HYDRAULIC MODEL STUDIES OF AMALUZA DAM SPILLWAY

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16. ABSTRACT

Studies were made on a 1:70 scale model of the spillway for Amaluza Dam on the Paute River, Ecuador. Two spillway designs were tested; an end pier was developed which reduced the drawdown along the outside retaining walls; and measures were made of the flow depth and pressures on the spillway face. The narrow river canyon downstream from the dam contains heavy overburden, and it was desired to direct the jets (from the flip buckets) as far from the dam and as near the center of the river as possible to reduce the danger of undermining the dam. The spillway is 114.5 m high, from the bucket invert to the crest, and discharges 7724 m^3/s at a head of 13.5 m above the crest. The bucket inverts are 15 m above the river channel. A design developed by the model study produced the desired results. At maximum discharge the jets from the 54.75-m-wide flip bucket are confined to 15 m wide and spread 90 m along the centerline of the river. The nearest jet impacts 127 m downstream from the lip of the bucket.

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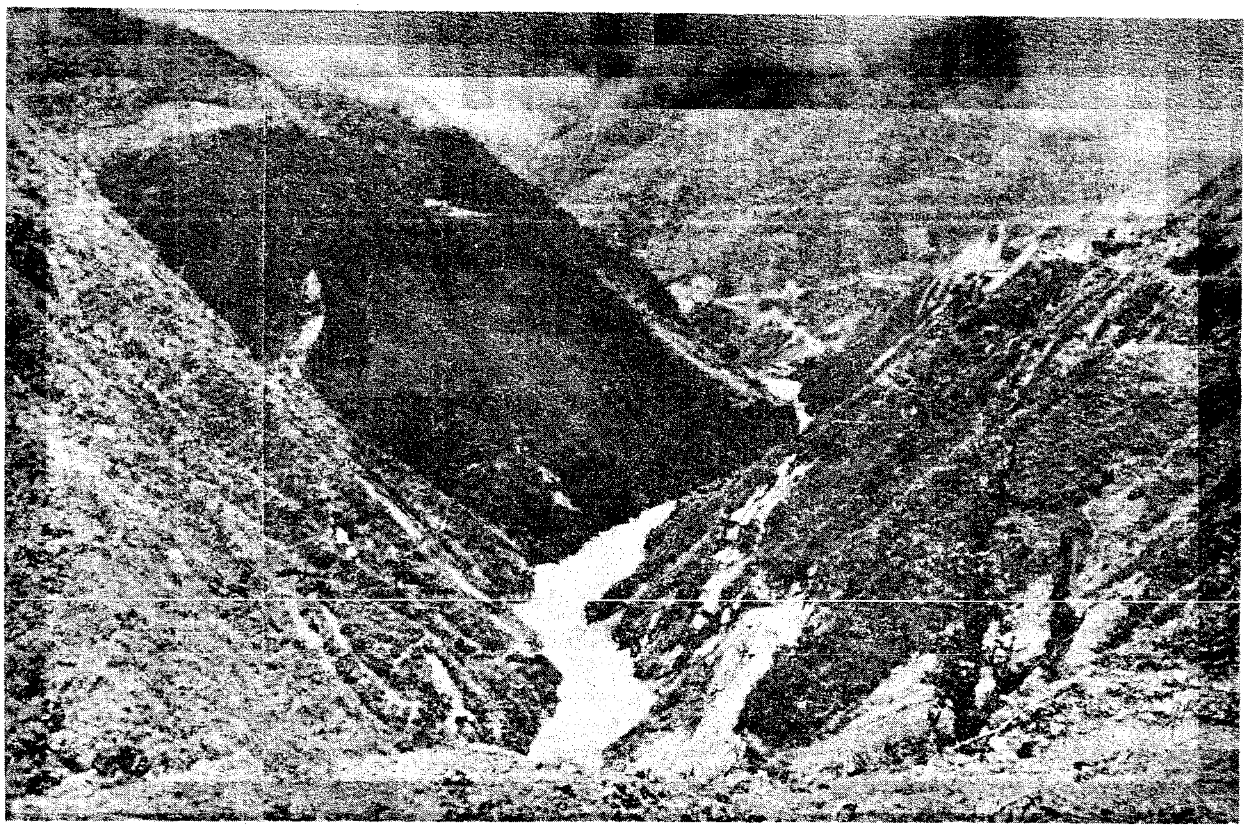

Frontispiece.–Paute River Valley in Ecuador at the Amaluza damsite. Photo P801-D-77405

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INTRODUCTION

The Paute River and its tributaries in Southeastern Ecuador drain the Cuenca Basin from the east slope of the Andes Mountains into the Amazon River. The Paute River is quite steep and presents an excellent opportunity for high-head hydroelectric development. The Ecuadorian Government has established the Paute Project to harness the flowing water of the Paute River for the production of much-needed electric energy.

The first stage of the Paute Project will be the construction of Amaluza Dam with an installed power capacity of 500 MW. Amaluza Dam will be a thin-arch structure 155 m high[1] with an overfall spillway and flip bucket near the midportion of the dam. At the Amaluza damsite the Paute River flows south. Downstream from the damsite the river swings east and then northeast in a large arc, and drops 515 m in elevation in a river run of 12 km. A power penstock tunnel 7 km long will be drilled directly through the mountain to the east of the Amaluza damsite to intersect the Paute River valley. Thus, the available head at the powerplant will be 670 m, 155 m backed up by the dam and 515 m of river fall between the dam and the powerplant.

In the locality of Amaluza Dam, the Paute River is confined in a deep canyon with steep side slopes. The overburden on the slopes and in the riverbed consists of sand, cobbles, boulders, and large rock blocks overlaying a zone of weathered grandodiorite. The overburden ranges up to 30 m thick, is not stable, and has produced small or large landslides wherever road and trail construction or investigative activities have disturbed the surface.

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[1] All dimensions (except for model construction as described later) in this report are metric. All elevations are given in metres above mean sea level.

One area, known as the "Gualpa slide," starts 300 m downstream from the dam axis, is about 600 m wide, as much as 30 m deep, and extends from the riverbed, elevation 1830, up the right canyon wall to elevation 2280 (metres above mean sea level). The Gualpa slide is of grave concern because it is in a very delicate state of equilibrium and contains several million cubic metres of material.

The Paute Project is under the direction of INECEL (Instituto Ecutoriano De Electrificacion), Quito, Ecuador. The design of Amaluza Dam is guided by a board of consultants for INECEL, and the responsibility for the design features rests wth IECO (International Engineering Company, Inc.), San Francisco, Calif. The USBR Hydraulics Laboratory was requested by INECEL to construct and test a hydraulic model of the Amaluza Dam spillway.

The purpose of the model study was to determine the spillway capacity for both free and gate-controlled flow and to determine the hydraulic adequacy of the spillway with regard to spillway face pressures, entrance flow conditions to the crest from the reservoir, and jet impact location downstream from the dam. Because of the presence of fractured rock and deep overburden in the area of the damsite, it was desired to force the jet from the spillway flip bucket to impact a minimum distance of about 100 m from the toe of the dam for 1500 m^3/s and to be confined to a narrow band near the centerline of the river to minimize the amount of overburden washed from the canyon walls into the river channel.

CONCLUSIONS

1. The profile and plan of the spillway recommended for construction are shown in figures 27 and 28 (referred to in this report as the "Lombardi spillway").

2. The discharge capacity curves for the recommended design are shown on figure 11.

3. The lowest pressure on the spillway face with the Lombardi design is near atmospheric. This pressure occurs throughout the lower 80 percent of the upper parabolic portion of the spillway. (Note: To reduce the danger of cavitation damage, great care must be exercised during construction of the prototype spillway to assure that inadvertent surface irregularities or deviations from the true parabolic alinement do not occur.)

4. The recommended end pier design is shown in figure 24. With these end piers installed, the radial gate trunnions may be placed as designed or: 14.26 m downstream from the crest centerline and 3.19 m above the elevation of the spillway crest. With the recommended end piers installed, the water surface will not impinge on the trunnions.

5. The flip bucket design shown in figure 27 will operate satisfactorily for free flow or controlled flow using any combination of gates.

6. The results of the scour tests in the downstream river channel are pictured in figures 37 through 48. From these studies the location of the maximum scour hole may be determined, and a qualitative comparison may be made of the areas of deposition of material scoured from the riverbed for various discharges. The action of the jets on the riverbed material and the potential for slides of the overburden on the steep canyon walls are unknown. A determination of the volumes of material which will be moved in the prototype may not be inferred from these studies.

7. Neither radial gate No. 2 nor No. 5 should be operated alone in their respective bays. The spreading jet on the spillway face will impinge on and perhaps overtop

the training wall, causing flows and potential scour along the side of the spillway structure.

8. Drawdown on the side of an intermediate pier in a bay operating with the gate fully opened may be expected if the gate in the bay on the opposite side of the intermediate pier is closed.

9. With one bay operating and an adjacent bay not operating, the side pressure on the dividing or retaining wall may be as high as 275 kPa in the bucket area.

10. With the left bay discharging less than about 600 m^3/s, the jet will wash the left canyon wall near the dam and produce deep riverbed scour against the left canyon wall bedrock. Onsite examination should be made to determine whether such washing would be hazardous to the dam.

THE MODEL

The initial model (designated spillway design "A") was constructed to a scale of 1:70. Subsequent to the fabrication of the initial model crest section to the transition at elevation 1907.56, as shown on the preliminary design drawings, new design drawings were received showing the transition starting at elevation 1925.34 and the parabolic bucket profile. To utilize the already constructed crest section (fig. 1), the laboratory designed and constructed a radii transition and bucket portion as shown in figure 2. This modified bucket followed very closely the profile shown on the revised design drawings. The lip of the flip bucket and the bucket invert in the model were located exactly as shown on the revised design drawings. Figures 3 and 4 show the model radial gates.

The model head box, tail box, and upstream face of the dam were constructed with plywood (fig. 5). The contours in the reservoir were formed of concrete and extended from elevation 1900 up to elevation 2005, about 11 m above high reservoir elevation (figs. 6a and 6b).

The tail box was constructed sufficiently large to include a 600-m segment of the downstream riverbed. The downstream river contours were not included in the preliminary study. The model installation of the spillway is shown in figure 7. The model reservoir extended upstream about 175 m. The water entered the model through a rock baffle which stilled the incoming flow and produced a wave-free reservoir (fig. 8).

MODEL SIMILITUDE PARAMETERS

The model was constructed to an undistorted linear scale of 1:70 and was evaluated with respect to the Froude laws of similitude. The model discharge, Qm, was determined by the relationship $(1/N^{5/2})(Qp)$ where N is the scale ratio of 70, and Qp is the prototype discharge. A model discharge of 0.188 m^3/s represented the maximum prototype discharge of 7724 m^3/s.

The spillway jet trajectories, direction, and free travel distances were accurately modeled at a true linear scale of 1:70. All pressures were measured in manometer tubes with vertical water columns and could be directly converted to pressure head in metres of water, prototype. Water surface elevations were similarly measured with respect to mean sea level as a datum.

The scour test results were qualitative only. The study indicated the relationships between various model discharges and the resultant model scour for the same initial conditions. The volume, duration, material, and frequency of occurrence of various landslides from the side slopes were unknown and could not be model studied.

SPILLWAY DESIGN "A"

Entrance

Flow past the intermediate piers was acceptable for all free or controlled flows up to the maximum design flood of 7724 m^3/s. Flow past the end piers was unacceptable for free-flow discharges greater than 4300 m^3/s. A drawdown occurred just downstream from the pier nose, followed by a large surface wave which impinged on the gate trunnion (fig. 9). Several modifications were made to the end piers to achieve acceptable flow without changing the trunnion location. A design consisting of a rounded pier nose, overhanging upstream, produced the desired flow (figs. 10a and 10b). Refinements and details of the pier were made after other spillway designs had been studied and a recommended design fabricated.

Calibration

The spillway capacity was determined for both free and controlled flow. The discharge capacity curves, figure 11, were drawn from the results of the study. The gate opening shown on the chart is for all six gates opened the same amount, and the gate open value is the vertical distance of the gate lip above the gate seat. The dashed line shows the elevation of the top of the gate for the gate opening shown.

Studies were made with individual gates or combinations of gates opened. The results indicated that each gate discharged one-sixth the amount shown on the discharge capacity curves for gate opening and reservoir elevation shown.

Flow on the Spillway

Piezometers were installed at intervals on the spillway face on the radial centerline of gate bay 3 (third from the left). Pressures were above atmospheric at all locations for all flows, free or controlled. Table 1 shows the location of each piezometer and pressures for typical flow conditions.

A survey was made of the maximum flow depth on the left training wall for near maximum discharge. These values, shown in table 1, may be used for the design of the height of the left and right training walls.

Flip Bucket and Jet Studies

With the lip of the flip bucket at 30°, and free flow down the spillway, the three jets intersected 200 m from the spillway crest on the center of radii of the spillway. The combined jet continued downstream in line with the spillway centerline. When each of the three bays operated individually, the jet continued on the centerline of the operating bay.

The trajectories of the jets were studied to determine the impact location for various flows (figs. 12a through 12d). It became apparent that the 200-m radius of the spillway did not achieve the desired results of keeping the jet impact near the center of the downstream river and the maximum distance away from the toe of the dam.

With all six gates open and with maximum reservoir, the jets merged and the impact area was near the river centerline. However, with either the right or left bays only, operating, the impact area was on the heavily overburdened left or right bank as shown in figure 13.

Table 1.–*Crest "A" piezometer locations, pressures, and water depths*

Piezometer No.	X[1]	Y[1]	[2] Gates 3.6m, Res. 1993	Gates open–free flow			Water depth, (vertical)
			Q = 2400	2000	5000	7700	7724
1	-3.38	1.12	13.66	5.76	6.83	5.97	
2	-1.85	0.26					
3	-0.09	-0-	11.95	3.41	3.63	2.77	
Crest	-0-	-0-	-	-	-	-	
4	1.69	0.14	10.67	3.20	3.63	3.20	
5	3.20	0.47	6.83	2.56	2.99	2.35	
6	4.62	0.93	1.92	2.13	2.13	1.49	
7	8.18	2.66	0.43	1.71	1.71	1.28	9.0
8	11.11	4.69	0.43	1.28	1.49	1.28	
9	13.96	7.15					
10	16.62	9.88	0.43	1.07	0.85	0.85	
11	18.85	12.47					10.2
12	23.74	19.11	0.43	0.64	0.85	0.85	
13	28.54	26.86					11.7
14	32.98	35.10	1.07	1.07	1.07	1.07	12.8
15	36.80	42.99					14.0
16	40.36	51.00	0.43	0.43	0.21	0.21	
17	43.65	58.96					17.1
18	46.85	67.20	0.85	0.43	1.28	1.71	18.8
19	54.46	85.70	2.56	1.71	4.91	8.11	19.0
20	57.26	89.26	3.20	2.56	5.97	9.81	17.0
21	60.34	93.59	2.56	1.92	6.61	10.46	12.6
22	63.71	97.72	2.77	2.35	6.83	10.03	9.0
23	67.35	101.63	1.92	2.13	5.97	10.67	6.8
24	71.24	105.27	4.23	4.48	13.23	21.55	5.7
25	74.08	107.41	8.53	8.32	21.76	33.07	
26	77.57	109.06	11.31	10.03	25.39	40.11	5.4
27	80.71	109.84	11.10	8.96	24.96	41.39	
Invert	83.39	110.04	-	-	-	-	6.0
28	84.26	110.02	10.88	7.25	22.40	39.90	
29	87.77	109.49	11.52	7.47	23.90	39.26	7.6
30	91.11	108.28	12.16	8.32	23.47	33.50	

[1] (X-Y)–Coordinates of spillway from crest axis
All dimensions are in metres.
[2] Gates open 3.6 metres; measured from seat to bottom edge of gates
All Q (discharge) in m^3/s

A study was made with the left bucket flip angle at 45°, the center bucket flip angle at 37.5° (fig. 14a), and the right bucket flip angle at 30°. Jet impact was unsatisfactory. The jets did not intersect and the combined impact was spread laterally far up on each side of the river.

The right bucket was molded into the shape of the Aldeadavila[2] flip bucket, with the left and center buckets remaining at 45° and 37.5° (fig. 14a). The impact area with this design was elongated in the direction of flow, but was also spread laterally, and the jet contained heavy splash and spray (fig. 14b). The Aldeadavila bucket did not produce satisfactory flow for the Amaluza installation.

<u>Results of Spillway "A" Studies</u>

The results of the study of spillway "A" indicated that:

1. The intermediate piers were satisfactory.

2. With slight additional modification, the end piers shown in figure 10a were satisfactory.

3. The gate trunnion location was satisfactory with the modified end piers.

4. The pressures on the spillway face were above atmospheric for all flow combinations. A steeper spillway could be tolerated.

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[2] Aldeadavila Dam is a gravity-arch dam built by Spain on the Rio Duero. The dam is on the Spanish-Portuguese international boundary about 275 km east of Madrid.

5. The jet flow into the downstream river channel was not acceptable. Further studies were required to develop a spillway and bucket shape that would produce jets near the center of the river and spread longitudinally but not laterally.

LOMBARDI SPILLWAY

Model Spillway

A model of a new spillway design, referred to as the "Lombardi spillway," was fabricated to replace the design "A" spillway model. Figures 15 through 19 and tables 2 and 3 are model shop drawings with dimensions in inches and are included in this report to aid INECEL in the fabrication of a spillway model in Quito, Ecuador. Figure 3 is the shop drawing for a model gate and figure 4 shows one of the six fabricated gates. The USBR sheet metal spillway section of the Lombardi spillway, fabricated to these drawings and shown in figure 20, will be sent to INECEL for their use.

Side Piers

The model side pier on the right side of the spillway was made excessively large to assure smooth flow during the model studies (fig. 21). Side pier studies and refinements were made to the left side pier. Studies on spillway "A" indicated that similar piers on either side of the spillway would operate the same for the same flow. Figure 22 shows the initial design pier for the Lombardi crest and the water surface traces for 4500 and 7700 m^3/s.

This pier was unacceptable because it allowed flow impingement on the gate trunnion at a discharge of 7700 m^3/s. The studies for pier refinements were

Table 2.—*Lombardi spillway skinplate model dimensions*

(X, Y)—Coordinates of crest templates, origin at crest centerline
S—Skinplate stretch out from (0, 0)
W—Skinplate width at (X, Y)
Dimensions in inches—model scale 1:70

X	Y	S	W	X	Y	S	W
0	0	0	8.698	13.048	11	17.729	7.950
0.978	0.075	0.981	8.642	13.652	12	18.897	7.916
1.275	0.125	1.282	8.625	14.231	13	20.053	7.883
1.827	0.250	1.848	8.593	14.790	14	21.199	7.851
2.256	0.375	2.295	8.568	15.330	15	22.335	7.820
2.619	0.500	2.679	8.548	15.852	16	23.463	7.790
2.941	0.625	3.024	8.529	16.359	17	24.584	7.761
3.233	0.750	3.342	8.513	16.852	18	25.699	7.732
3.503	0.875	3.640	8.497	17.332	19	26.808	7.705
3.755	1.000	3.921	8.483	17.801	20	27.913	7.678
4.216	1.25	4.445	8.456	18.257	21	29.012	7.652
4.635	1.50	4.933	8.432	18.704	22	30.107	7.626
5.021	1.75	5.393	8.410	19.141	23	31.198	7.601
5.382	2.00	5.832	8.389	19.569	24	32.286	7.577
5.722	2.25	6.254	8.370	19.988	25	33.370	7.553
6.044	2.50	6.662	8.352	20.047	25.141	33.522	7.550
6.350	2.75	7.057	8.334				
6.644	3.00	7.443	8.307		Upstream from crest		
7.198	3.50	8.189	8.285	0	0	0	8.698
7.715	4.00	8.908	8.256	1.333	0.242	1.362	8.774
8.202	4.5	9.606	8.228	2.104	0.877	2.382	8.818
8.663	5.0	10.286	8.202	2.148	1.035	2.548	8.821
9.103	5.5	10.952	8.176				
9.524	6.0	11.606	8.152				
9.928	6.5	12.249	8.129				
10.318	7.0	12.883	8.107				
10.694	7.5	13.509	8.085				
11.059	8.0	14.128	8.064				
11.757	9.0	15.348	8.024				
12.418	10.0	16.547	7.986				

Conversion: 25.4 x in = mm

Table 3.—*Lombardi bucket skinplate model dimensions*

(X, Y)—Coordinates of bucket templates, origin at bucket invert
S—Skinplate stretch out from (21.389, 25.287)
W—Skinplate width at (X, Y)
Dimensions in inches model scale 1:70

X	Y	S	W	X	Y	S	W
21.389	25.287	0	14.429	7	2.584	27.150	12.782
21	24.355	1.010	14.385	6	1.886	28.370	12.667
20	22.046	3.526	14.270	5	1.300	29.529	12.553
19	19.854	5.935	14.156	4	0.824	30.637	12.438
18	17.778	8.239	14.041	3	0.458	31.702	12.324
17	15.820	10.437	13.927	2	0.200	32.735	12.209
16	13.978	12.533	13.812	1	0.048	33.746	12.095
15	12.252	14.528	13.698	0	0	34.747	11.980
14	10.642	16.423	13.583	-1	0.048	35.748	11.866
13	9.147	18.222	13.469	-2	0.200	36.759	11.751
12	7.768	19.925	13.354	-3	0.458	37.792	11.637
11	6.503	21.538	13.240	-4	0.824	38.857	11.522
10	5.353	23.062	13.125	-5	1.300	39.965	11.408
9	4.317	24.502	13.011	-5.191	1.403	40.182	11.386
8	3.394	25.863	12.896				

Conversion: 25.4 x in = mm

continued and the pier shown in figures 23 and 24 was developed. The pier is simple in design and contains the least amount of material of any hydraulically acceptable pier tested. The details shown in figure 24 are recommended for both side piers for the prototype construction.

Intermediate Piers

Flow past the intermediate piers was satisfactory for all free-flow discharges and for all flows with the six gates opened uniformly. Some drawdown on the side of an intermediate pier (and trunnion impact for large flows) may be expected if the gate in one bay is fully open and the gate on the opposite side of the pier is closed.

Calibration

The discharge capacity curve developed for crest "A" was checked for both free and controlled flow and found also to be accurate for the Lombardi crest. Figure 11 presents the discharge capacity curves.

Spillway Pressures and Flow Depth

Piezometers were installed on the spillway face along the radial centerline of gate bay 3. Pressures were recorded for various free-flow and gate-controlled discharges. Piezometer locations and typical pressures are shown in table 4. Piezometers 9 and 13 were installed just downstream from inadvertent surface irregularities in the model and were considered to be invalid. Pressures were very near atmospheric for much of the length of the parabolic crest above the transition, indicating that the Lombardi profile is near optimum.

Table 4.—*Lombardi crest piezometer locations and pressures*

Piezometer number	X[1]	Y[1]	[2] Gates 3m Q = 1500	[2] 4m 2445	Gates open—free flow 1500	4550	7700
Upstream nose	-3.82	1.84	-	-	-	-	-
1	-3.41	1.11	7.47	9.39	5.12	6.40	5.76
2	-1.85	0.23	6.83	8.32	4.05	5.55	4.69
3	-0.09	0.01	5.97	7.89	3.20	4.05	3.20
Crest	-0-	-0-	-	-	-	-	-
4	1.69	0.05	5.12	6.19	2.56	3.20	2.35
5	3.11	0.41	2.99	3.20	1.92	2.13	1.28
6	5.03	1.03	1.71	1.92	1.92	2.35	1.92
7	6.75	1.81	0.64	0.64	1.28	1.49	0.85
8	8.30	2.70	0.85	0.85	1.28	1.49	1.07
9	9.77	3.70			NG		
10	11.12	4.75	0.85	0.85	1.28	1.49	1.07
11	13.72	7.11	0.43	0.21	0.64	0.43	-0-
12	16.05	9.62	0.85	0.85	0.85	0.85	0.65
13	18.39	12.50			NG		
14	20.44	15.33	0.43	0.42	0.64	0.64	0.64
15	22.38	18.24	0.85	0.64	0.64	0.85	1.07
16	27.70	27.52	0.43	0.42	0.85	0.85	1.92
17	32.30	36.98	0.85	1.49	1.28	1.28	1.49
18	35.09	43.38	0.85	0.85	0	0	-0.43
End of crest	35.64	44.70	-	-	-	-	-
Start of bucket	45.96	69.59	-	-	-	-	-
19	46.65	71.25	3.20	4.05	2.35	4.27	5.76
20	50.20	79.25	0.64	1.28	0.85	2.77	4.91
21	53.76	86.53	1.71	2.35	0.43	2.13	3.84
22	57.32	92.77	1.92	2.77	0.43	2.56	4.69
23	60.87	98.29	3.41	4.69	2.99	7.68	10.88
24	64.43	102.99	3.20	4.69	2.35	7.47	12.37
25	67.98	106.88	4.69	6.83	1.92	8.32	14.29
26	71.54	109.96	6.19	8.53	5.76	15.15	23.47
27	75.10	112.24	9.81	13.23	6.40	19.42	31.15
28	78.65	113.74	8.58	12.59	8.32	22.40	34.99
29	82.21	114.47	10.03	16.43	7.68	21.76	36.06
Invert	83.98	114.55	-	-	-	-	-
30	85.76	114.47	8.96	13.87	6.83	21.55	35.63
31	89.32	113.74	5.33	11.52	2.77	14.51	28.16
32	92.86	112.24	0.21	0.64	2.35	11.52	13.44
33	93.10	112.12	-	-	-	-	-
34	93.10	112.12	-	-	-	-	-
Flip lip	93.22	112.06	-	-	-	-	-

[1] (X-Y)—coordinates of spillway from crest axis
All dimensions are in metres.
[2] Gates open 3 and 4 metres respectively; measured from seat to bottom edge of gates.
All Q (discharge) in m^3/s

Caution must be exercised during prototype construction to assure that inadvertent surface irregularities or deviations from the theoretical alinements do not occur. Any surface blemish in this area of high velocity and near atmospheric pressures would be conducive to producing cavitation pressures and subsequent spillway surface damage.

The water surface on the left training wall was measured for the maximum discharge (fig. 25). The results of the survey and the pressure profile for maximum discharge are tabulated and graphed in figure 26. These values may be used in the structural design of the training walls. The spillway operated satisfactorily for all flows, free or gate controlled. Dimensions for the recommended Lombardi spillway, as tested in the model, are shown in figures 27 and 28.

Flip Bucket Studies

The spillway was acceptable from the reservoir down to the start of the flip buckets. The remaining model studies were concerned with the action of the jets issuing from the flip buckets, the jet impact area in the river channel, and the scour downstream. With the flip lip of all three bays at 30° the jets from the flip buckets converged 270 m from the crest as expected (fig. 29). To continue the study of the jets from the flip buckets, it was necessary to install the river valley contours downstream from the dam. The model concrete contours were made to the estimated bedrock in the riverbed up to elevation 1840 and to the ground surface contours above that elevation (figs. 30 and 31). An overall view of the completed model installation including the downstream contours is shown in figure 32.

A study was made to produce a jet impingement as narrow as possible in the riverbed and elongated in the direction of flow. The simplest construction which resulted in acceptable jet impingement consisted of reshaping the flip lip of the left bay to 20°, leaving the center bay at 30°, and reshaping the right bay to 15° (figs. 27 and 33). The impingement areas shown are the same for individual or combined flow (with the same discharge per bay) since the jets do not interfere with adjacent jets (figs. 34 and 35).

Spillway Radial Gate Operation

The most desirable operation of the spillway is with all six radial gates opened equally to achieve any desired discharge. This method of flow control will result in smooth, even flow throughout the spillway, will cover the maximum impact area, and therefore, will have the minimum impact energy per unit surface area in the jet impact zone.

Conditions may exist where operation other than with six equally opened gates might be required. The spillway may be operated with any combination of openings of the radial gates. With unequal radial gate openings, adverse flow conditions will increase in severity as the reservoir elevation and the discharge increase. The following conditions should be recognized when operating other than all six gates equally (repeated as conclusions number 7 through 10).

1. Neither radial gate No. 2 nor No. 5 should be operated alone in their respective bays. The spreading jet on the spillway face will impinge on and perhaps overtop the training wall, causing flows and potential scour along the side of the spillway structure.

2. Drawdown on the side of an intermediate pier in a bay operating with the gate fully opened may be expected if the gate in the bay on the opposite side of the intermediate pier is closed.

3. With one bay operating and an adjacent bay not operating, the side pressure on the dividing or retaining wall may be as high as 275 kPa in the bucket area.

4. With the left bay discharging less than about 600 m^3/s, the jet will wash the left canyon wall near the dam and produce deep riverbed scour against the left canyon wall bedrock. Onsite examination should be made to determine whether such washing would be hazardous to the dam.

Scour Tests

River scour tests were made to compare qualitative indications of the scour patterns for various discharges. These scour tests may not be used to determine prototype quantities. Erodible material in the prototype includes the canyon wall overburden and erodible materials in the riverbed which are unknown and could not be modeled.

For scour tests with flows of 1500 m^3/s and greater, the river channel was first filled to about elevation 1840 with graded gravel from 6- to 40-mm diameter (model), figure 36. The channel was then slowly filled with water to about elevation 1855. Next, with all gates fully opened, the reservoir was filled rapidly to the proper elevation to produce the desired discharge. The river water surface downstream (beyond 600 m) was maintained at about elevation 1855 during the test. The scour test at 1500 m^3/s was 15 minutes model time duration. The scour hole did not reach the solid riverbed surface (figs. 37 and 38).

The scour test at 4500 m^3/s was 15 minutes duration, model time. The scour hole reached the solid riverbed, and the eroded material built a dike 450 m downstream that raised the river surface to about elevation 1858. The results of the 4500 m^3/s scour study are shown in figures 39 and 40.

Figures 41a and 41b show the jet conditions during the 7700 m^3/s test. The force of the jet swept the water out of the upstream river channel, formed a hydraulic jump downstream from the area of jet impingement, and reduced the river water surface upstream from the area of the jet impingement to about elevation 1840. The jet removed the erodible material quite rapidly, and the scour test was terminated after 10 minutes, model time. Figures 42, 43a, and 43b show the results of the 7700 m^3/s scour test.

Additional scour studies were made with flows of 200, 500, and 800 m^3/s. The scour material was placed to elevation 1840, the tailwater maintained at elevation 1847.5, and the model test duration for each flow was 15 minutes. Two tests were made for each flow rate. In one test all the flow was discharged through gates 3 and 4 in the center bay only, and in the other test all six gates were fully opened. At a flow rate of 200 m^3/s with all six gates opened, there was no noticeable scour of the riverbed material. The 200 m^3/s passing through the center bay only, caused a scour hole about 7 m deep against the left canyon wall about 175 m downstream from the crest axis (fig. 44). The results of the scour tests for 500 and 800 m^3/s are shown in figures 45 through 48.

Flip Bucket Operation With Very High Tailwater

A massive movement of the Gualpa slide into the river downstream from Amaluza Dam could form a barrier sufficiently large to raise the tailwater above the spillway bucket. A study was made to determine the hydraulic action at the exit of the flip bucket with the tailwater at or above the elevation of the flip lip.

A mildly unsteady flow condition existed when the mean tailwater elevation was slightly above the elevation of the lip of the bucket (elev. 1868.0). The water surface near the bucket first interfered with the underneath surface of the jets from the buckets (figs. 49a and 49f), then momentarily swept down, freeing the jets (fig. 49e), and then rushed back to the bucket area again interfering with the jets. This unsteady flow condition caused large random waves along the riverbanks and pressure fluctuations on the downstream face of the bucket corbel.

As the tailwater elevation increased, the flow became quite rough but stable (fig. 49b) until the tailwater reached an elevation that forced the surface of the jet to roll back into the bucket (figs. 49c and 49g). This flow condition caused a perceptible movement of the model guide walls and training walls.

As the tailwater continued to rise, the surging action in the bucket increased (figs. 49d and 49h) and the pulsation of the model increased. The hydraulic action in, and downstream from, the bucket was quite unstable and there would be a possibility of structural damage to the prototype spillway in the bucket area. The tailwater at which noticeable surging of the structure could be felt was about 2.5 m above the elevation of the lip of the flip bucket.

Figure 1.–Construction of Crest "A," seven templates with one skinplate in place. Photo P801-D-77374

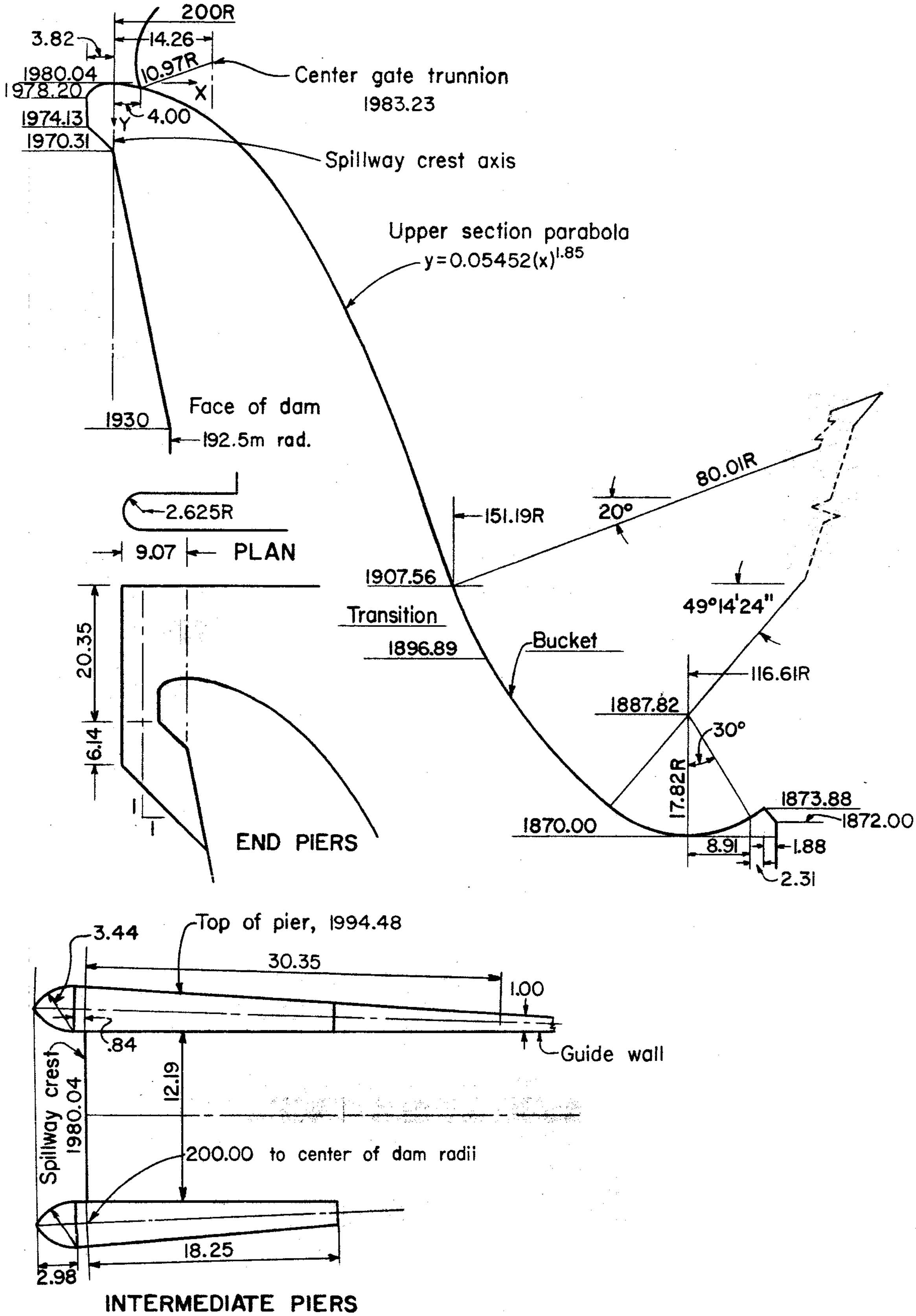


Figure 2.–Spillway "A" prototype dimensions.

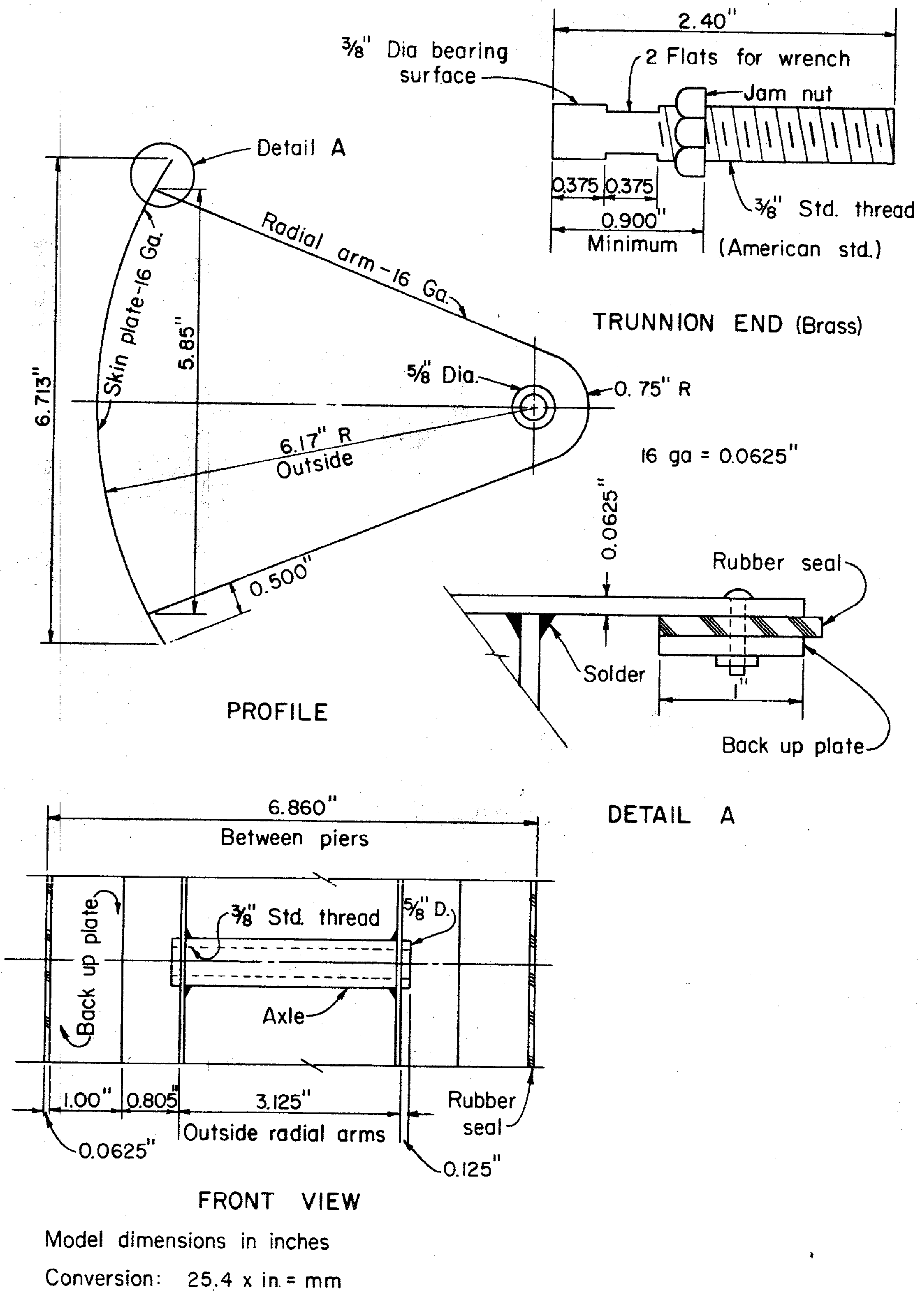
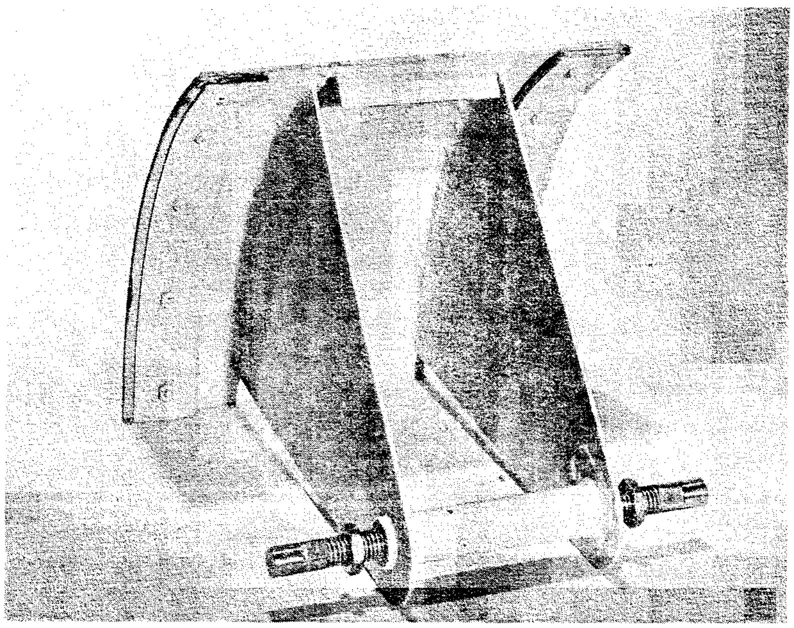
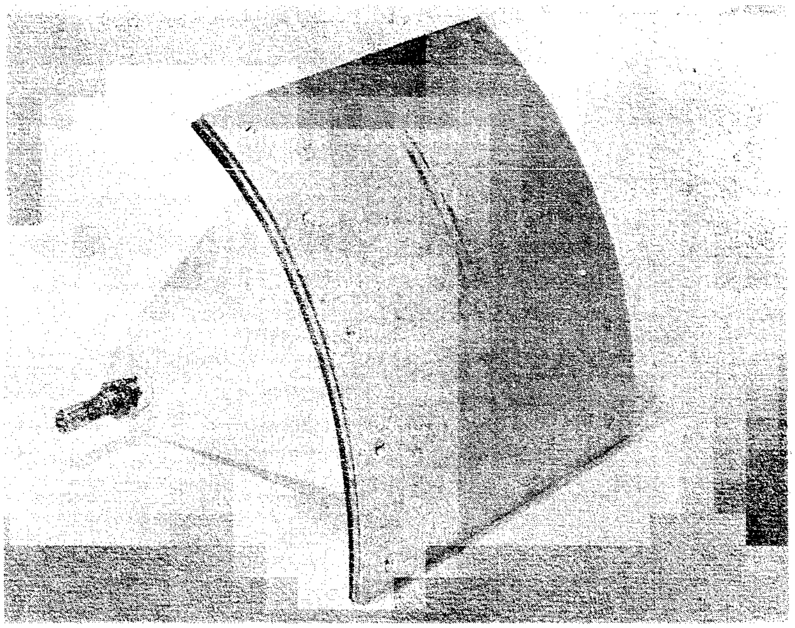


Figure 3.–Spillway gates, model dimensions used for both crests.



a.–Rear view, showing adjustable trunnions. Photo P801-D-77387



b.–Front view, showing seal. Photo P801-D-77388

Figure 4.–Model gate used for both crests.

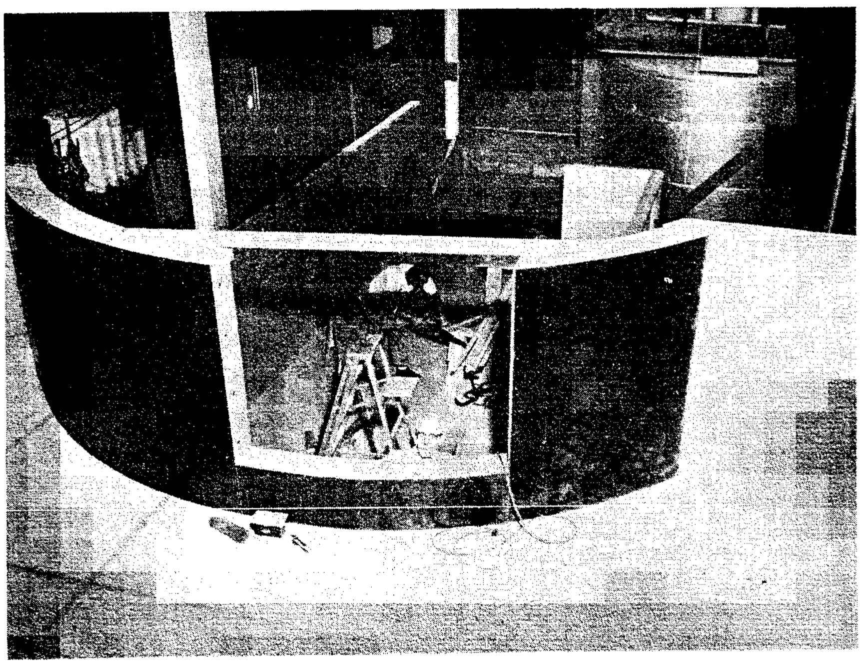
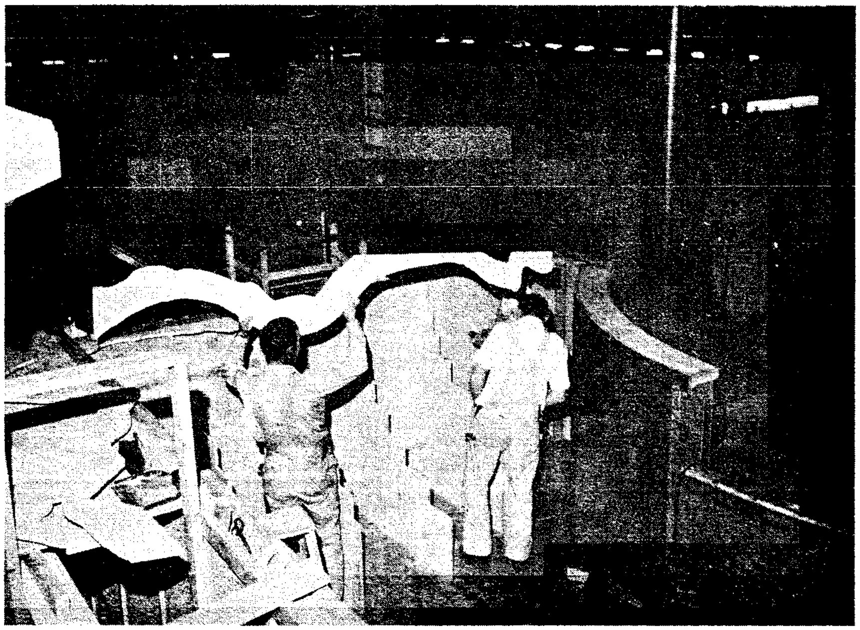
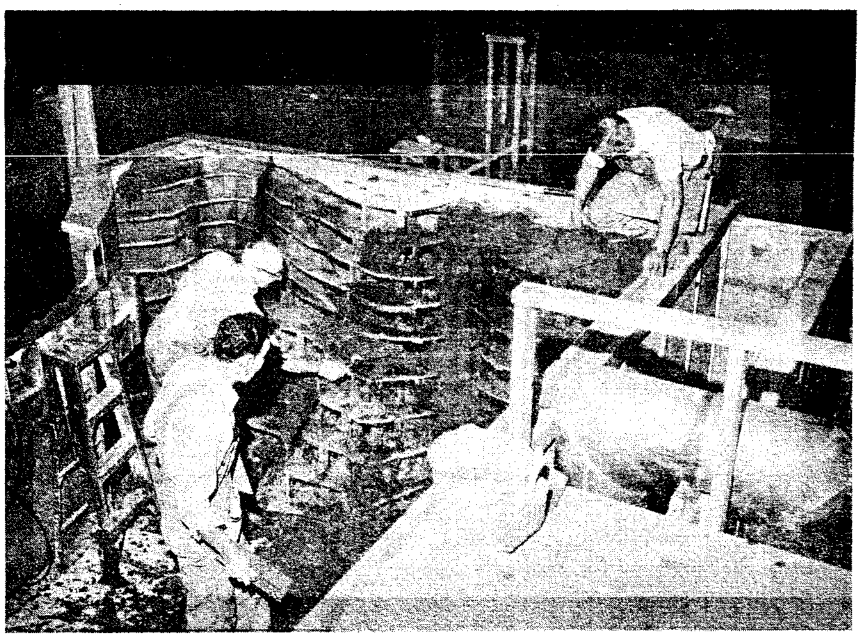


Figure 5.–Model headbox construction, with upstream face of dam. Photo P801-D-77373



a.–Installation of reservoir contours. Photo P801-D-77375



b.–Construction of upstream canyon wall reservoir contours, with concrete on metal lath. Photo P801-D-77376

Figure 6.–Installation of model topography.

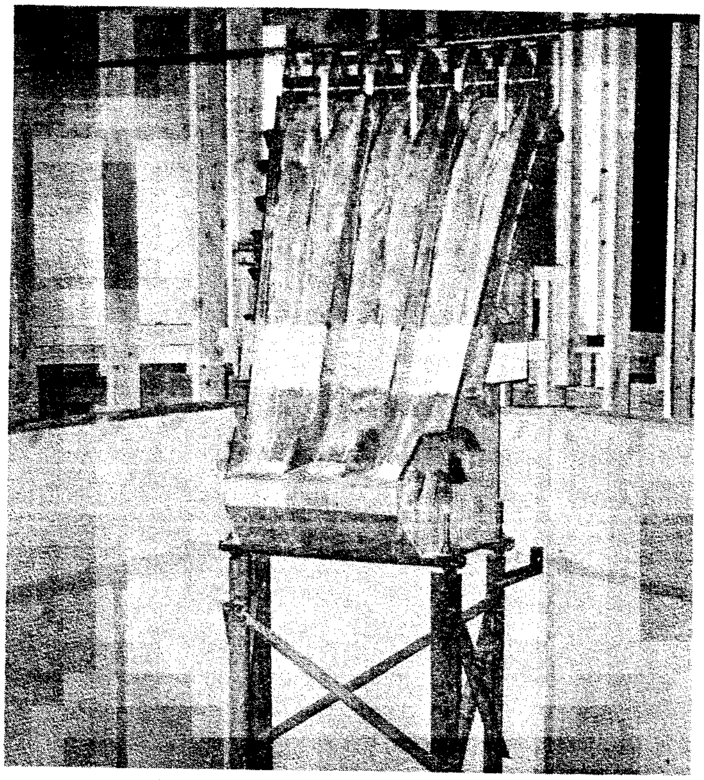


Figure 7.–Crest "A" installed. Photo P801-D-77406

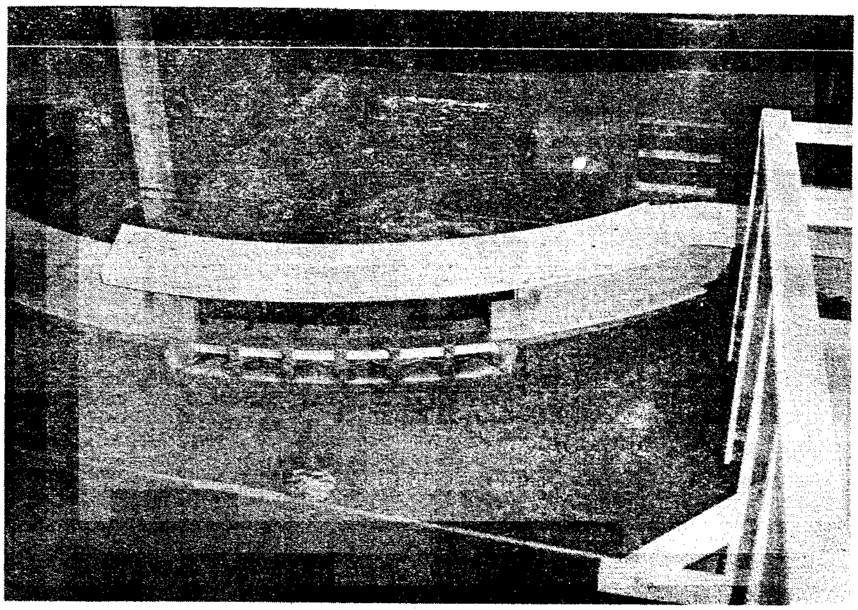


Figure 8.–Crest "A"– Q = 7700 m^3/s with all gates open. Photo P801-D-77378

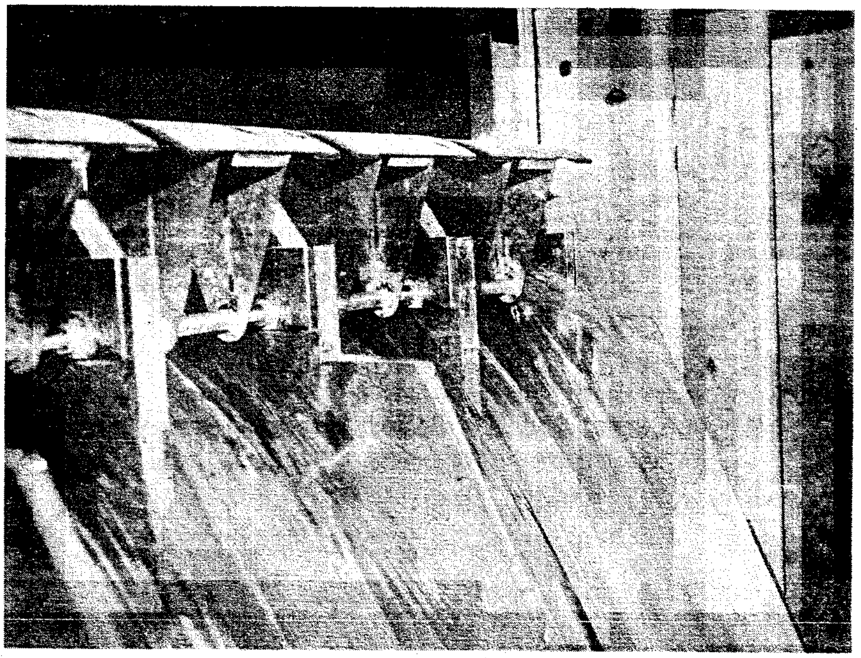


Figure 9.–Crest "A"– Q = 7700 m^3/s, note jet impingement on left bay trunnion (right of center in photograph). Photo P801-D-77377

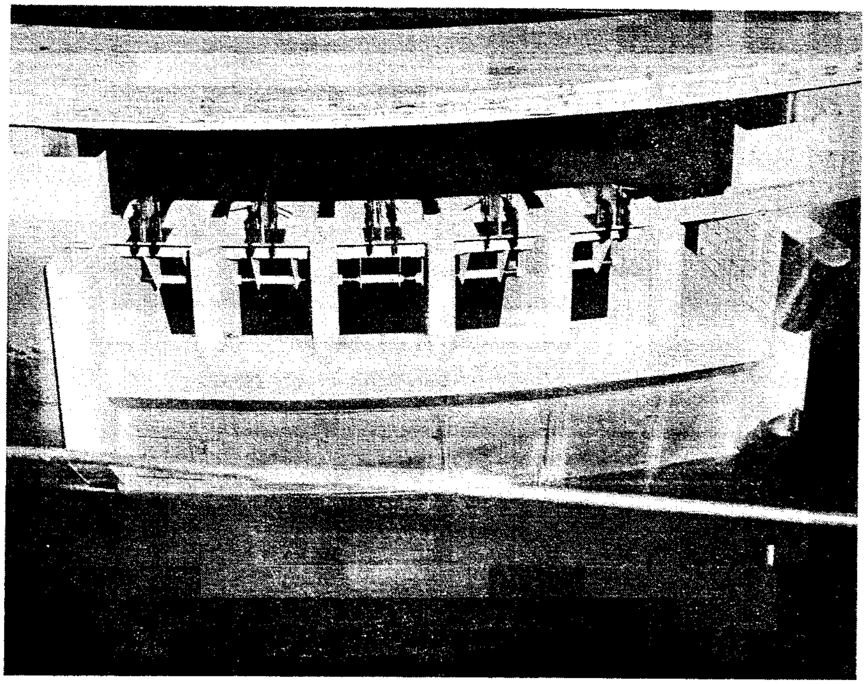


Figure 10a.–Crest "A" looking downstream, modified right pier. Photo P801-D-77383

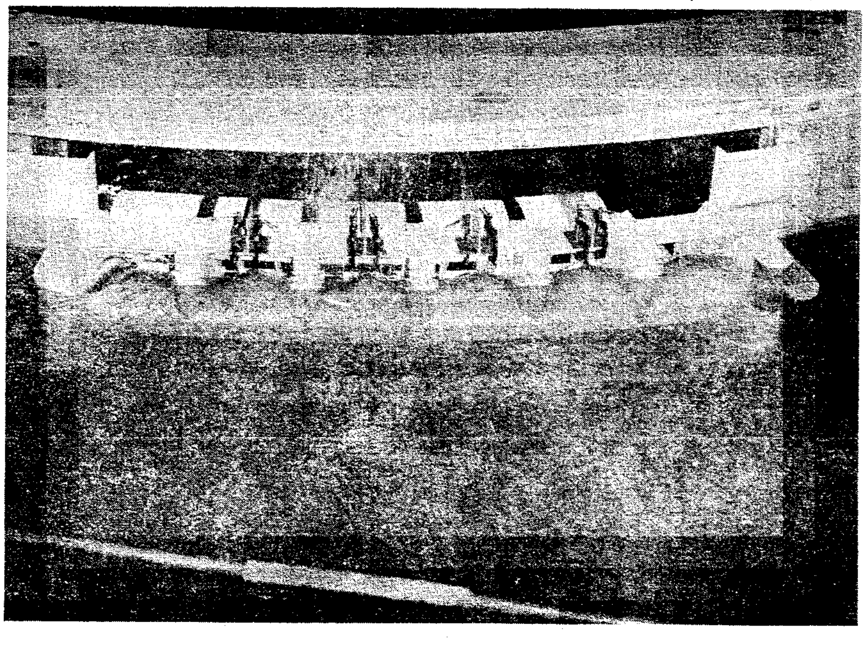


Figure 10b.–Q = 7700 m^3/s, showing flow on modified right pier. Photo P801-D-77384

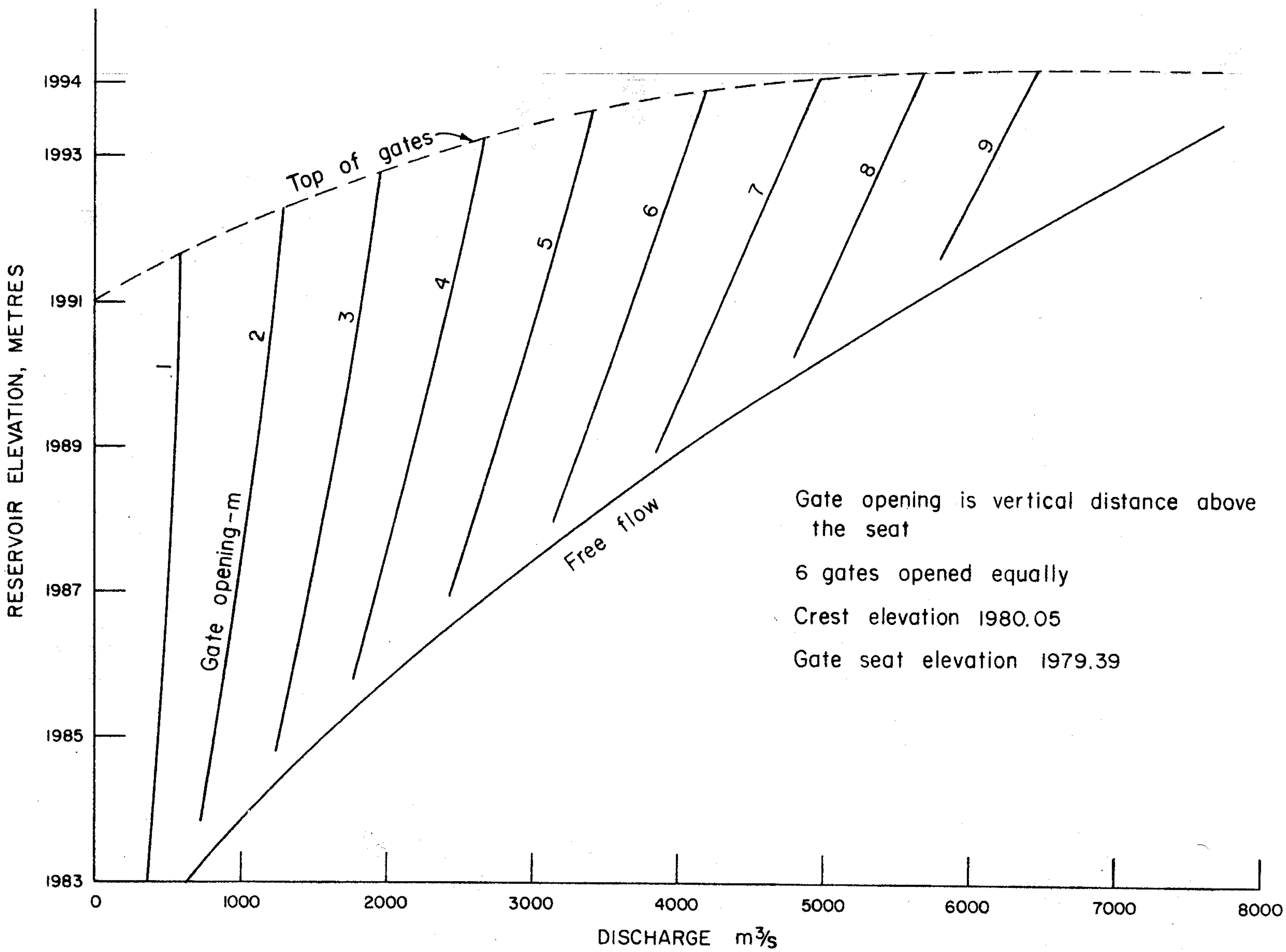
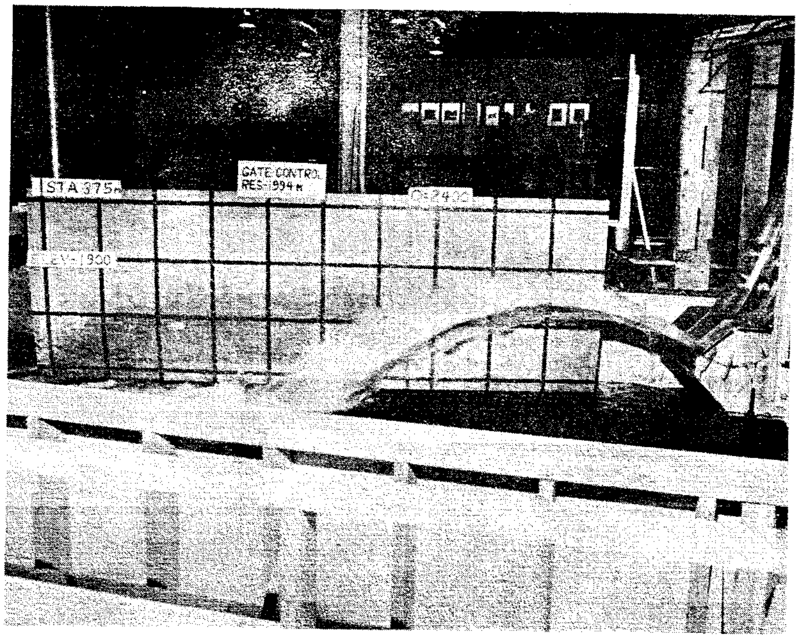
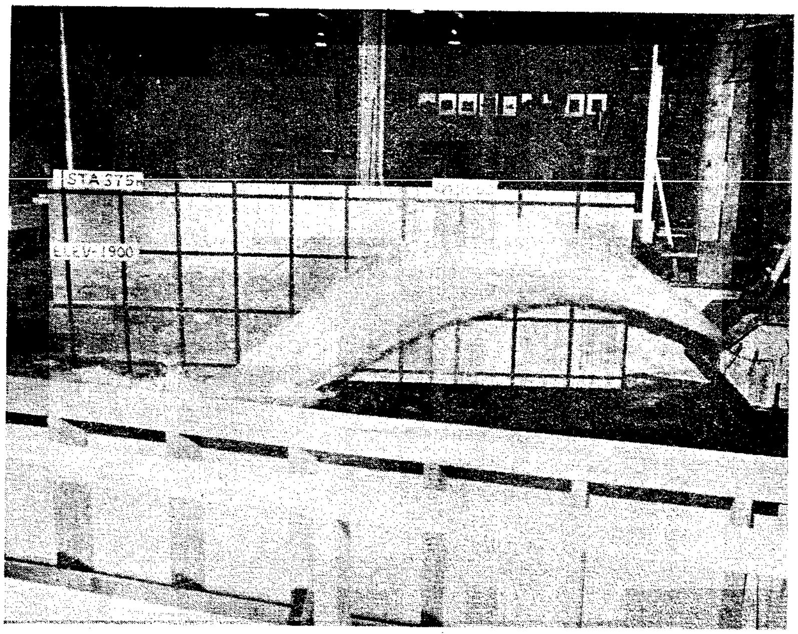


Figure 11.–Spillway discharge curves.

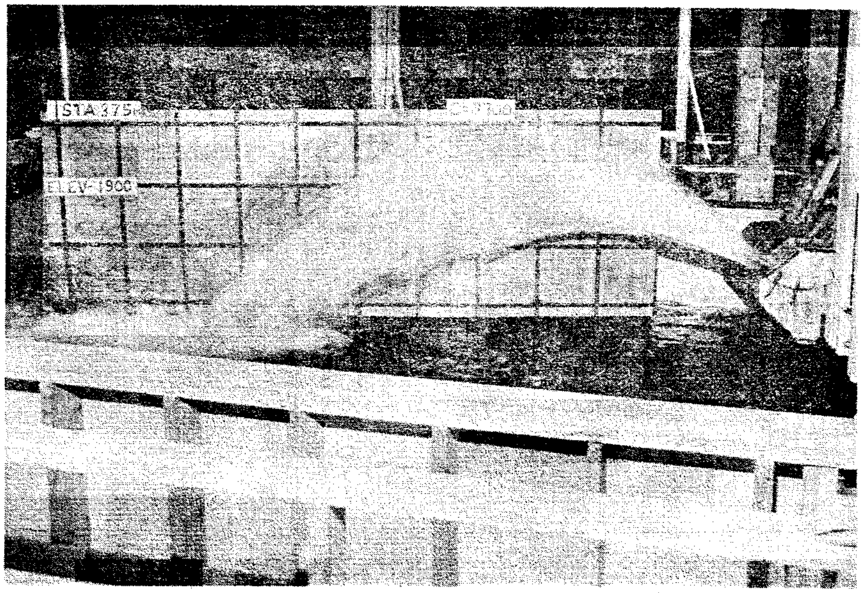


a.–Jet trajectory at Q = 2400 m^3/s (25-metre grid behind jet). Photo P801-D-77379

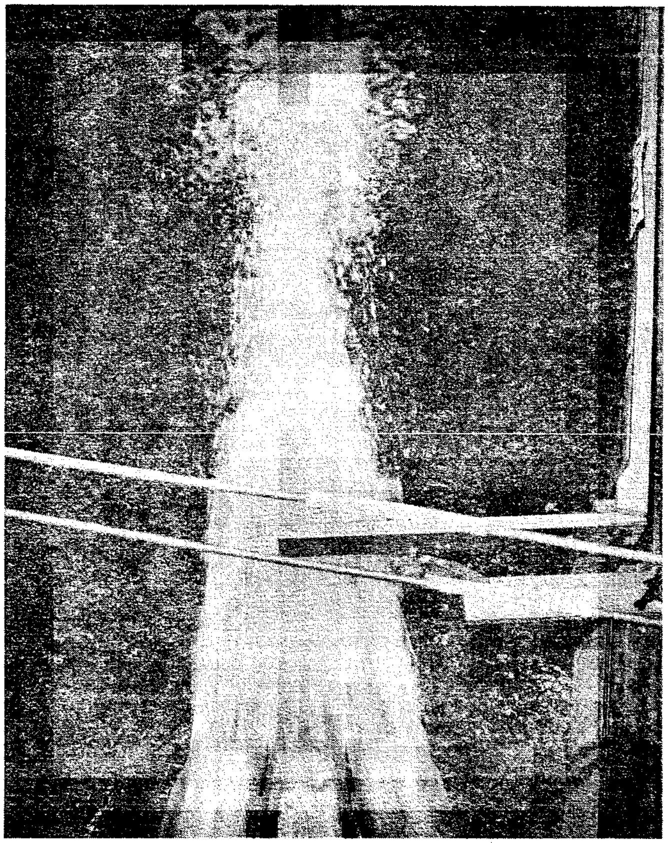


b.–Jet trajectory at Q = 5000 m^3/s. Photo P801-D-77380

Figure 12.–Crest "A," jet trajectories at various discharges as shown.



c.–Jet trajectory at Q = 7700 m^3/s. Photo P801-D-77381



d.–Jet trajectory at Q = 7700 m^3/s. The three jets intersect 200 m from the crest, and impinge in the pool 300 m from the crest. Photo P801-D-77382

Figure 12.–Crest "A" jet trajectories at various discharges as shown.–Continued

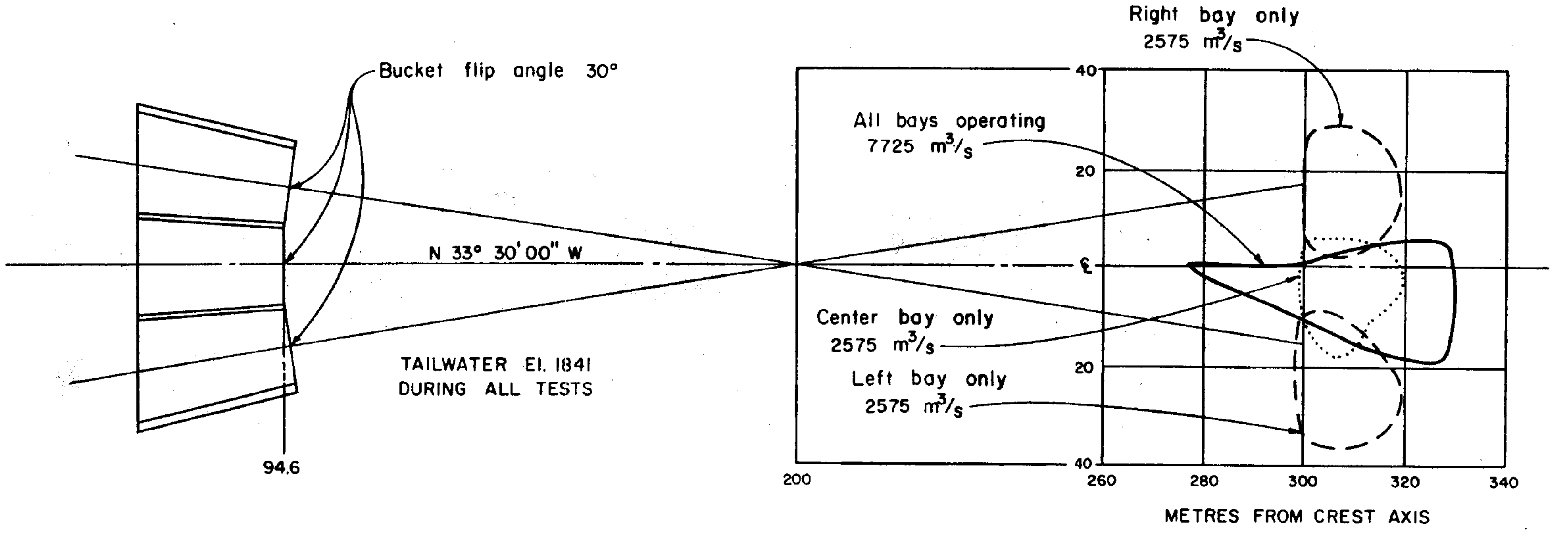
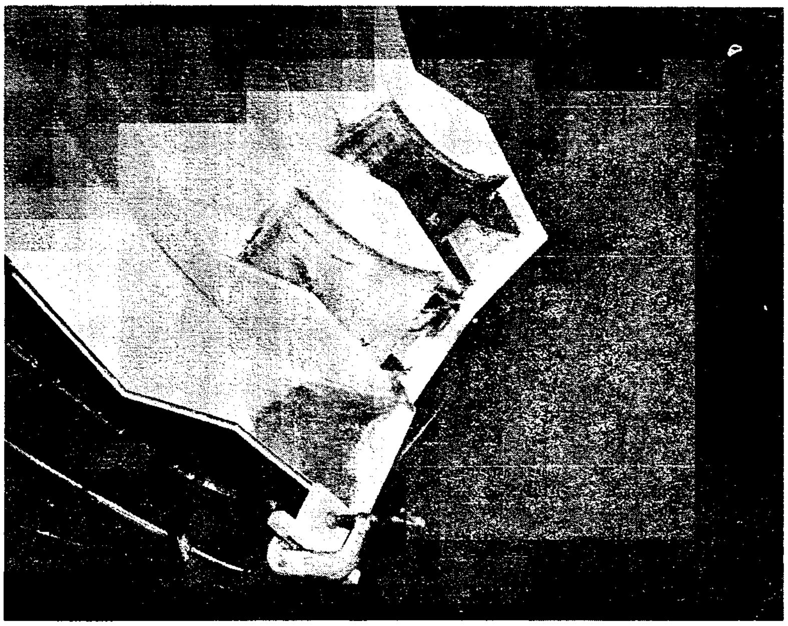
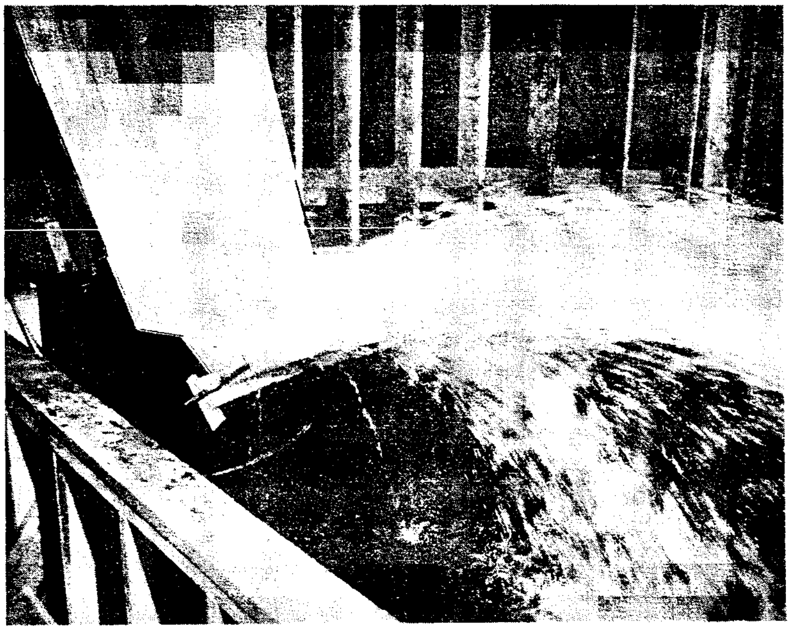


Figure 13.–Spillway "A" downstream jet-impact pattern, Q as shown.

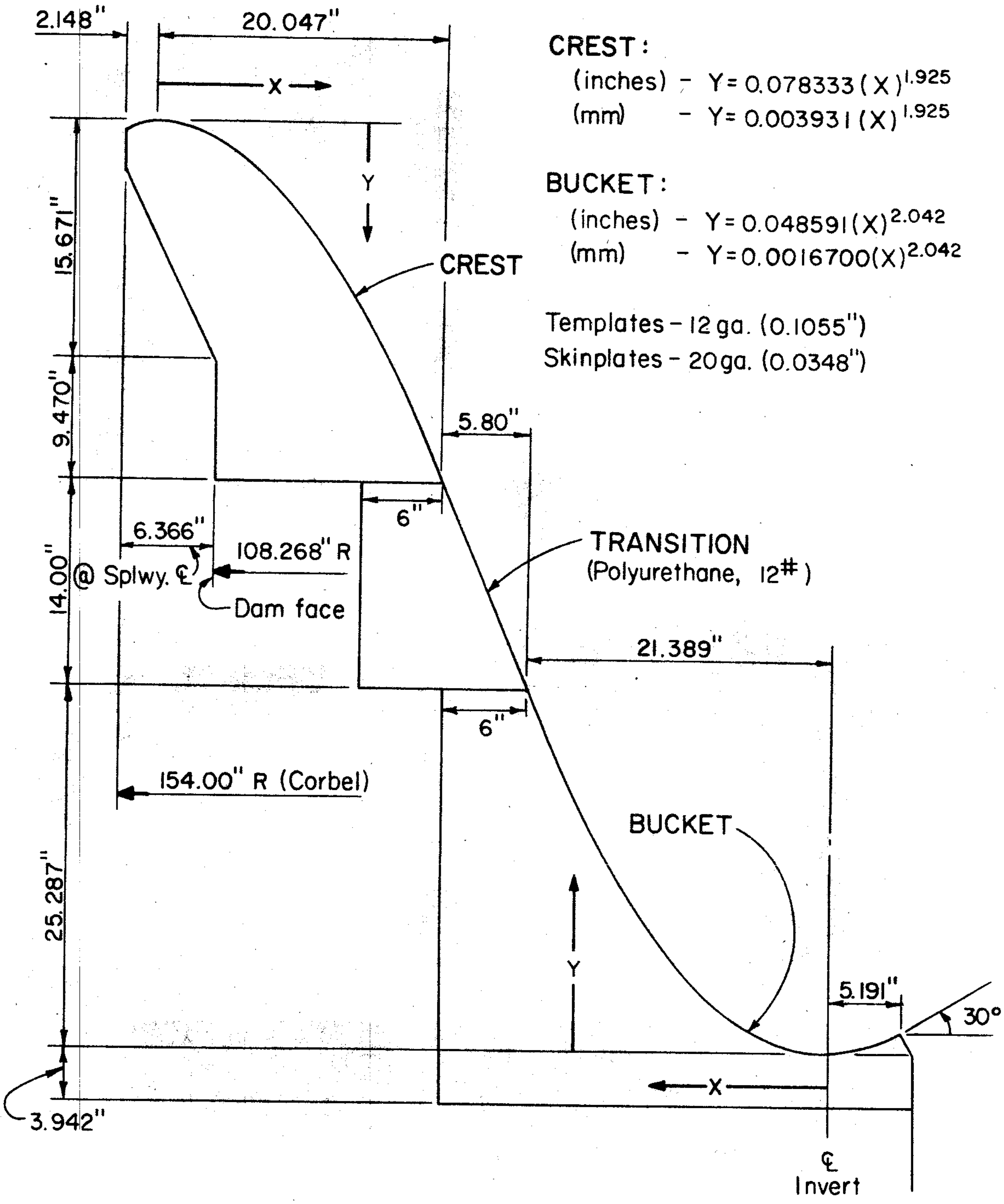


a.–Modified buckets. Photo P801-D-77385



b.–Discharge of 7700 m^3/s over modified buckets. Photo P801-D-77386

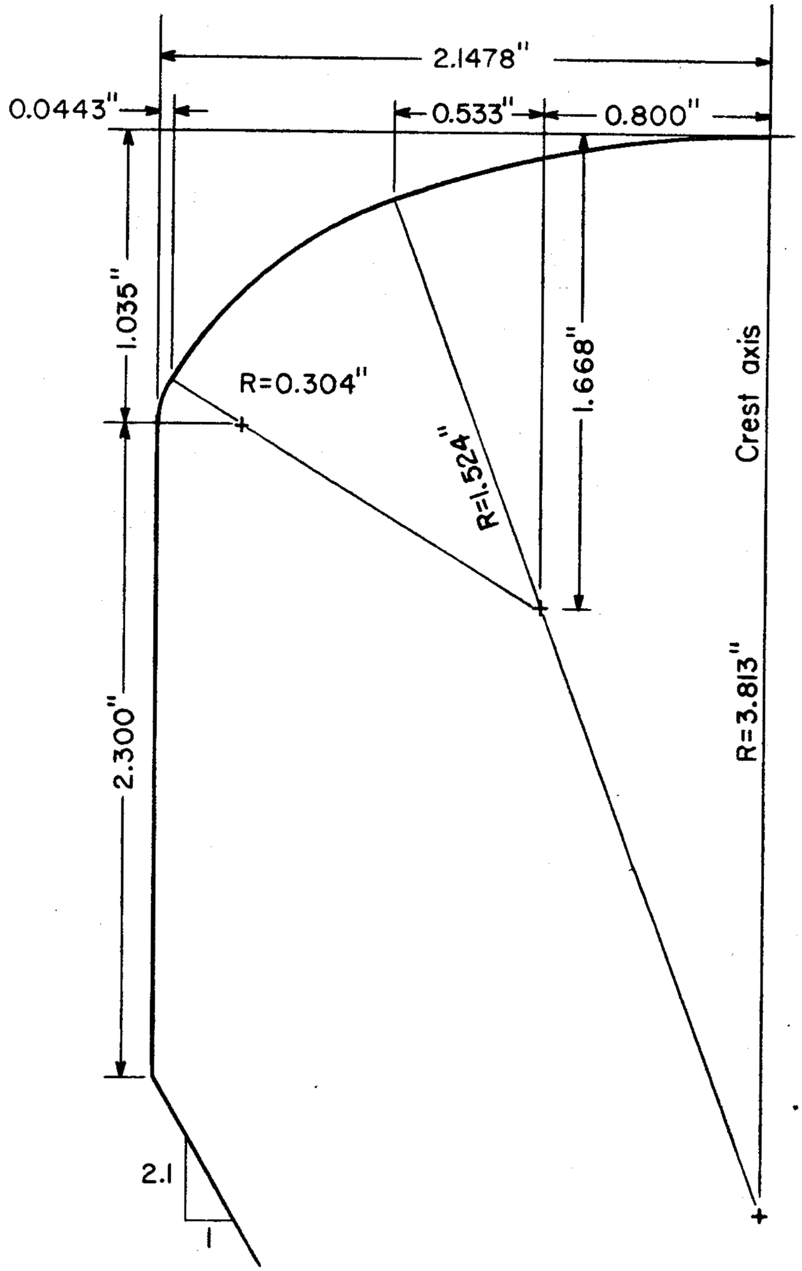
Figure 14.–Crest "A" and modified buckets (left bucket 45°, center bucket 37.5°, right bucket after Aldeadavila).



Model dimensions in inches

Conversion: 25.4 x in = mm

Figure 15.–Lombardi spillway profile, model dimensions.



X	Y	S	W
0	0	0	8.698
-1.333	0.242	1.362	8.774
-2.104	0.877	2.382	8.818
-2.148	1.035	2.548	8.821

Model dimensions in inches
Conversion: 25.4 x in = mm

Figure 16.–Lombardi spillway corbel and crest, model dimensions.

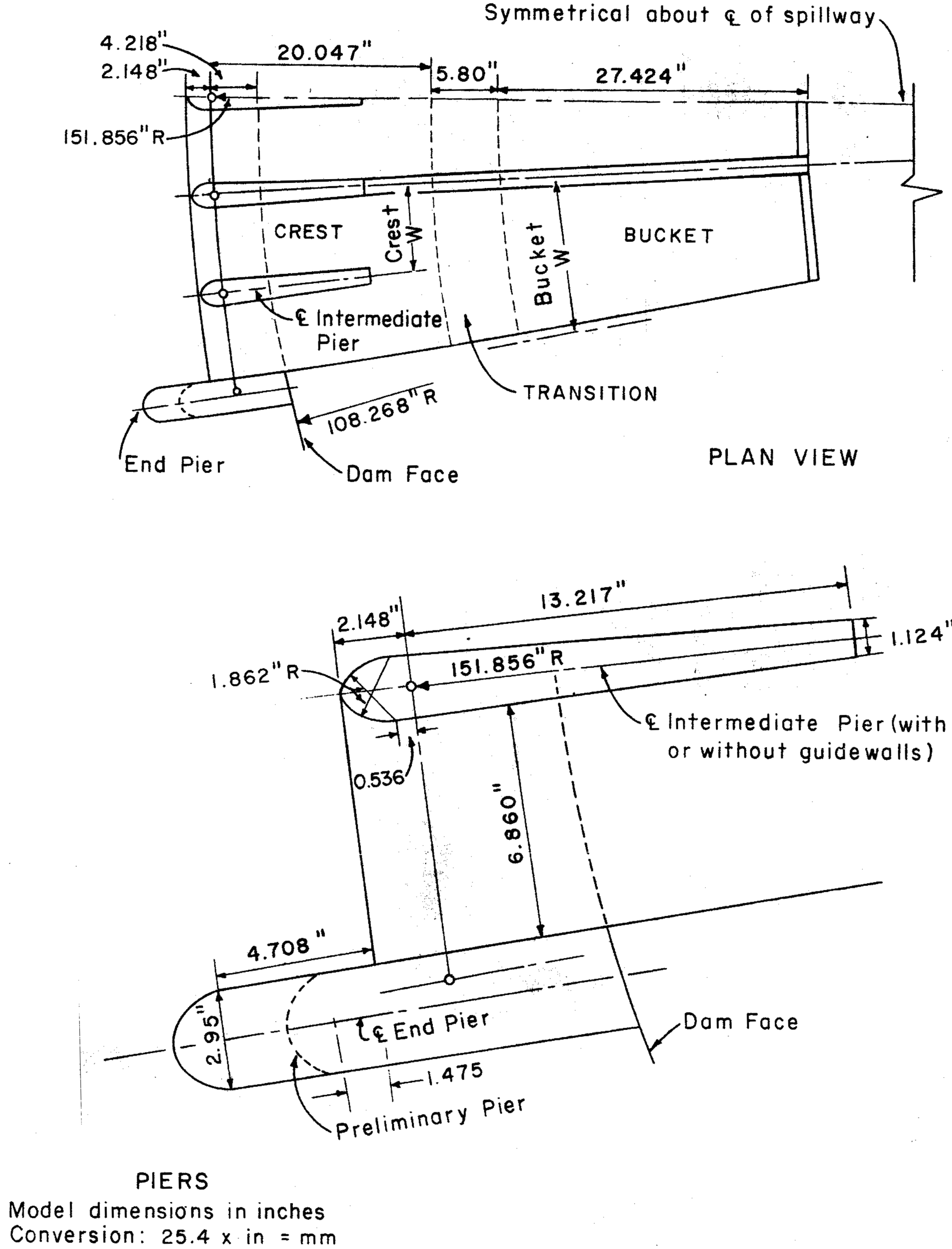


Figure 17.–Lombardi spillway pier, model dimensions.

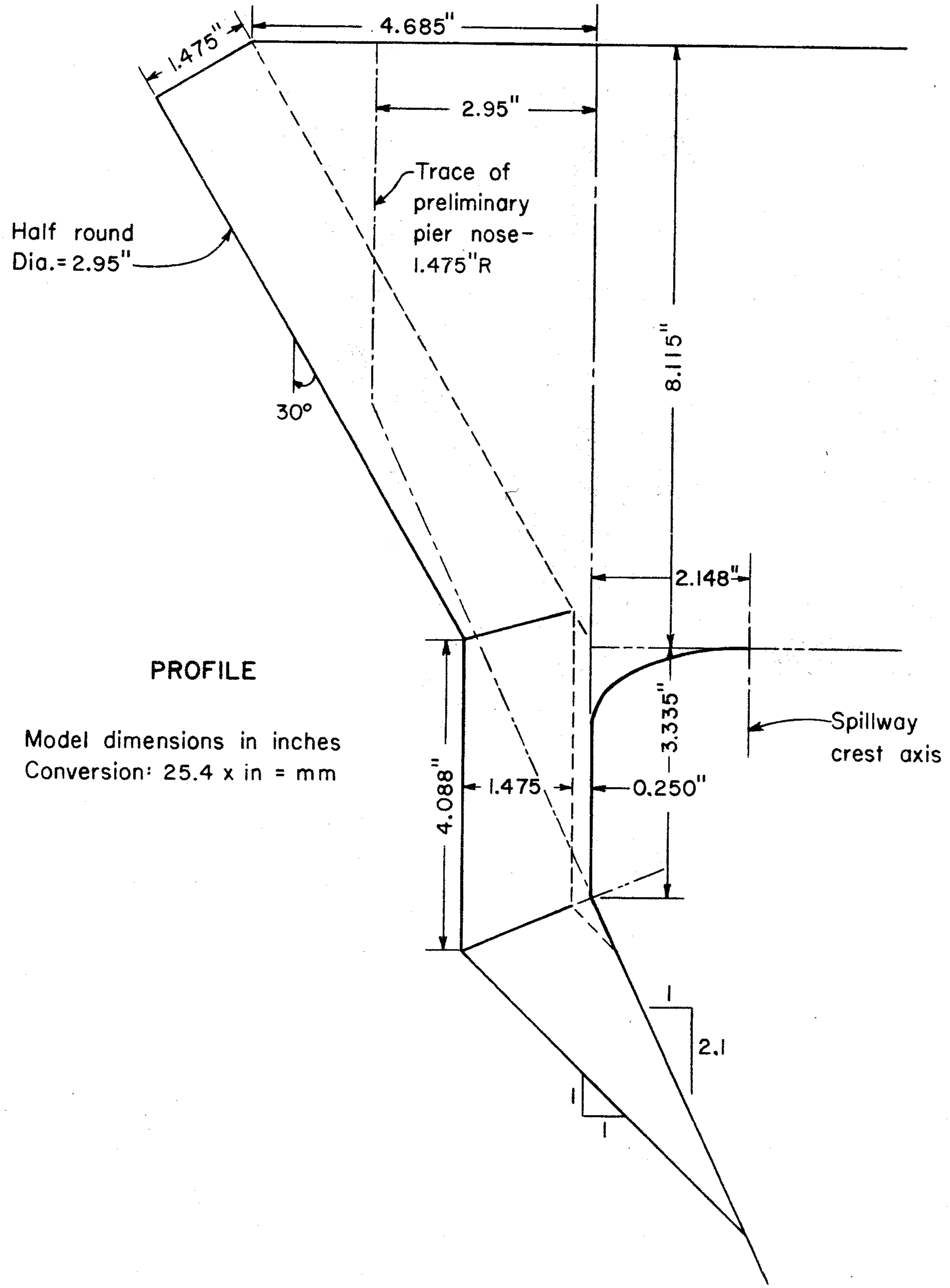


Figure 18.–Lombardi spillway recommended end pier, model dimensions.

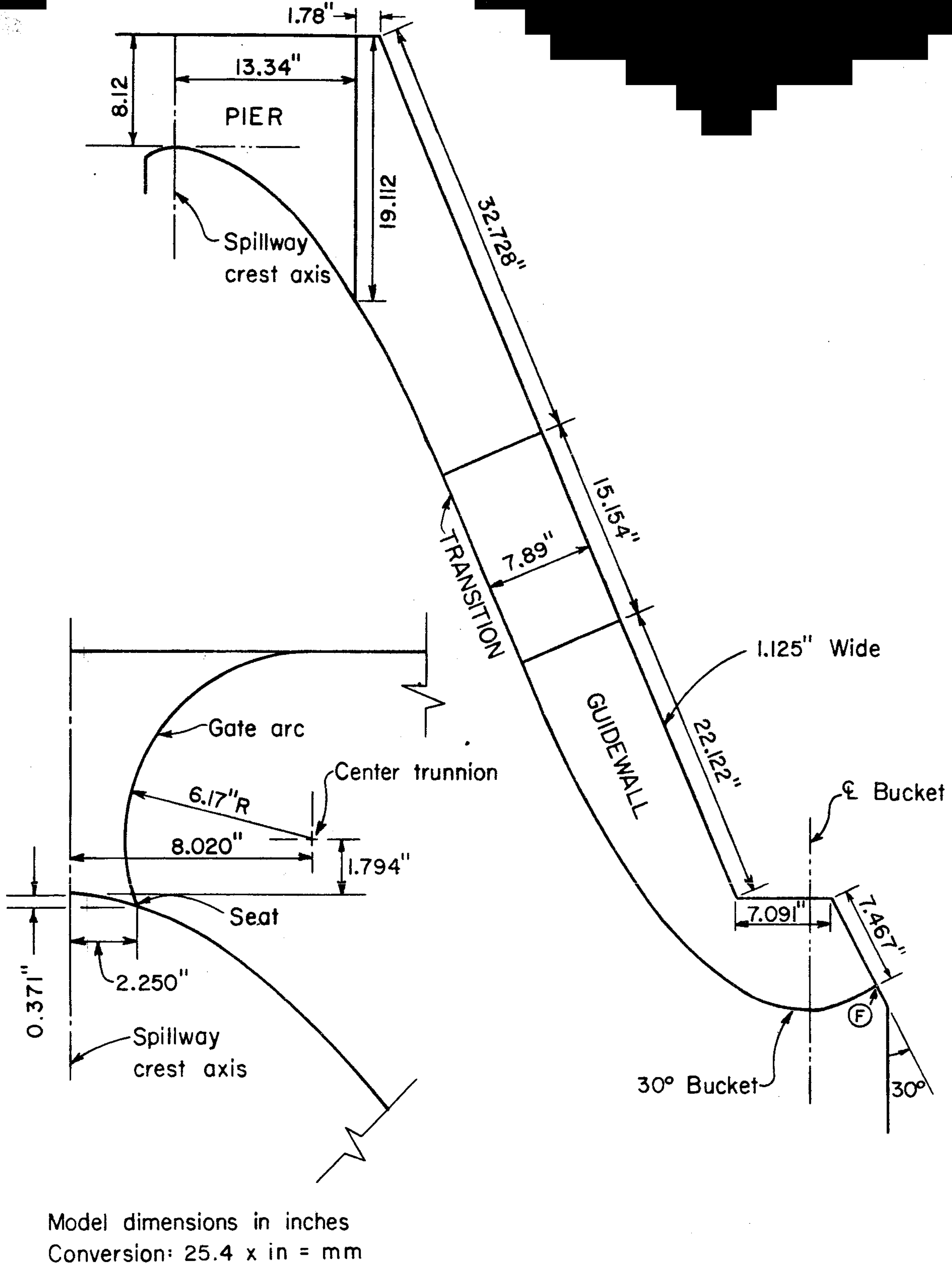


Figure 19.–Lombardi spillway guidewall, model dimensions.

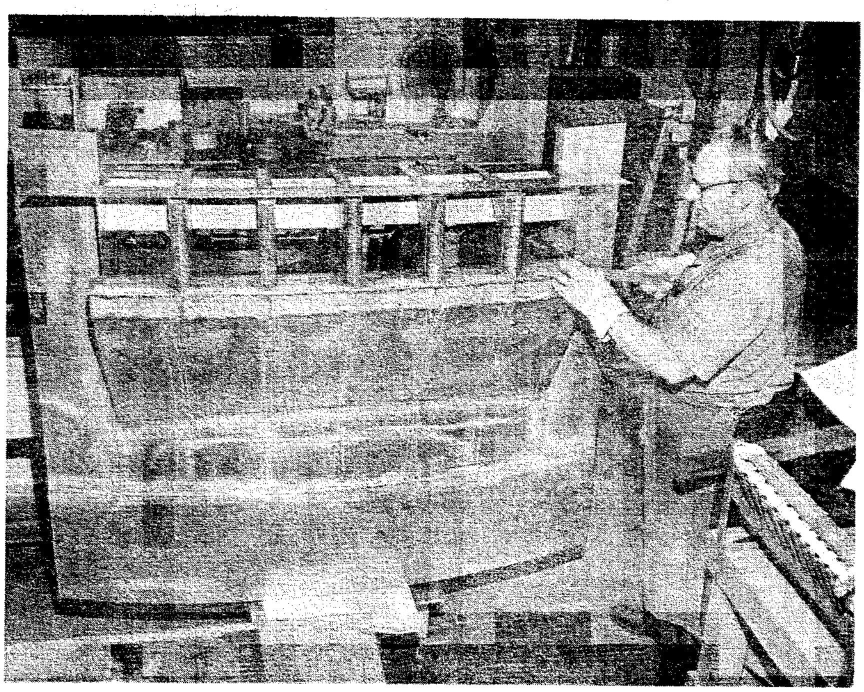


Figure 20.–Upstream face of the finished Lombardi crest. Photo P801-D-77389

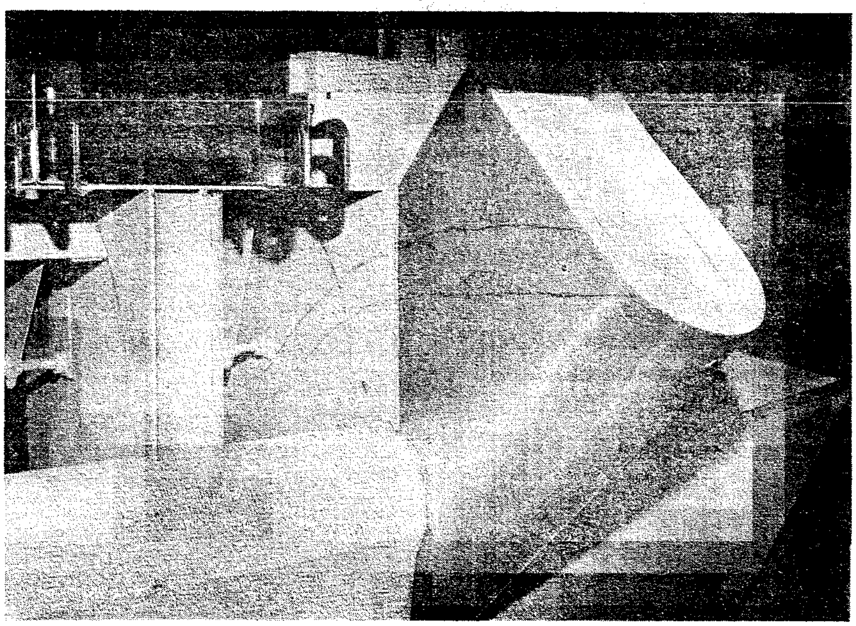


Figure 21.–Extra-large pier to demonstrate flow possibilities. Photo P801-D-77393

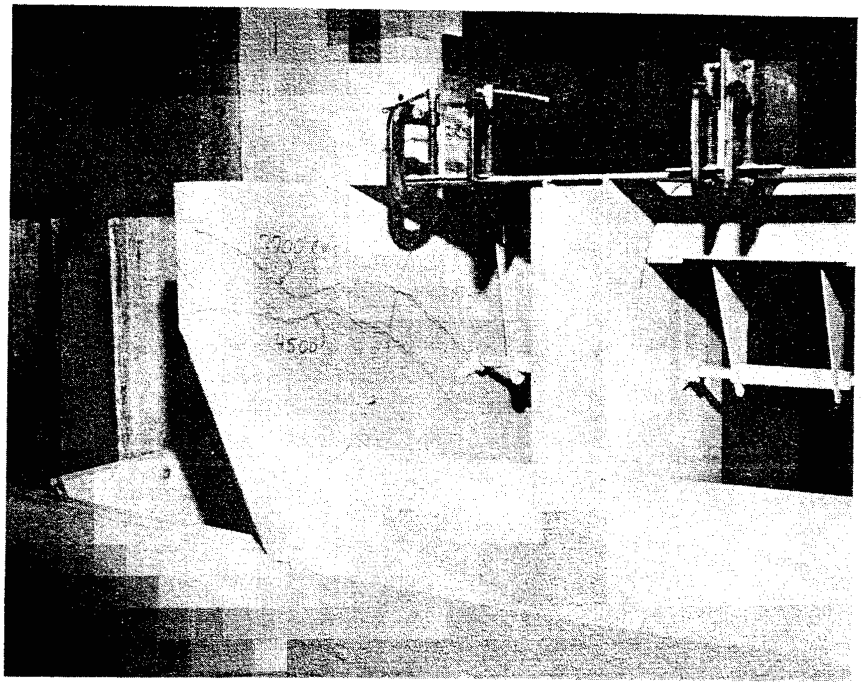


Figure 22.–Side pier study, note excessive drawdown.
Photo P801-D-77392

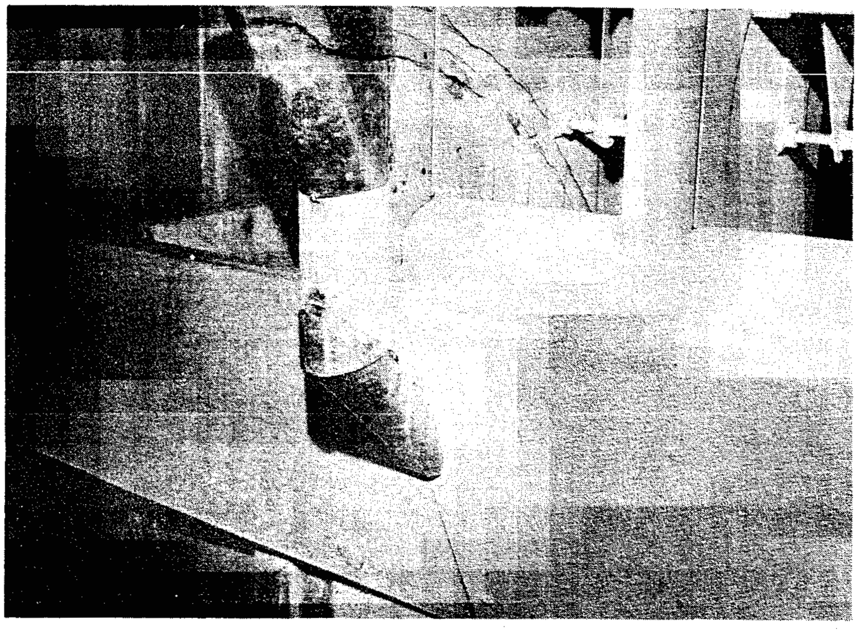


Figure 23.–Recommended side pier, Lombardi spillway.
Photo P801-D-77400

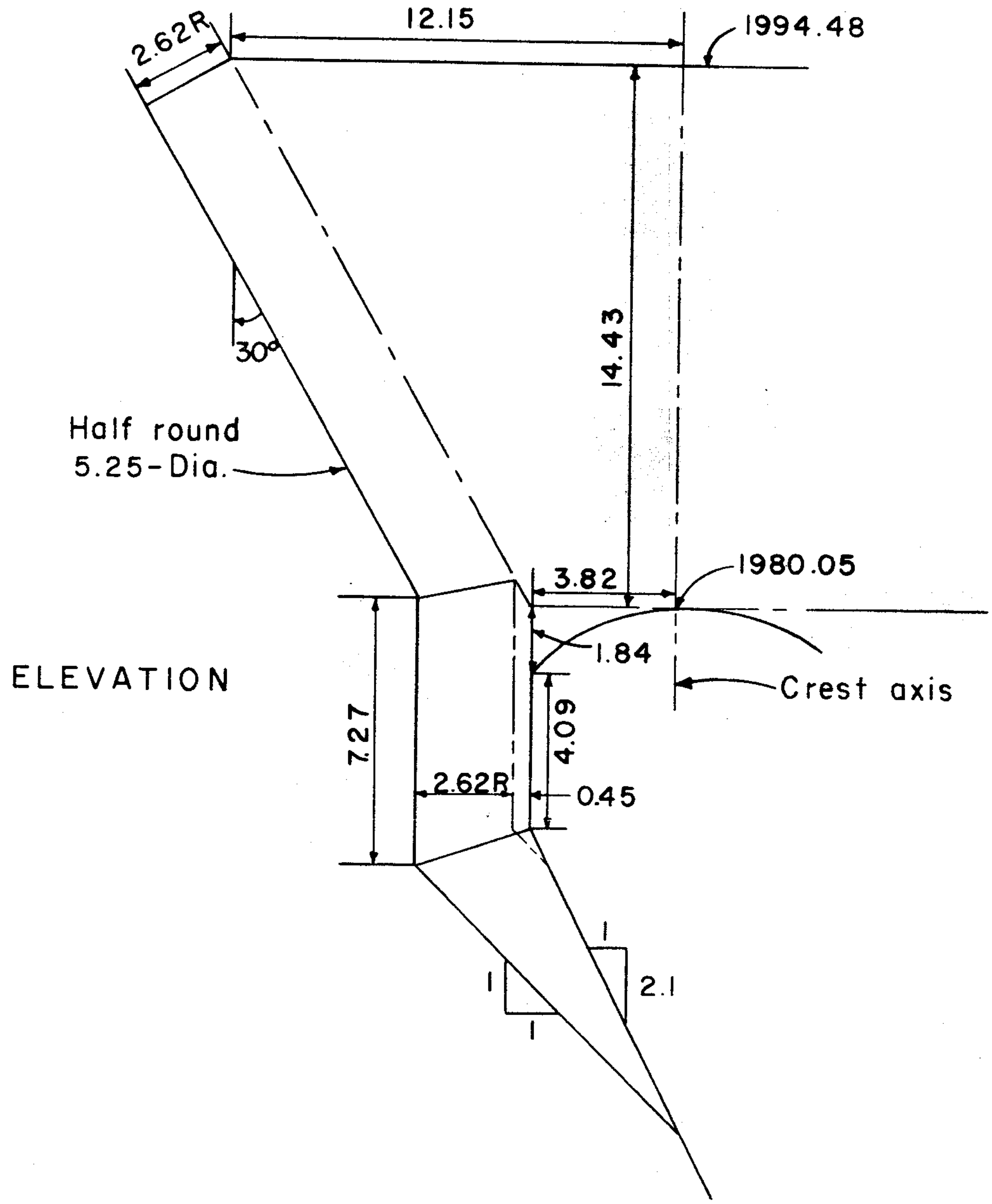
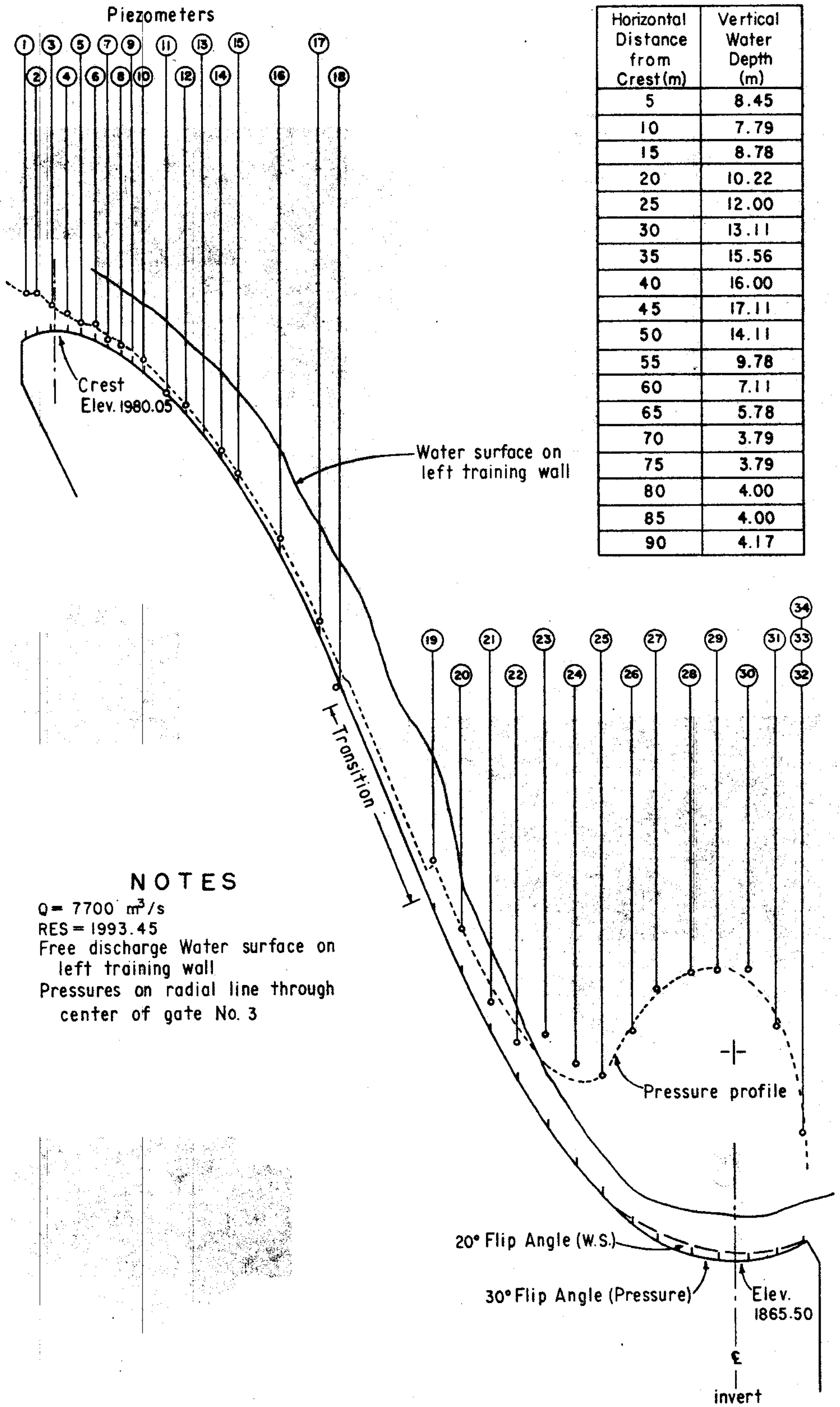


Figure 24.–Lombardi spillway recommended end pier, prototype dimensions.

Figure 25.–Mapping the water surface profile on the training wall, for the Lombardi spillway with the recommended side pier. Photo P801-D-77394



Horizontal Distance from Crest (m)	Vertical Water Depth (m)
5	8.45
10	7.79
15	8.78
20	10.22
25	12.00
30	13.11
35	15.56
40	16.00
45	17.11
50	14.11
55	9.78
60	7.11
65	5.78
70	3.79
75	3.79
80	4.00
85	4.00
90	4.17

Piez No.	Pressure Metres	Remarks
1	5.76	
2	4.69	
3	3.20	
4	2.35	
5	1.28	
6	1.92	
7	0.85	
8	1.07	
9	—	NG
10	1.07	
11	-0-	
12	0.65	
13	—	NG
14	0.64	
15	1.07	
16	1.92	
17	1.49	
18	-0.43	
19	5.76	
20	4.91	
21	3.84	
22	4.69	
23	10.88	
24	12.37	
25	14.29	
26	23.47	
27	31.15	
28	34.99	
29	36.06	
30	35.63	
31	28.16	
32	13.44	
33	—	
34	—	

Figure 26.–Lombardi spillway pressures and water surface profile.

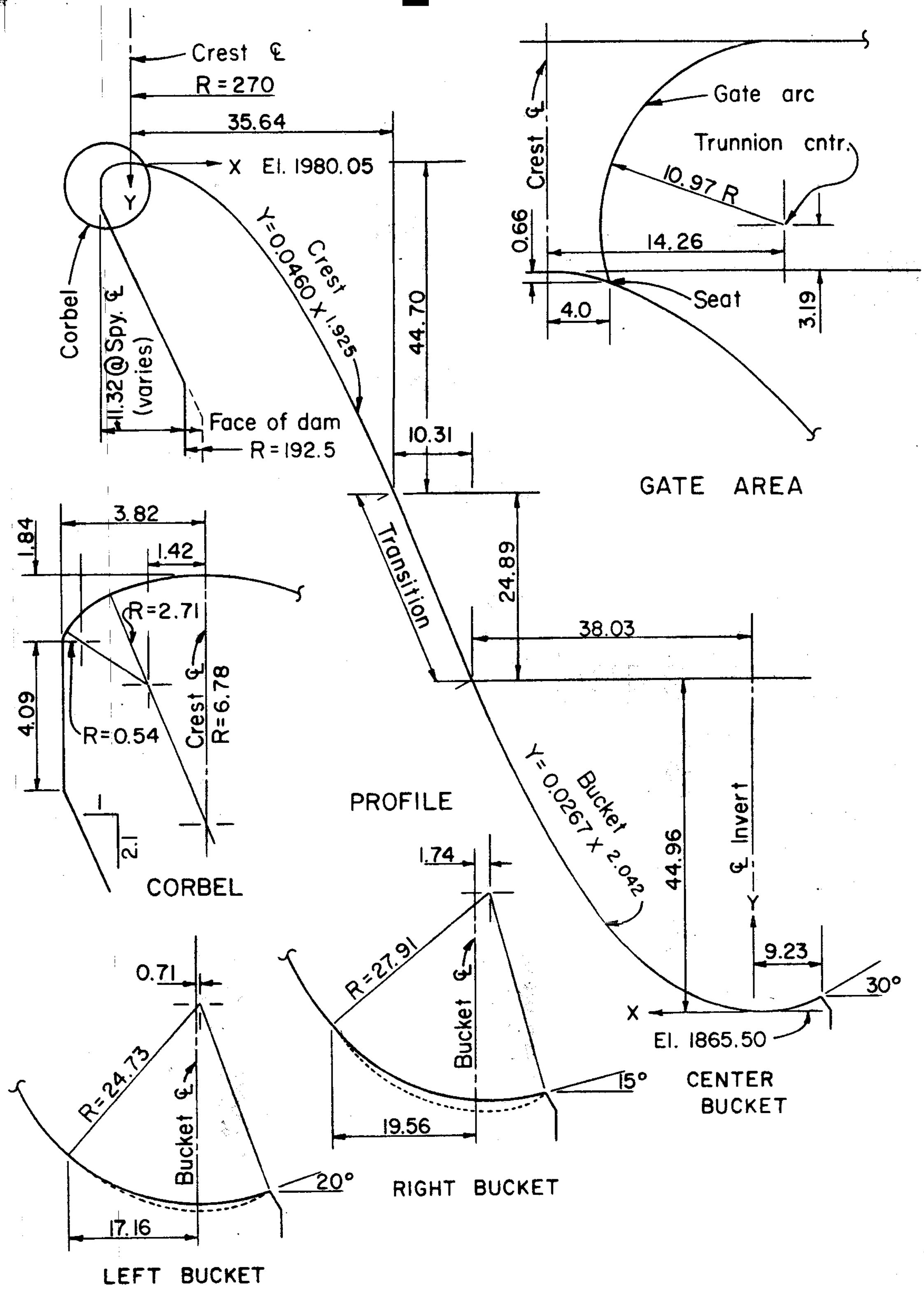


Figure 27.–Lombardi spillway profile, prototype dimensions.

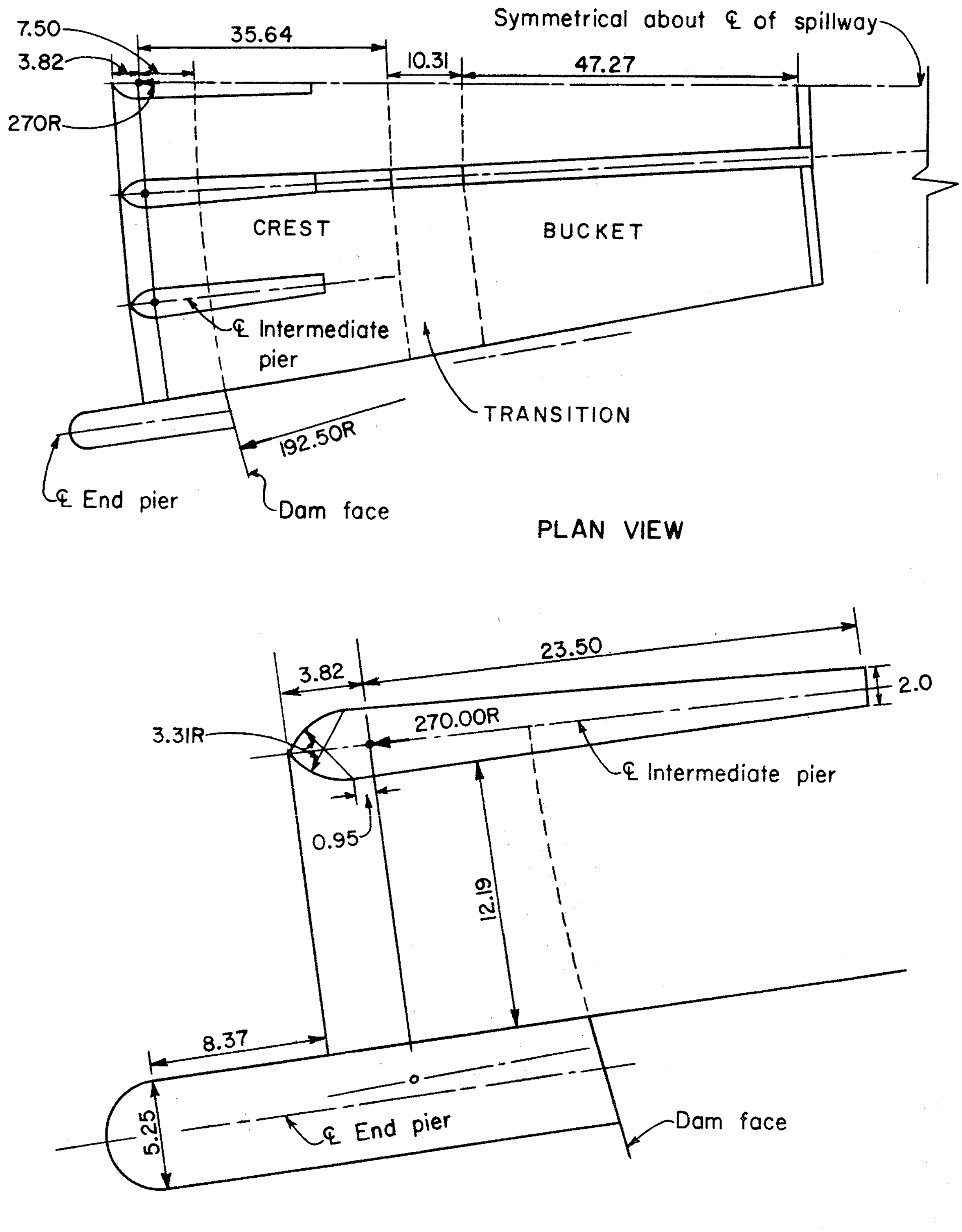


Figure 28.–Lombardi spillway piers, prototype dimensions.

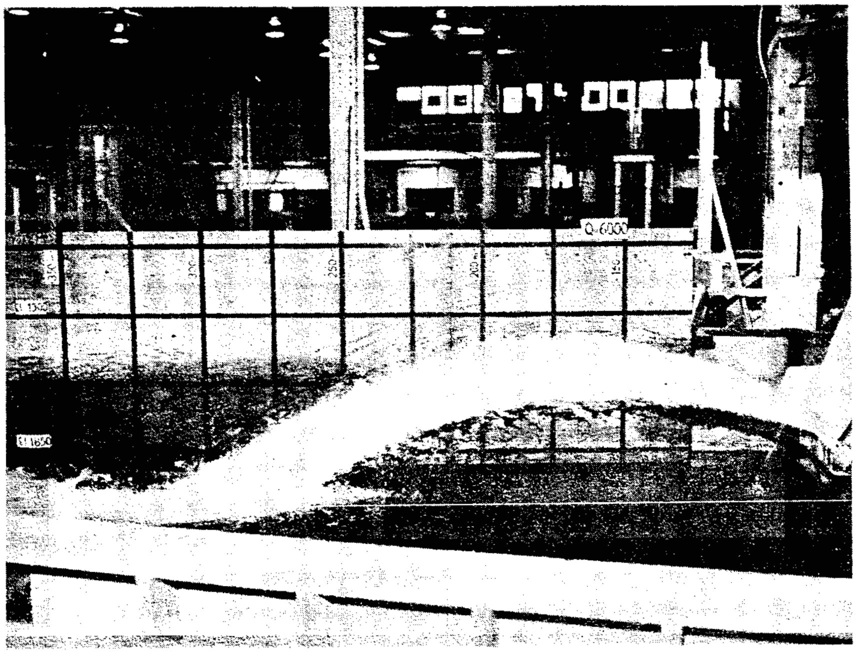


Figure 29.–Jet trajectory, Lombardi spillway, Q = 6000 m^3/s, all buckets 30°. Photo P801-D-77390

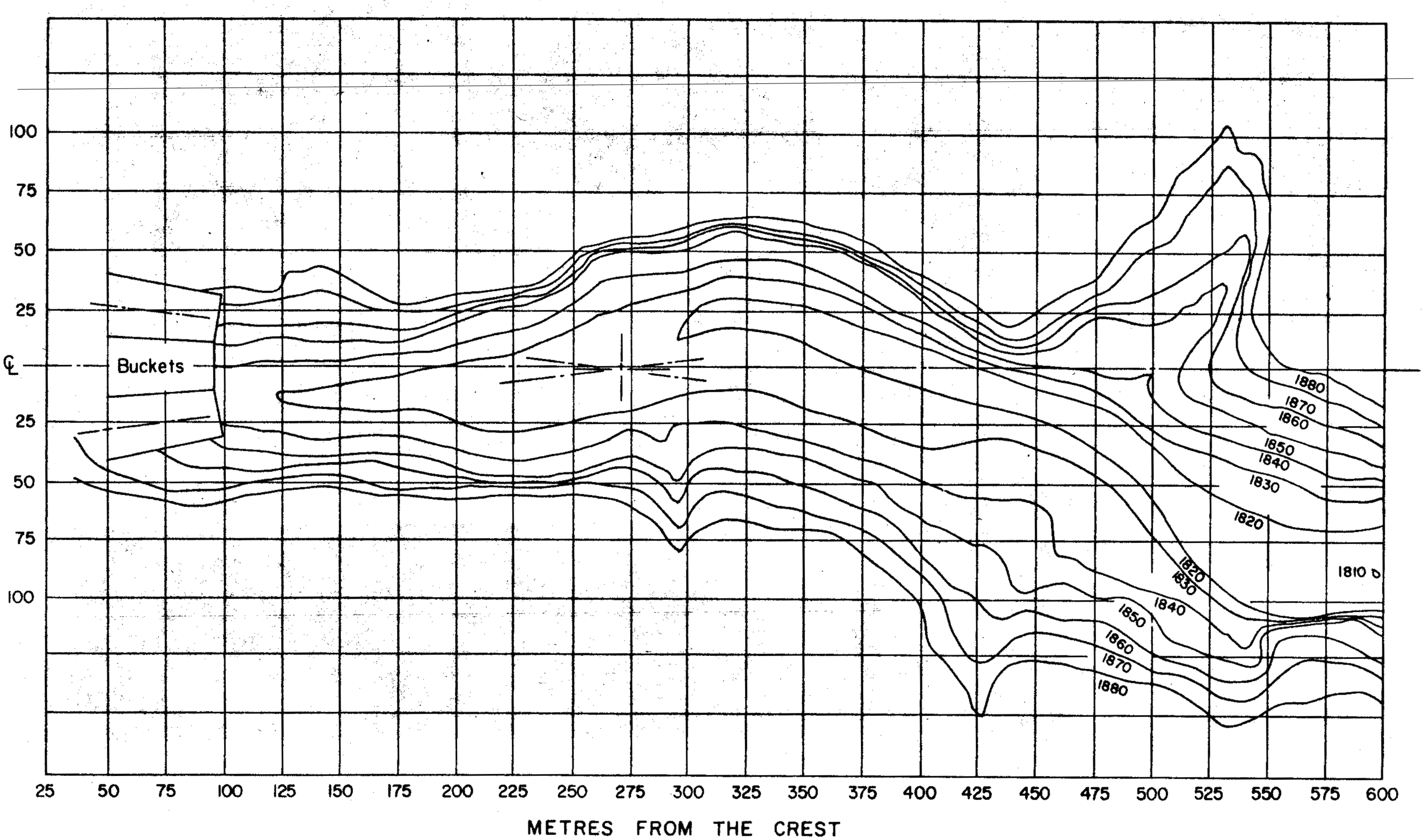
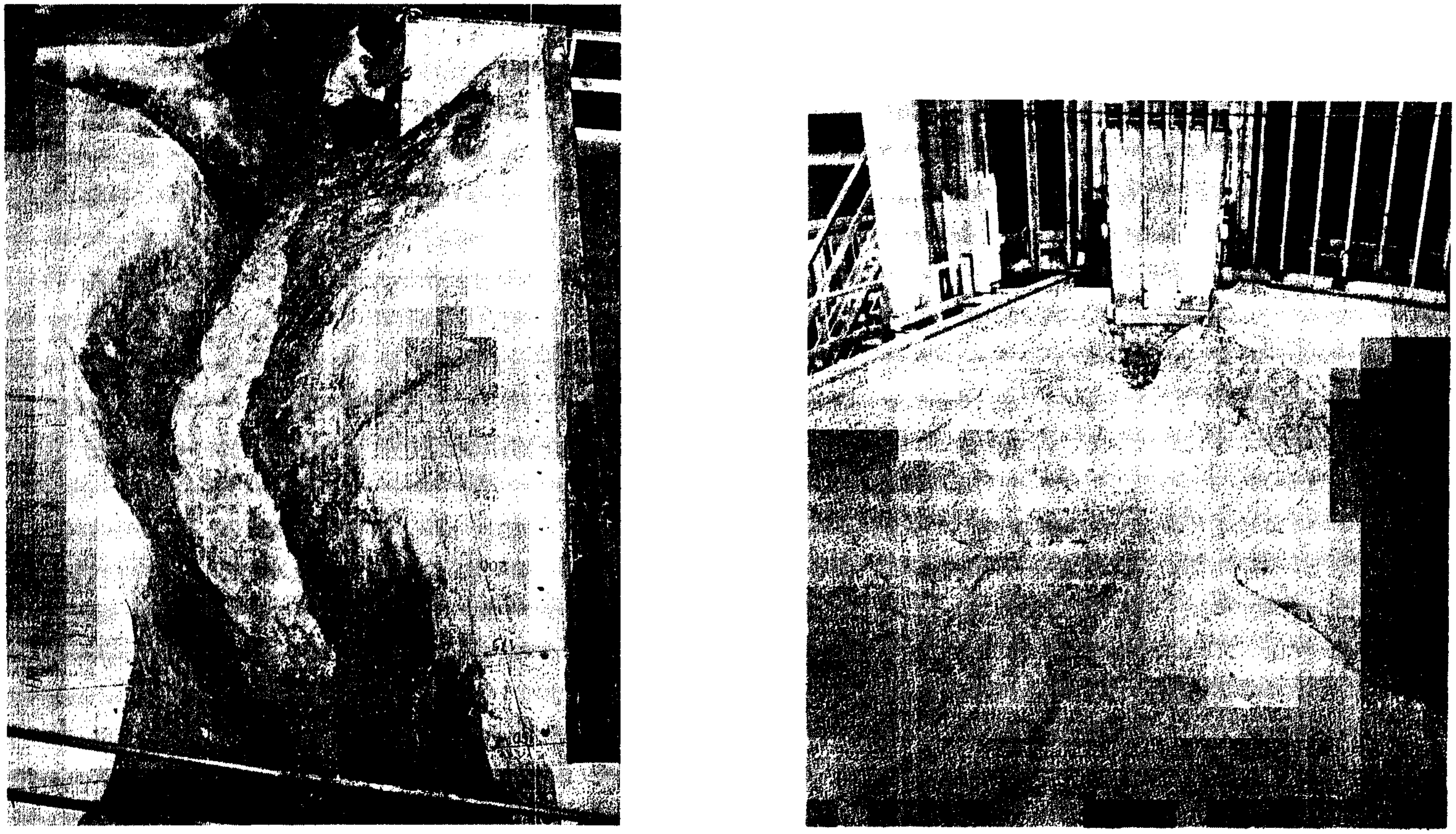


Figure 30.–Lombardi spillway, downstream river channel, model concrete contours.



a.–Looking downstream. Photo P801-D-77391

b.–Looking upstream. Photo P801-D-77403

Figure 31.–Concrete representing riverbed bedrock downstream from the dam.

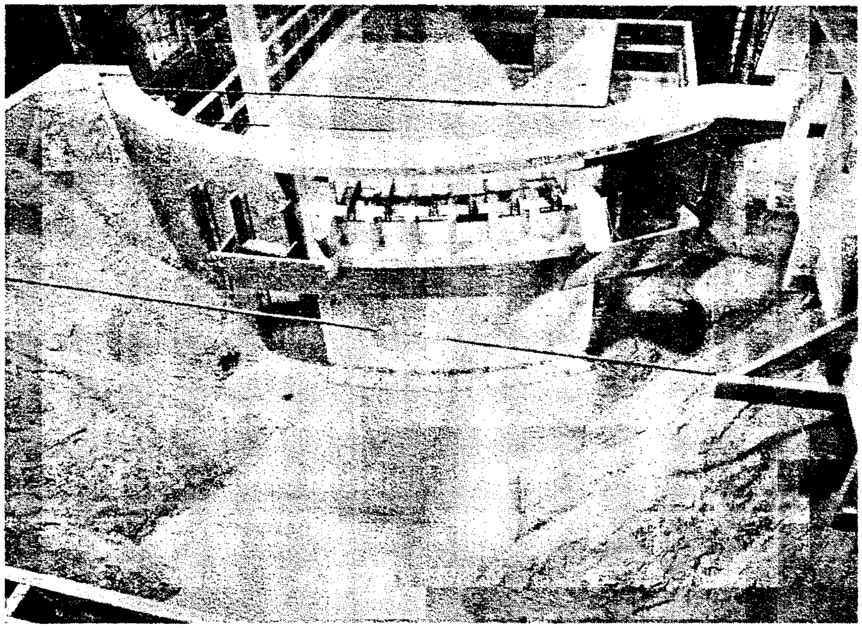


Figure 32.–Overall view of the model looking downstream. Photo P801-D-77404

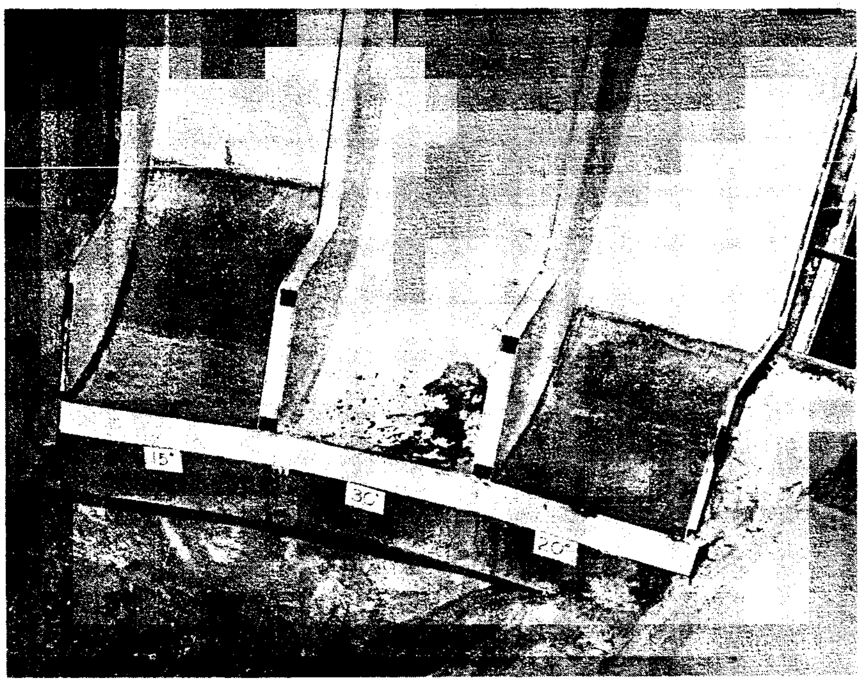


Figure 33.–Recommended flip buckets. Photo P801-D-77409

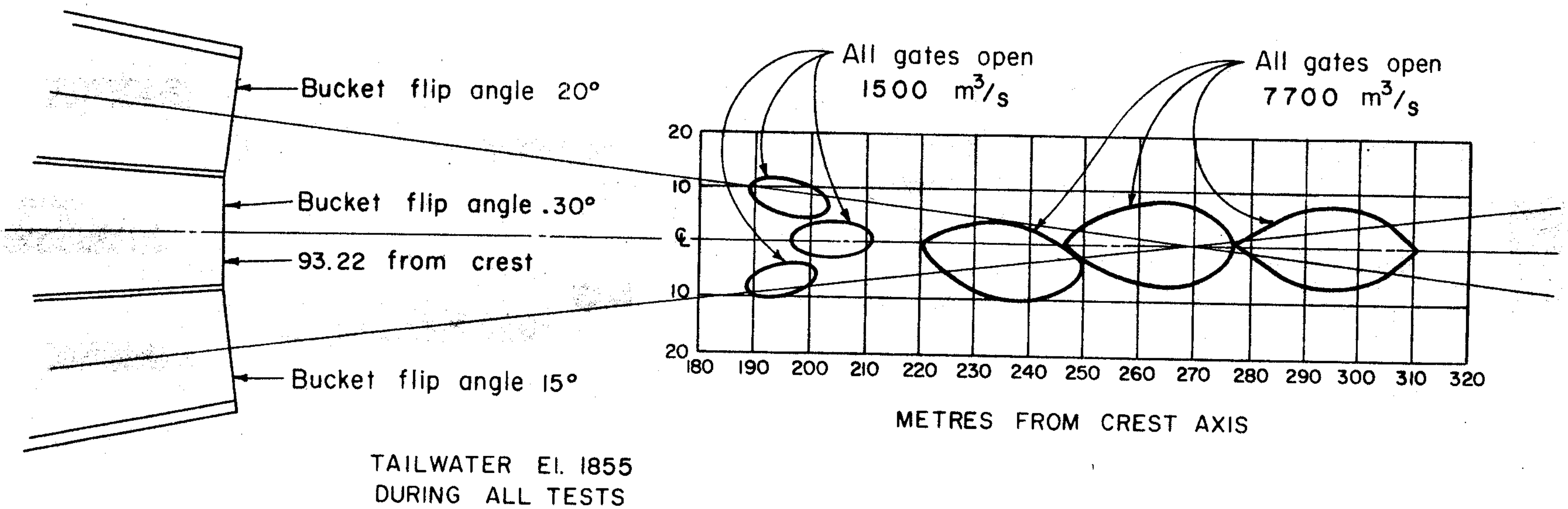


Figure 34.–Lombardi spillway, downstream river channel, jet impingement locations.

Figure 35.–The 20° left flip bucket, and the 15° right flip bucket each discharging 1500 m^3/s. Photo P801-D-77393

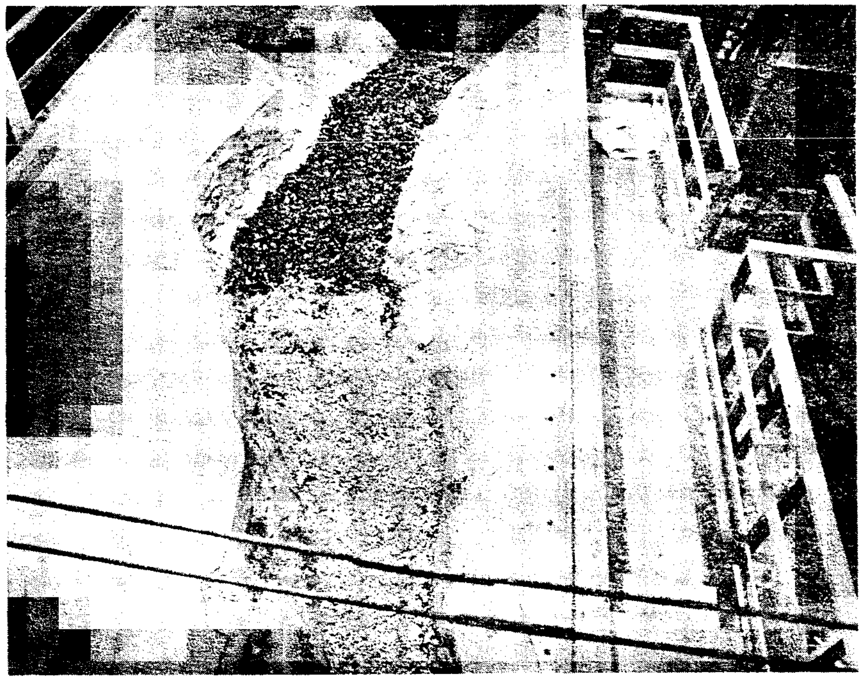


Figure 36.–Scour gravel placed to elevation 1840. Photo P801-D-77410

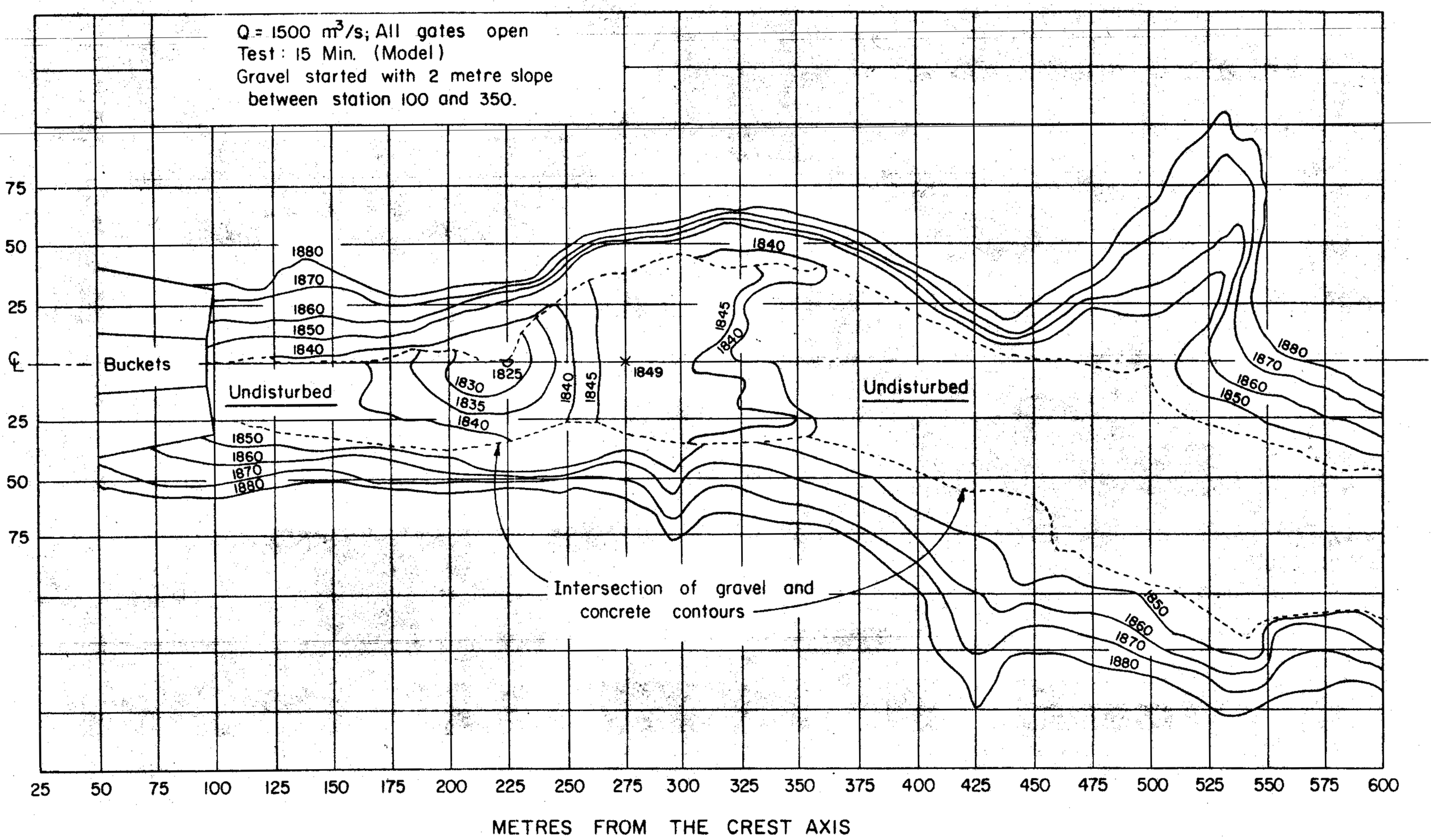


Figure 37.–Lombardi spillway, downstream river channel, model scour pattern, Q = 1500 m³/s 15 minutes (model time).

a.–Looking downstream. Photo P801-D-77397

b.–Looking upstream. Photo P801-D-77396

Figure 38.–Scour after 15 minutes (model time) with free discharge of 1500 m^3/s.

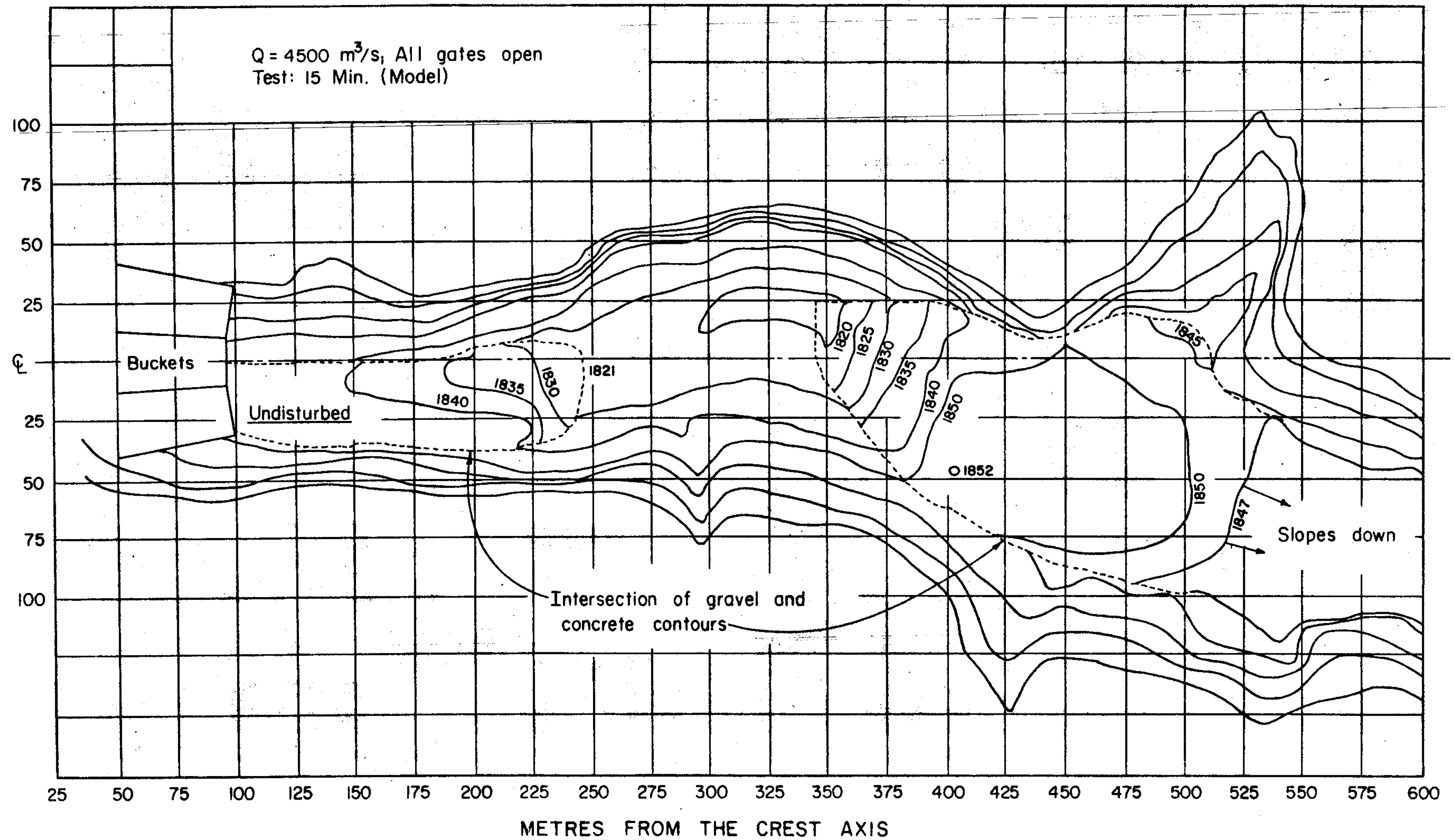


Figure 39.–Lombardi spillway, downstream river channel, model scour pattern, $Q = 4500\ m^3/s$ 15 minutes (model time).

a. Photo P801-D-77398

b. Photo P801-D-77399

Figure 40.–Looking downstream at the scour after 15 minutes (model time) with free discharge of 4500 m^3/s through the Lombardi spillway.

a.–Looking downstream. Photo P801-D-77402

b.–Looking upstream. Photo P801-D-77401

Figure 41.–Recommended design, Lombardi spillway, scour test with maximum discharge of 7700 m^3/s.

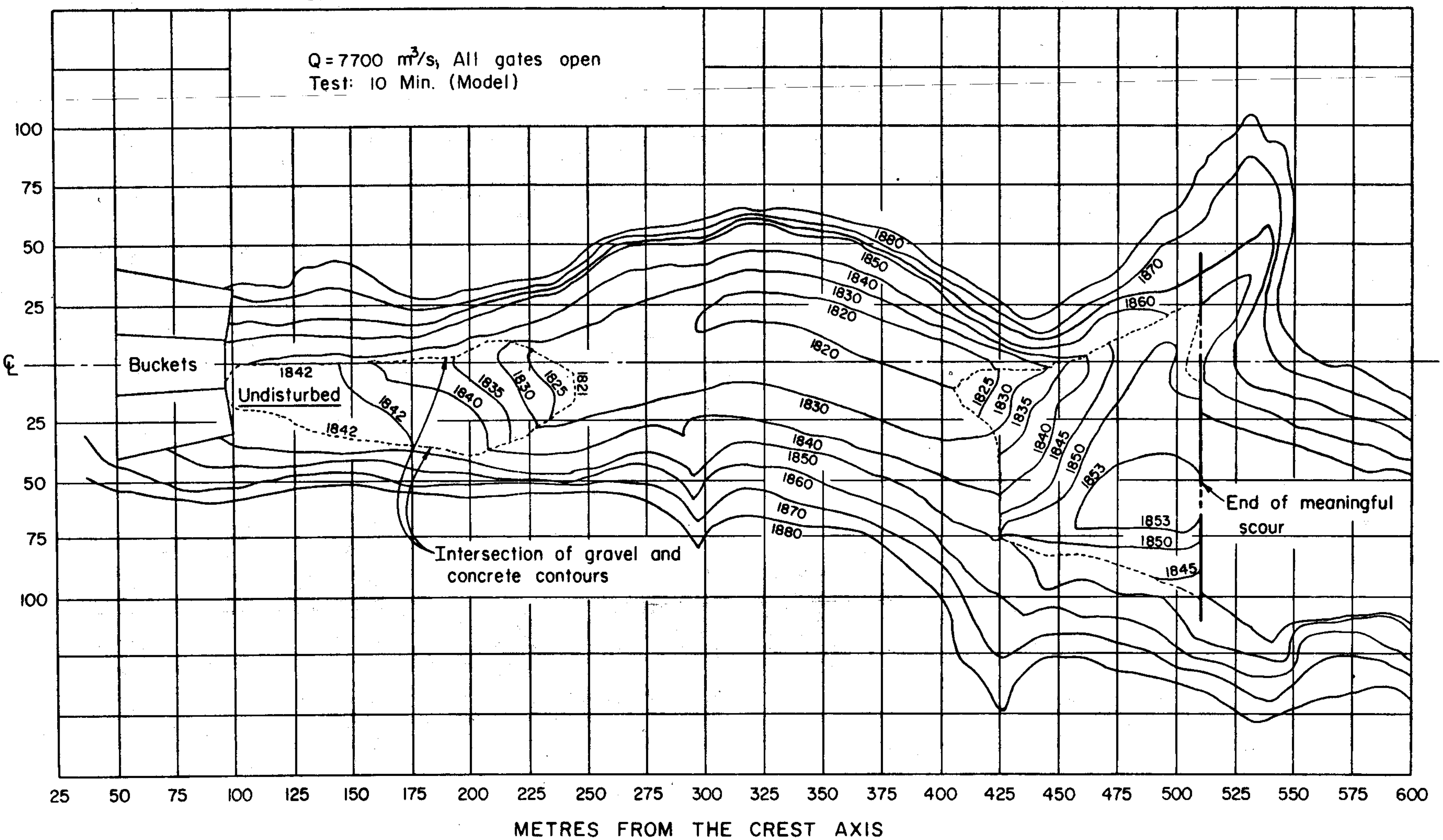


Figure 42.–Lombardi spillway, downstream river channel, model scour pattern, Q = 7700 m³/s 10 minutes (model time).

a.–Looking upstream. Photo P801-D-77407

b.–Looking downstream. Photo P801-D-77408

Figure 43.–Scour resulting from 7700 m^3/s discharge for 10 minutes (model time), Lombardi spillway.

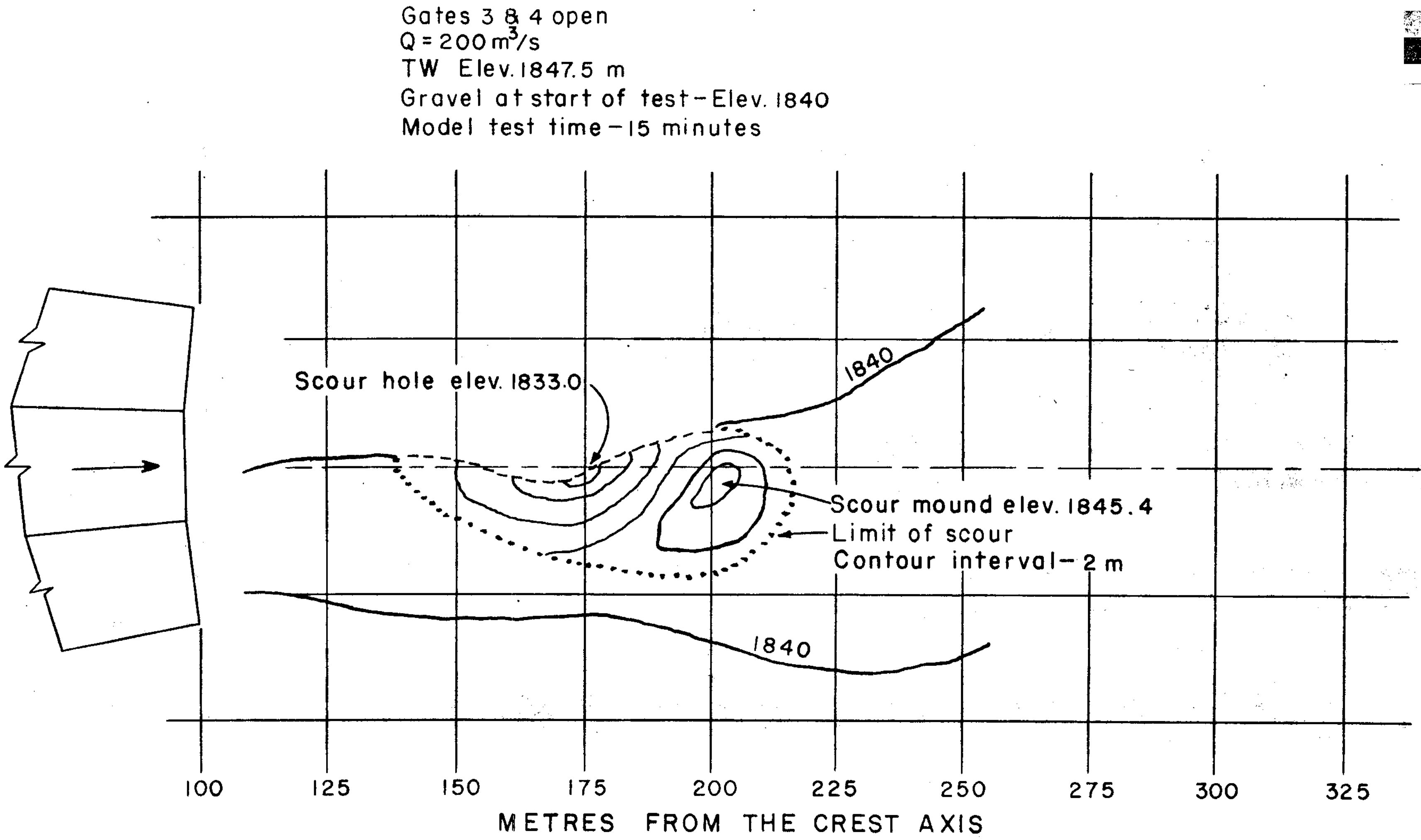


Figure 44.–Lombardi spillway, downstream river channel, model scour pattern, $Q = 200\ m^3/s$ 15 minutes (model time), gates 3 and 4 open.

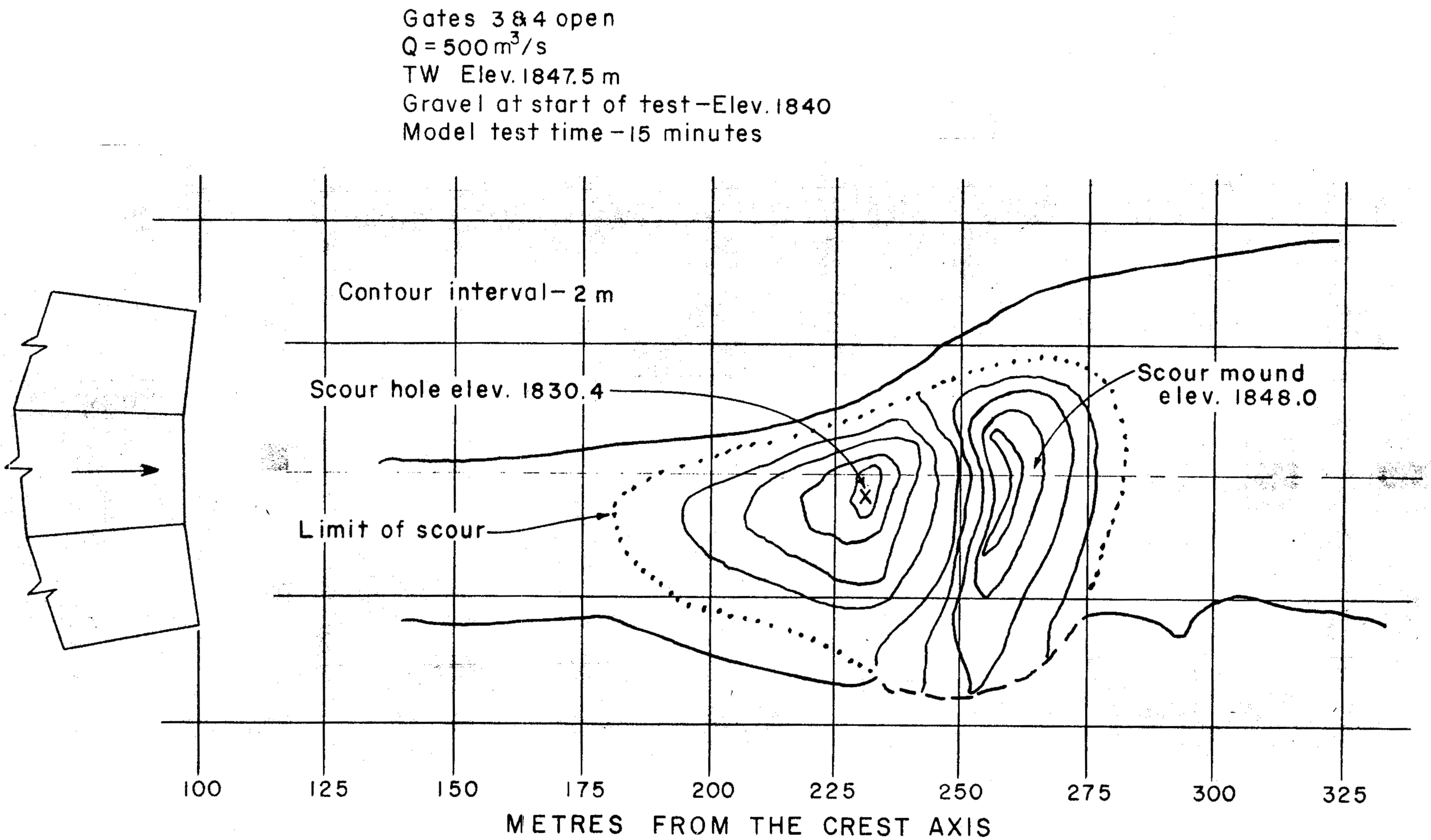


Figure 45.–Lombardi spillway, downstream river channel, model scour pattern, $Q = 500\ m^3/s$ 15 minutes (model time), gates 3 and 4 open.

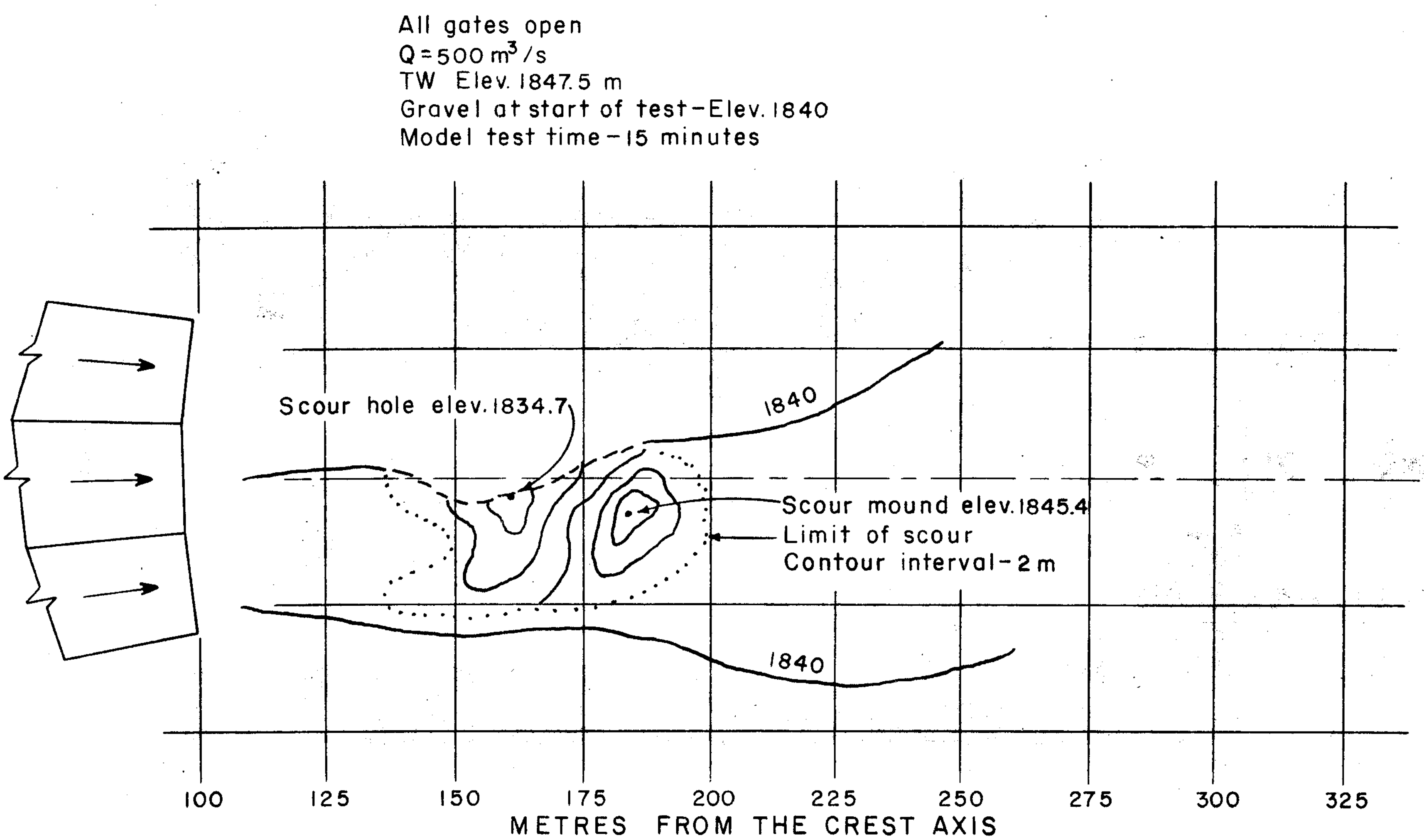


Figure 46.–Lombardi spillway, downstream river channel, model scour pattern, $Q = 500\ m^3/s$ 15 minutes (model time), all gates open.

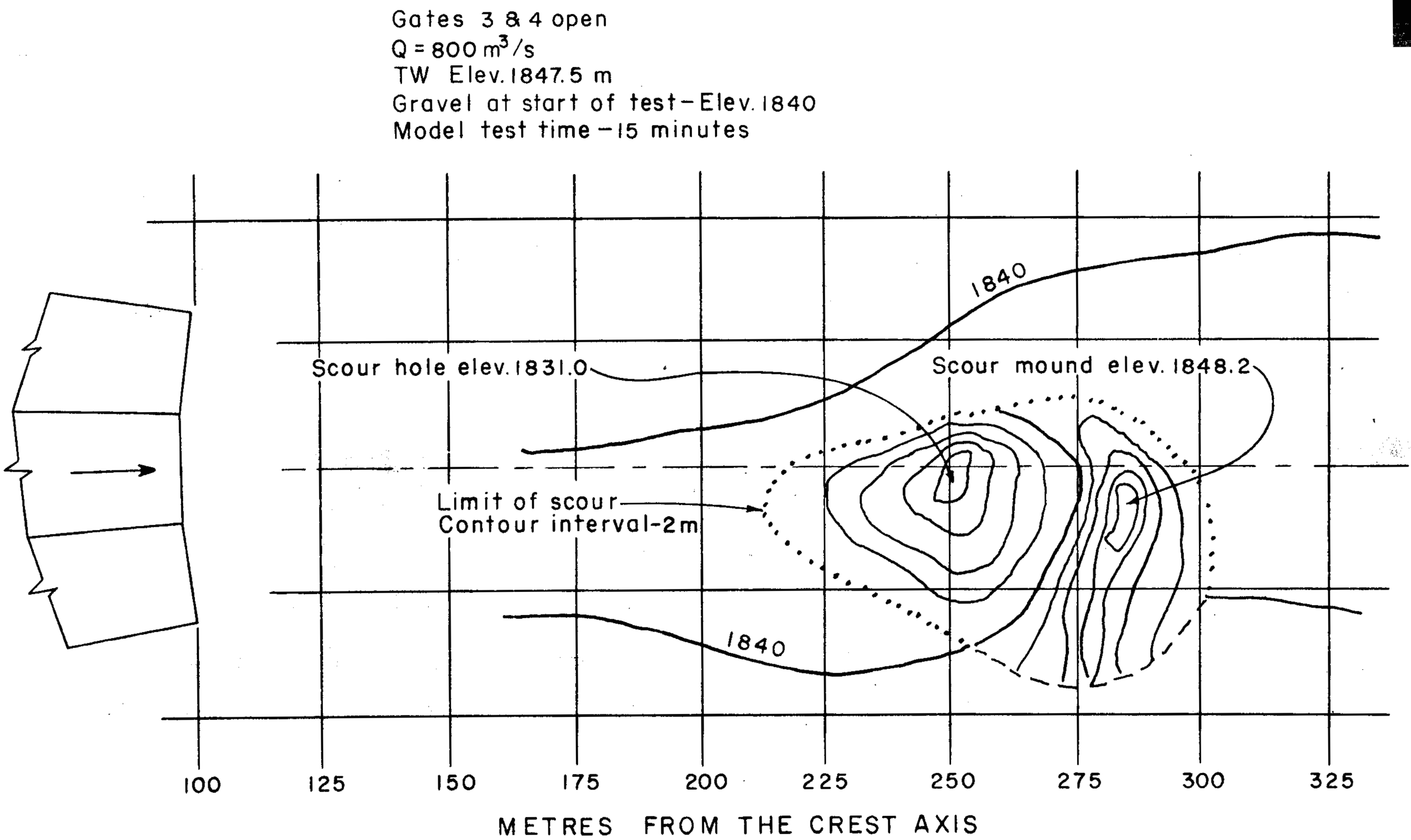


Figure 47.–Lombardi spillway, downstream river channel, model scour pattern, $Q = 800\ m^3/s$ 15 minutes (model time), gates 3 and 4 open.

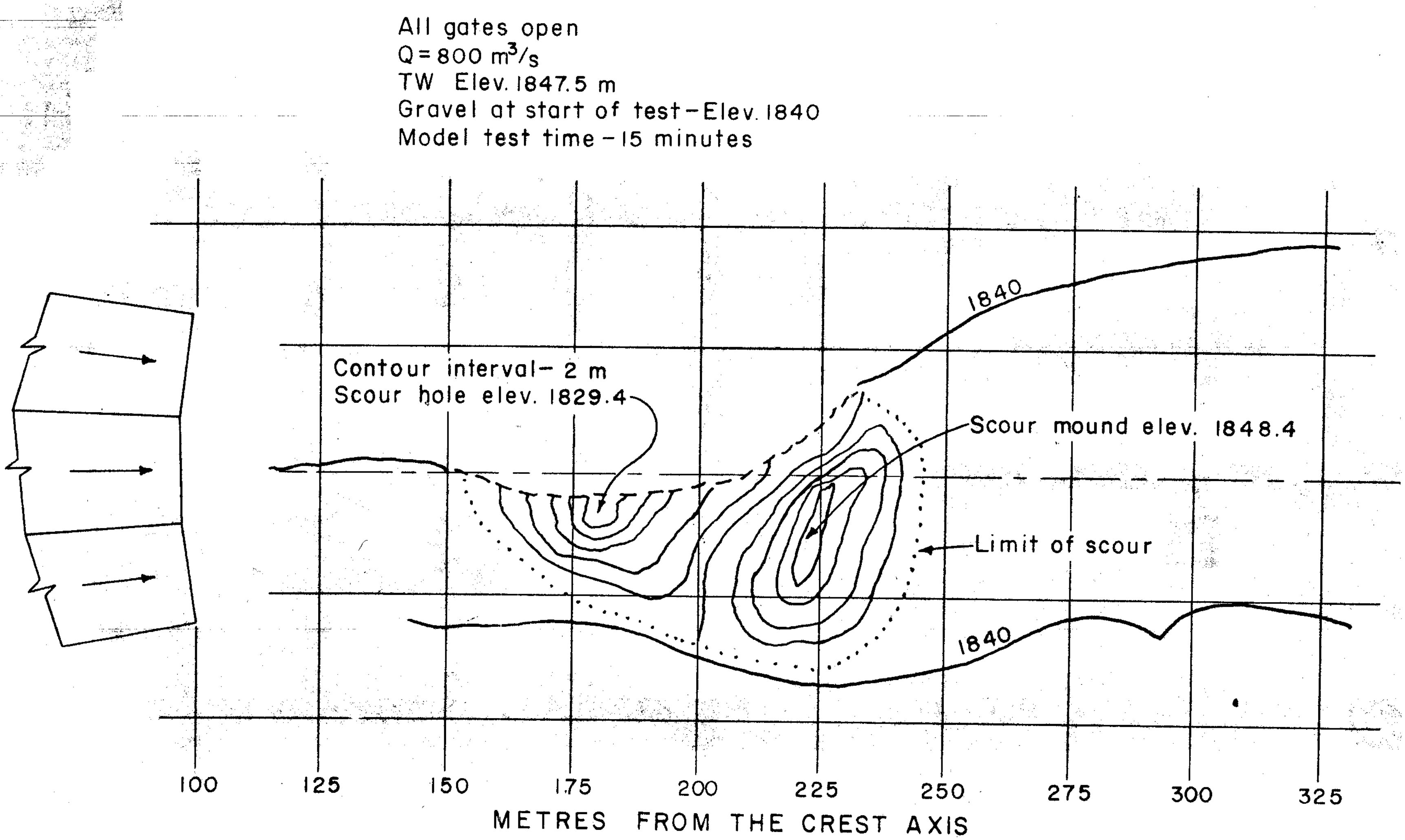
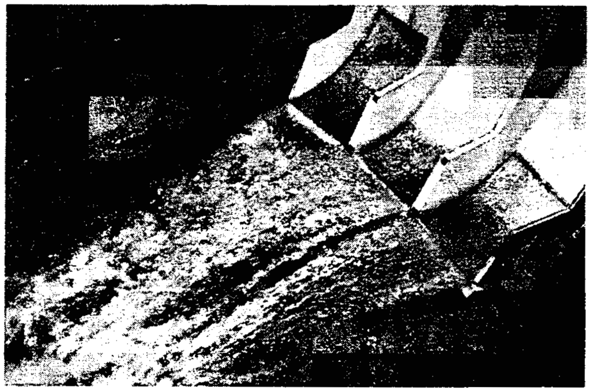
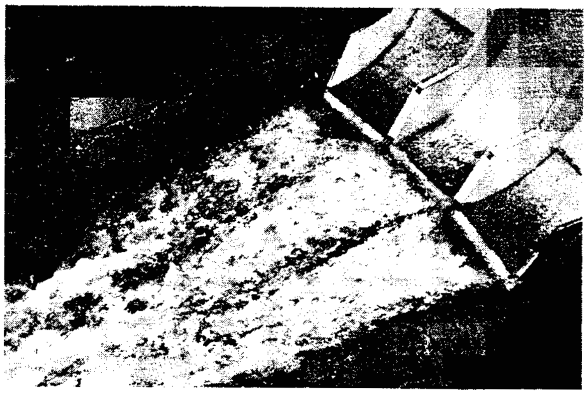


Figure 48.–Lombardi spillway, downstream river channel, model scour pattern, Q = 800 m^3/s 15 minutes (model time), all gates open.



a.– Q = 1000 m^3/s
TW = 1868.3
Photo P801-D-77418



b.– Q = 1000 m^3/s
TW = 1869.5
Photo P801-D-77417

c.– Q = 1000 m^3/s
TW = 1871.0
Photo P801-D-77416

d.– Q = 1000 m^3/s
TW = 1872.4
Photo P801-D-77415

Figure 49.–Flow at lip with high tailwater, Lombardi spillway, for several discharges and tailwater elevations.

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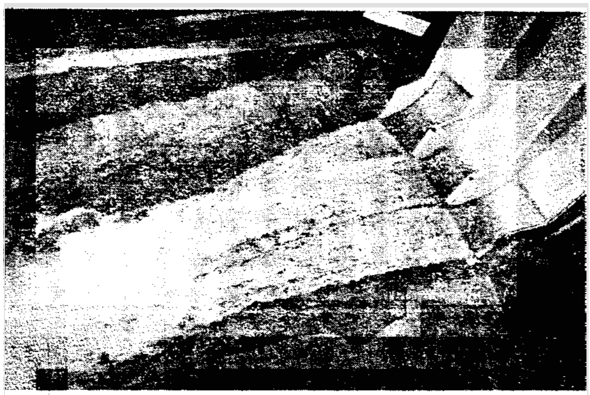
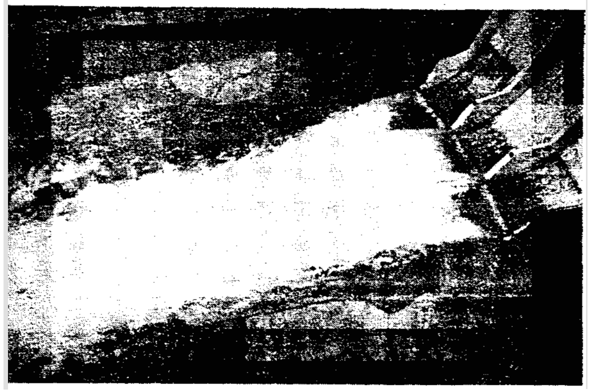


e.– Q = 3050 m^3/s
TW = 1868.5
Photo P801-D-77414



f.– Q = 3050 m^3/s
TW = 1868.5
Photo P801-D-77413

g.– Q = 3050 m^3/s
TW = 1871.5
Photo P801-D-77412

h.– Q = 3050 m^3/s
TW = 1874.5
Photo P801-D-77411

Figure 49.–Flow at lip with high tailwater, Lombardi spillway, for several discharges and tailwater elevations.–Continued