HYDRAULIC MODEL STUDY

OF THE

KREMASTA DAM SPILLWAY

KREMASTA HYDROELECTRIC PROJECT
ACHELOOS RIVER DEVELOPMENT
GREECE

ADDENDUM I
FLIP BUCKET RELOCATION

Colorado State University Research Foundation
Civil Engineering Section
Hydraulics Laboratory
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September 1962 CER62SSK19a

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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

ADDENDUM I
FLIP BUCKET RELOCATION

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INTRODUCTION

Subsequent to the publication of "Hydraulic Model Study of the Kremasts Dam Spillway" in March, 1962, additional foundation investigations revealed the advisability of relocating the spillway flip bucket. It was proposed that the chute spillway be shortened a horizontal distance of 25 meters with no change in slope so that the floor of the flip bucket would be at elevation 195.21 meters and the flip bucket would terminate at station 0+580.56 meters.

The general model of the Kremasta Dam spillway which had been constructed in the hydraulics laboratory at Colorado State University was altered to meet the above conditions. Additional model tests were performed to check the flip bucket performance in the revised location. Specifically, the objectives of the study were to:

1. Determine the jet impact area of the flip bucket in the revised location and modify the flip bucket if necessary.

2. Determine the effect of the jet impact on the river bed and on the water level in the power plant tailrace.

3. Determine the slope protection necessary at the flip bucket to prevent erosion of the hillside.

Flip Bucket -- The chute spillway of the general model was shortened a horizontal distance of 25 meters with no change in the spillway slope so that flip bucket terminated at station 0+580.56 meters and the bucket floor was at elevation 195.21 meters.

The recommended flip bucket described in "Hydraulic Model Study of the Kremasta Dam Spillway" was tested in the revised location on the model for prototype spillway discharges ranging from zero to 3000 cms. At all discharges greater than approximately 200 cms, the flip bucket performed satisfactorily. The resulting flow conditions are shown in Figures 63(a) through (j) .

Figure 63(a)

Figure 63(b)

Figure 63(c)

Figure 63(d)

Figure 63(e)

Figure 63(f)

Figure 63(g)

Figure 63(h)

Figure 63(i)

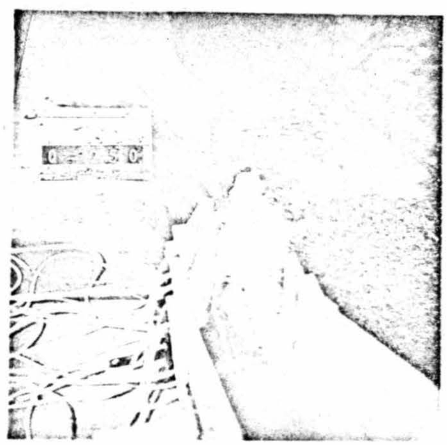


Figure 63(j)

Discharges less than 200 cms impacted on the hillside before cascading into the river below. The recommended measures to prevent erosion are discussed in the section on slope protection.

Due to a slightly lower velocity of flow through the flip bucket in the revised location and the resulting greater flow depth, the left wall height of the recommended flip bucket has been increased. A higher flow line along the left was also caused by interception of the standing wave at a different point from the original flip bucket location. The pertinent dimensions of the recommended flip bucket are shown in Figure 64.

Dynamic pressure heads were measured at selected points in the model flip bucket at a prototype discharge of 3000 cms. The pressure point locations and measured pressure heads are shown in Figure 65. No negative pressures were recorded. Figure 65 also shows the water surface profile along the right wall of the flip bucket for discharges of 1000, 2000, and 3000 cms.

Slope Protection -- At spillway discharges less than approximately 200 cms, the jet emerging from the model flip bucket impacted on the hillside before falling to the river below. Because spillway discharges of this magnitude will occur frequently in the prototype, some preventative measures should be taken to protect the hillside impact area from erosion. The preventive measures consisted of slope protection by paving with retaining walls.

The first method investigated, as proposed by Engineering Consultants Inc., consisted of a paved area bounded by the flip bucket, two curved training walls and the cliff edge as shown in Figures 66 and 67. The flow at the right was contained by a wall tapering in height from 2.0 meters at the flip bucket to 4.5 meters at the edge of the cliff. A freeboard of

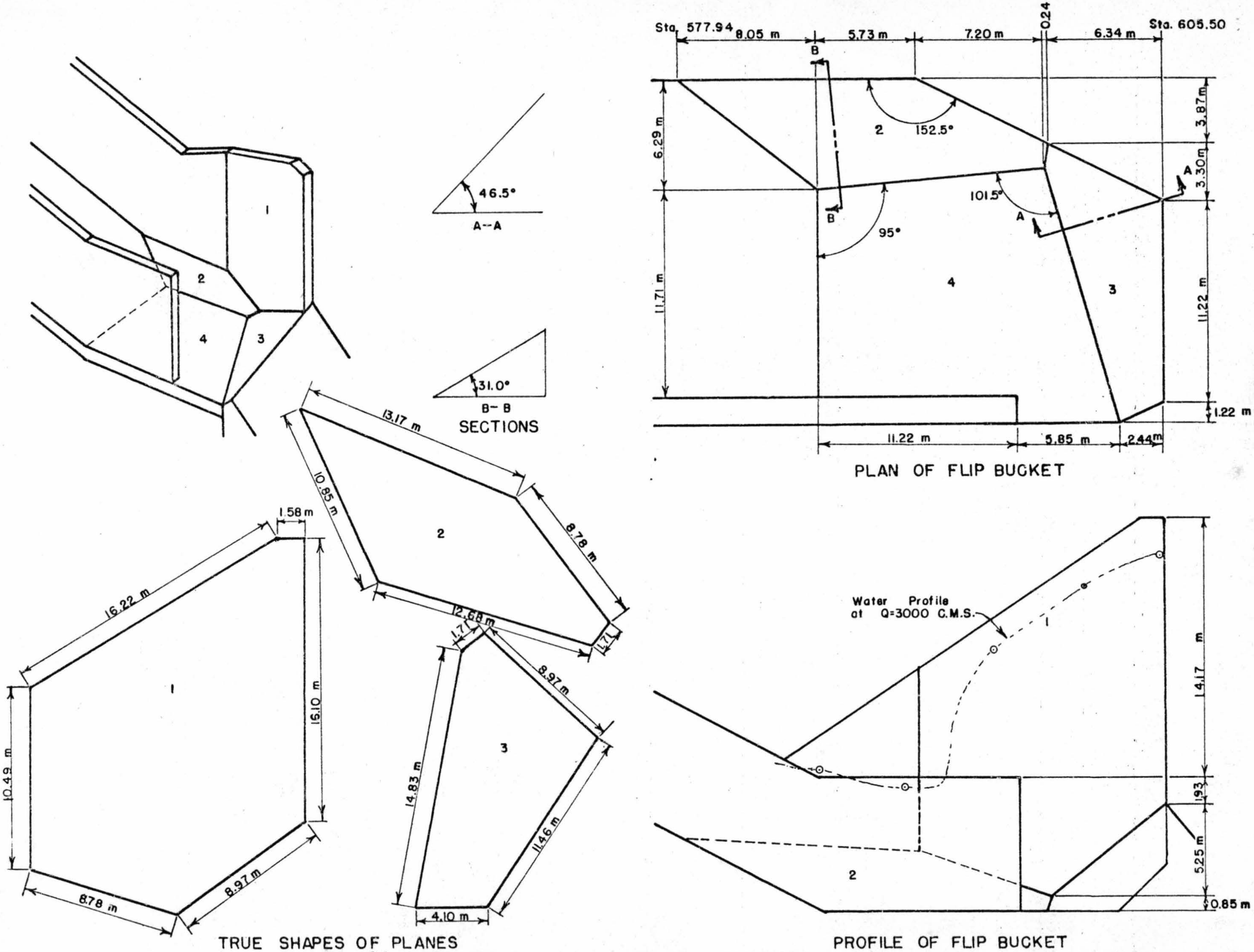
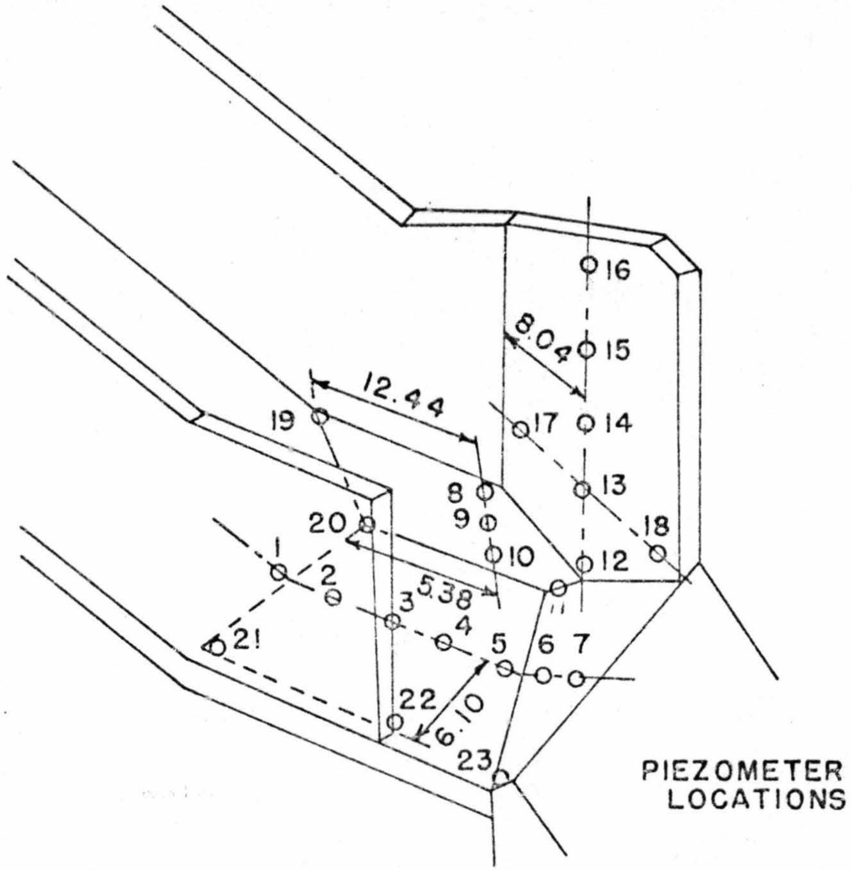
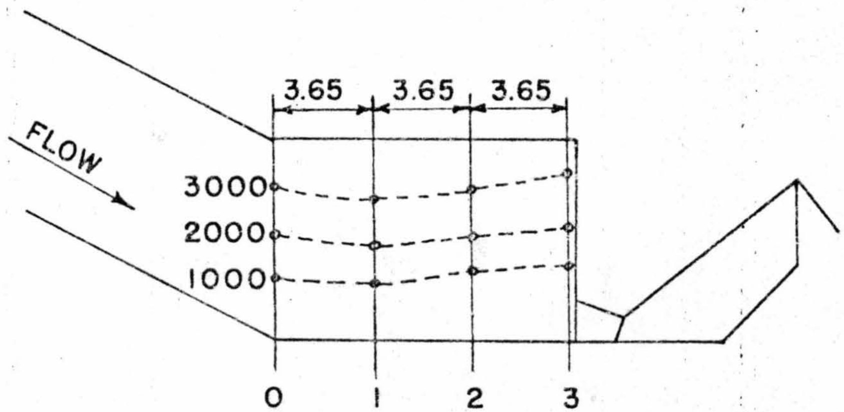


Fig. 64 RECOMMENDED FLIP BUCKET GEOMETRY



WATER SURFACE PROFILE ALONG RIGHT WALL OF FLIP BUCKET

	FLOW DEPTH ABOVE FLIP BUCKET FLOOR IN METERS		
Q	1000 CMS	2000 CMS	3000 CMS
0	2.55	4.25	6.33
1	2.30	3.76	5.84
2	2.78	4.38	6.21
3	3.03	4.75	6.94



RIGHT WALL PROFILE

PRESSURE HEADS IN FLIP BUCKET IN METERS OF WATER PROTOTYPE

PROTO-TYPE Q CMS	PIEZOMETER NUMBERS																					
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
1000																							
2000																							
3000	35.4	32.6	26.4	31.0	41.2	33.3	10.1	27.1	26.6	29.5	46.3	—	33.3	9.6	2.6	—	33.3	12.1	26.0	33.8	34.0	15.6	11.8

Figure 65 Water Surface Profiles and Pressure Heads in Flip Bucket

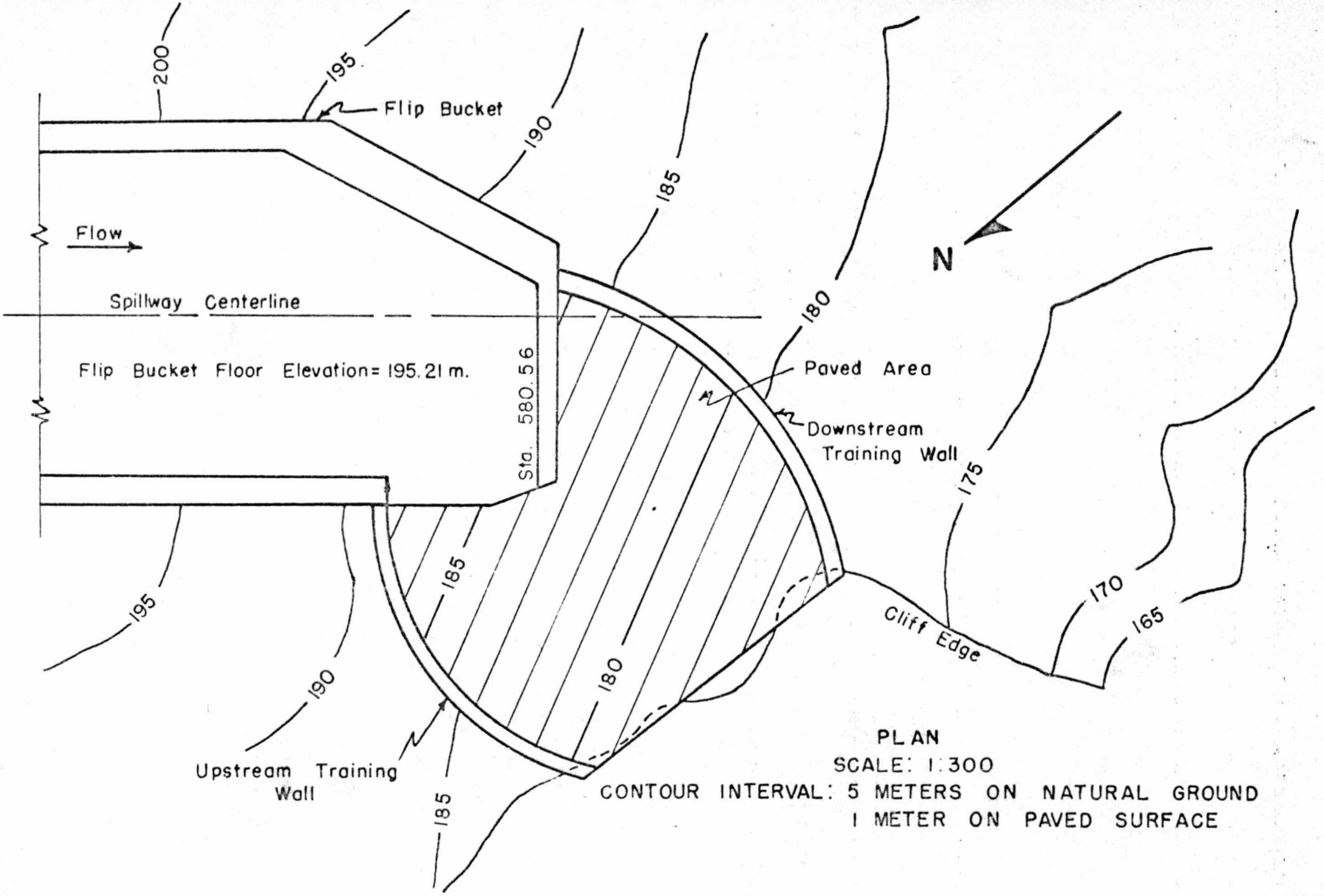


FIGURE 66 SLOPE PROTECTION, PRELIMINARY DESIGN

Figure 67 Preliminary slope protection below flip bucket.

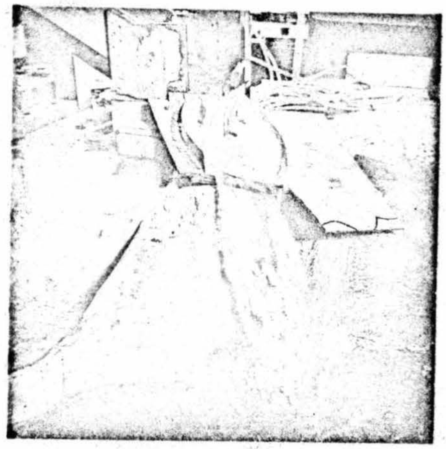


Figure 68(a) Flow over preliminary paved protection at Q = 50 cms.

Figure 68(b) Flow over preliminary paved protection. Q = 100 cms.

Figure 68(c) Flow over preliminary paved protection. Q = 150 cms.

1.0 meter is afforded by this wall height. A fillet at the base of the wall was provided. Although this fillet is not required for hydraulic reasons it might be used to structurally stabilize the wall. The left wall was 4.0 meters in height which contained discharges up to 50 cms, but was not adequate for higher discharges. Discharges above 50 cms but less than 200 cms impacted on the hillside downstream of the wall. Figures 68(a) through 68(c) illustrate the flow conditions for discharges of 50, 100, and 150 cms. The jet for discharges above approximately 200 cms cleared the cliff edge as shown by Figure 69.

Figure 69 Flip bucket jet clears paving edge and cliff.

Downstream walls of various heights and alignments were tested to determine an economical and effective solution to the problem. The recommended slope protection consists of a large paved area bounded by the upstream curved wall described above, the flip bucket, the cliff edge, and a straight vertical wall 2.0 meters in height extending along the left edge of the paving from the flip bucket to the cliff edge as shown in Figures 70 and 71. In order to prevent overtopping near the downstream end of the left wall without increasing the wall height, it is recommended that a curved deflector be constructed 16 meters long, along the downstream length of the wall as shown in Figure 72.

Figure 71 Recommended paved slope protection.

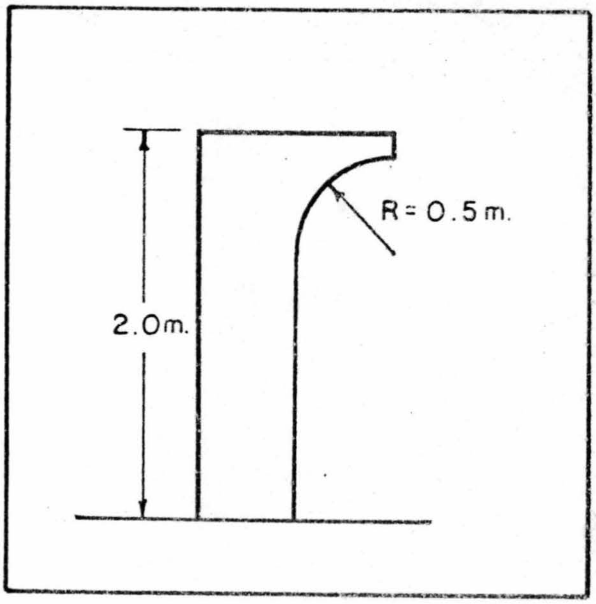


Figure 72 Vertical section of left wall at the downstream end of paving.

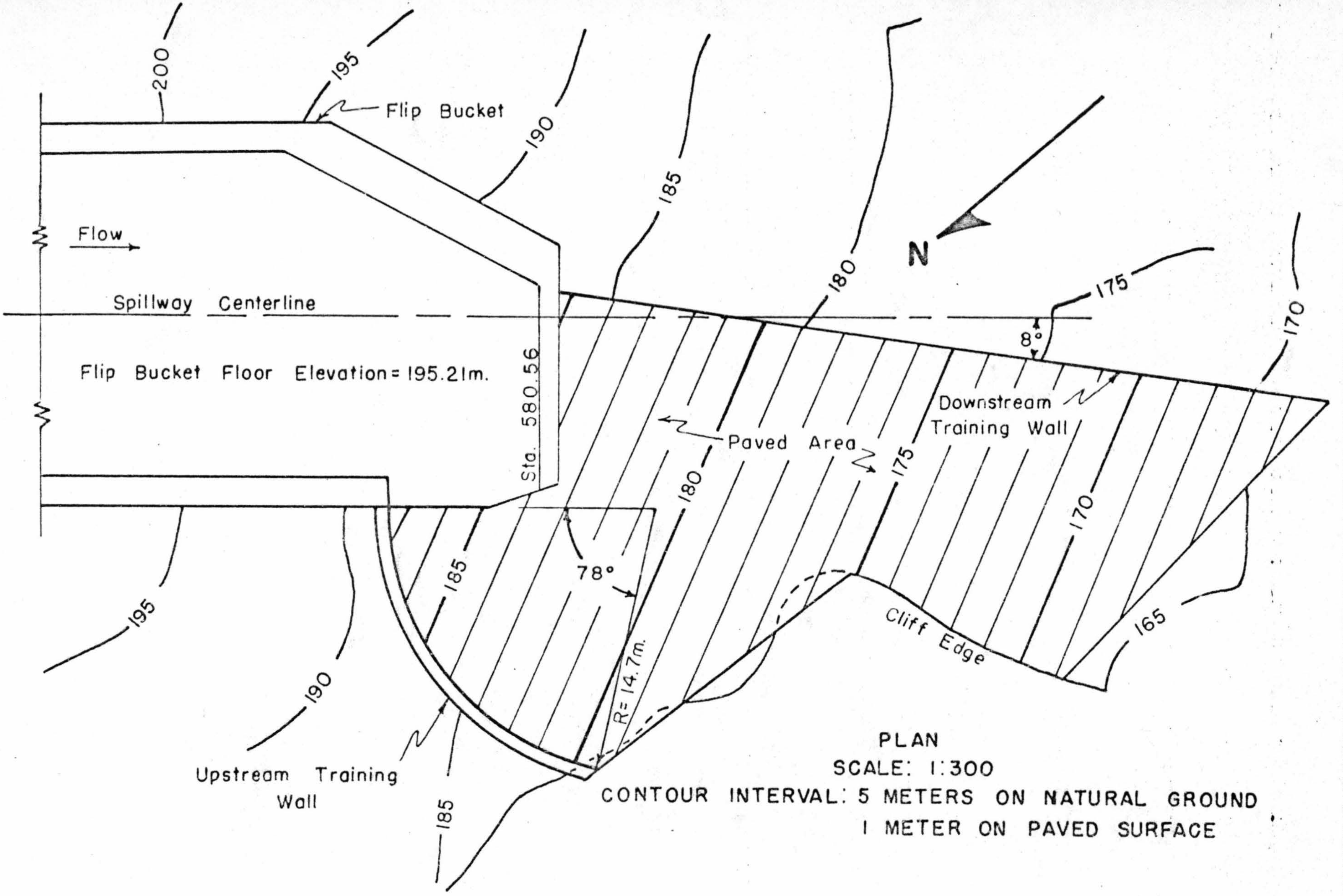


FIGURE 70 SLOPE PROTECTION, RECOMMENDED DESIGN

Figure 73 Recommended paving
Q = 100 cms

Figure 74 Recommended paving
Q = 150 cms

Figures 73 and 74 show model flow conditions over the paving with discharges of 100 and 150 cms respectively.

River Bed Erosion and Power Plant Tail Water Levels

The stage-discharge curves at the location of the model gaging station are reproduced as Figure 75 which coincides with Figure 53 of the parent report.

To provide a range of possible conditions of the river bed in the prototype, as was done in the parent study, the model tests included two different bed material sizes and the elevation of the "bed rock" was studied at two levels (115.0 meters and 139.0 meters). Description of the model river bed materials can be found on page 30 of the original report.

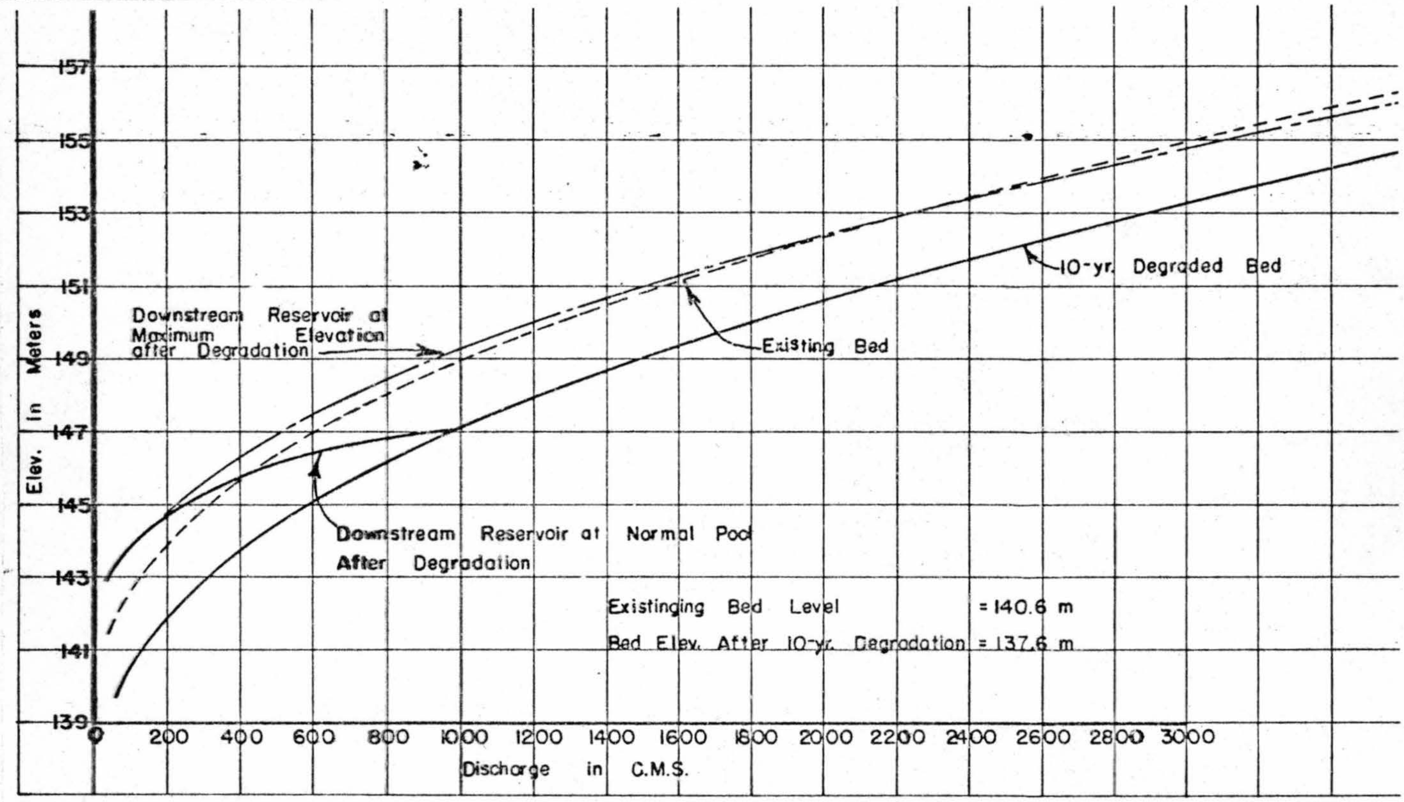


Figure 75 STAGE DISCHARGE RATING CURVES AT MODEL GAGING STATION

The development of scour in the river bed with the 3/8-inch gravel and spillway discharge of 1000 cms is shown in Figure 76(a). The tailwater levels at the power plant are shown in Figure 76(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 128.0 m. Test results for discharges of 2000 and 3000 cms are shown in Figures 77 and 78 respectively. The power plant wall was not overtopped in any of the tests.

Scour development after a flow cycle, the hydrograph of which is shown in Figure 79, is shown in Figure 80(a). The scour area was more extensive and a greater amount of river bed material was transported downstream. Variation of tailwater levels with discharge are shown in Figure 80(b). The maximum tailwater level was 153.1 m providing 2.9 meters of freeboard to the top of the wall. On the recession cycle of the hydrograph the power plant was shut down and tailwater levels were measured with only the spillway flows. The result is shown in Figure 80(c).

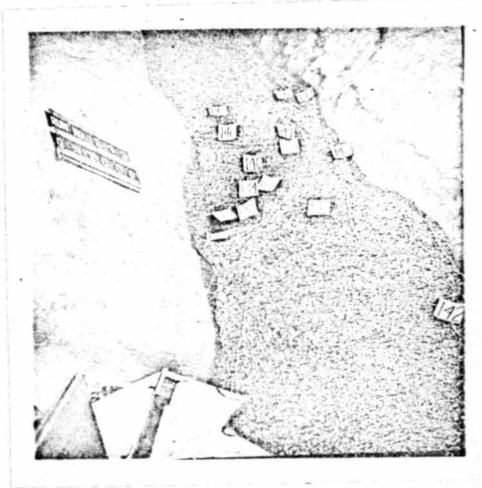


Figure 76(a) Scour pattern with 3/8-inch gravel
Q = 1000 cms
Power Plant Q = 400 cms
Model time = 25 min.
Maximum scour = Elevation 128.0 m

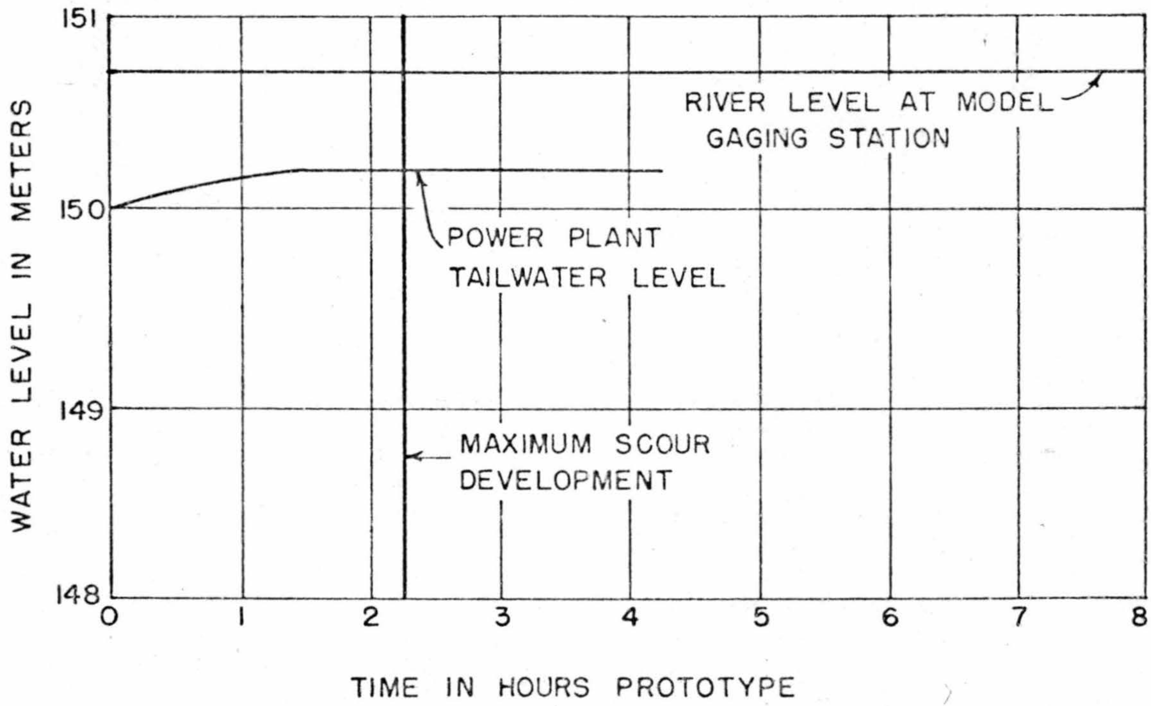


Figure 76(b)

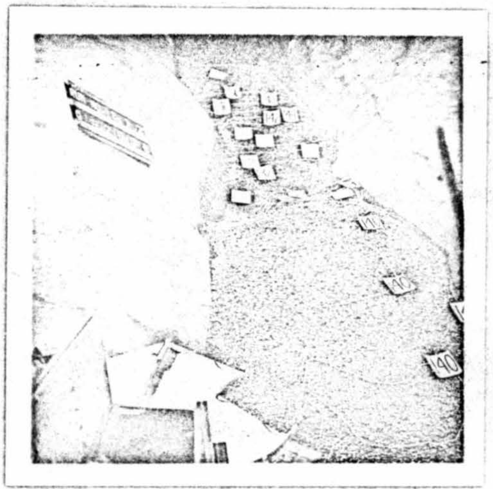


Figure 77(a) Scour pattern with 3/8-in. gravel
Q = 2000 cms
Power plant Q = 400 cms
Model time = 30 min.
Maximum scour El. = 130 m

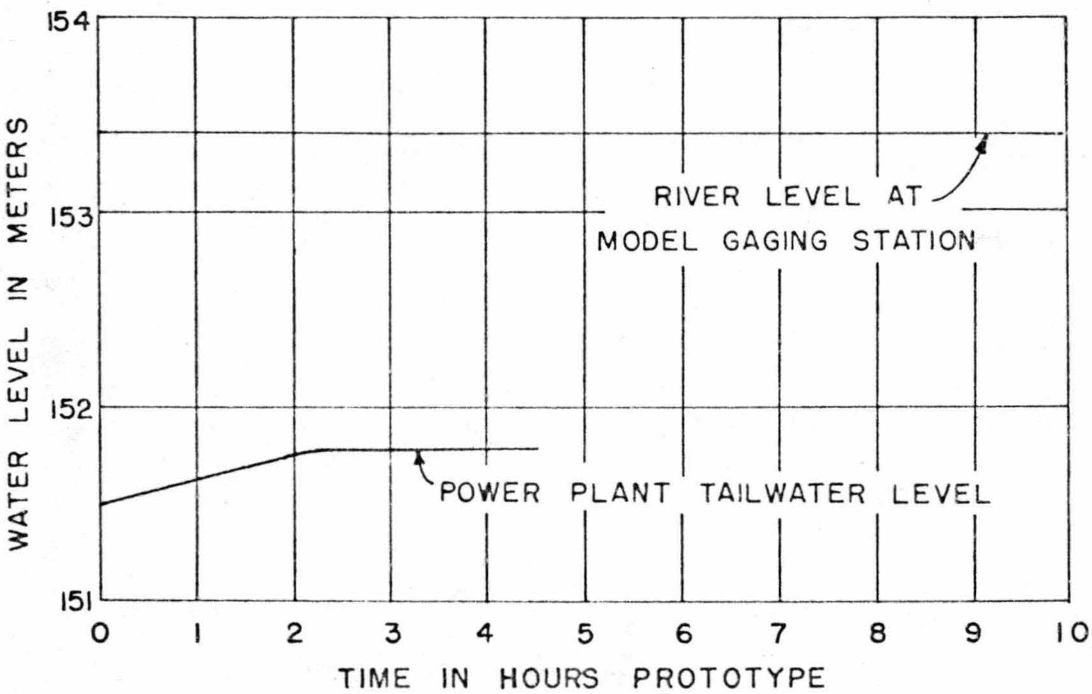


Figure 77(b)

Figure 78(a) Scour pattern with 3/8-in. gravel
Spillway Q = 3000 cms
Power plant Q = 400 cms
Model time = 40 min.
Maximum scour El. = 124 m

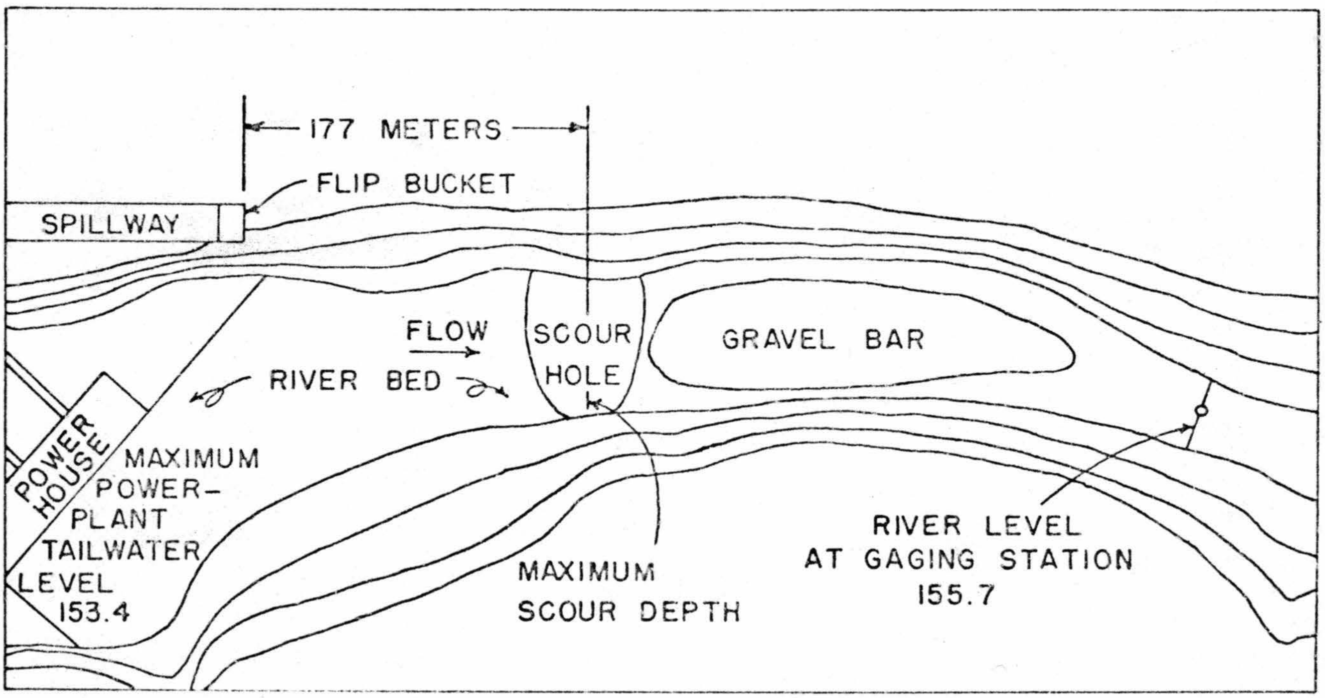


Figure 78(b)

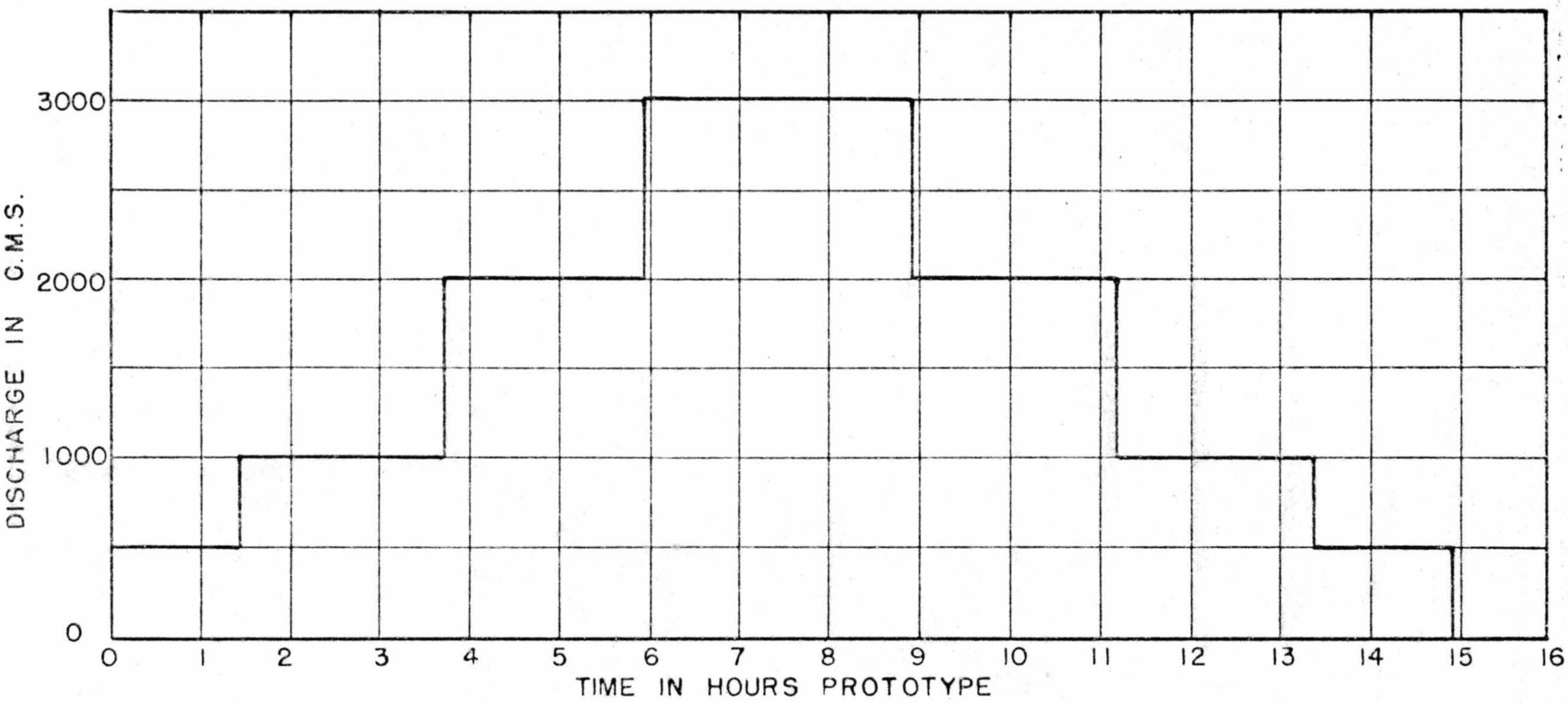


Figure 79 STEP HYDROGRAPH

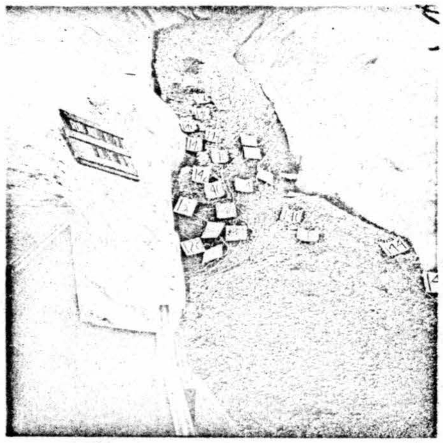


Figure 80(a) Scour pattern with hydrograph of Figure 79.

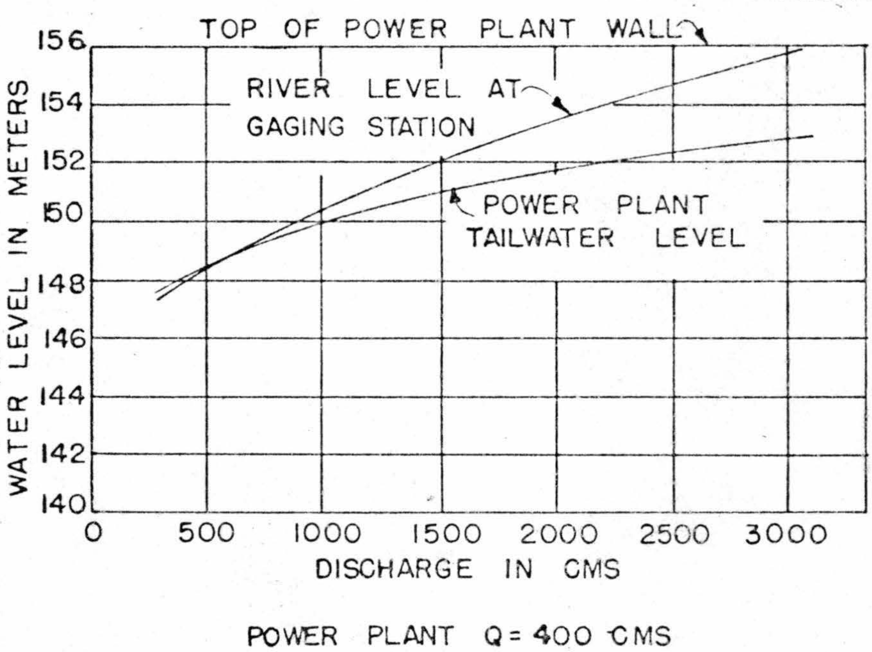


Figure 80(b) Power plant tailwater level.

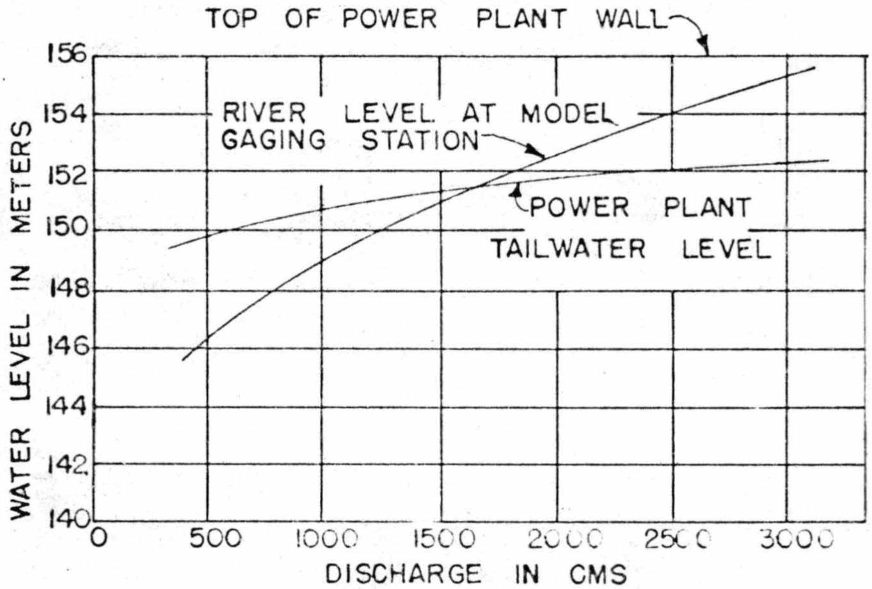


Figure 80(c) Power plant tailwater level. No power plant discharge

Because of the gravel bar formed downstream of the scour area, the tailwater levels at discharges less than 2000 cms 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.

To determine at what river stage 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 Figure 81. There was at least 5.5 meters of allowance in depth between the calculated stage and that which would cause overtopping.

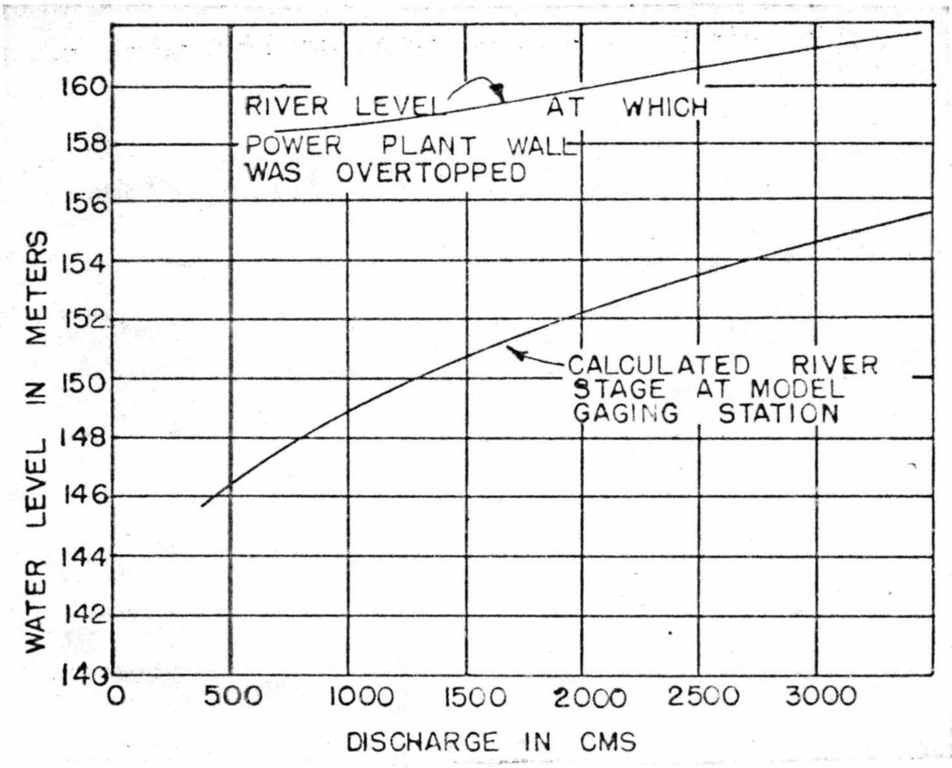


Figure 81 River stage studies

The results of tests with 3/4-inch gravel in the model river bed are shown in Figures 82(a) through (c) for discharges of 1000, 2000, and 3000 cms. The power plant tailwater levels were the same as tests with the 3/8-inch gravel bed.

Tests results with river bed rocks located at elevation 139.0 m are shown in Figures 83(a) through (c) for discharges comparable to Figure 82. Measured tailwater levels at the power plant are shown in Figure 84. The tailwater at the power plant was lower for corresponding discharges during tests with the bed rock at elevation 139.0 m than during tests with the bed rock at elevation 115.0 m. This was because of the draw-down effect of the flip bucket jet. No apparent tailwater problem will arise if bed rock is near the present river bottom.

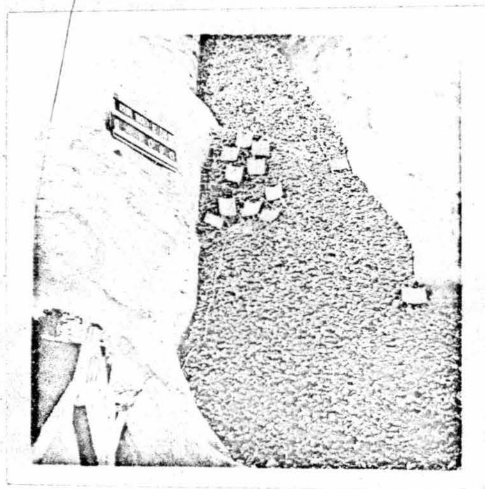


Figure 82(a) Scour patterns with 3/4-inch gravel Q = 1000 cms

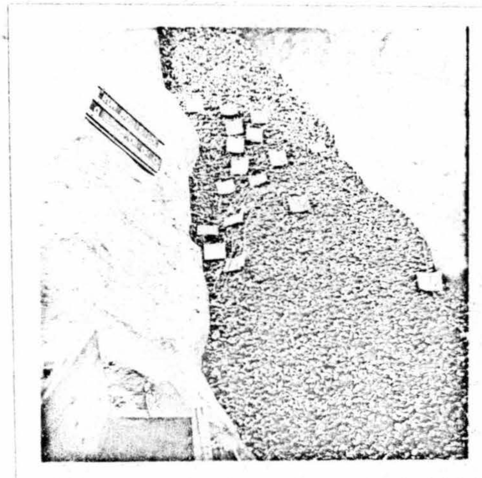


Figure 82(b) Scour patterns with 3/4-inch gravel Q = 2000 cms

Figure 82(c) Scour patterns with 3/4-inch gravel Q = 3000 cms

Figure 83(a) Scour patterns with bed rock at elevation 139.0 m Q = 1000 cms

Figure 83(b) Scour patterns with bed rock at elevation 139.0 m Q = 2000 cms

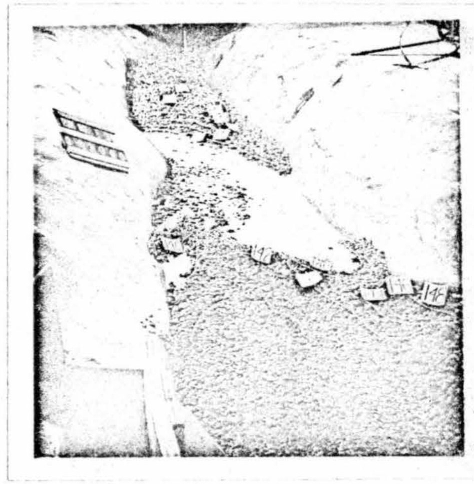


Figure 83(c) Scour patterns with bed rock at elevation 139.0 m Q = 3000 cms

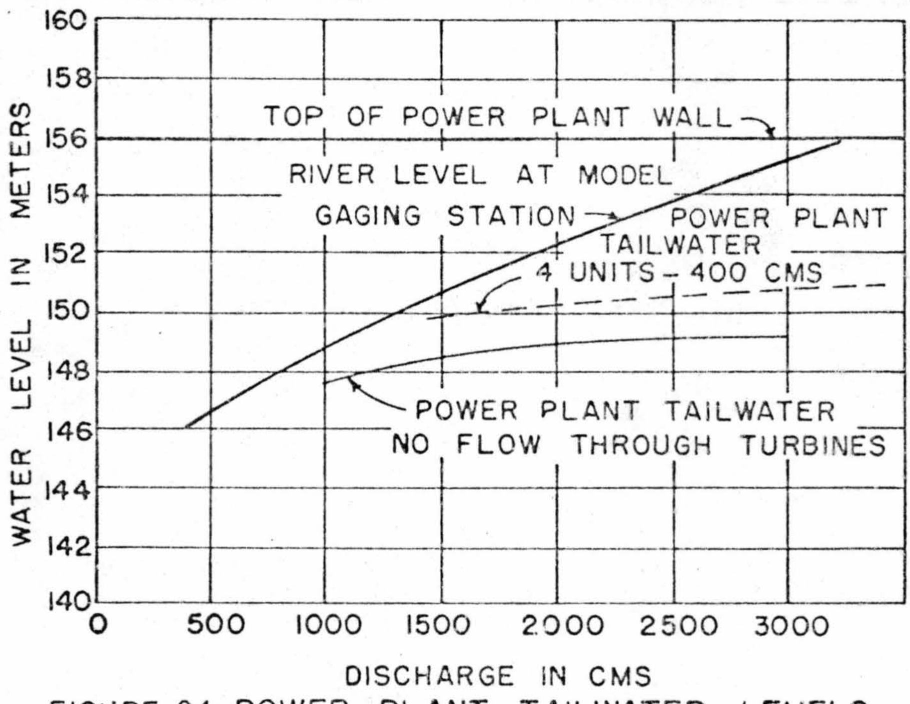


FIGURE 84 POWER PLANT TAILWATER LEVELS
BED ROCK AT ELEV. 139.0