shear component is developed on the bed and the water level upstream of the jet impingement area is drawn down as a result. No apparent tail water problem at the power plant will arise if bed rock is near the present river bottom. It might be mentioned, however, that with the high velocities of the jet, should the bed rock scale off in the form of large boulders or slabs and block the river channel, a rise in tail water is to be expected. Unless the entire river channel is completely blocked off, however, some channeling will result around the plug and overtopping should not occur at the power plant. In the event blocking should occur, naturally remedial measures should be taken as soon as practicable.

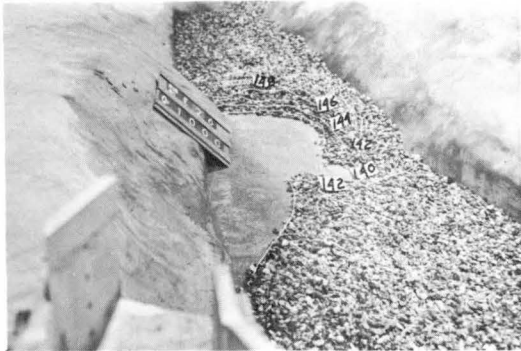


Figure 61 (a). Scour patterns with bed rock at elevation 139.0 meters. $Q = 1000$ cms.

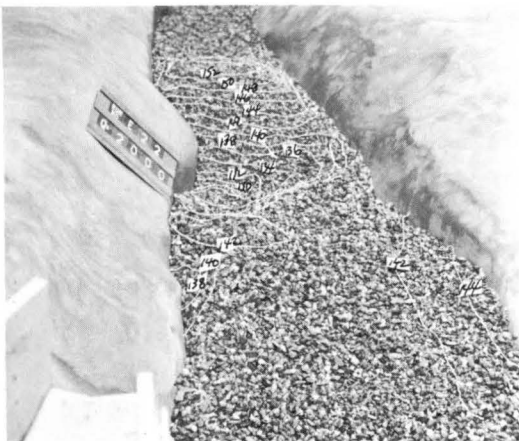


Figure 61 (b). Scour patterns with bed rock at elevation 139.0 meters. $Q = 2000$ cms.



Figure 61 (c). Scour patterns with bed rock at elevation 139.0 meters. $Q = 3000$ cms.

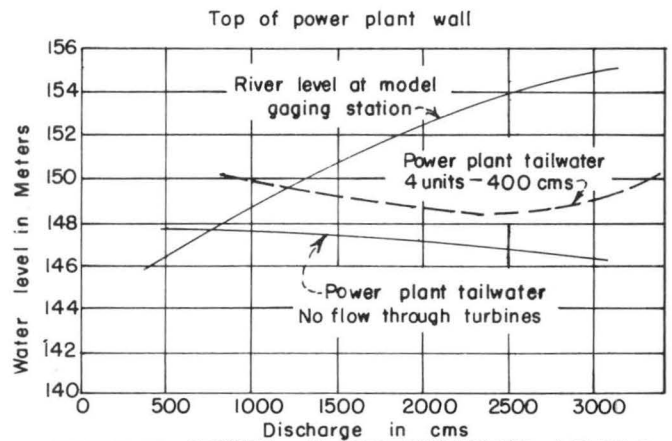


FIGURE 62 POWER PLANT TAILWATER LEVELS BED ROCK AT ELEV. 139.0

CONCLUSIONS AND RECOMMENDATIONS

Several modifications to the preliminary design of the spillway are recommended as a result of the model studies.

1. Approach Channel -

- (a) The right bank should be excavated to elevation 173.0 meters to improve the approach flow.
- (b) The left wing wall should be continued on a circular arc with 20-meter radius to intersect the top of the cut at elevation 287.0 meters.
- (c) The right wing wall should be constructed as a quarter circle of 20-meter radius and the surface of the cut adjacent to the wing wall should be paved for protection from high velocities.
- (d) It is suggested that the entrance to the approach channel on the left bank be rounded to eliminate separation.

2. Crest and Gate Structure -

- (a) The preliminary shape of the crest was satisfactory. No negative pressures were measured on the crest.
- (b) The preliminary shape of pier nose was satisfactory. However, if an extended pier is necessary for structural reasons, a 4:1 elliptical nose would be hydraulically satisfactory.
- (c) The gate pins should be raised to elevation 277.04 meters to prevent inundation at high discharges.

3. Transition -

- (a) An extension wall 20 meters long in the shape described in Fig. 24 should be added downstream from the pier at the centerline of the spillway to reduce the standing waves in the chute. At discharges near 3000 cms the walls would otherwise be overtopped. With the pier extension added, no increase in chute wall height is necessary. The height of the extension wall should be the same as the transition wall heights.
- (b) The length of the transition need not be altered from the preliminary design length. There were no serious standing waves generated from the change in wall

direction at the end of the transition.

4. Vertical Curve -

No change is necessary to the equation for the vertical curve. Pressures on the bottom were positive. In view of the change in the chute slope downstream from the vertical curve, necessitated by a longer flip bucket, the vertical curve should be extended by the same curve equation to meet the changed slope tangentially.

5. Chute Slope -

Downstream from the vertical curve, the chute slope should be revised to meet the extended vertical curve tangentially.

6. Flip Bucket -

- (a) A functionally satisfactory deflecting flip bucket was developed involving only plane surfaces. The recommended dimensions and arrangement of the planes comprising the deflector are shown in Fig. 51.
- (b) Pressures on the surfaces of the flip bucket were all positive.
- (c) The flip bucket structure was 4.2 meters longer than the preliminary design.

7. Scour and Power Plant Tail Water Levels -

- (a) The river bed scour will have decided effects on the power plant tail water levels, especially due to formations of gravel bars downstream of the scour area. Extremely adverse scour conditions were tested in the general model but in no case was the tail water level found to overtop the power plant wall. As gravel bars form, the tail water level will rise and some loss in effective head of the turbines is to be anticipated.
- (b) Scour studies in the model were made with non-erodible river banks. If, in the prototype, sound rock is not prevalent through the expected scour area along the river banks, some deterioration of the banks can be expected. The seriousness of bank scour will depend upon stability of bank material, flood discharge duration and frequency.

- (c) The water levels in the tailrace of the power plant, during periods with no spillway discharge, will be controlled either by the height and size of gravel bar downstream of the scour area or by the river bed level immediately downstream from the power plant. Time-degradation studies of the river bed were not made in the model, but it would be difficult to envision very serious degradation of the river bed level between jet impact area and the power plant. Consequently, it is anticipated the tail water levels would always be above the river bed levels at the control point near the power plant.

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APPENDICES

APPENDIX A

SPILLWAY CREST PRESSURES

Table A-1
No Gated Control

Run No.	Q Prototype cms	Piezometer Numbers* Pressure in Prototype Meters of Water								
		1	2	3	4	5	6	7	8	9
1	575	8.25	5.20	2.50	1.95	2.20	2.45	2.85	3.60	3.25
2	750	9.35	6.00	2.65	2.05	2.60	3.15	3.80	4.65	4.25
3	1000	10.55	6.85	2.75	2.20	3.00	3.90	5.00	5.90	5.45
4	1550	12.65	8.20	2.85	2.50	3.85	5.40	7.10	8.15	7.75
5	2000	14.35	9.00	2.90	2.60	4.60	6.65	8.70	9.90	9.50
6	2500	16.00	10.15	3.15	3.05	5.40	7.90	10.25	11.55	11.15
7	2975	17.55	10.70	2.55	2.80	6.00	8.95	11.40	12.95	12.80

* For location of piezometers refer to Figure 7

Table A-2
With Gated Control

Run No.	Prototype Q cms	Gate Opening Meters	Piezometer Numbers* Pressure in Prototype Meters of Water						
			3	4	5	6	7	8	9
44	990	10.22	2.85	1.75	3.75	6.05	8.15	9.45	
45	1050	10.93	2.75	2.25	4.00	6.35	8.45	9.75	
46	1150	11.56	2.00	2.25	4.75	7.00	9.00	10.25	
47	1200	12.26	2.30	2.20	4.75	7.25	9.20	10.60	
48	980	9.55	2.70	1.45	3.35	5.70	7.85	9.20	
49	880	8.85	2.85	1.30	2.95	5.00	7.65	8.80	
50	825	8.20	3.00	1.10	2.55	4.75	7.45	8.50	9.00
51	765	7.50	3.20	1.10	2.30	4.30	7.10	7.85	8.75
52	715	6.82	3.50	0.75	1.85	3.85	6.90	7.65	8.70
53	640	6.12	3.90	0.75	1.65	3.60	6.70	7.20	8.60
54	585	5.49	4.30	0.75	1.35	3.20	6.40	6.70	8.35
55	510	4.82	5.15	0.65	1.15	2.80	6.25	6.20	8.50
56	455	4.17	6.15	0.85	0.85	2.40	6.00	5.50	8.50
57	370	3.51	7.35	1.25	0.80	2.10	5.50	4.70	8.25
58	330	2.91	9.00	1.75	0.70	1.75	5.20	3.95	8.20

* Gate opening is the vertical distance from the bottom of the gate to the gate seat.

APPENDIX B
PRESSURES ON VERTICAL CURVE

Table B-1
 Without Gated Control

Run No.	Prototype Q cms	Piezometer Location* Pressure in Prototype Meters of Water								
		1	2	3	4	5	6	7	8	9
1	575	2.20	1.15	0.90	0.90	0.70	0.60	0.40	0.95	0.65
2	750	2.45	1.25	1.10	1.00	0.70	0.75	0.55	1.20	0.90
3	1000	2.90	1.60	1.35	1.20	1.10	1.10	0.75	1.40	1.10
4	1550	3.90	2.20	1.80	1.50	1.40	1.40	0.95	1.65	1.40
5	2000	4.65	2.60	2.10	1.70	1.60	1.60	1.10	1.80	1.65
6	2500	5.35	2.90	2.30	1.90	1.75	1.70	1.15	1.90	1.85
7	2975	5.95	3.25	2.45	2.00	1.90	1.85	1.25	2.00	2.00

* For piezometer locations refer to Figure 8

APPENDIX C
PRESSURES IN THE FLIP BUCKET

Table C-1

Prototype Q cms	Piezometer Numbers* Pressure in Prototype Meters of Water															
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
1000	36.8	30.9	24.2	31.3	43.5	32.7	10.5	30.7	29.2	30.5	55.5	58.5	29.2	4.9	0.4	0.2
2000	21.1	21.3	13.0	22.7	38.4	28.0	9.5	16.7	15.4	16.6	45.9	48.1	14.4	2.0	--	--
3000	20.9	10.3	4.3	8.9	28.6	14.8	3.4	5.5	4.7	4.3	30.0	30.2	1.6	--	--	--

* For piezometer locations refer to Figure 52