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HYDRAULIC MODEL STUDY OF UTE DAM LABYRINTH SPILLWAY

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16. ABSTRACT

At the request of the New Mexico Interstate Stream Commission, studies were made to develop a spillway to increase the discharge capacity at Ute Dam near Logan, New Mexico. This increase will be accomplished using a labyrinth spillway. In plan, the spillway is a series of trapezoidal shapes. The Ute Dam spillway will be 9.14 m (30.0 ft) high and 1024.13 m (3360.0 ft) in length, with 14 cycles. This spillway will provide the desired storage capacity and still pass the IDF (inflow design flood). The spillway is designed for 5.79 m (19.0 ft) of head which produces a flow magnification of 2.4. Impact pressures taken in the downstream channel will be used in the stability design of the spillway. At low discharges, subatmospheric pressures develop under the nappe near the top of the crest. Two splitter piers on each cycle alleviate these pressures. Velocities measured along the embankment provide information on slope protection required adjacent to the labyrinth.

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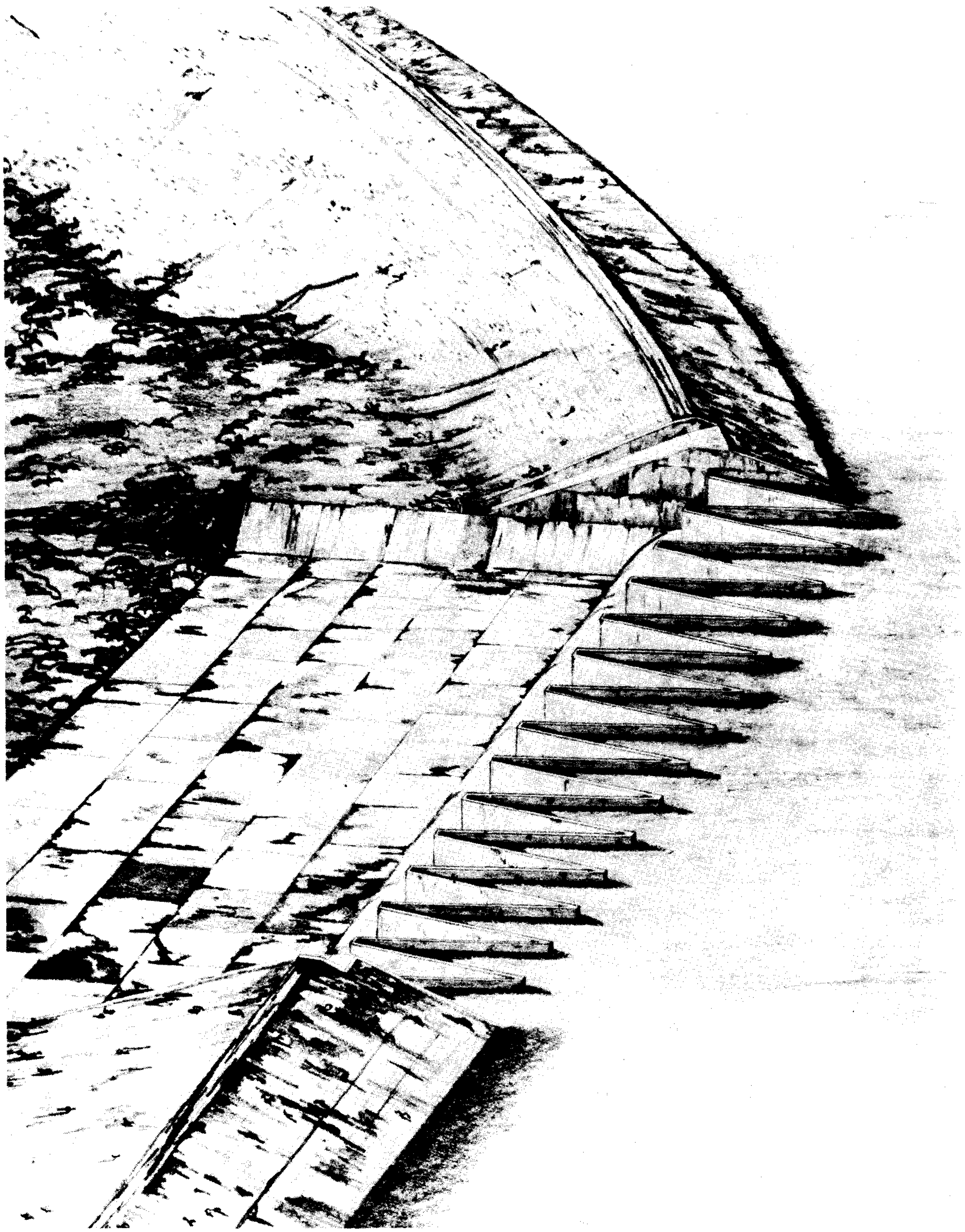
The original work was done using inch-pound units and converted to SI units.



Frontispiece.—Artist's conception of Ute Dam labyrinth spillway modification.

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CONTENTS—Continued

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GLOSSARY

a	=	One-half of the apex of the weir
b	=	Side length of weir
h_m	=	Measured upstream head over weir
h_t	=	Total upstream head over weir
ℓ	=	Developed length of one cycle = $4a + 2b$
ℓ/w	=	Length magnification
L	=	Total developed length of spillway
n	=	Number of weir cycles in plan
p	=	Crest height
Q_N	=	Discharge over linear weir, with $Q_N = CLH^{3/2}$ [C (coefficient of discharge) $= 3.22 + 0.4\ h/p$, L = length of crest minus 0.003 ft, H = head plus 0.003 ft, and h_m and p correspond to the labyrinth weir discharge, Q_L]
Q_L	=	Discharge over weir
Q_L/Q_N	=	Discharge magnification
t	=	Crest thickness
w	=	Width of one cycle of weir
w/p	=	Vertical aspect ratio
W	=	Total width of spillway
α	=	Sidewall angle

INTRODUCTION

Ute Dam, located on the Canadian River 3.2 km (2 mi) west of Logan, New Mexico, was completed in 1962. The 36.88-m (121.0-ft) high embankment dam has a gated outlet works and a 256.03-m (840.0-ft) wide free overflow spillway. Presently, the dam crest is at elevation 1158.54 m (3801.0 ft) and the spillway crest at elevation 1146.05 m (3760.0 ft). The original spillway design included 8.23-m (27.0-ft) high gates which were not installed. Without the gates to increase the reservoir storage capacity, the requirements of future water users could not be fulfilled.

The Bureau of Reclamation was requested by the NMISC (New Mexico Interstate Stream Commission) to increase the storage capacity of the reservoir by (1) installing spillway gates or (2) developing a more economical alternative. An economical labyrinth spillway, in plan a spillway of repeated triangular or trapezoidal shapes, was selected because a much longer length is provided for a given channel width. The Bureau had not previously constructed this type of structure; therefore, a model study was requested by the designers in the Concrete Dams, Spillways, and Outlets Section, Dams Branch, with concurrence from NMISC. The study was required to confirm the 9.14-m (30.0-ft) high spillway's ability to discharge the IDF (inflow design flood) of 16 042 m^3/s (566 509 ft^3/s) at reservoir elevation 1160.07 m (3806.0 ft).

CONCLUSIONS

1. The originally designed 10-cycle spillway based on Hay and Taylor [1]* design curves did not pass the desired IDF within the maximum reservoir elevation. The discharge at reservoir elevation 1160.07 m (3806.0 ft) was 13 479 m^3/s (476 000 ft^3/s) instead of 16 042 m^3/s (566 509 ft^3/s). To pass the maximum discharge, the reservoir elevation was 1161.17 m (3809.6 ft). A 14-cycle spillway, designed using criteria from the

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* Numbers in brackets indicate entries in Bibliography.

laboratory flume testing, successfully met the required discharge and water surface elevation.

2. Design curves developed by Bureau laboratory personnel more accurately predict the behavior of labyrinth spillways because they include the total head present in a reservoir; whereas, the Hay and Taylor curves apparently are not adaptable to a reservoir situation.

3. An attempt was made to improve the approach flow conditions adjacent to the spillway. Spur dikes were constructed of pea gravel perpendicular to the embankment and adjacent to the first and fourteenth cycles. These dikes did not improve the flow and would have required a large quantity of material.

4. To determine the amount of protection required along the embankment, flow velocities were measured adjacent to each end cycle of the spillway. Velocities along the embankment range from 2.07 m/s (6.8 ft/s), 85.34 m (280.0 ft) from the left abutment, to 3.78 m/s (12.4 ft/s) at the abutment; and 0.58 m/s (1.9 ft/s), 73.15 m (240.0 ft) from the right abutment, to 3.11 m/s (10.2 ft/s) at the abutment.

5. To prevent nappe oscillation and provide aeration, two splitter piers should be placed on the crest of each cycle. The optimum location for the piers was 3.35 m (11.0 ft) upstream from the downstream apex of the cycle. This provides aeration along the full length of each sidewall in the cycle. The piers are overtopped when a discharge of 2407 m^3/s (85 000 ft^3/s) is reached.

6. Pressure heads were measured in the downstream channels of cycles 12 and 14. The highest pressure heads were associated with the greatest discharge. The pressure heads are given in meters (feet) of water above the base of the spillway. Pressure heads ranged from approximately 4.6 m (15 ft) to 12.2 m (40 ft) along the centerline of the channel. The pressure heads all tended to decrease as the downstream channel width expanded.

APPLICATION

Labyrinth spillways are particularly well suited for rehabilitation of existing spillway structures because the developed crest length may be greatly increased for a given width. The increased crest length allows passage of a greater or updated IDF than the existing structure. A free overflow labyrinth spillway provides additional reservoir storage capacity as an alternative to the traditional gated structure which requires manual or mechanical operation. Labyrinth structures may also be economically built, provided there is adequate foundation available.

THE MODELS

A flume test facility was used in phase 1 of the model studies. The labyrinth weirs were tested in a 0.76- by 0.61- by 10.97-m (2.5- by 2.0- by 36.0-ft) flume. The upstream water surface was measured using a hook gage.

The crest shape used was the same as Hay and Taylor's [1] to verify and extrapolate their design curves. The model weirs were 152.4 mm (6.0 in) high and 12.7 mm (0.5 in) thick, with a 45° angle forming a sharp crest on the upstream edge. This shape is shown on figure 1. The weirs were made of aluminum and sealed to the floor and sides of the flume. The suppressed linear weirs were tested under both aerated and nonaerated conditions.

The 1:80 scale model of Ute Dam included the upstream approach channel and dam embankment, overflow side channels on top of the existing dam on both sides of the existing ogee crest, and sloped spillway chute. The labyrinth is the same width as, and immediately upstream of, the existing crest. The topography was constructed of plywood and pea gravel, the existing ogee crest of polyurethane, the side channels of sheet metal, and the spillway chute of plywood. The labyrinth spillways were made of mahogany. Water was supplied to the models by the permanent laboratory system and measured with venturi meters.

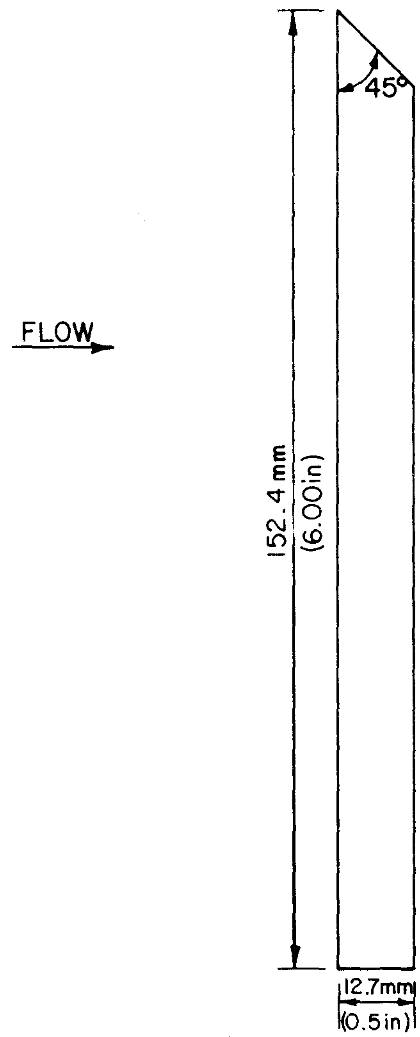


Figure 1.—Sectional view of sharp crested weir.

The model structures were based upon the prototype dimensions. The ogee crest was 256.03 by 15.24 m (840.0 by 50.0 ft) with a 256.03- by 76.20-m (840.0- by 250.0-ft) spillway chute. The labyrinth spillways were each 9.14 m (30 ft) high with a vertical upstream face and a 1:10 batter on the downstream face. The labyrinth crest was 0.61 m (2.0 ft) wide at the top with a 0.30-m (1.0-ft) radius forming a rounded upstream edge. The side channels on either side of the spillway were approximately 17.68 m (58.0 ft) wide and located on the existing dam at elevation 1158.84 m (3802.0 ft). An overall view of the 1:80 scale model is shown on figure 2 with the 14-cycle spillway installed.

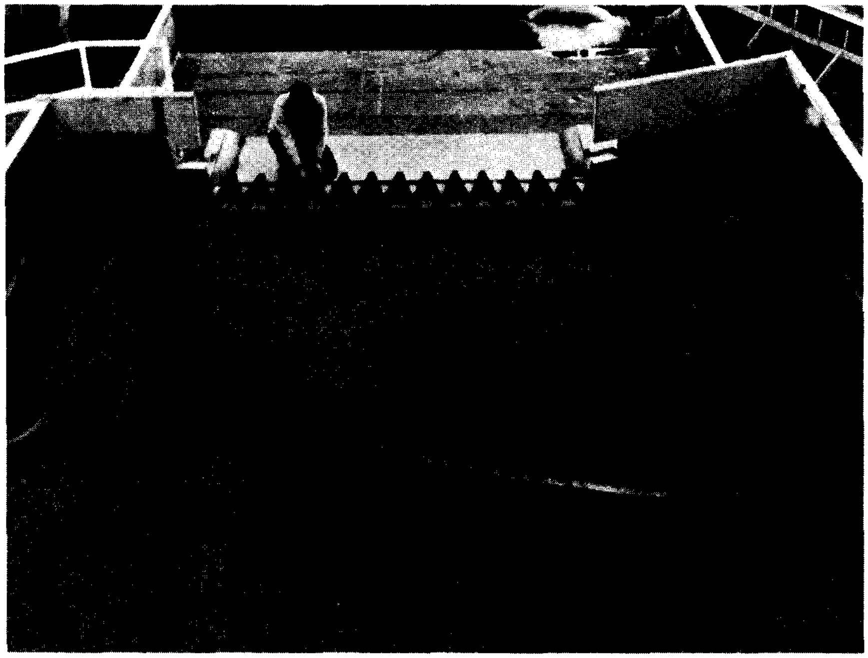


Figure 2.—Scale model (1:80) of Ute Dam spillway. P801-D-79837

SIMILITUDE AND TEST DISCHARGES

The model was designed to a linear scale using Froude law relationships. The model variables were computed as follows:

$$Q_m = \frac{1}{(80^{2.5})} Q_p$$

$$V_m = \frac{1}{(80^{0.5})} V_p$$

where:

Q_m = Model discharge
Q_p = Prototype discharge
V_m = Model velocity
V_p = Prototype velocity

For example, a prototype discharge of 2832 m³/s (100 000 ft³/s) equals a model discharge of:

$$\frac{2832 \text{ m}^3/\text{s}}{80^{2.5}} = 0.049 \text{ m}^3/\text{s or } \frac{100\,000 \text{ ft}^3/\text{s}}{80^{2.5}} = 1.7469 \text{ ft}^3/\text{s}$$

For documentation purposes, discharges of 2832, 7787, 15 206, and 16 042 m³/s (100 000, 275 000, 537 000, and 566 509 ft³/s) were tested and photographed.

FLUME TESTS

Hay and Taylor Design Curves

Hay and Taylor's design curves, figure 3, show sharp crested weir length magnifications of 3, 4, 5, 6, 7, and 8 over the h_m/p range from 0 to 0.5. Each ℓ/w curve starts at the

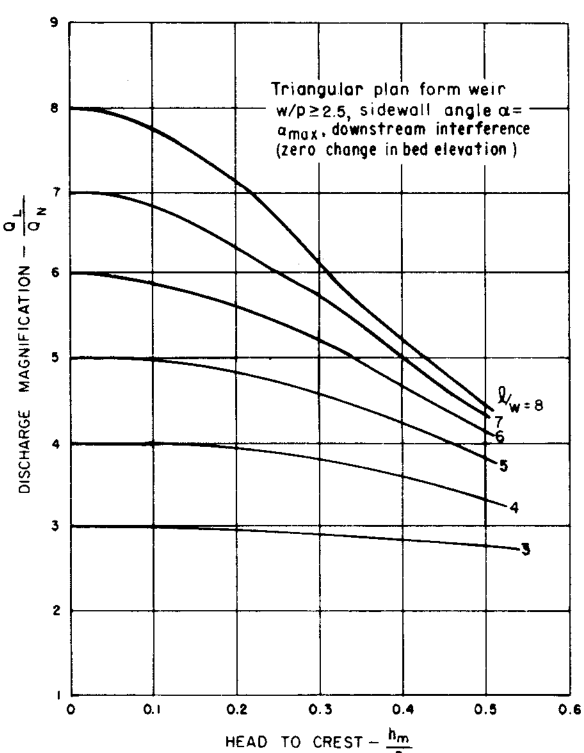


Figure 3.—Hay and Taylor design curves.

Q_L/Q_N ratio corresponding to the ideal condition of discharge increasing in direct proportion to an increase in ℓ/w. For example, an increase in length of four times would ideally produce a discharge increase of four times that of a straight weir. Other major labyrinth parameters (defined in the Glossary) include the vertical aspect ratio and the sidewall angle. The design curves (fig. 3) were developed based on $w/p \geq 2.5$ and $\alpha \leq 0$ with the weir performance increasing as α increases to the maximum of a triangular shape weir. The trapezoidal shape weir should be designed for $\alpha = 0.75\ \alpha_{max}$.

Labyrinth Design Procedure

The general design procedure for labyrinth spillways is outlined below. The procedure is discussed in detail by Hay and Taylor [1]. The site constraints of Ute spillway determine the values for p (spillway height), Q_L (maximum discharge), h_m (reservoir head), and W (existing spillway width). The corresponding initial design values were 9.14 (30.0 ft), 16 042 m^3/s (566 509 ft^3/s), 5.79 m (19.0 ft), and 256.03 m (840.0 ft), respectively. The design is based upon a set of curves plotted in dimensionless ratios of discharge magnification, Q_L/Q_N, versus head to crest height, h_m/p (fig. 3). The Q_N value is based upon the Q_L parameters defined in the Glossary.

The Ute design ratios are $Q_L/Q_N = 2.4$ and $h_m/p = 0.63$. Extrapolating the design curves, figure 3, to include these values determines the ℓ/w of a sharp crested weir. The ratio of the discharge coefficient of the prototype labyrinth crest to the sharp crest is used to determine the ℓ/w for the prototype spillway. The number of cycles, n, formed from the total length, L, may be determined using the cycle width, w, and maximum sidewall angle, α.

Expansion and Attempted Verification of Existing Design Curves

Plotting $Q_L/Q_N = 2.44$ and $h_m/p = 0.63$ on figure 3 shows that the design values were not encompassed by the design curves. Therefore, it was necessary to extend the existing curves to include this design region. To ensure continuity of the curves between the previous range of $h_m/p \leq 0.5$ and the extended range $0.5 \leq h_m/p \leq 1.0$, it was first necessary to verify the existing curves.

These tests were done in the flume described earlier. As background information on Hay and Taylor's work had not yet been obtained, it was assumed that the head, h_t, included the velocity head. Using this assumption, verification tests were made with a triangular weir of $\ell/w = 3$. The Q_L/Q_N versus h_t/p plot of these test data did not corroborate the Hay and Taylor plot. The curve developed by the flume testing showed a considerably lower discharge magnification as seen by comparing the curves of figures 3 and 4 for $\ell/w = 3$. This discrepancy between the design curves of $\ell/w = 3$ prompted additional flume testing. The sharp crested labyrinth weir shapes tested in the flume are shown schematically on figure 5. Each weir was tested in 2 cycles with $0 \leq h_t/p \leq 1$. Figure 6 shows a triangular shape weir of $\ell/w = 4$ operating at $h_t/p = 0.57$. The tests emphasized triangular shape weirs as they are the most efficient. The Ute spillway closely approximates the triangular shape by having narrow apexes (2a).

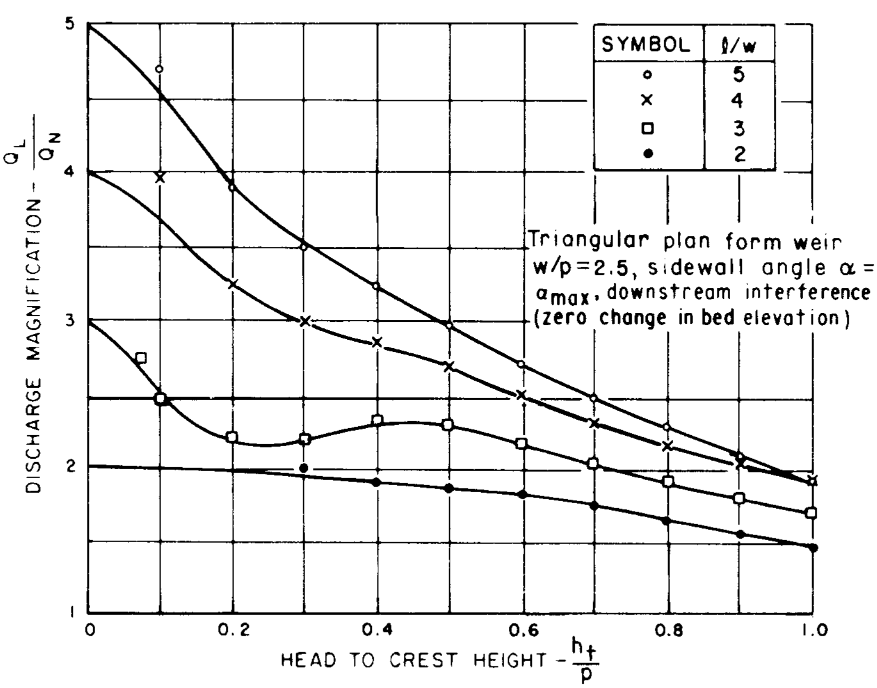


Figure 4.—Bureau design curves.

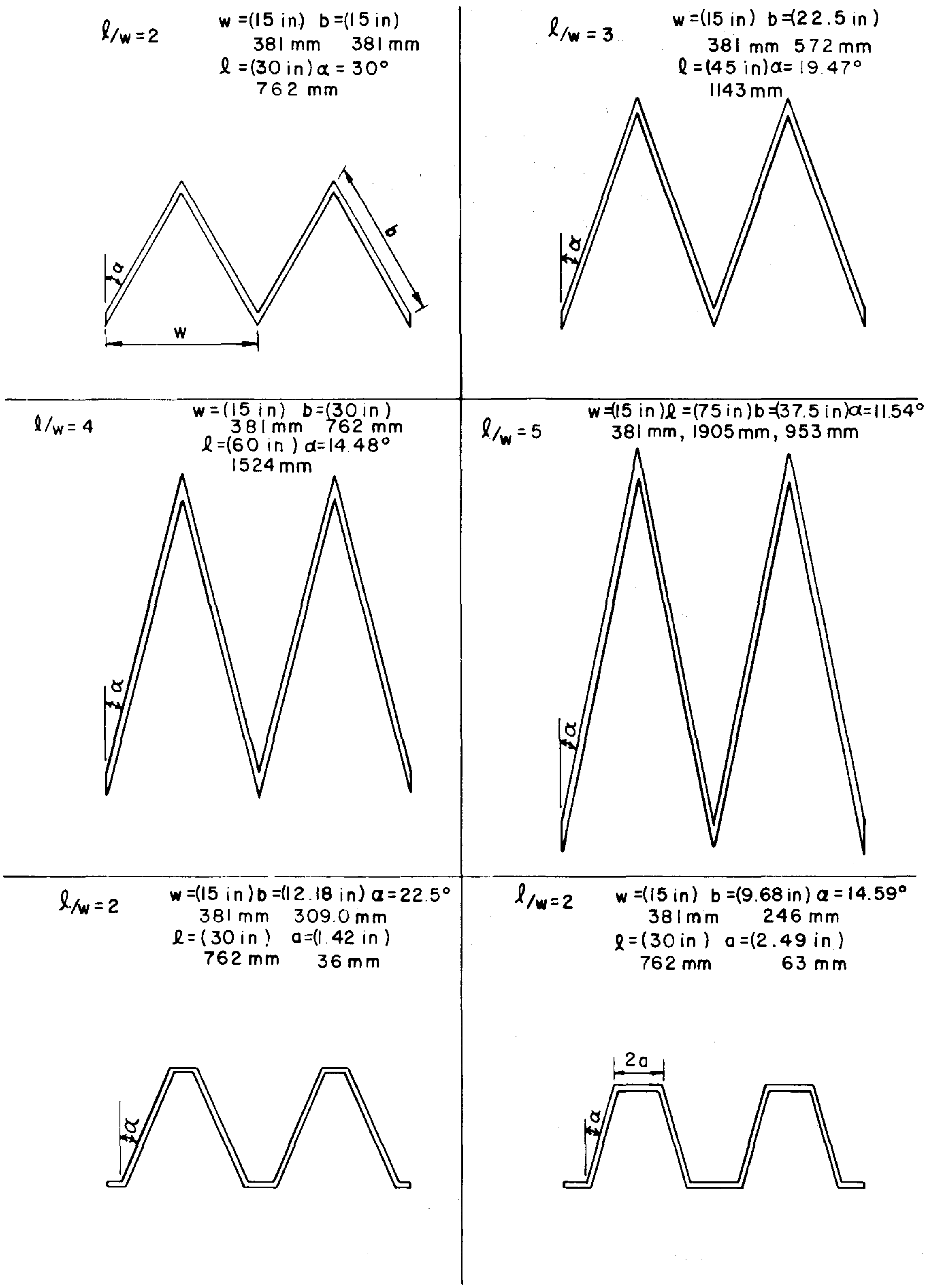


Figure 5.—Schematic of the sharp crested weir shapes tested in the flume.

Figure 6.—Labyrinth weir in flume, $\ell/w = 4$, $w/p = 2.5$, $h_t/p = 0.57$.
P801-D-79838

Bureau Design Curves

Test results of triangular shape sharp crested weirs for $2 \leq \ell/w \leq 5$ are shown on figure 4, plotting Q_L/Q_N versus h_t/p. These curves were developed using the total head, h_t, which includes the measured head, h_m, plus velocity head $v^2/2g$. Hay and Taylor's design curves apparently do not include total head and, therefore, are not applicable to a reservoir situation without the addition of the velocity head to the measured head value. This difference in head definition appears to be the difference between the Hay and Taylor and Bureau design curves. The design procedure remains essentially the same; however, the h_m term should be replaced with h_t.

If the Ute spillway was designed using the Bureau's curves, the length would have to be much greater than initially calculated. A construction cost analysis indicated that the required spillway length would be most economically obtained with 14 cycles. This required a vertical aspect ratio, w/p, of 2, which is less than the minimum value suggested by Hay and Taylor. The triangular plan form weir of $\ell/w = 4$ was tested with $w/p = 2$ in the flume by increasing the weir height. The results showed only a slight reduction in discharge magnification, as shown by the comparison of figures 4 and 7. This is, however, a parameter that needs more definition and appears to be dependent upon other parameters, such as ℓ/w.

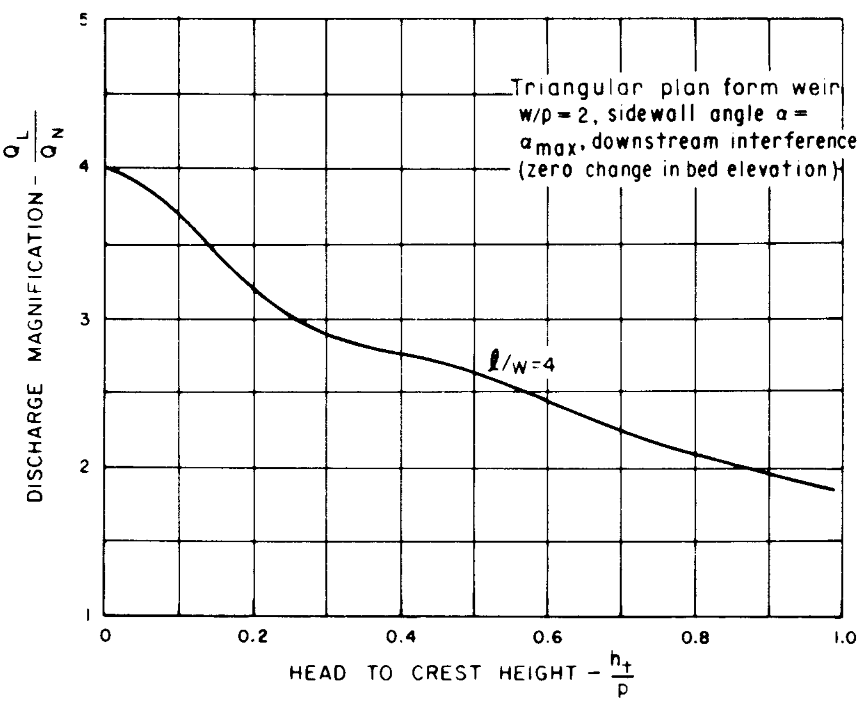


Figure 7.—Effect of vertical aspect ratio, $w/p = 2$ with $\ell/w = 4$.

UTE DAM SPILLWAY MODEL

Ten-Cycle Labyrinth Spillway

Although the head definition appeared to be causing the discrepancy between the design curves, confirming data had still not been obtained. To determine which set of curves should be used in a reservoir situation, the initially designed 10-cycle spillway, based on Hay and Taylor's work, was tested in the 1:80 scale model. The design head and weir height resulted in an h_m/p which made it necessary to extrapolate the design curves.

Dimensions of the spillway appear on figure 8. Other parameters are ℓ/w = 2.74, w/p = 2.8, and a total developed length, L, of 701.04 m (2300.0 ft).

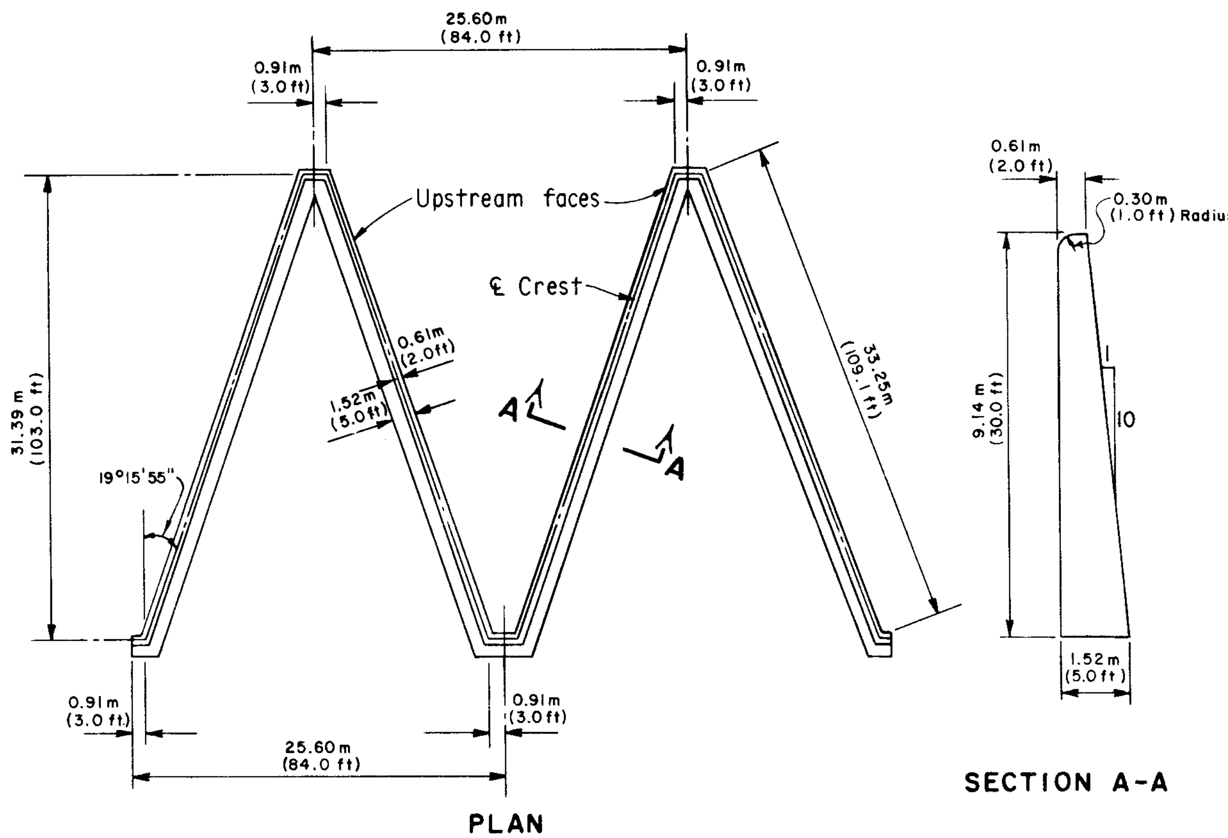


Figure 8.—Dimensions of the 10-cycle labyrinth spillway.

The spillway is shown on figure 9 discharging 16 042 m^3/s (566 509 ft^3/s) which included flow through the side channel spillways at the elevation of the existing dam. This maximum discharge required a reservoir elevation of 1161.17 m (3809.6 ft) as indicated on the discharge curve on figure 10. This reservoir elevation was 1.10 m (3.6 ft) higher than the design maximum water surface and therefore unsatisfactory. The spillway has a discharge coefficient of 1.383 (2.505) at design reservoir elevation 1160.07 m (3806.0 ft) which is based upon the total developed length.

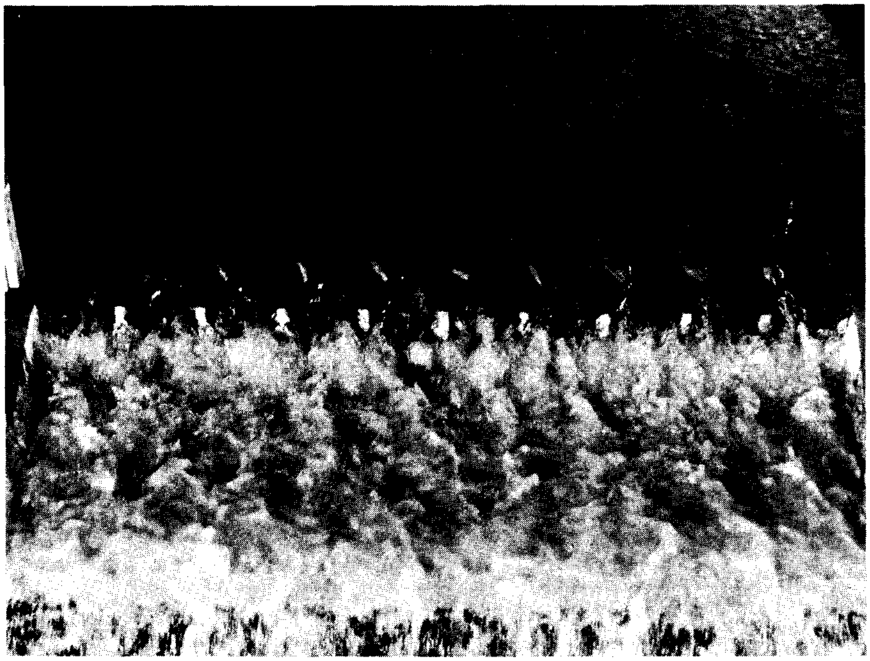


Figure 9.—Ten-cycle labyrinth spillway, Q = 16 042 m^3/s (566 509 ft^3/s), reservoir elevation 1161.17 m (3809.6 ft). P801-D-79839

This test showed that the maximum discharge could not be passed over the spillway within the stipulated design head of 5.79 m (19.0 ft). This implied that the total developed spillway length required would be greater than that predicted by Hay and Taylor. The design discharge magnification and head-to-crest height ratio when plotted on the Bureau curves (fig. 4) showed a length magnification of 4 instead of 2.74 as predicted by Hay and Taylor. This result prompted design of another spillway based on the Bureau ℓ/w = 4 curve.

Investigation of approach conditions.—A flow disturbance was noted upstream of the two cycles on each end of the spillway. Dikes were placed at various angles to the embankment to improve the flow conditions and perhaps increase the discharge capacity. The dikes were constructed of pea gravel to elevation 1160.98 m (3809.0 ft). The volume of a dike would be about 24 103 m^3 (31 526 yd^3). Figure 11 shows a discharge of 16 042 m^3/s (566 509 ft^3/s), with flow over the left side of the spillway without the dike in place; figure 12 shows the same discharge with the dike. The dikes caused a large drop in the water surface due to an increase in velocity and actually worsened the flow conditions adjacent to the spillway. There was negligible effect on the capacity of the spillway.

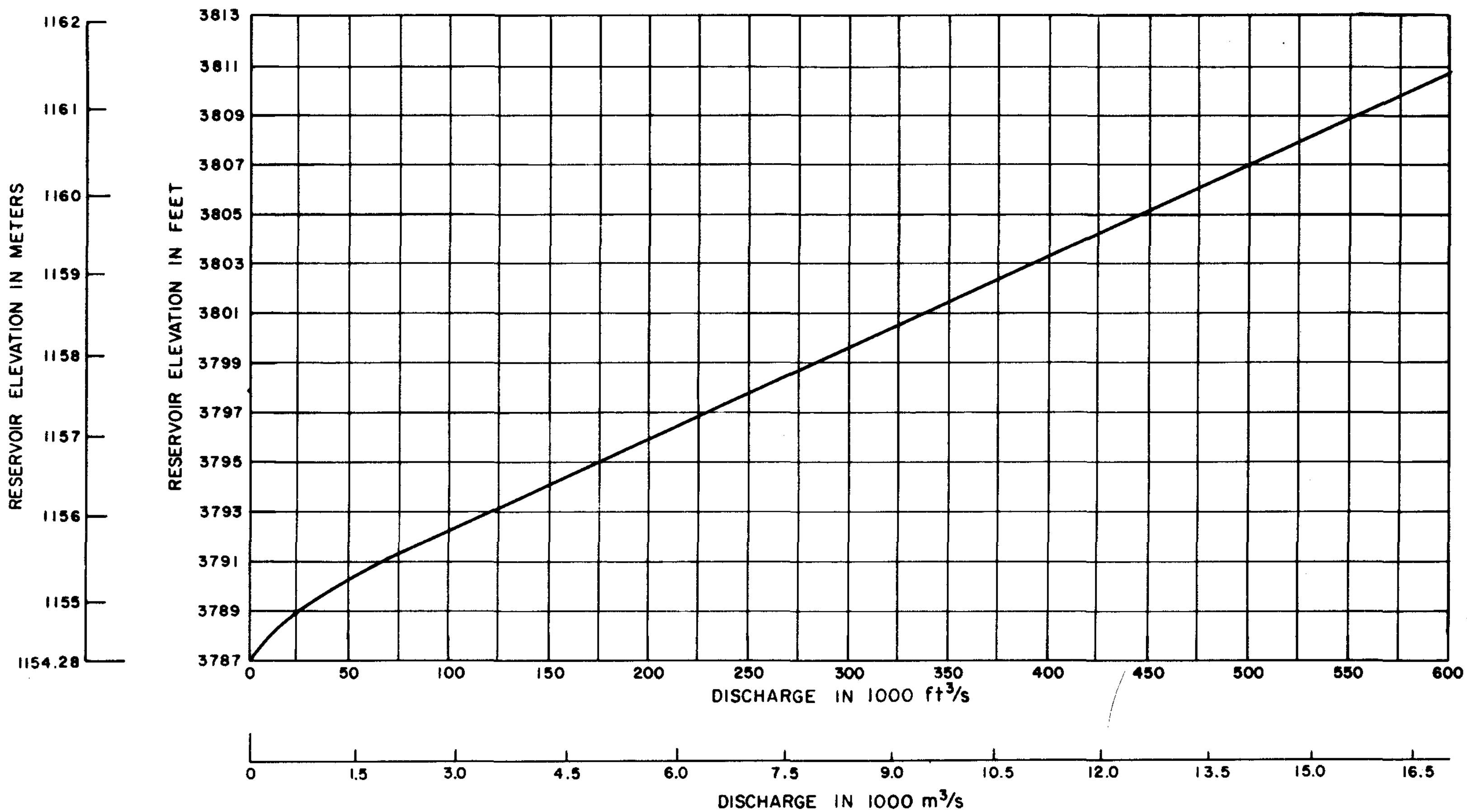


Figure 10.—Discharge curve for the 10-cycle labyrinth spillway.

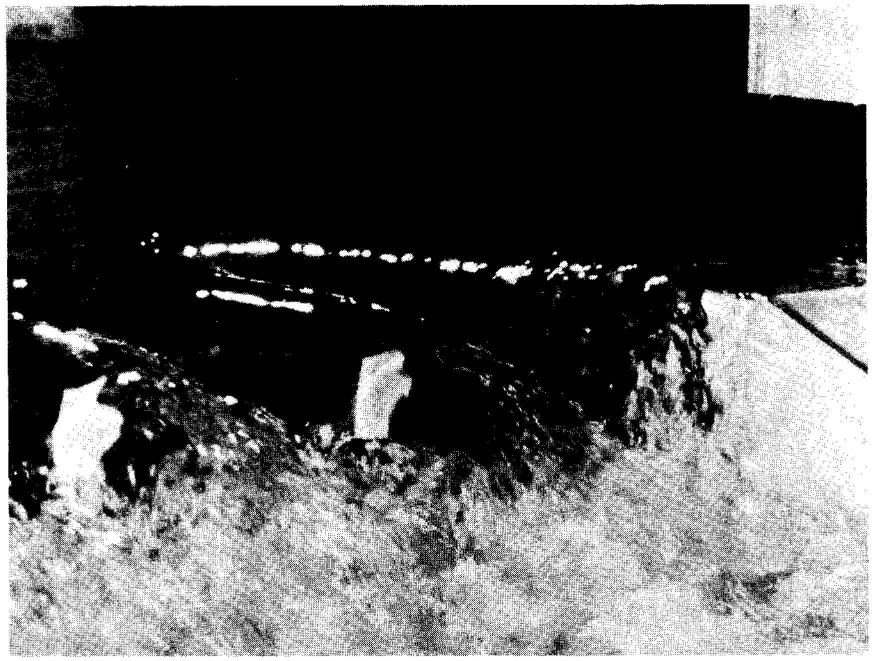


Figure 11.—Flow over left side of spillway, Q = 16 042 m³/s (566 509 ft³/s). P801-D-79840

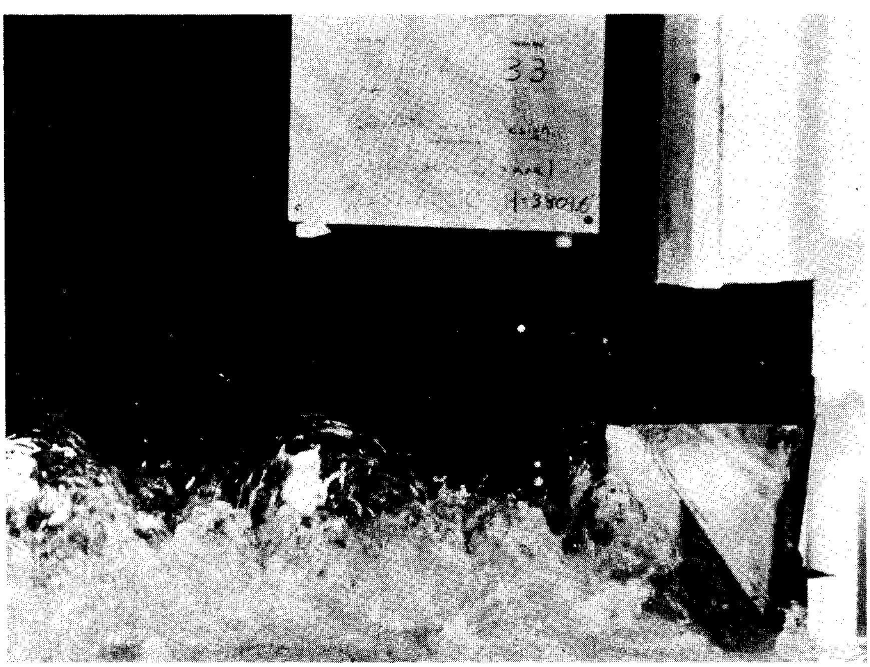


Figure 12.—Flow over left side of spillway with the modified approach, Q = 16 042 m³/s (566 509 ft³/s). P801-D-79841

Fourteen-Cycle Labyrinth Spillway

The second and recommended design for the Ute spillway was a 14-cycle spillway. The design was based upon criteria developed from the flume tests for sharp crested triangular labyrinth weirs. The 14-cycle spillway was the most economical shape that would meet the maximum water surface elevation and discharge requirements. The dimensions of the spillway are shown on figure 13. Other parameters are $\ell/w = 4$, $w/p = 2$, and $L = 1024.13$ m (3360.0 ft).

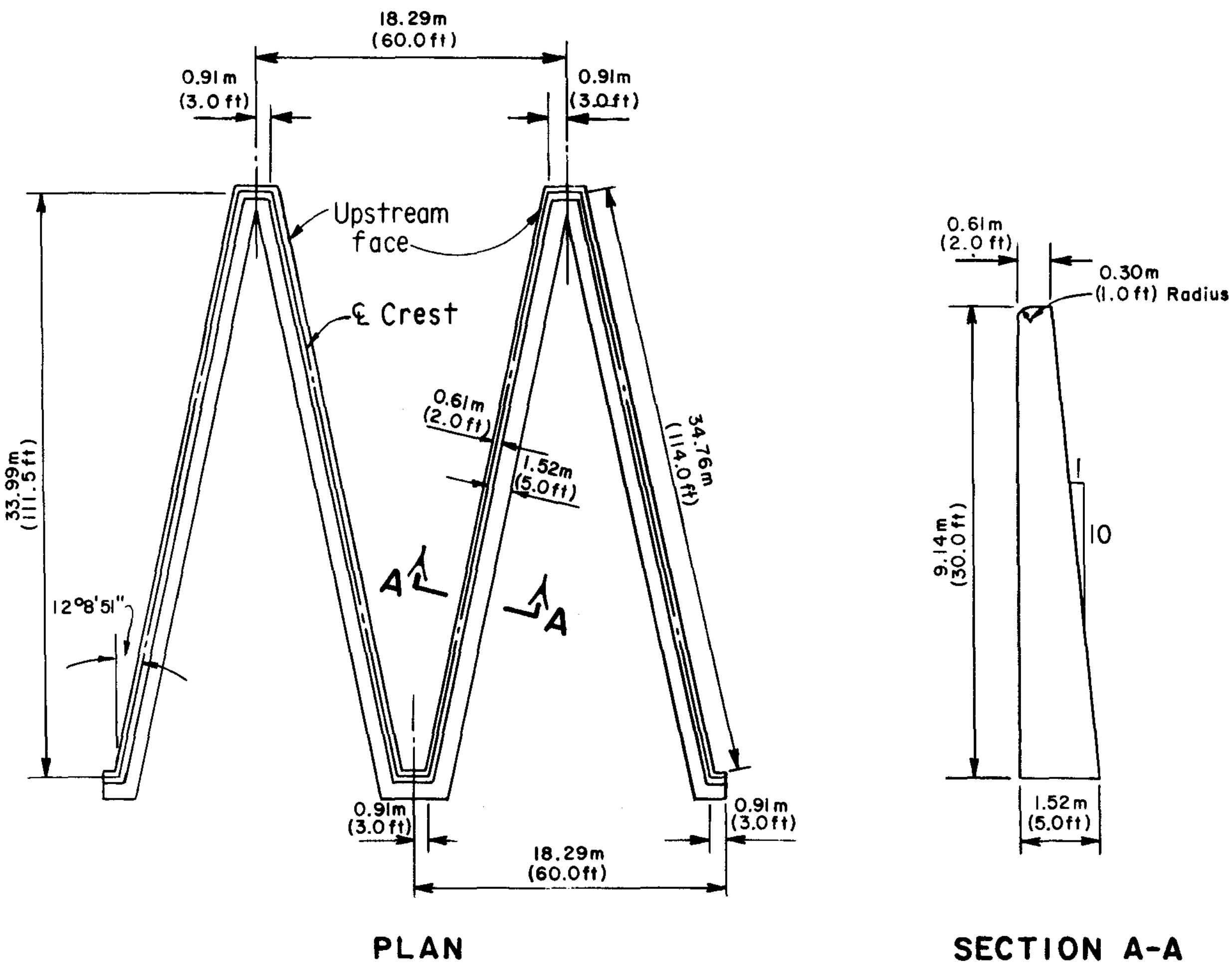


Figure 13.—Dimensions of the 14-cycle labyrinth spillway.

Initial testing of the 14-cycle spillway showed that more of the IDF could be passed at lower reservoir elevations. This allowed the maximum discharge at reservoir elevation 1160.07 m (3806.0 ft) to be lowered to 15 574 m^3/s (550 000 ft^3/s) from the previous 16 042 m^3/s (566 509 ft^3/s). The discharge coefficient at this reservoir elevation was 1.060 (1.920). The discharge curve is shown on figure 14.

A discharge of 2832 m^3/s (100 000 ft^3/s) is shown on figure 15. The overall view clearly shows the shape of the labyrinth as well as the flow pattern in the chute. The head over the spillway at 2832 m^3/s (100 000 ft^3/s) is only 1.34 m (4.4 ft). This appears to be a small discharge over a spillway of such length.

Figure 16 shows the spillway operating at 7787 m^3/s (275 000 ft^3/s), approximately one-half the maximum discharge. This discharge corresponds to 2.96 m (9.7 ft) of head over the spillway. At this discharge, the spillway operates efficiently with no localized head drop occurring as is observed at higher discharges.

At maximum discharge, indicated on figure 17, there was flow over the side channels and a high boil in the downstream channel of each cycle. Figure 18 shows the increase in velocity and corresponding drop in water surface caused by the small w/p ratio and the high reservoir head. A combination of nappe interference occurring over the upstream ends of the spillway and the scale effects at lower discharges was probably the cause of the slightly lower maximum discharge of 15 206 m^3/s (537 000 ft^3/s) obtained at reservoir elevation 1160.07 m (3806.0 ft). This discharge is only 368 m^3/s (13 000 ft^3/s) or 2.36 percent less than the maximum discharge. This should be acceptable as the maximum discharge is reached at reservoir elevation 1160.22 m (3806.5 ft).

The side channels were later modified to decrease the construction costs. The channels will be replaced by concrete walls located at the dam centerline normal to each abutment and occupying the same width as the initial side channels. The discharge remains essentially the same with no change in maximum reservoir elevation. The maximum discharge is directed by the wall into the chute downstream of the labyrinth spillway (fig. 19).

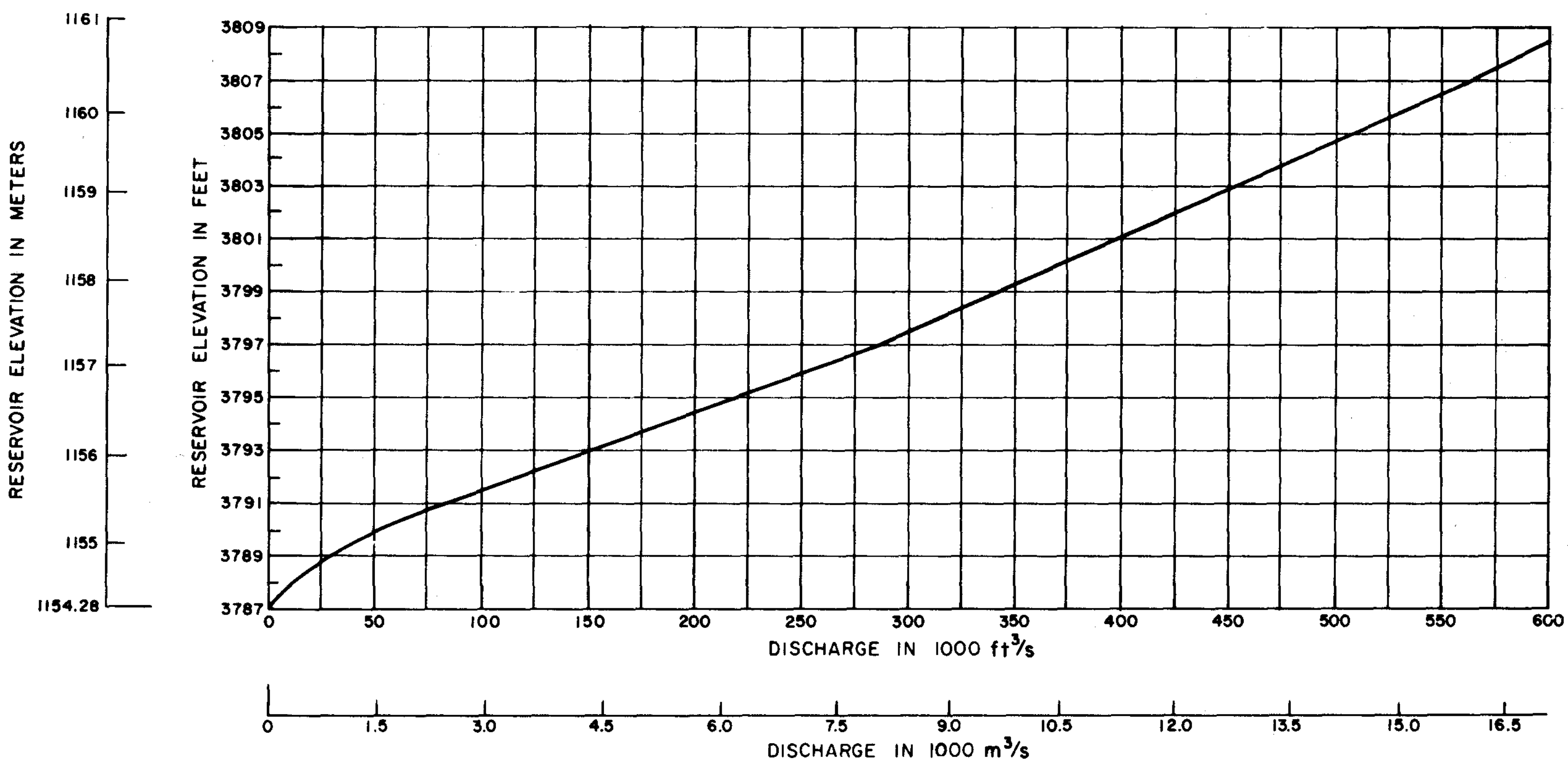
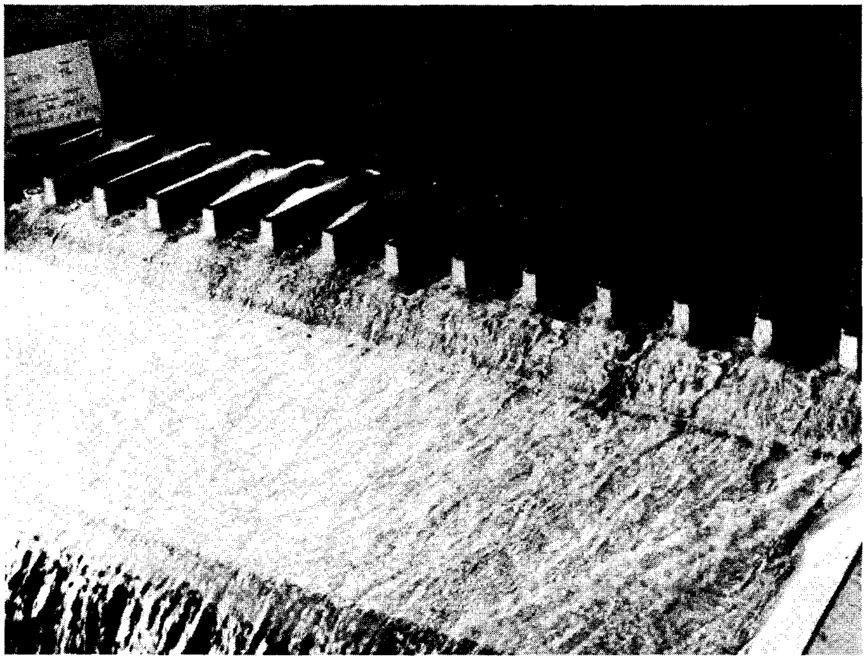
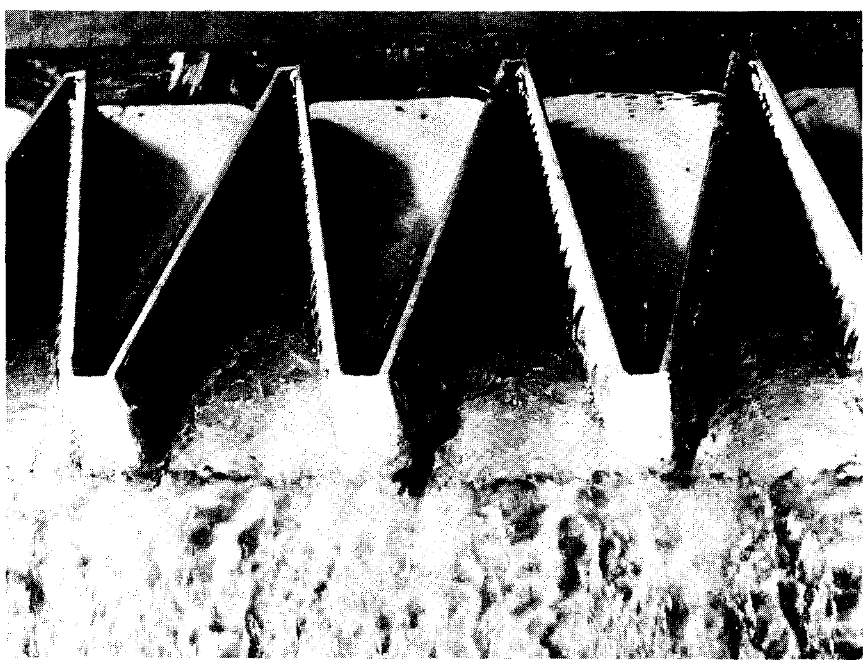


Figure 14.—Discharge curve for the 14-cycle labyrinth spillway.

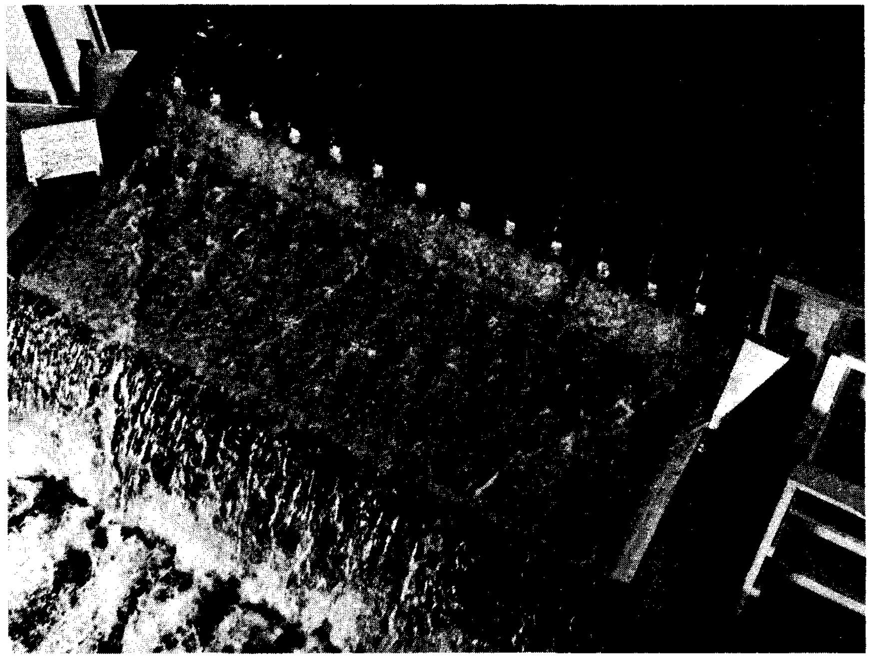


a. Overall view. P801-D-79842



b. Detailed view. P801-D-79843

Figure 15.—Fourteen-cycle labyrinth spillway, Q = 2832 m^3/s (100 000 ft^3/s), reservoir elevation 1155.62 m (3791.4 ft).



a. Overall view. P801-D-79844

b. Detailed view. P801-D-79845

Figure 16.—Fourteen-cycle labyrinth spillway, Q = 7787 m^3/s (275 000 ft^3/s), reservoir elevation 1157.23 m (3796.7 ft).

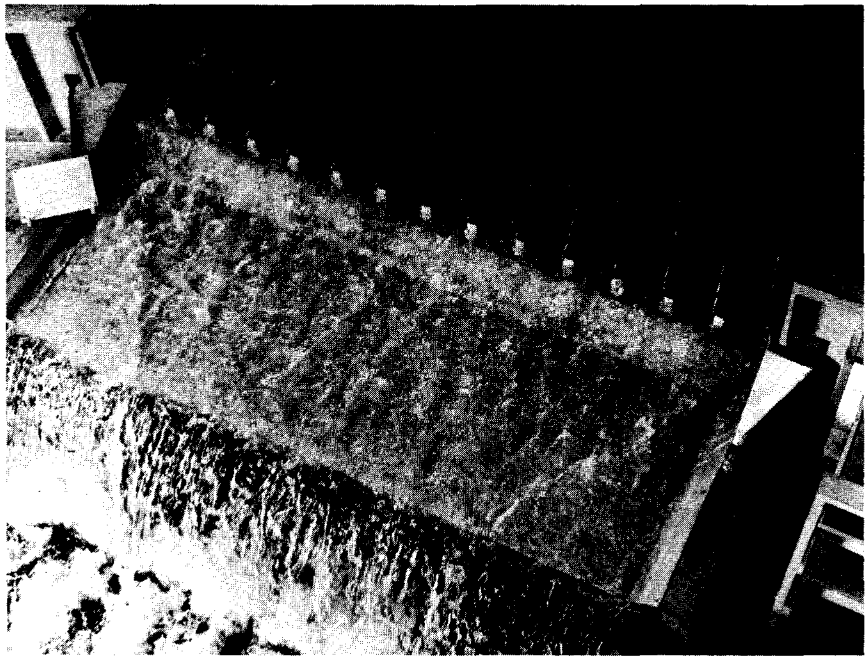


Figure 17.—Overall view of the 14-cycle labyrinth spillway, Q = 15 206 m³/s (537 000 ft³/s), reservoir elevation 1160.07 m (3806.0 ft). P801-D-79846

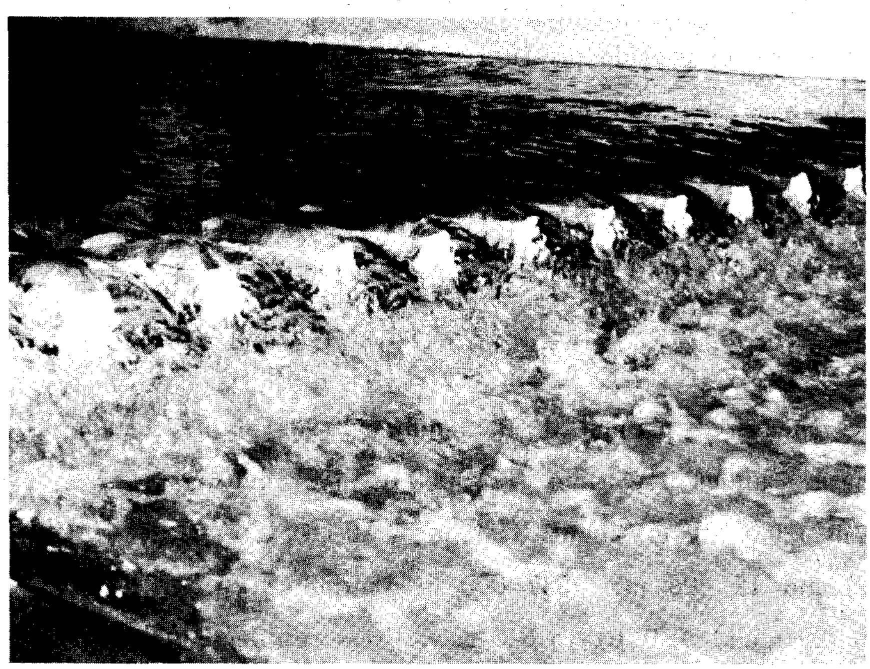


Figure 18.—Effect of the vertical aspect ratio, Q = 15 206 m³/s (537 000 ft³/s), reservoir elevation 1160.07 m (3806.0 ft). P801-D-79847

Figure 19.—Flow around wall and into chute for Q = 15 206 m³/s (537 000 ft³/s). P801-D-79848

Impact pressures in the downstream channels.—Piezometers were installed in the downstream channels to provide pressure profiles for use in the design of the structural support for the base and walls of the labyrinth spillway. The piezometers were placed along the centerline and parallel to the sidewalls in cycles 12 and 14 (fig. 20). Table 1 gives the pressure values and the graphical representations are shown on figures 21, 22, and 23. All pressures were measured using water manometers and showed very slight fluctuations.

The pressures measured parallel to the spillway walls showed similar trends in both cycles. Discharges under 3143 m³/s (111 000 ft³/s) produced pressure heads from just below 6.10 m (20.0 ft) to about 3.05 m (10.0 ft). Discharges between 6654 m³/s (235 000 ft³/s) and 15 206 m³/s (537 000 ft³/s) produced pressure heads between 7.62 and 13.72 m (25.0 and 45.0 ft), respectively. Pressures decreased as the channel width expanded.

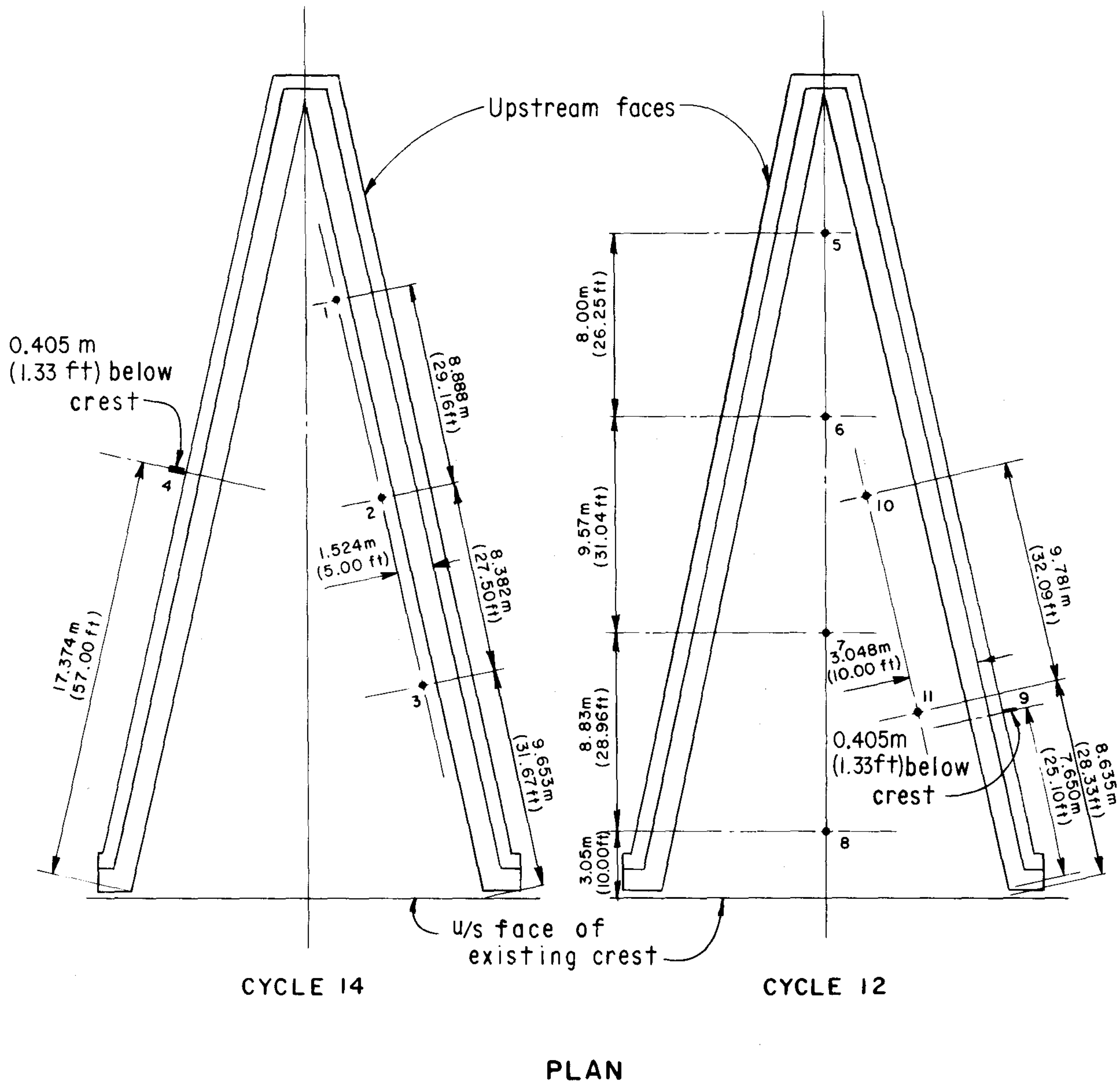
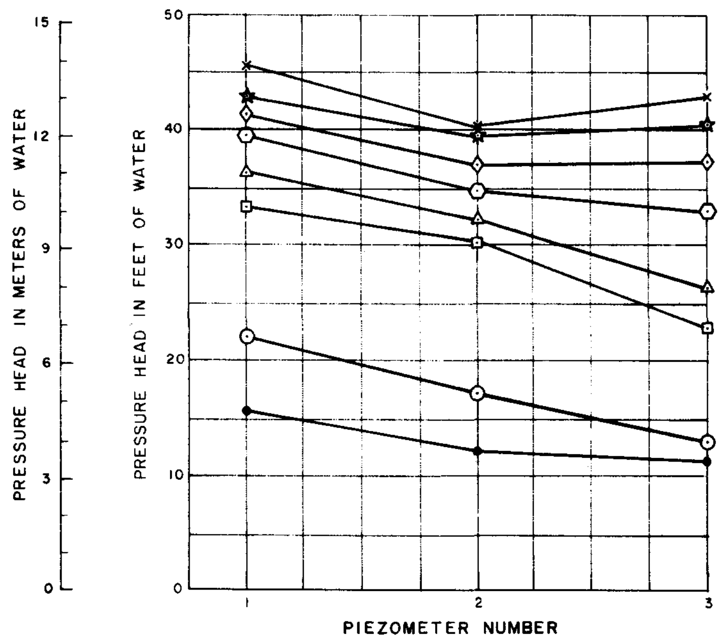


Figure 20.—Piezometer placement location map.

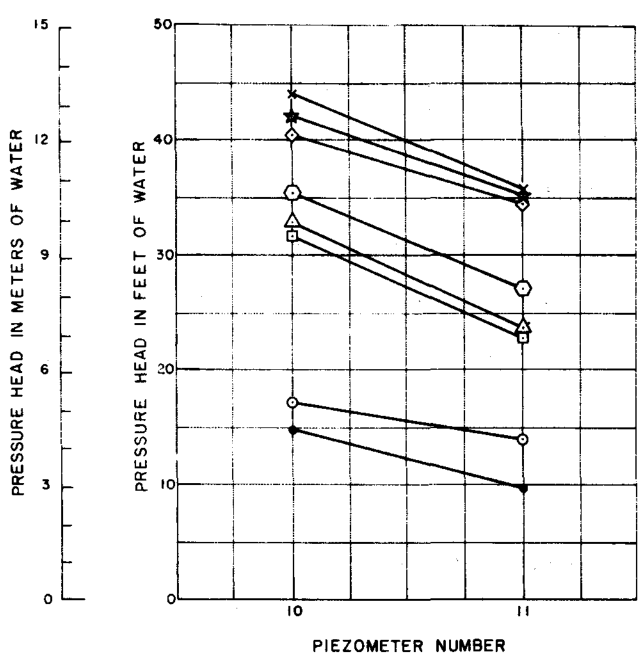
Table 1.—*Average water manometer pressure heads in downstream channels of cycles 12 and 14*

Reservoir elevation, m (ft)	1155.44 (3790.84)	1155.74 (3791.80)	1156.86 (3795.48)	1157.37 (3797.16)	1157.89 (3798.84)	1158.62 (3801.24)	1159.25 (3803.32)	1160.06 (3805.96)
Discharge, m^3/s (1000 ft^3/s)	2265 (80)	3143 (111)	6654 (235)	8155 (288)	9571 (338)	11 412 (403)	13 097 (462.5)	15 206 (537)
Piezometer	Pressure head in meters (feet) of water							
1	4.75 (15.60)	7.14 (23.44)	10.36 (34.00)	11.46 (37.60)	12.09 (39.68)	12.68 (41.60)	13.19 (43.04)	13.83 (45.36)
2	3.76 (12.32)	5.34 (17.52)	9.20 (30.20)	9.95 (32.64)	10.57 (34.68)	11.41 (37.44)	12.05 (39.52)	12.29 (40.32)
3	3.61 (11.84)	4.05 (13.28)	7.05 (23.12)	8.10 (26.56)	10.19 (33.44)	11.48 (37.68)	12.32 (40.40)	13.25 (43.48)
5	4.46 (14.64)	7.36 (24.16)	9.08 (29.80)	8.90 (29.20)	8.17 (26.80)	8.05 (26.40)	7.74 (25.40)	7.47 (24.52)
6	4.39 (14.40)	5.30 (17.40)	9.19 (30.16)	9.88 (32.40)	10.36 (34.00)	11.50 (37.72)	12.00 (39.36)	12.58 (41.28)
7	4.27 (14.00)	4.71 (15.44)	8.51 (27.92)	9.28 (30.44)	10.56 (34.64)	10.24 (33.60)	10.68 (35.04)	11.08 (36.36)
8	3.54 (11.60)	4.24 (13.92)	7.50 (24.64)	8.69 (28.52)	9.75 (32.00)	10.00 (32.80)	10.61 (34.80)	11.27 (36.96)
10	4.85 (15.92)	5.57 (18.28)	9.75 (32.00)	10.23 (33.56)	11.09 (36.40)	12.37 (40.60)	12.97 (42.56)	13.63 (44.72)
11	3.00 (9.84)	4.47 (14.68)	7.19 (23.60)	7.34 (24.08)	8.62 (28.28)	10.68 (35.04)	11.11 (36.44)	11.27 (36.96)



DISCHARGE SYMBOL	RESERVOIR HEAD ABOVE SPILLWAY CREST		DISCHARGE	
	ft	m	ft³/s	m³/s
•	3.84	1.17	80 000	2 265
⊙	4.80	1.46	111 000	3 143
⊡	8.48	2.58	235 000	6 654
△	10.16	3.10	288 000	8 155
⬡	11.84	3.61	338 000	9 571
◇	14.24	4.34	403 000	11 412
✩	16.32	4.97	462 500	13 097
×	18.96	5.78	537 000	15 206

Figure 21.—Graphical representation of pressure head measurements, cycle 12, piezometers 1 through 3.



DISCHARGE SYMBOL	RESERVOIR HEAD ABOVE SPILLWAY CREST		DISCHARGE	
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•	3.84	1.17	80 000	2 265
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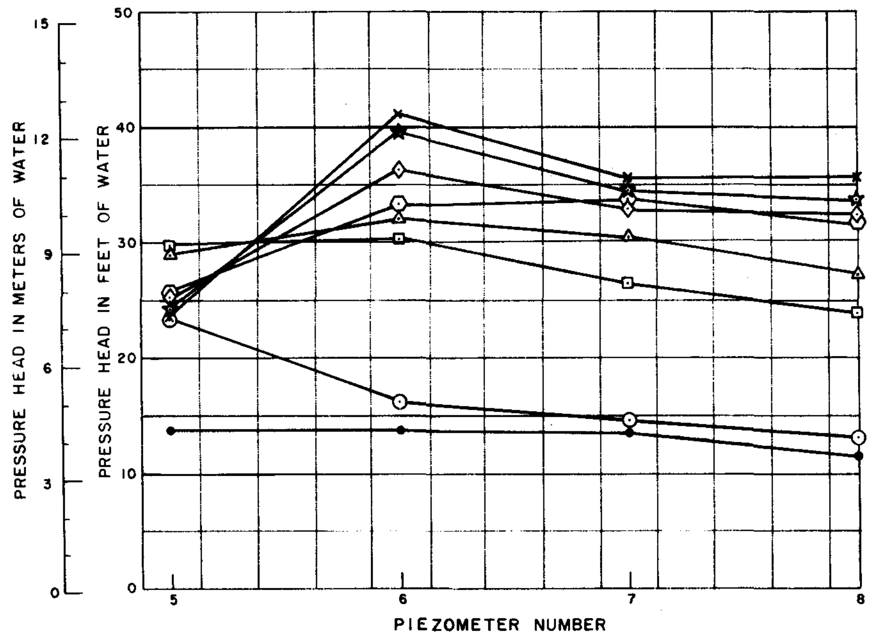
⊡	8.48	2.58	235 000	6 654
△	10.16	3.10	288 000	8 155
⬡	11.84	3.61	338 000	9 571
◇	14.24	4.34	403 000	11 412
✩	16.32	4.97	462 500	13 097
×	18.96	5.78	537 000	15 206

Figure 22.—Graphical representation of pressure head measurements, cycle 12, piezometers 10 and 11.



DISCHARGE SYMBOL	RESERVOIR HEAD ABOVE SPILLWAY CREST		DISCHARGE	
	ft	m	ft³/s	m³/s
•	3.84	1.17	80 000	2 265
⊙	4.80	1.46	111 000	3 143
⊡	8.48	2.58	235 000	6 654
△	10.16	3.10	288 000	8 155
⊙	11.84	3.61	338 000	9 571
◇	14.24	4.34	403 000	11 412
★	16.32	4.97	462 500	13 097
×	18.96	5.78	537 000	15 206

Figure 23.—Graphical representation of pressure head measurements, cycle 10, piezometers 5 through 8.

Pressures along the centerline of cycle 12 showed the same tendencies as those parallel to the wall. The only exception was piezometer 5 located near the upstream apex. The water height above the piezometer stayed fairly constant due to the contraction at the apex of the spillway. The small *w/p* ratio produces nappe interference and loss of efficiency at the higher flows. For all piezometers, the pressure heads ranged from about 4.57 to 12.19 m (15.0 to 40.0 ft), with the highest pressures usually associated with the largest discharges. Again, the pressures diminished as the channel expanded downstream.

Structural supports and water surface profiles in the upstream channels.—To add structural stability to the spillway walls, stiffeners were added in the upstream channels of each cycle. The stiffeners will be symmetrical about the centerline of the channels, with dimensions of 0.61 by 0.61 m (2.0 by 2.0 ft) and extending across the width of the

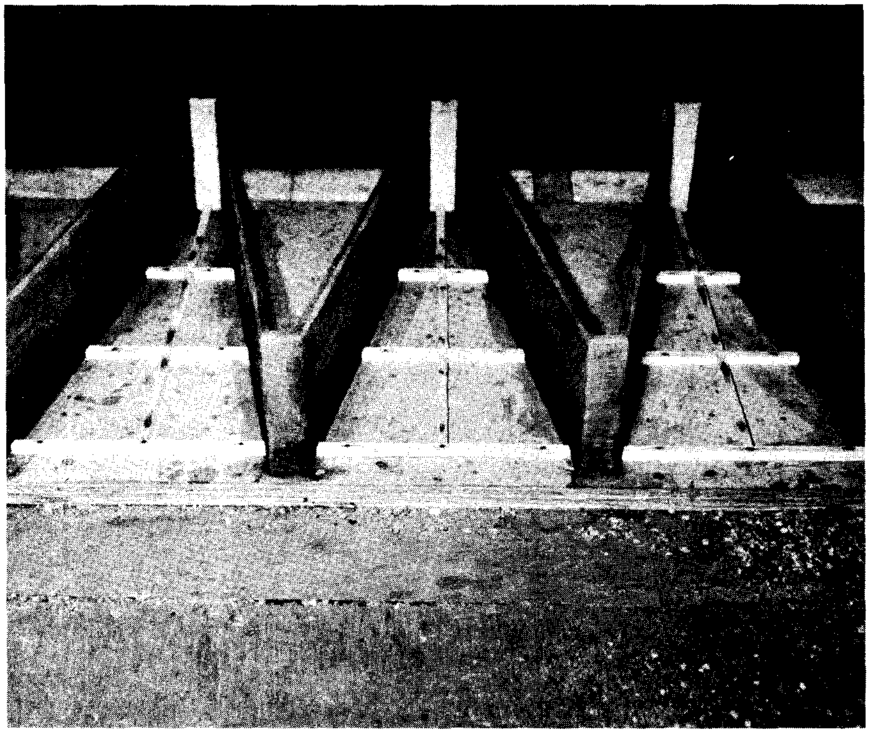


Figure 24.—Structural supports in the upstream channels of the spillway. P801-D-79849

channel in three locations. The layout is shown on figure 24. These supports did not affect the discharge capacity.

Water surface profiles at maximum discharge were measured along the centerline of the upstream channels of the spillway between cycles 7 and 8 and cycles 13 and 14. The water surface above the spillway may be seen on figure 18 and the measured profiles are shown on figure 25. The drop in water surface as flow approaches the spillway is due to the area contraction as flow enters the upstream channel, the head loss associated with the velocity head, and nappe interference. Nappe interference occurs over the upstream apexes of the spillway due to the small vertical aspect ratio. The irregularities in the water surface in the upstream channel between cycles 13 and 14 were caused by lateral flow from the embankment. This same phenomenon also was observed at the other end of the spillway.

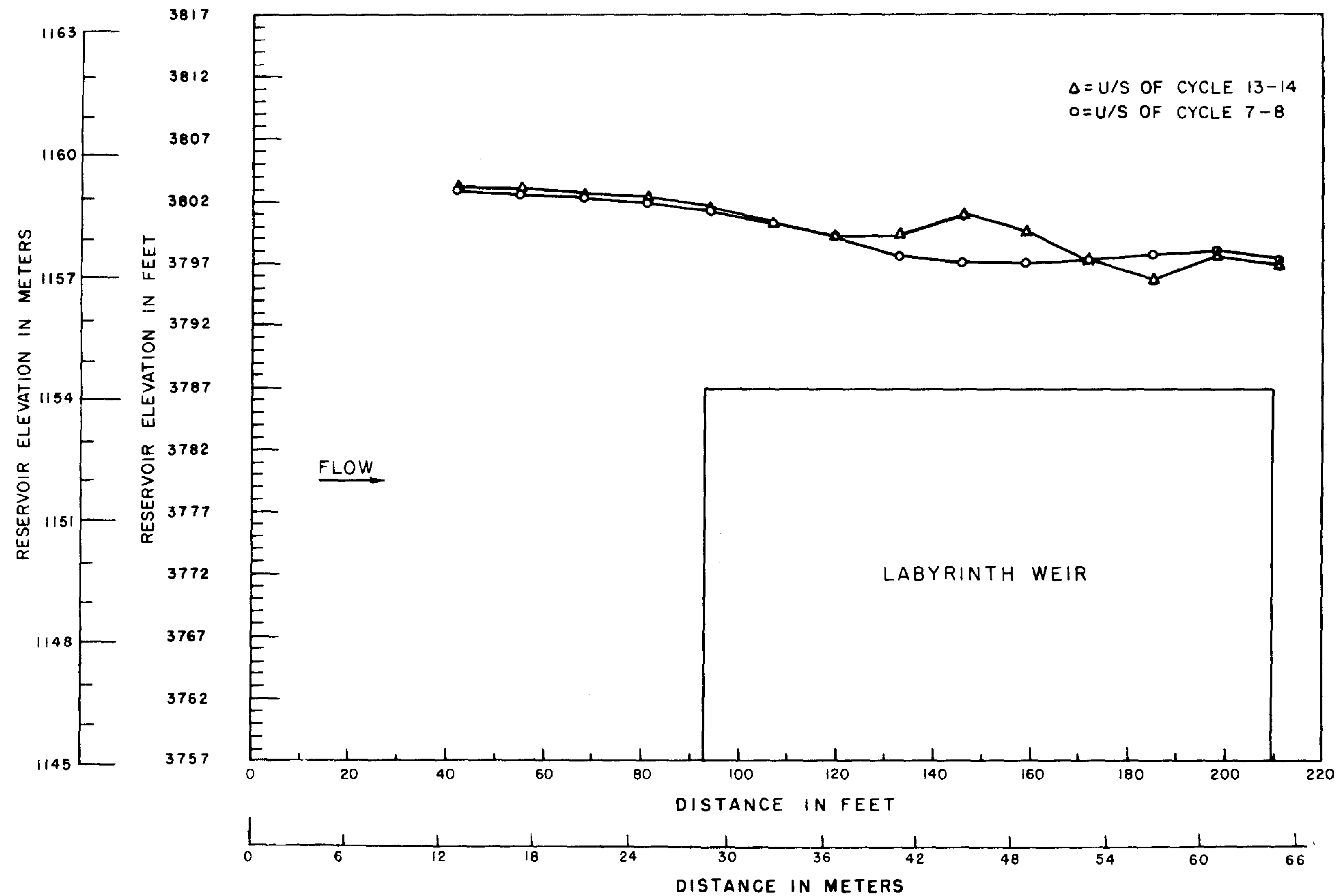


Figure 25.—Water surface profile at maximum discharge along the center line of the upstream channels.

Velocities adjacent to the embankment.—Flow velocities along the embankment on both sides of the spillway were measured. The velocities will be used to estimate the amount, size, and type of slope protection required. The locations of the measurements are shown on figure 26.

The plotted velocities are an average of readings taken at approximately two-tenths and eight-tenths of the depth. All measurements were taken at the maximum reservoir elevation of 1160.07 m (3806.0 ft). In general, the velocities increased as the spillway was approached. This trend may be seen on the graphs on figure 27. Maximum velocities, as high as 3.78 m/s (12.4 ft/s), were recorded near the spillway. Approximately 30.48 m (100.0 ft) away from the spillway along the embankment the velocities were considerably lower. Velocities were higher on the left side than on the right side, probably because of the topography.

Splitter piers.—At Avon Dam in Australia [1], strong nappe vibrations have occurred at low discharges over the spillway. This was accompanied by low frequency noise and explosive bangs. As a result of this situation, pressures were taken 0.405 m (1.33 ft) below the top of the crest on the downstream face of the 12th and 14th spillway cycles. The locations are shown as piezometers 4 and 9 on figure 20. Pressures in the 14th cycle were subatmospheric for the low discharge range. The largest subatmospheric pressure head, 1.32 m (4.3 ft), occurred at a discharge of about 3143 m^3/s (111 000 ft^3/s) under 1.46 m (4.8 ft) of head. All the pressure heads are given in table 2 with a graphical representation shown on figure 28. Even though none of the pressure heads were substantially subatmospheric, aeration of the nappe is recommended.

Splitter piers to provide aeration were modeled. Several locations for the piers were tested. These included the center of the side lengths, b, the downstream apexes, and $0.25b$ upstream of the downstream tips. These locations did not provide complete aeration of the cycle. The recommended location and dimensions for the piers are shown on figure 29. This location, 3.35 m (11.0 ft) upstream from the downstream apexes along

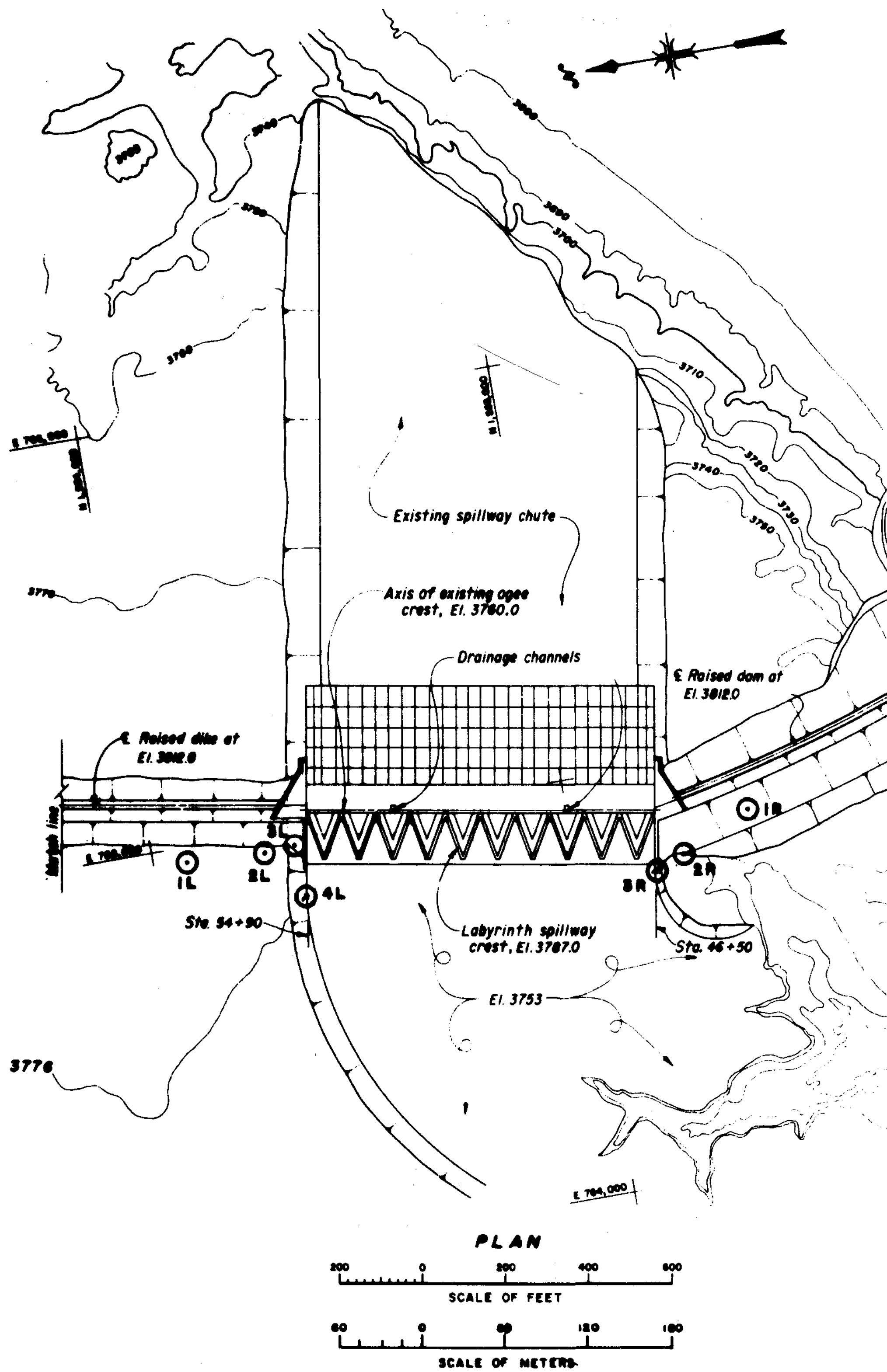
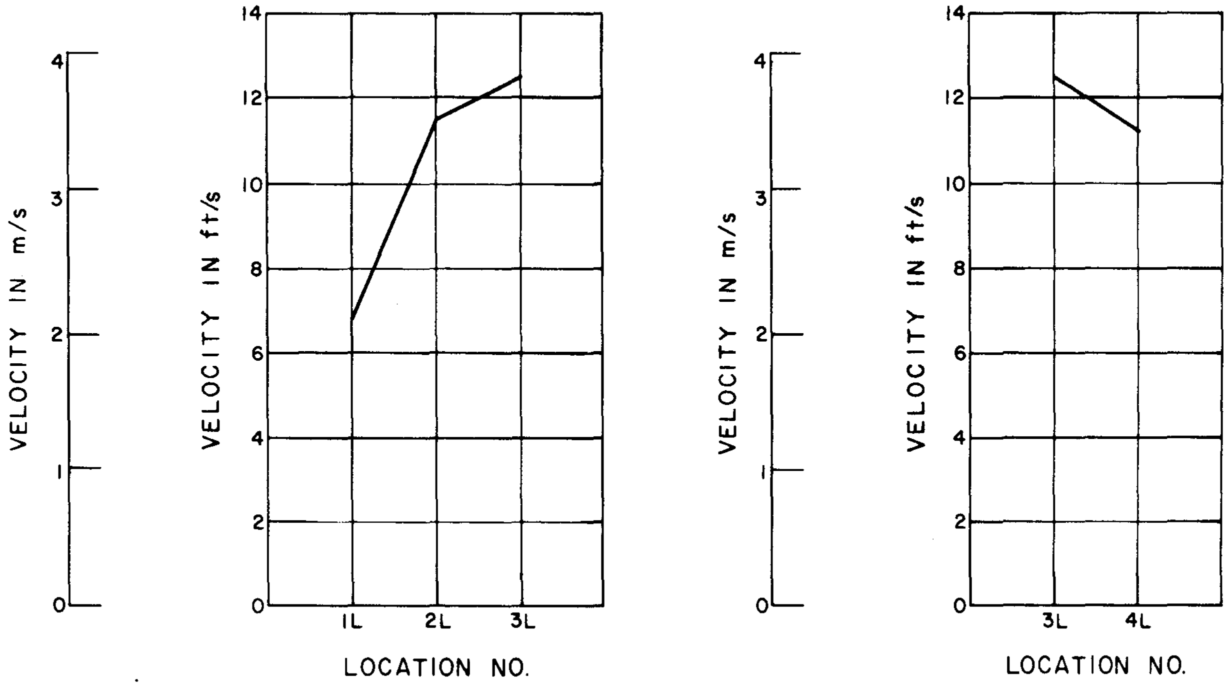
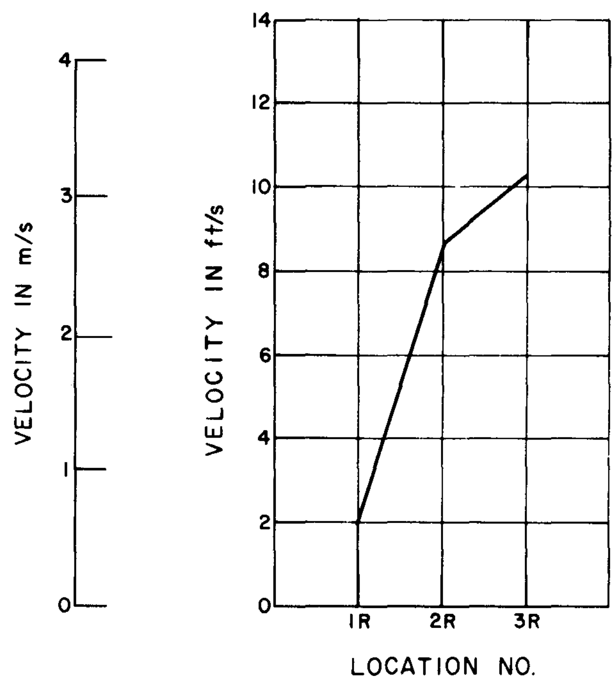


Figure 26.—Velocity measurement location map.

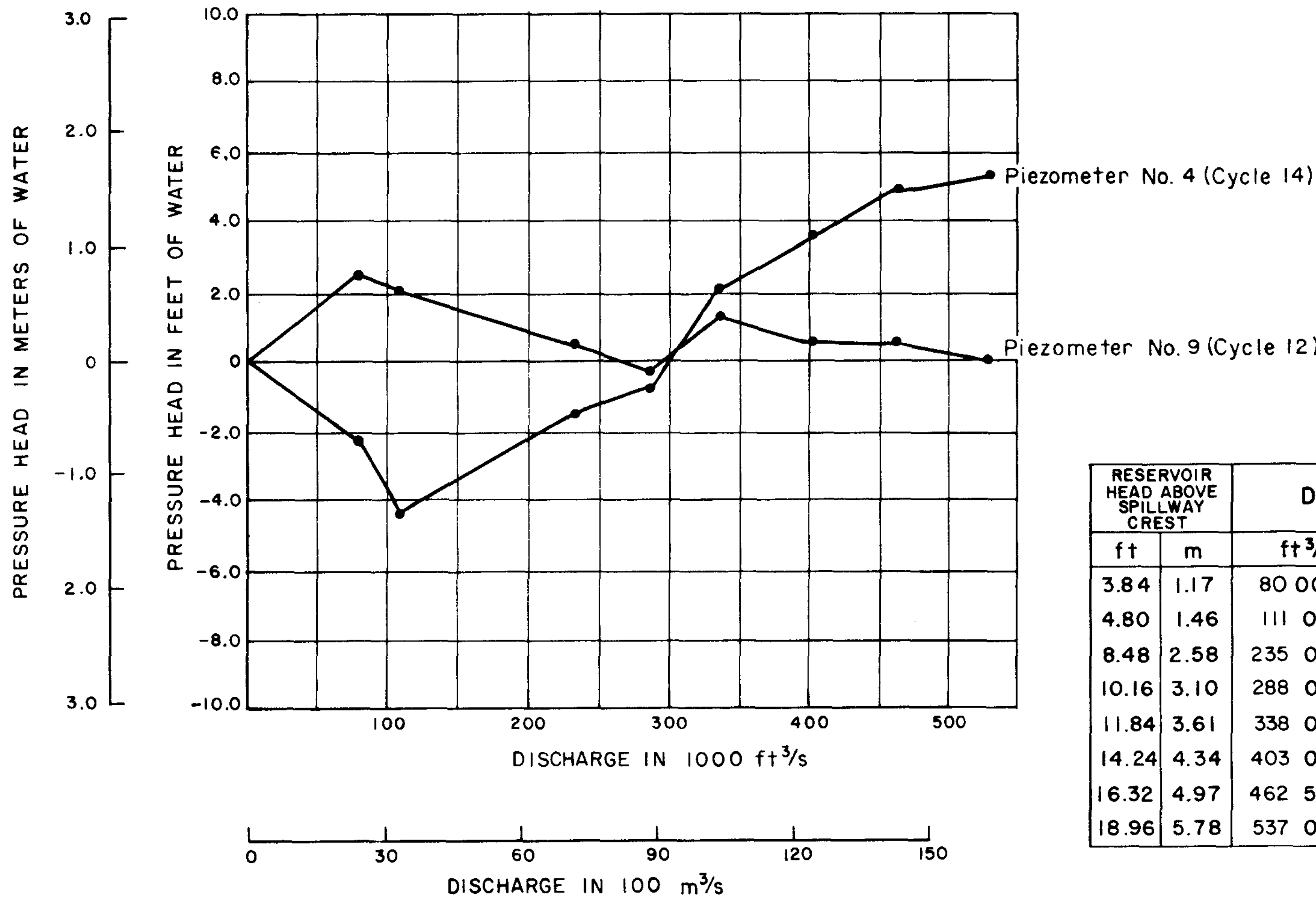




LOCATION* NO.	VELOCITY ft/s	VELOCITY m/s
1L	6.78	2.07
2L	11.52	3.51
3L	12.39	3.78
4L	11.17	3.40
1R	1.92	0.59
2R	8.69	2.65
3R	10.22	3.12

* See location map, figure 26.

Figure 27.—Graph of velocities adjacent to the embankment.



RESERVOIR HEAD ABOVE SPILLWAY CREST		DISCHARGE	
ft	m	ft^3/s	m^3/s
3.84	1.17	80 000	2 265
4.80	1.46	111 000	3 143
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10.16	3.10	288 000	8 155
11.84	3.61	338 000	9 571
14.24	4.34	403 000	11 412
16.32	4.97	462 500	13 097
18.96	5.78	537 000	15 206

Figure 28.—Graphical representation of pressure measurements, piezometers 4 through 9.

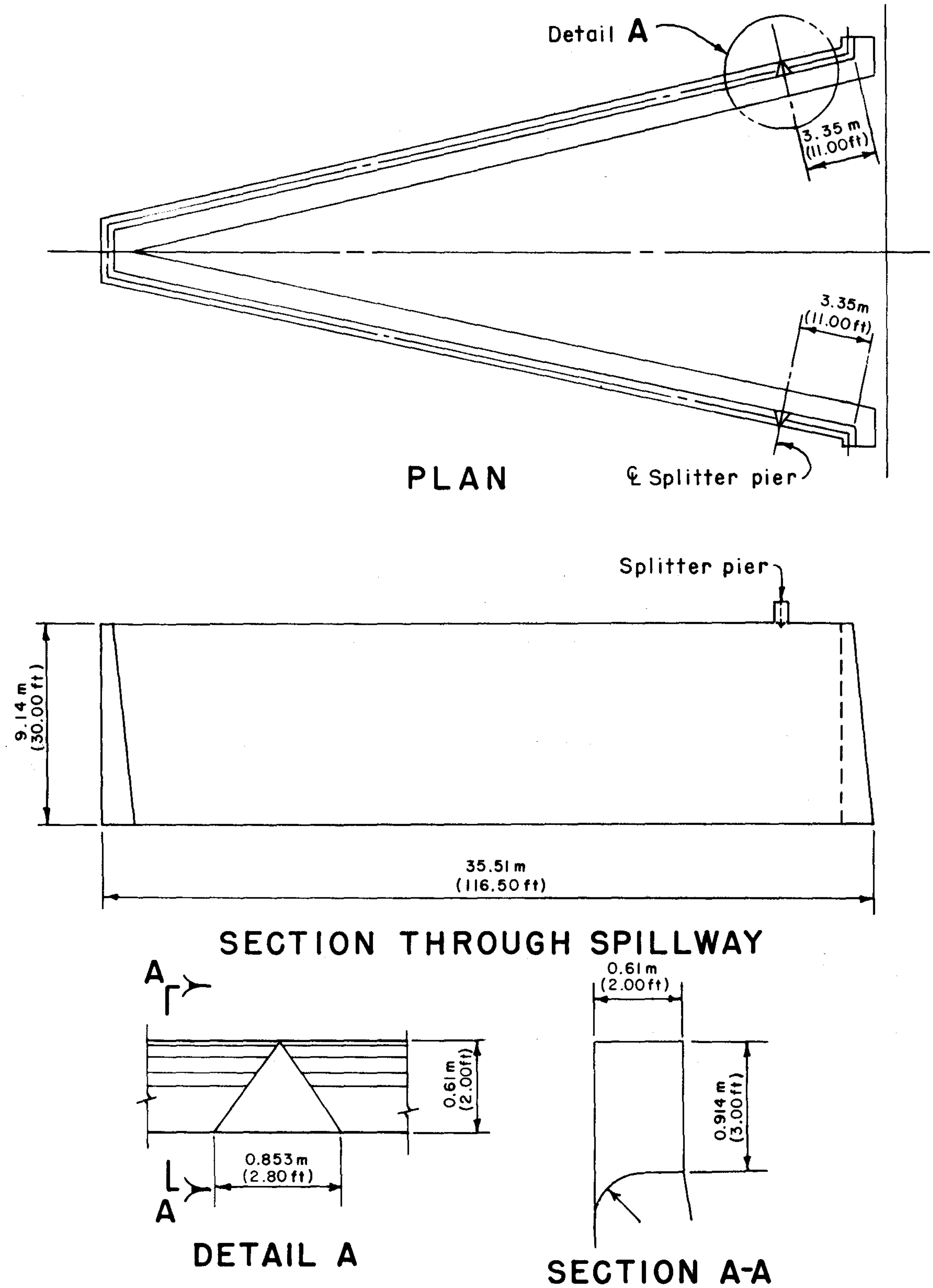

Figure 29.—Splitter pier location and dimensions.

each side length, b, provide complete aeration up to a discharge of 2406.9 m^3/s (85 000 ft^3/s) when the piers are overtopped. Their effectiveness may be seen on figure 30 where the spillway is operating under a discharge of 1841 m^3/s (65 000 ft^3/s). (Note the nappe clinging to the spillway wall on the adjacent cycle.)

Table 2.—*Average transducer impact pressure heads on downstream faces of cycles 12 and 14*

Reservoir elevation, m (ft)	1155.44 (3790.84)	1155.74 (3791.80)	1156.86 (3795.48)	1157.37 (3797.16)	1157.89 (3798.84)	1158.62 (3801.24)	1159.25 (3803.32)	1160.06 (3805.96)
Discharge, m^3/s (1000 ft^3/s)	2265 (80)	3143 (111)	6654 (235)	8155 (288)	9571 (338)	11 412 (403)	13 097 (462.5)	15 206 (537)
Piezometer	Pressure head in meters (feet) of water							
4	−0.71 (−2.32)	−1.32 (−4.32)	−0.46 (−1.52)	−0.22 (−0.72)	−0.63 (2.08)	1.06 (3.48)	1.49 (4.88)	1.61 (5.28)
9	0.76 (2.48)	0.63 (2.08)	0.15 (0.48)	−0.10 (−0.32)	0.39 (1.28)	0.15 (0.48)	0.15 (0.48)	0.00 (0.00)

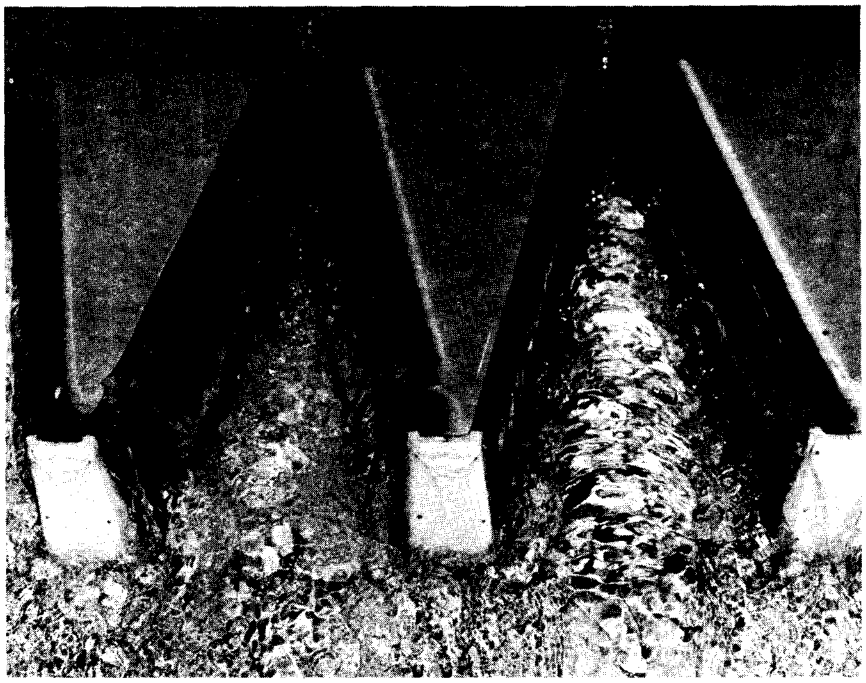

Figure 30.—Discharge of 1841 m^3/s (65 000 ft^3/s) aerated by splitter piers. P801-D-79850

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[2] Darvas, Louis A., Discussion on "Performance and Design of Labyrinth Weirs," J. of the Hyd. Div., ASCE, pp. 1246-1251, August 1971.

[3] Taylor, Geoffrey, "The Performance of Labyrinth Weirs," Ph. D. thesis, University of Nottingham, October 1968.

[4] Hay, Nessim, and Geoffrey Taylor, "A Computer Model for the Determination of the Performance of Labyrinth Weirs," 13th Congress of the International Association of Hydraulic Research, pp. 36-378, Kyoto, Japan, September 1969.

APPENDIX

SUPPLEMENTAL MODEL TESTS

Stability of end cycle sidewalls

The structural stability of the long labyrinth sidewalls of the end cycles (1 and 14) was a major design concern. Hydrodynamic forces tend to pull the walls away from the abutments at the point of attachment. Several solutions to this problem were investigated by the designers and the two that required physically changing the structure were tested in the model. The modifications were (1) a backfilled section 4.57 m (15.0 ft) deep and approximately 22.35 m (73.0 ft) long between the sidewall and the abutment and (2) a gravity wall replacing the downstream apex, 10.36 m (34.0 ft) long and 3.75 m (12.3 ft) wide, sloping from the labyrinth crest down to the floor. The vertical upstream face of the wall was 11.28 m (37.0 ft) upstream from the original apex.

The prototype backfill was modeled with pea gravel. The maximum width of the sections was at the upstream apexes between the sidewalls and the abutments and diminished with the sidewall angle until an approximate width of 3.05 m (10.0 ft) was attained. The 4.57-m (15.0-ft) depth of those backfilled sections reduced the spillway sidewall height in those areas by one-half [p = 4.57 m (15.0 ft)]. This caused high velocity flow over the walls and a large horizontal vortex formed that at times was the full length of the cycle. This is shown on figure A1 for a discharge of 15 206 m^3/s (537 000 ft^3/s). The vortex was a sign of unstable flow which made the modification unsatisfactory.

Gravity walls were installed along the abutments in place of the apexes of the end cycles. The gravity wall is shown on figure A2. These walls reduced the crest length by approximately 18.9 m (62 ft). The spillway length reduction may be seen by comparing figures A1 and A2. This reduction in length increased the reservoir elevation by 0.116 m (0.38 ft) at the maximum discharge. The designers decided not to use the gravity walls because of this increase in reservoir elevation and made other structural design modifications to ensure the stability of the end walls.

Figure A1.—Vortex in cycle 1. Backfill placed between the sidewall and the abutment, $Q = 15\,206\ m^3/s$ ($537\,000\ ft^3/s$), reservoir elevation 1160.07 m (3806.0 ft). P801-D-79851

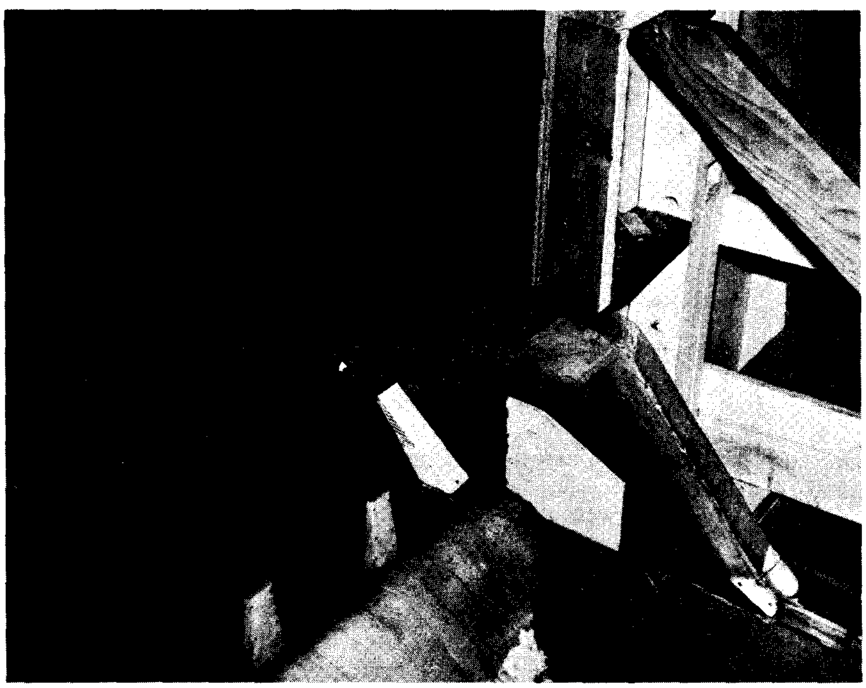

Figure A2.—Gravity wall replacing the downstream apex of the labyrinth spillway. P801-D-79852

GPO 836-909

Tailwater and velocity measurements downstream of the spillway

Tailwater profiles and velocity measurements were taken because of the concern over the type of erosion protection required in the downstream channels between the labyrinth spillway and the ogee crest. The area is composed of very strong sandstone that is not highly fractured. The tests were to determine whether the entire area should be protected with a concrete slab or just the area 1.83 m (6.0 ft) immediately downstream of the labyrinth spillway parallel to the sidewalls as originally designed.

Tailwater elevations were obtained at the left abutment and along the centerlines of cycles 9 and 10. The highest tailwater elevation was 1154.28 m (3787.0 ft) at the left abutment. A depression forms in the center of the downstream channel area where the two nappes impinge with tailwater elevation about 1152.57 m (3781.4 ft). The tailwater elevation increases to about 1153.70 m (3785.1 ft) as the flow approaches the ogee crest.

Velocities were measured in cycle 11 in the same locations as the tailwater measurements. The velocity decreased as the channel width expanded. Velocities were 9.45 m/s (31.0 ft/s) midway into the cycle and 4.45 m/s (14.6 ft/s) at the downstream apex.

On the basis of these data, the designers decided to excavate all loose material. A protective slab will be placed over the area 1.83 m (6.0 ft) immediately downstream of the spillway as originally designed. Concrete backfill will be placed only in those areas where strong sandstone was found below elevation 1143.91 m (3753.0 ft).

Mission of the Bureau of Reclamation

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Bureau programs most frequently are the result of close cooperation with the U.S. Congress, other Federal agencies, States, local governments, academic institutions, water-user organizations, and other concerned groups.