GR-82-13

HYDRAULIC MODEL STUDY OF HYRUM DAM AUXILIARY LABYRINTH SPILLWAY

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16. ABSTRACT

The updated IDF (inflow design flood) for Hyrum Reservoir could not be passed with the existing hydraulic structures. The labyrinth was the most economical alternative for an auxiliary spillway. The labyrinth spillway, a series of trapezoidal shapes in plan form, would provide the necessary spillway length and capacity within a comparatively small width. The spillway configuration was based on design curves developed in the Bureau of Reclamation Hydraulic Laboratory.

The 1:30 scale model included the upstream approach channel, the labyrinth spillway, and a transition section leading to a long sloping chute. The model confirmed the maximum discharge of the spillway and investigated different approach conditions and spillway orientations. Water surface profiles were measured over the spillway and in the chute downstream of the spillway. Splitter piers were installed to provide aeration under low flow conditions.

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UNITED STATES DEPARTMENT OF THE INTERIOR ★ BUREAU OF RECLAMATION

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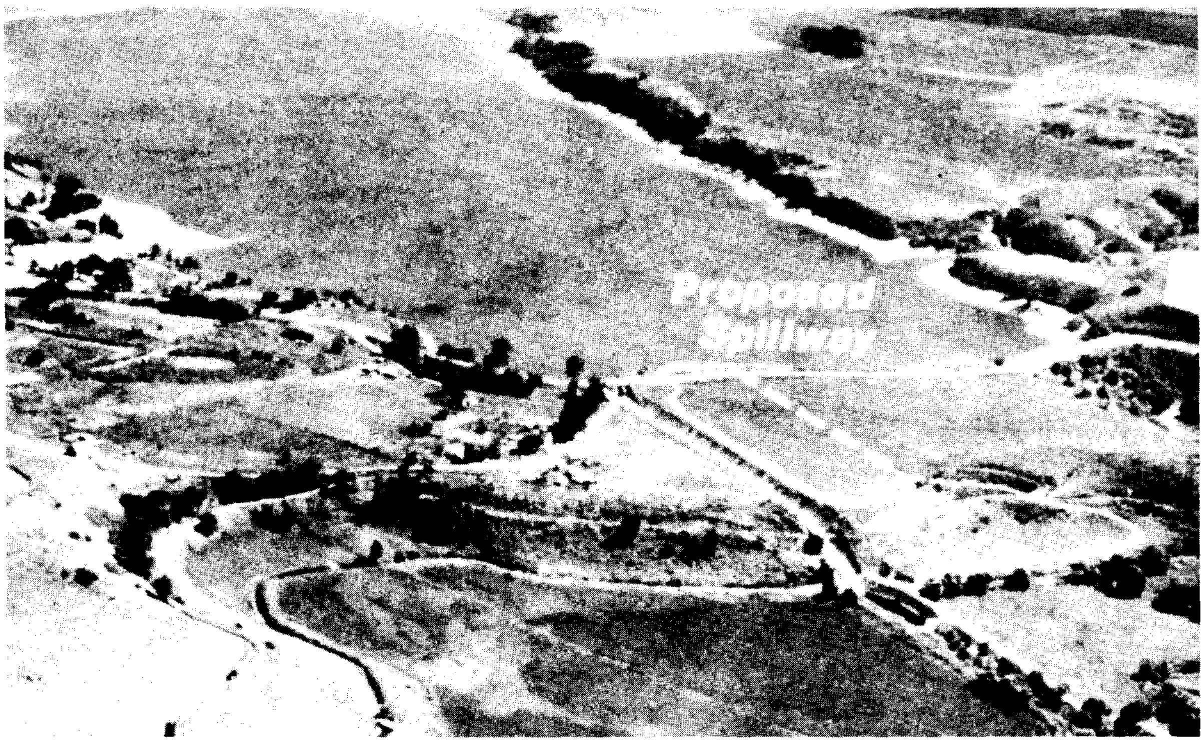

Frontispiece.—Hyrum Dam, with location of proposed auxiliary spillway. P801-D-80189

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INTRODUCTION

Hyrum Dam, located on the Little Bear River 10 miles south of Logan, Utah, was completed in 1935. The earthfill dam is 540 ft long with a 50-ft-wide gated spillway and a concrete-lined tunnel outlet works. The capacity of the spillway is 5600 ft^3/s and the outlet works capacity is 300 ft^3/s, at the normal reservoir El. (elevation) of 4672.0 ft. The primary purpose of the reservoir is to provide irrigation water.

The IDF (inflow design flood) for Hyrum Dam was revised in 1981. The existing hydraulic structures could not pass this revised flood. The preferred alternative to pass the increased IDF was to construct an ungated auxiliary labyrinth spillway. The spillway will be located approximately 500 ft from the right abutment of the dam, nearly parallel to the existing spillway (fig. 1). The 300-ft-long labyrinth spillway will be built in two cycles within a 60-ft width. A 40-ft-wide, 683-ft-long concrete chute and a 143-ft-long stilling basin will be downstream. The spillway will be 12 ft high with a crest El. of 4672.0 ft. When discharges are required, the existing spillway will be operated initially, with a flow over the labyrinth spillway occurring when the normal water surface of 4672.0 ft is surpassed. The capacity of the labyrinth spillway is 9830 ft^3/s at maximum reservoir El. 4678.0.

The hydraulic model study was performed to ensure passage of the maximum discharge at maximum reservoir El. 4678.0, and to study the spillway approach conditions, water surface profiles, and flow patterns in the chute.

The laboratory work associated with this study was begun in December 1981 and completed in March 1982.

CONCLUSIONS

1. The flume tests for length magnification of five showed that the labyrinth weir orientation of two apexes downstream was slightly more efficient hydraulically than that of two apexes upstream. However, tests done with the Hyrum spillway model showed that a good entrance condition had more effect on spillway efficiency than spillway orientation.

2. The square-cornered labyrinth spillway entrance used in tests 1, 1A, 2, and 3 produced unacceptable flow conditions. This entrance reduced the efficiency of the spillway's upstream apexes and side lengths near the entrance corners.

3. The recommended labyrinth spillway entrance, placement, and orientation are shown on figures 1 and 2. The curved entrance is formed by a 21-ft radius between Stas. (survey stations) 1+44 and 1+65. At Sta. 1+65, the walls begin a 1:0.12 convergence that continues through the transition section to the 40-ft wide spillway chute at Sta. 2+80. The spillway will be placed with the two upstream apexes extending into the reservoir to Sta. 1+25.

This spillway configuration passed the maximum discharge, 9050 ft^3/s at reservoir El. 4677.5 (H = 5.5 ft), which is 0.5 ft below the maximum reservoir El. 4678.0.

4. For the recommended spillway design, cross waves and turbulence in the chute caused overtopping of both sidewalls during operation at maximum discharge. Most sidewall overtopping occurred between Stas. 2+73 and 2+87, the end of the chute transition section. The wall height should be raised 1.5 ft between these stations. Overtopping also occurred between Stas. 3+34 and 3+55 and between Stas. 4+47 and 4+82. The wall height should also be raised in these areas, or erosion protection should be placed along both sides of the chute. The alternatives to raising the wall heights in these areas are to: (a) lengthen the transition section, or (b) widen the downstream chute which will lessen the contraction. Neither would be cost effective. Turbulent flow immediately downstream of the spillway would cause considerable spray in the prototype.

5. Splitter piers on the spillway crest will provide aeration and prevent nappe oscillation during low discharge operation. These piers should be located 6.0 ft upstream of the downstream apexes on each spillway side length. A pier height of 1.5 ft will provide adequate aeration for discharges up to at least 1765 ft^3/s.

APPLICATION

Although the model study was site specific, the concept of the labyrinth spillway has many applications. Labyrinth spillways are economical because a long spillway crest may be compressed into a narrow width which allows reduction of the head required to pass a large discharge. The spillway height provides reservoir storage as an alternative to a gated structure. The Hyrum spillway utilizes all of these features. The required 300-ft spillway crest length is contained in a 60-ft width with the spillway height providing storage up to the present normal water surface and free overflow of the updated IDF as the reservoir level exceeds 4672.0 ft.

THE MODEL

The 1:30 scale model included the auxiliary two-cycle labyrinth spillway, about 240 ft of reservoir topography on either side of the spillway, the transition section, and the spillway chute. The model included only the prototype features that would affect the spillway performance to allow the labyrinth spillway to be modeled as large as possible in the available laboratory space. The reservoir topography was modeled using concrete, with areas where changes might be required modeled in plywood, metal, or pea gravel. The spillway chute and walls were modeled in plywood. All the labyrinth spillways were modeled with mahogany. Water was supplied to the model by the permanent laboratory system and measured with venturi meters.

The prototype labyrinth spillway will be 12 ft high with a vertical upstream face and a 1:16 batter on the downstream face. The crest will be 1.0 ft wide at the top with 0.5-ft radius forming a rounded upstream edge. The dimensions of the two-cycle spillway varied with the several designs tested; however, the total length remained constant. Two transition shapes were tested, both leading to a 40-ft-wide chute with a 0.04 slope. The chute terminates in a stilling basin, which was not modeled. An overall view of the model is shown on figure 3 with the recommended labyrinth spillway, reservoir approach, and transition section installed.

SIMILITUDE AND TEST DISCHARGES

The hydraulic model was designed with identical Froude numbers between the model and prototype reflecting similitude of gravitational effects. The model scale ratio of 1:30 resulted in the following parameters:

Parameter	*Ratio*
Vertical and horizontal lengths	$L_r = 1:30$
Velocity	$V_r = L_r^{1/2} = (1:30)^{1/2} = 1:5.477$
Area	$A_r = L_r^2 = (1:30)^2 = 1:900$
Volume	$V_r = L_r^3 = (1:30)^3 = 1:27000.$
Discharge	$Q_r = V_rA_r = L_r^{1/2}L_r^2 = (L_r)^{5/2} = 1:4929.5$

For example, a prototype discharge of 9050 ft³/s equals a model discharge of:

$$\frac{9050 \text{ ft}^3/\text{s}}{30^{5/2}} = 1.836 \text{ ft}^3/\text{s}$$

For documentation purposes, representative discharges of 2000, 4000, 6000, and 9050 ft³/s were tested for each model change.

TEST PLAN

The primary purpose of the model study was to verify the ability of the spillway to pass the maximum discharge, 9050 ft³/s, within the maximum head of 6.0 ft which occurs at reservoir El. 4678.0. Discharge capacity data were taken for each orientation and dimension change of the labyrinth spillway and each spillway entrance configuration. Each change had a significant effect on the flow distribution and the spillway efficiency. Water surface profiles and photographs were taken upstream of the spillway and in the transition and chute downstream of the spillway to document the flow distribution caused by the spillway orientation and entrance changes.

The design of the two-cycle labyrinth spillway was based on the procedure developed by Hay and Taylor [1]* and modified design curves developed in the USBR

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* Numbers in brackets indicate entries in Bibliography.

Hydraulic Laboratory [2]. Flume tests to determine the effect of the labyrinth spillway orientation (one or two apexes downstream) were completed prior to installing a labyrinth spillway in the Hyrum model. These tests showed that the spillway orientation with two apexes downstream was slightly more hydraulically efficient. The Hyrum model provided the opportunity to study the effect of approach conditions on the spillway orientation. The flume test results were compared to the results obtained in the Hyrum spillway model.

A schematic of each approach condition and spillway orientation is shown on figure 2, referred to as tests 1, 1A, 2, 2A, 3, and 4. The recommended design is the one used in test 4.

LABYRINTH SPILLWAY TESTS

Initial Labyrinth Spillway Design (Tests 1 and 1A)

The initial design, test 1, is shown on figures 2 and 4. Tests 1 and 1A were done with the same labyrinth orientation and dimensions. The difference between the two tests was the placement of the 2:1 sloped area adjacent to the square-edged entrance. In test 1 the slope started 18 ft back from the corners of the entrance and in test 1A the slope began at the entrance corner. The toe of the slope was at El. 4660.0 in both tests. The upstream apexes of the spillways were located 5.0 ft downstream from the square-edged entrance at Sta. 1+49.

The approach configuration in test 1A provided a slightly better flow distribution which produced a slightly higher spillway discharge capacity. The difference in discharge was only 4.0 percent; however, neither test configuration passed the maximum design discharge of 9050 ft^3/s. At reservoir El. 4678.0 (H = 6.0 ft), the maximum discharges for tests 1 and 1A were 8625 and 8970 ft^3/s, respectively. Test 1A was close to passing the maximum discharge; however, the square-edged entrance caused contractions at the sidewalls and turbulent flow distribution in the upstream channels (fig. 5).

The contraction of the flow had two adverse effects: the flow was directed away from the wall toward the center of the spillway channel, and the water surface in the upstream spillway channels showed unusual flow distribution. The effective length of spillway crest was reduced by about 22 ft, as little flow occurred over each apex and spillway length near the channel sidewalls. The water surface profile showed two drops: one at the entrance to the spillway channel, and another in both upstream channels of the labyrinth. These drops were due to an increase in velocity as the flow accelerated. The first drop in water surface was due to the area contraction as subcritical flow from the reservoir entered the narrower width of the spillway opening. Normally, this initial drop in water surface would be followed by a slight increase and then a gradual decrease in water surface as the water flowed over the sidewalls of the spillway. However, the square-cornered entrance caused another noticeable drop in water surface (fig. 5) which did not occur with the other entrance configurations of tests 2A and 4.

Tests 2 and 2A

Tests 2 and 2A were performed with the labyrinth spillway of the previous tests, but with the orientation reversed. As shown on figure 2, this orientation had two apexes upstream whereas the previous tests had two apexes downstream. Also, the spillway apexes were moved upstream to Sta. 1+44. Test 2 had the same square-cornered entrance and upstream sloping embankment section as test 1A. In test 2A, a curved entrance replaced the square-cornered entrance with the 2:1 sloping embankment section following the curvature (fig. 2).

As predicted by the flume tests, the reversed orientation of the spillway resulted in a lower discharge capacity. At the maximum reservoir El. 4678.0 (H = 6.0 ft), the maximum discharge for test 2 was 8200 ft^3/s. This discharge was 8.6 percent less than test 1A and 9.4 percent less than the required maximum discharge. The maximum discharge was not attained until reservoir El. 4678.8 (H = 6.8 ft). Figure 6 shows the spillway operating with a discharge of 8200 ft^3/s at reservoir El. 4678.0 (H = 6.0 ft). This figure shows the adverse effect of the spillway channel sidewalls on the side lengths of each cycle and the contraction due to the square-cornered spillway entrance. The upstream water surface profile was similar to that of test 1A.

Test 2A had the same labyrinth position as test 2, except the entrance was curved with a radius of about 27 ft starting at Sta. 1+71. The curved entrance produced a significant increase in the spillway discharge capacity. The required maximum discharge of 9050 ft^3/s was attained at reservoir El. 4678.0 (H = 6.0 ft). The water surface upstream of the spillway also showed an improvement in the flow distribution compared to the previous square-cornered entrances (fig. 7). A comparison of the water surface profiles for tests 2 and 2A is shown on figure 8.

Test 3

Tests 3 and 4 were done with a new labyrinth spillway design. The total length was the same, but the spillway had a smaller sidewall angle and wider upstream apexes. The wider upstream apexes were necessary for field construction purposes. The dimensions of the spillway are shown on figure 4. The spillway layout is shown on figure 2. Other than the dimensional change and the placement of the spillway at Sta. 1+44, tests 3 and 1A were similar.

The maximum discharge for test 3 was 8490 ft^3/s at a reservoir El. 4678.0. This was 6.19 percent less than the required spillway capacity. The failure of the spillway to pass the maximum discharge was a function of the square-cornered entrance, the placement of the spillway, and the wider upstream apexes. Figure 9 shows spillway operation at a flow rate of 8490 ft^3/s and reservoir El. 4678.0 (H = 6.0 ft).

Although the same spillway orientation was used for this test and test 1A (fig. 2), the discharge was 480 ft^3/s less than test 1A. This difference was attributed to the placement of the spillway at Sta. 1+44 and the wider upstream apexes. The placement of the spillway in line with the square-cornered entrance reduced the efficiency of the upstream apexes and side lengths near the walls. This efficiency reduction was caused by the contraction of the flow from the square entrance. The wider upstream apexes also reduced the efficiency of the spillway. Previous studies had shown that a triangular shape in plan is the most efficient [1].

Comparison of tests 2 and 3 showed that the spillway orientation (two apexes downstream) in test 3 provided a higher discharge capacity than test 2. The spillways were located at the same station (1+44), but the better spillway orientation in test 3 allowed passage of a discharge that was 290 ft^3/s greater than test 2. The wider upstream apexes did not have as much effect on the spillway discharge capacity as the spillway orientation.

Comparison of tests 2A and 3 showed that spillway orientation had less effect on the discharge capacity than a good entrance condition. Test 2A (with the less efficient spillway orientation but good entrance condition) passed the maximum discharge, whereas test 3 was 560 ft^3/s less than the discharge required at reservoir El. 4678.0.

The following is a summary of the test comparisons:

Test	Discharge (ft^3/s) at reservoir El. 4678.0	Discharge comparison	Remarks
1	8625		Sloping embankment began 18 ft back from spillway square-cornered entrance.
1A	8970	1A > 1	For test 1A, sloping embankment began at base of spillway sidewall.
2	8200	2 < 1A	Test 2 spillway orientation less efficient than test 1A.
2A	9050	2A > 1A	Curved entrance had greater effect than spillway orientation.
3	8490	3 < 1A	Wider upstream apexes and spillway placement reduced efficiency.
		3 > 2	More efficient spillway orientation had a greater effect than wider upstream apexes.
		3 < 2A	Spillway entrance more important than spillway orientation.

Final Design, Test 4

The final labyrinth spillway design attempted to maximize the hydraulic efficiency and decrease the field construction costs. The comparisons of the tests discussed in the previous section showed that spillway efficiency was a function of entrance conditions, spillway placement, and orientation. Comparison of tests 2 and 2A showed the importance of the entrance condition under the same spillway operation. Comparison of tests 2A and 3 showed that the entrance condition was even more vital than spillway orientation.

The spillway dimensions for the final design are shown on figure 10. The placement differed from any of the previous tests. The two upstream apexes extended 19 ft into the reservoir to Sta. 1+25. The downstream apex of the spillway was located at Sta. 1+97. The entrance shape was formed by a 21-ft radius between Stas. 1+44 and 1+65. At Sta. 1+44 the walls began a 1:0.12 convergence that continued through the transition section to the 40-ft wide spillway chute at Sta. 2+80. The 1:0.12 sloping floor of the chute transition section started at Sta. 2+00 and continued to Sta. 2+80 where the slope changed to 0:0.04 for the remaining chute. Lessening the slope of the transition section would decrease the construction costs. Details of this design are shown on figures 1a and 1b in plan and section. The schematic is shown on figure 2 for direct comparison with the other configurations tested.

The final design provided the best entrance condition and spillway placement. The wide curved entrance with the spillway extending farther into the reservoir provided excellent flow distribution and minimal head loss. The maximum discharge, 9050 ft^3/s, was reached at reservoir El. 4677.5 (H = 5.5 ft), 0.5 ft below the maximum allowable water surface. At the maximum water surface, El. 4678.0, the discharge was 9830 ft^3/s, 8.6 percent greater than required. The discharge rating curve is shown on figure 11. The coefficient of discharge at reservoir El. 4678.0 was 2.2596. Figure 12 shows the spillway operating at reservoir El. 4677.5 (H = 5.5 ft) with a discharge of 9050 ft^3/s. This figure shows the even flow distribution produced by the placement of the spillway and the curved entrance.

It was necessary to use the less efficient spillway orientation (two apexes upstream) to attach the spillway to the sidewalls when it was moved upstream into the reservoir. Upstream water surface profiles for discharges of 6000 and 9050 ft^3/s showed the change in water surface as flow approached the spillway and the relatively constant water surface in the labyrinth (fig. 13). The curved spillway entrance and extension of the spillway into the reservoir provided optimum hydraulic conditions and the most efficient overall design.

CHUTE WATER SURFACE PROFILES

The flow in the chute downstream of the spillway was supercritical. Waves that formed downstream of the spillway traveled the length of the chute forming a crossing pattern as they reflected off the chute walls (fig. 14). The height of the waves tended to diminish as they traveled downstream.

The flow over the labyrinth spillway and its placement immediately upstream of the transition produced an unusual flow distribution downstream. Flow downstream of the spillway was extremely rough due to the spillway shape, with the depth of flow at the beginning of the transition near critical. The waves originated at the entrance of the transition section where the channel width began decreasing from 60 to 40 ft. To minimize the downstream disturbance, these cross waves should be directed toward the opposite walls at the end points of the contraction [3]. To do this, the length of the transition section would have to be considerably longer or the downstream chute wider. Neither of these alternatives would be cost effective.

The cross waves caused overtopping of the chute walls at the maximum discharge for each test configuration. For the recommended spillway configuration, the worst overtopping occurred at the end of the chute transition section where the wall height should be raised by about 1.5 ft from Sta. 2+73 to Sta. 2+87 on both sides.

SPLITTER PIERS

Spillway operation under low flow conditions could cause a problem with nappe oscillation. This phenomenon is shown on figure 15 where the nonaerated nappe clings to the downstream face of the spillway. At median flows, the nappe intermittently became aerated at the apex and sprang free. The nappe then reattached as the air was exhausted. Installation of splitter piers on the spillway crest will provide aeration to prevent this.

Because the nappe becomes submerged at higher discharges, the piers need only be designed for low flows. From the discharge rating curve (fig. 11), a pier height of 1.5 ft would be adequate to provide aeration for flow rates up to 1765 ft^3/s. Overtopping of the piers at higher discharges did not cause any problems. The piers tested were 2.5 ft wide and were placed in several locations near the downstream apexes. The recommended pier location (the centerline of the pier) was 6.0 ft upstream of the downstream apexes on each side length. The pier shape tested was a thin metal plate; however, this shape could be different. Nonaerated and aerated flows may be seen on figures 15 and 16, respectively.

The four splitter piers decreased the crest length by about 10 ft or about 3.3 percent. Prior to installing the piers, the discharge at maximum reservoir elevation was 8.6 percent greater than required. Therefore, this decrease in length still allows passage of the maximum discharge, 9050 ft^3/s, at or below reservoir El. 4678.0.

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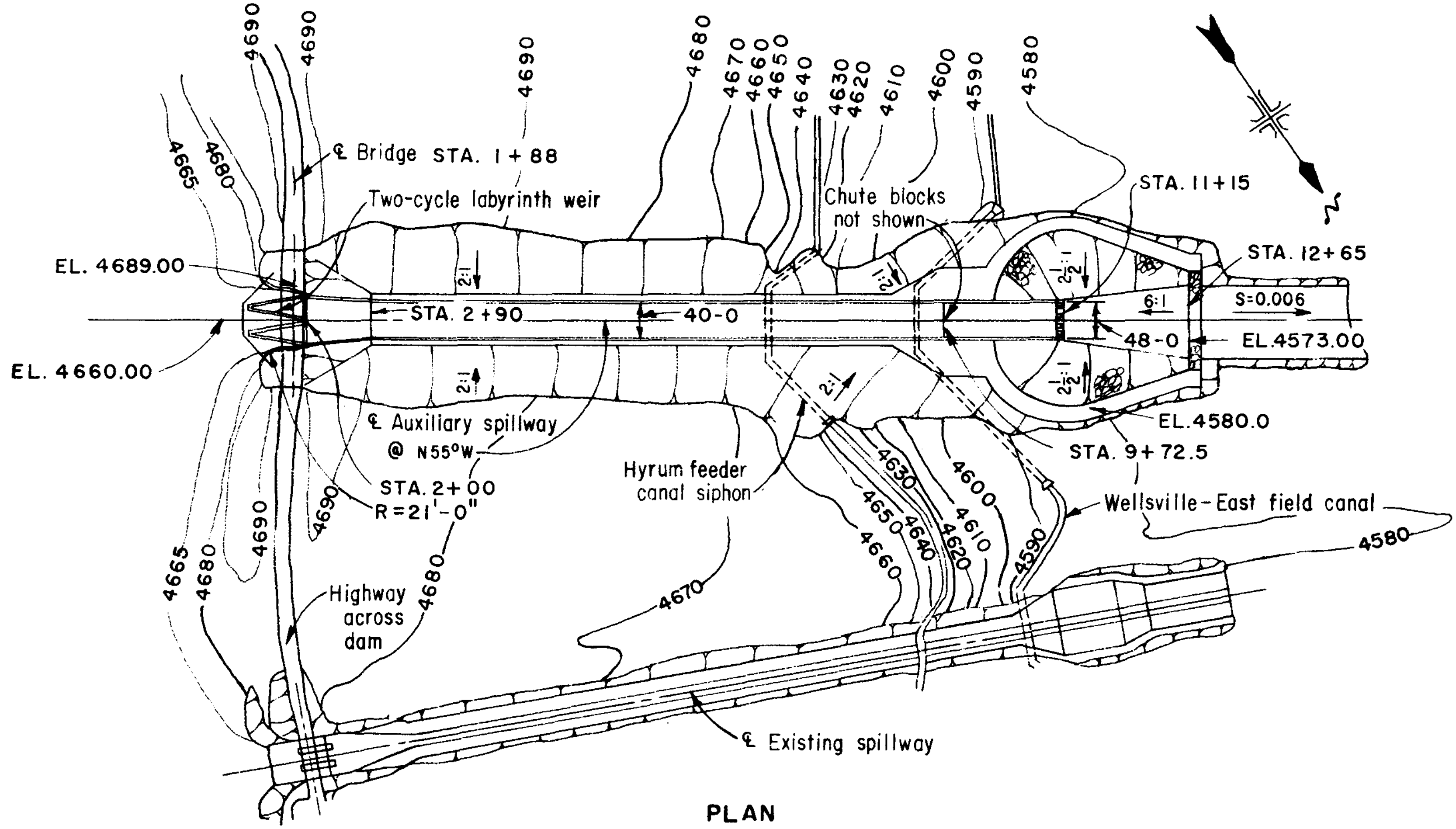
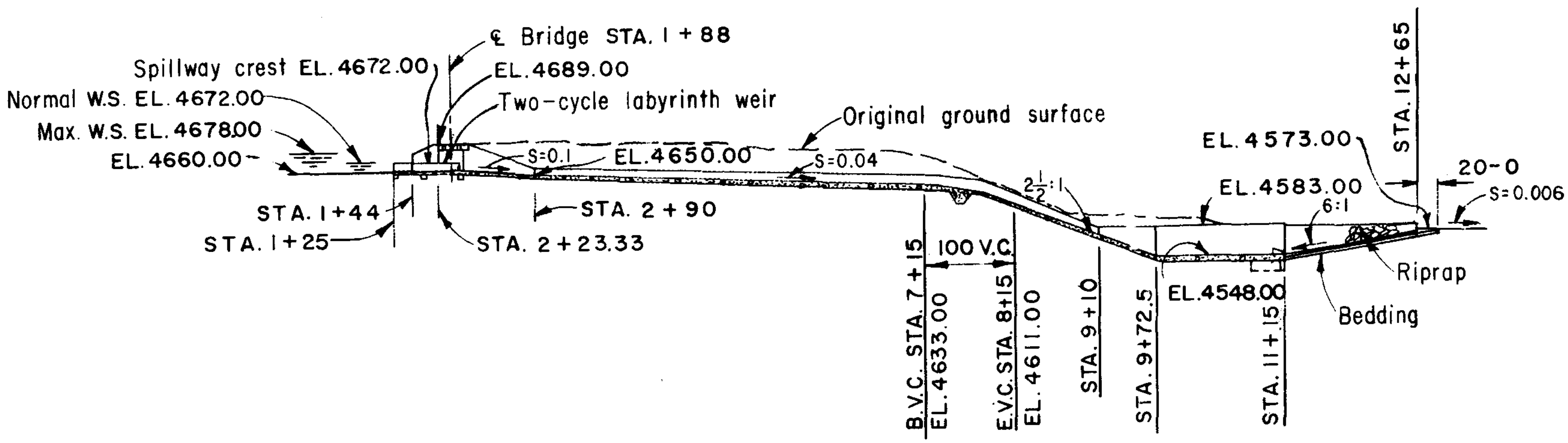


Figure 1a.—Plan of existing and auxiliary spillways.



PROFILE ALONG ℄ AUXILIARY SPILLWAY

Figure 1b.—Section of the auxiliary spillway.

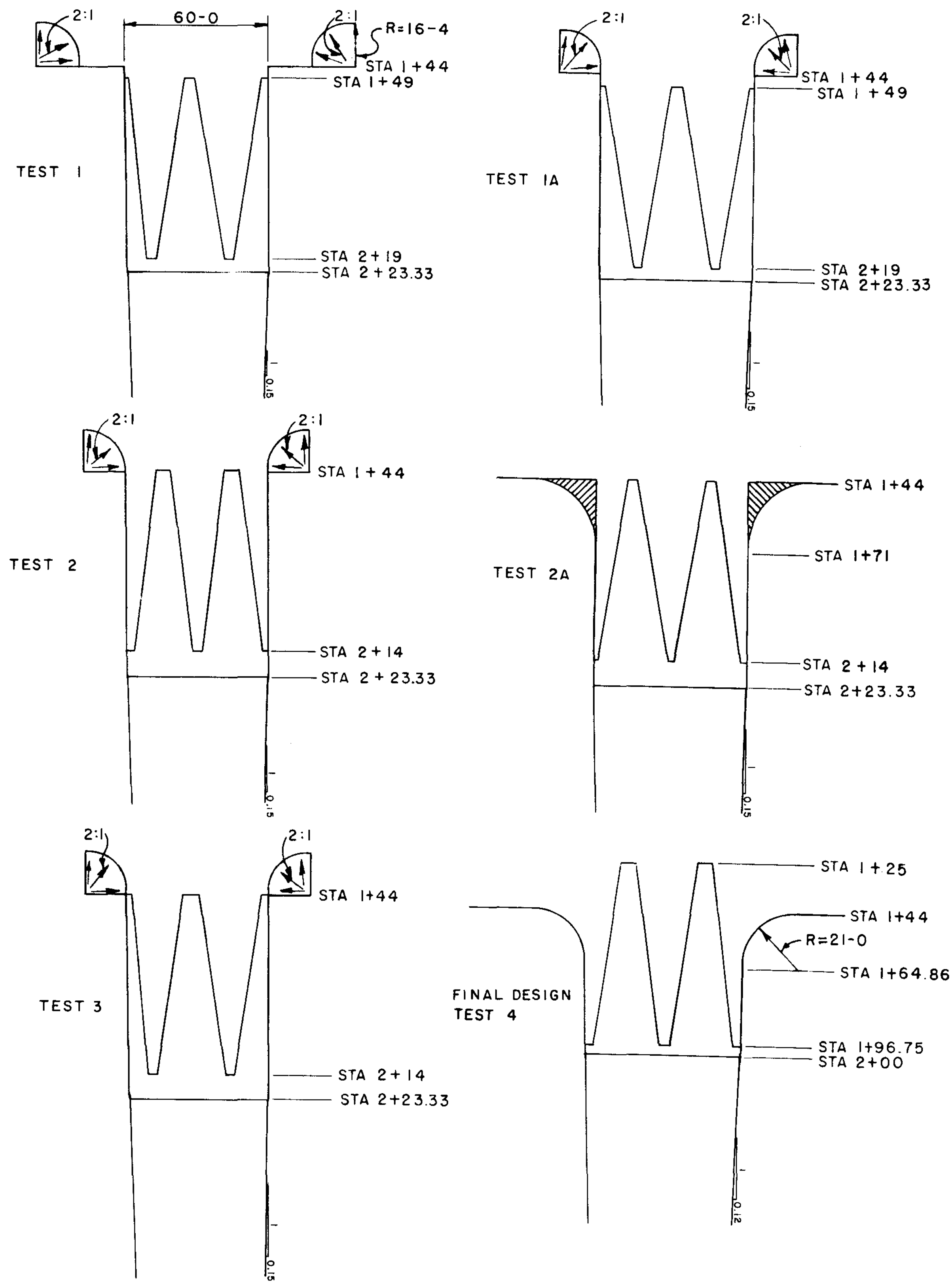


Figure 2.—Schematic of the spillway approach conditions and labyrinth spillway orientations tested. (Flow direction from top to bottom).

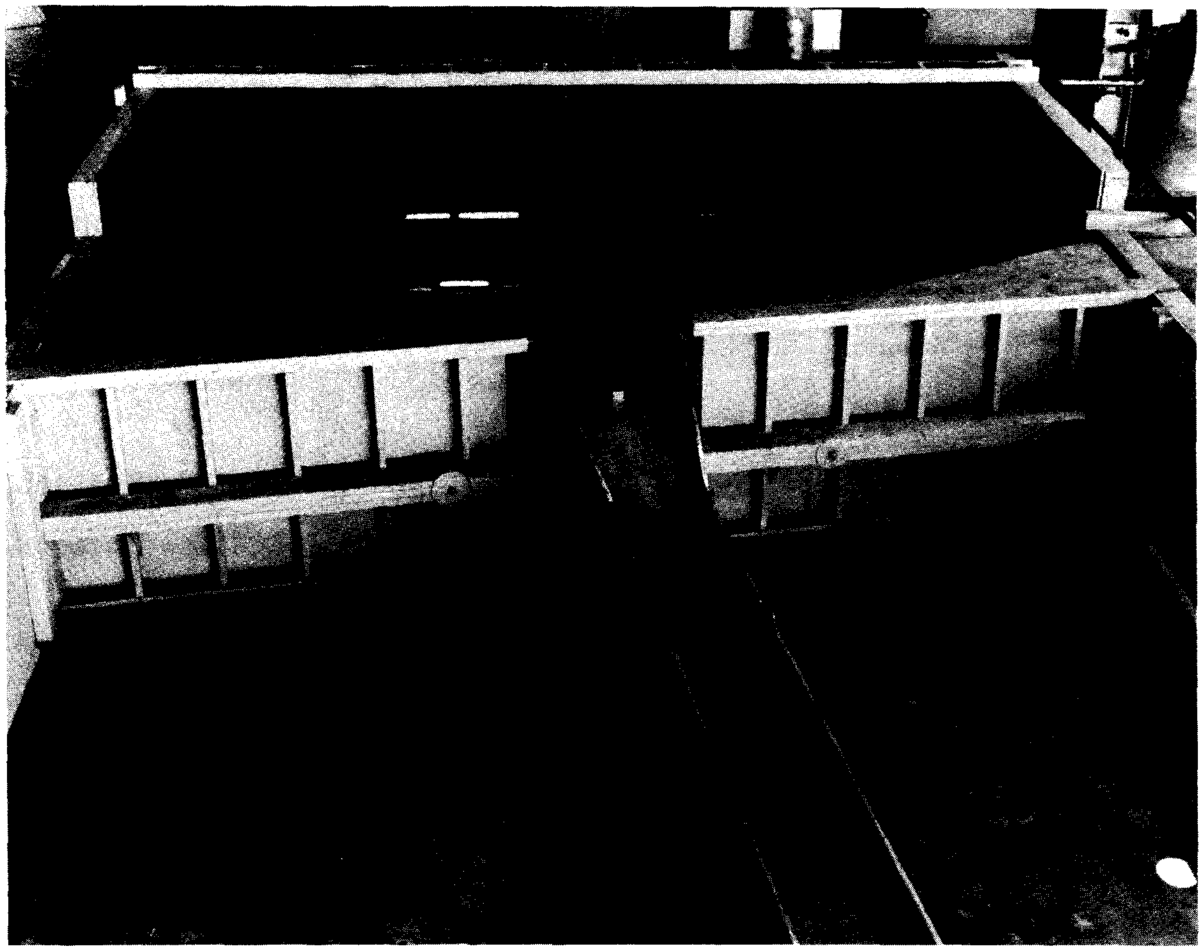


Figure 3.—Scale model (1:30) of the Hyrum auxiliary labyrinth spillway. P801-D-80190

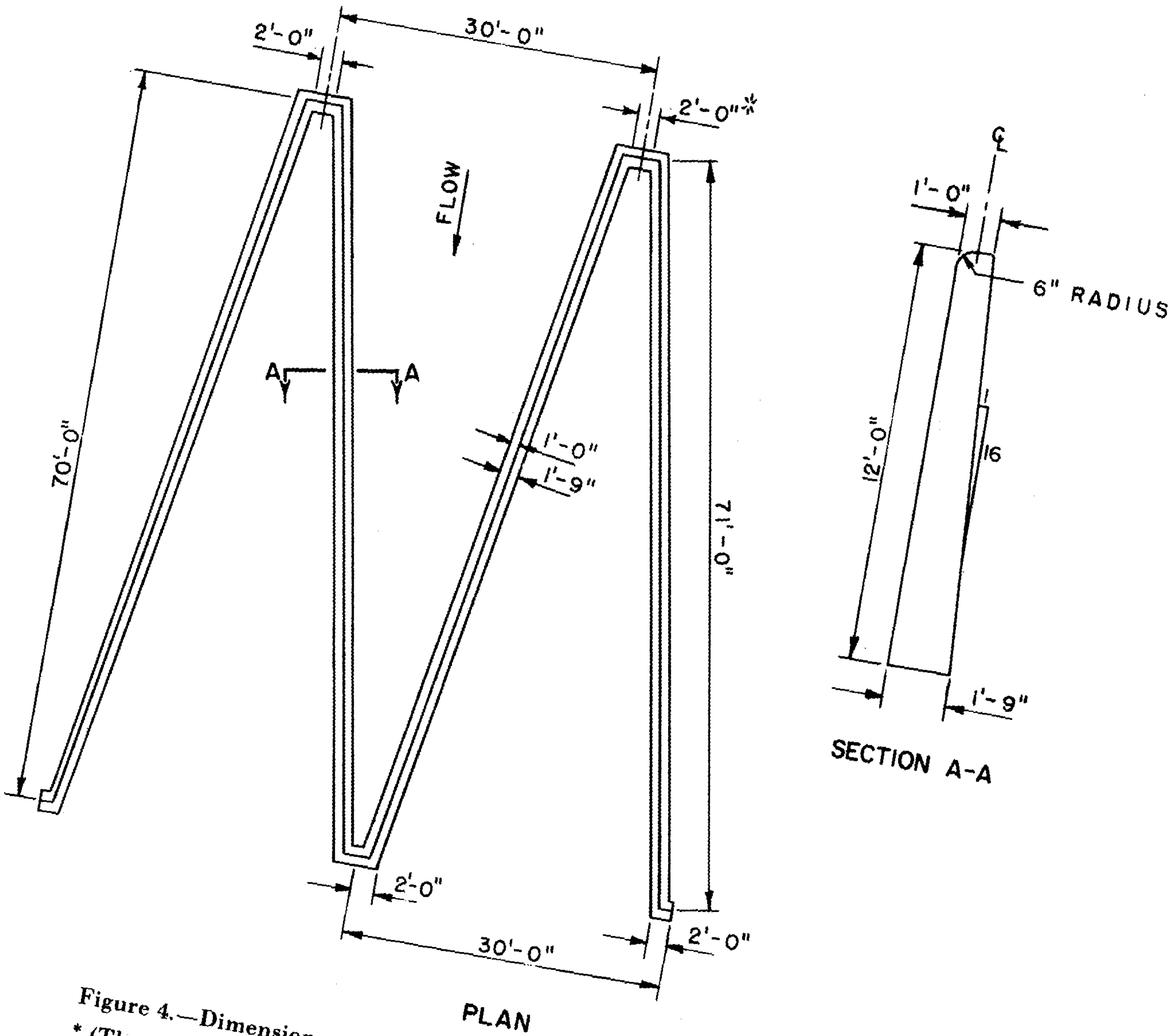


Figure 4.—Dimensions of the labyrinth spillway used in tests 1, 1A, 2, 2A, and 3.
* (The dimensions for the upstream apexes in test 3 were 3.0 ft)

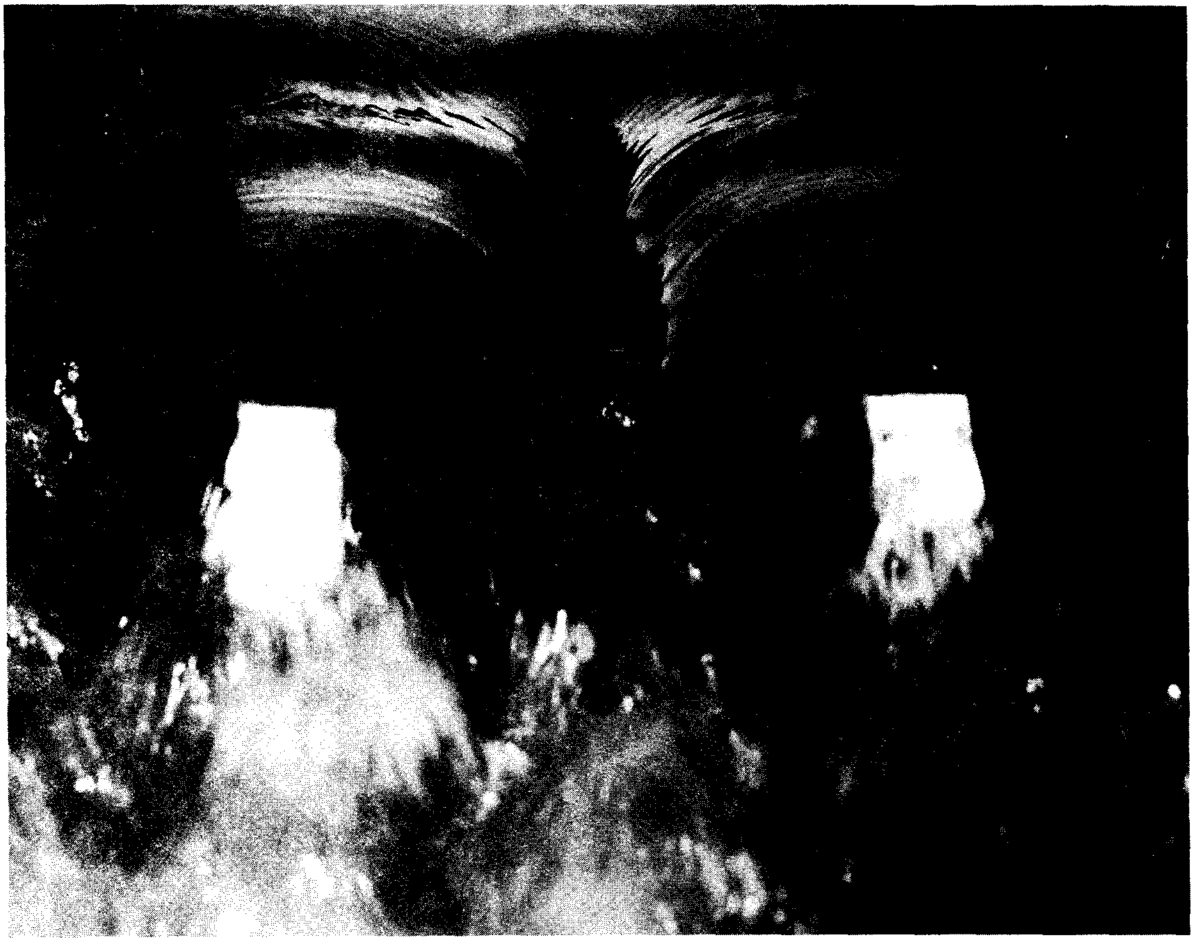


Figure 5.—View of labyrinth spillway (test 1A), Q = 8970 ft^3/s, reservoir elevation El. 4678. P801-D-80191

Figure 6.—View of labyrinth spillway (test 2), Q = 8200 ft^3/s, reservoir El. 4678. P801-D-80192

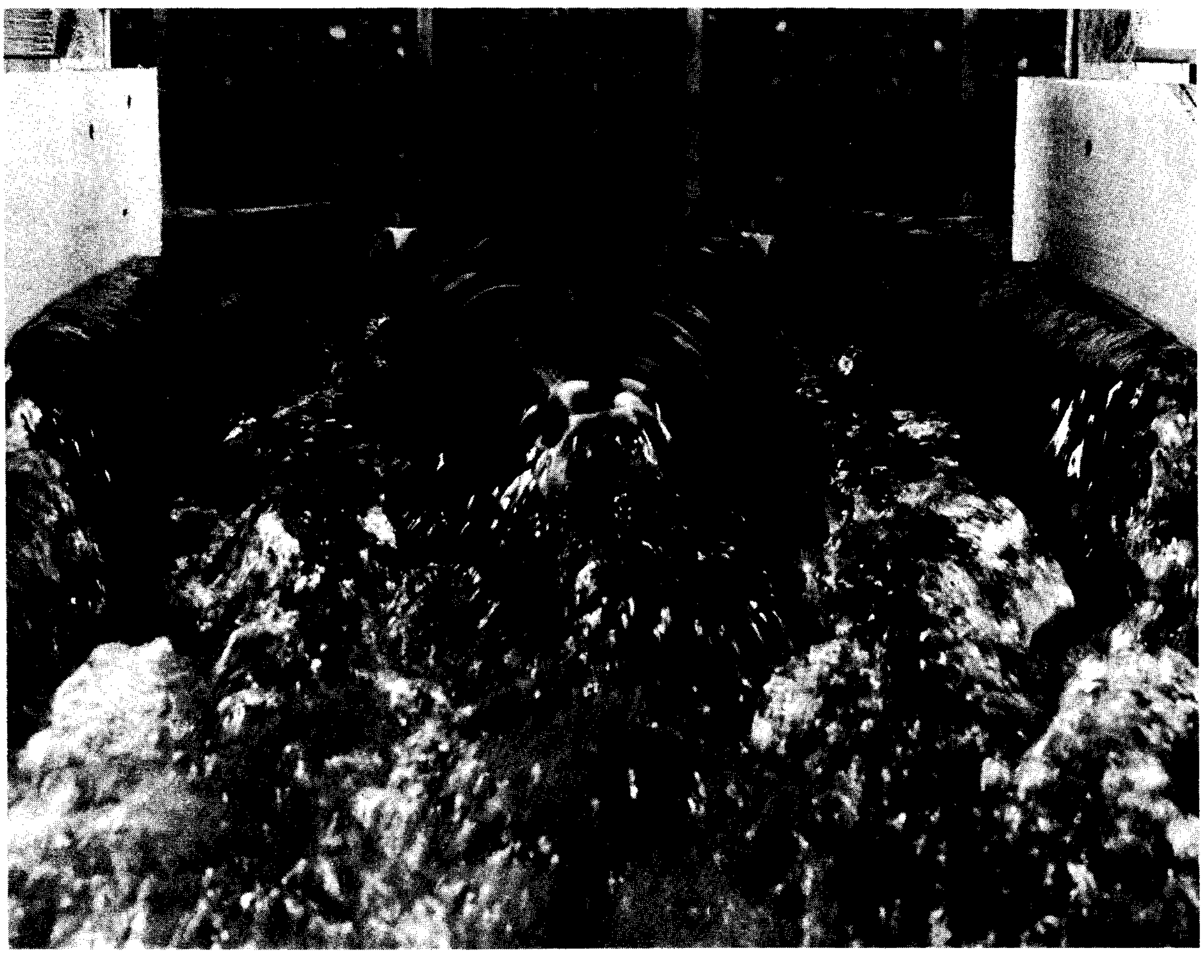


Figure 7.—View of labyrinth spillway (test 2A), Q = 9050 ft^3/s, reservoir El. 4678. P801-D-80193

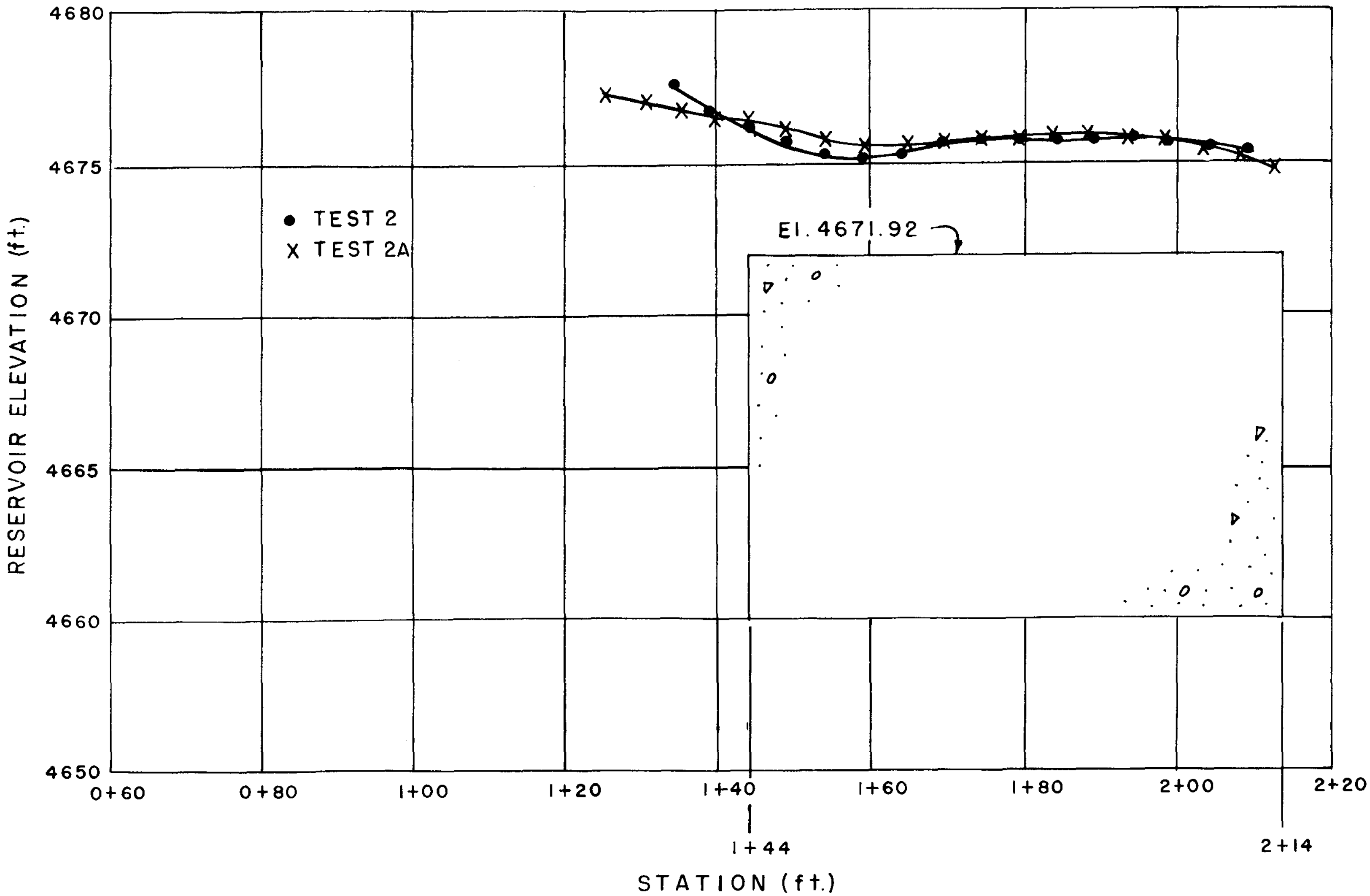


Figure 8.—Comparison of upstream water surface profiles (tests 2 and 2A).

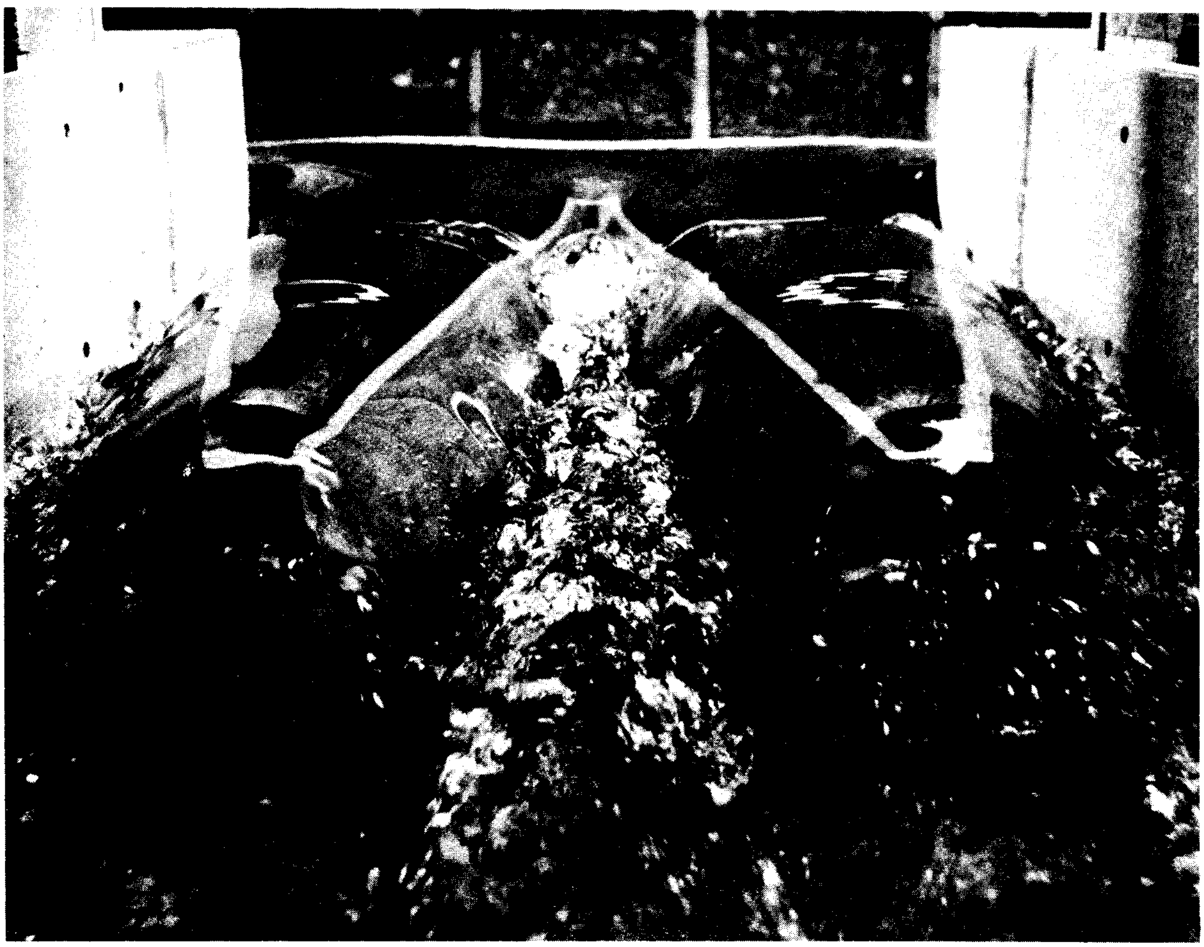


Figure 9.—View of labyrinth spillway (test 3), Q = 8490 ft^3/s, reservoir El. 4678. P801-D-80194

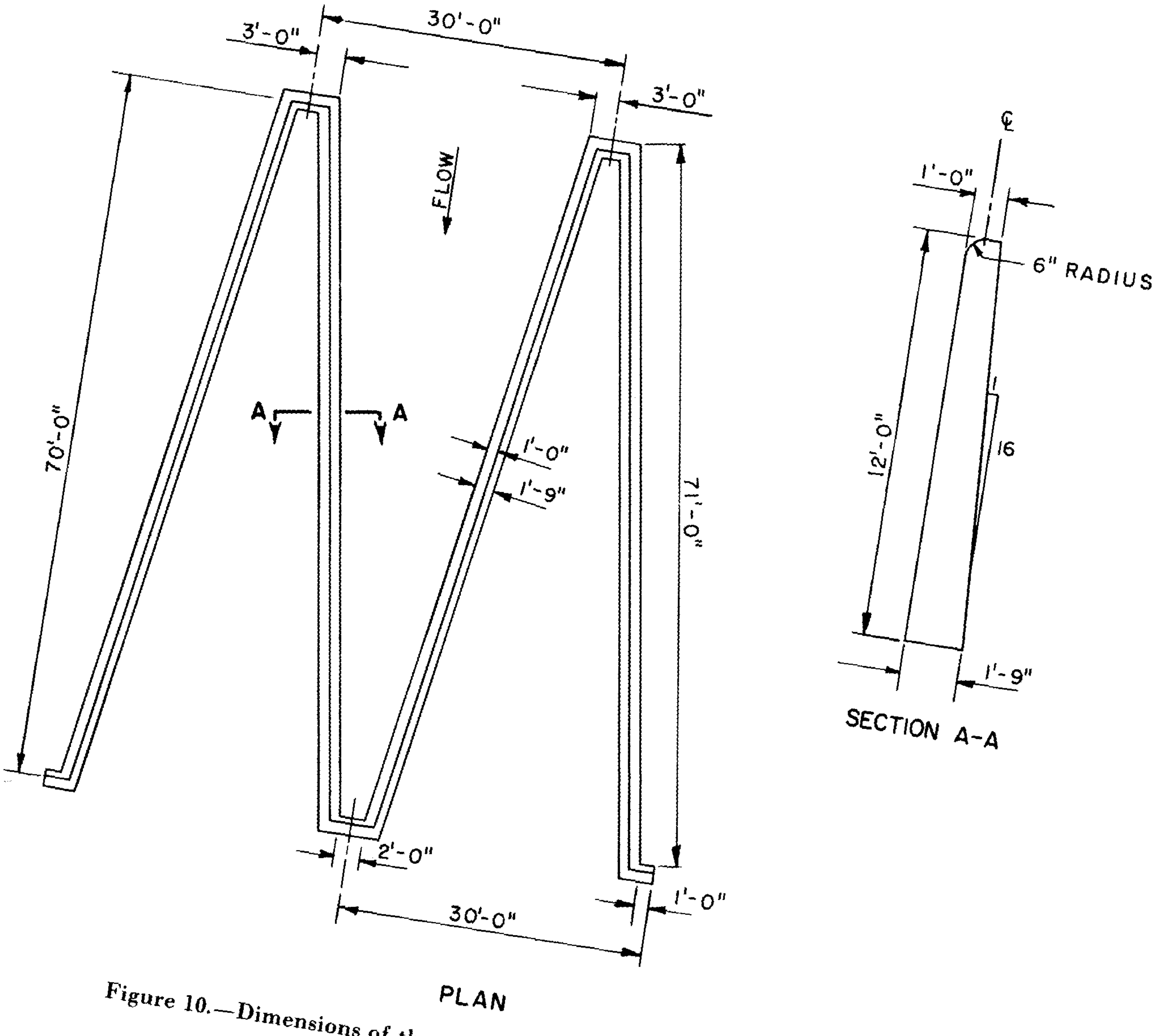


Figure 10.—Dimensions of the recommended labyrinth spillway (test 4).

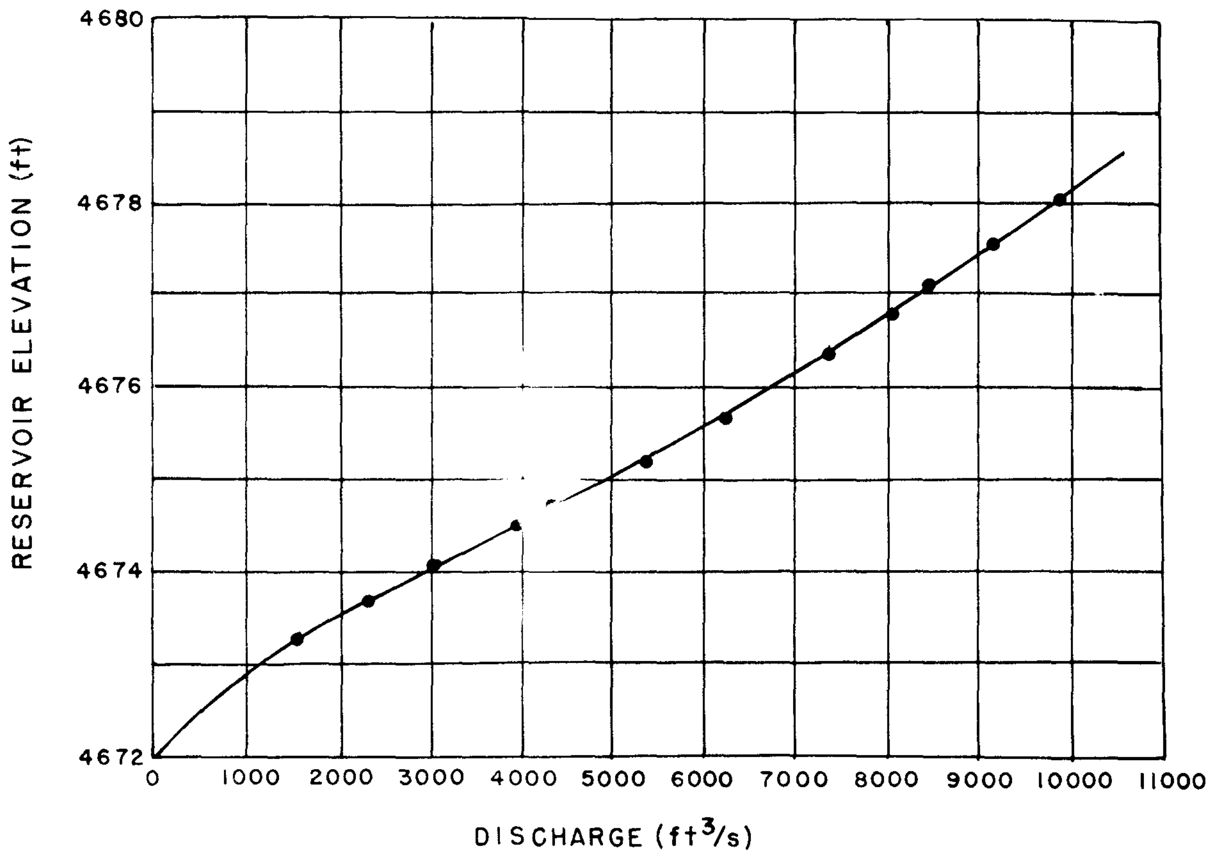


Figure 11.—Discharge curve for the recommended labyrinth spillway design (test 4).

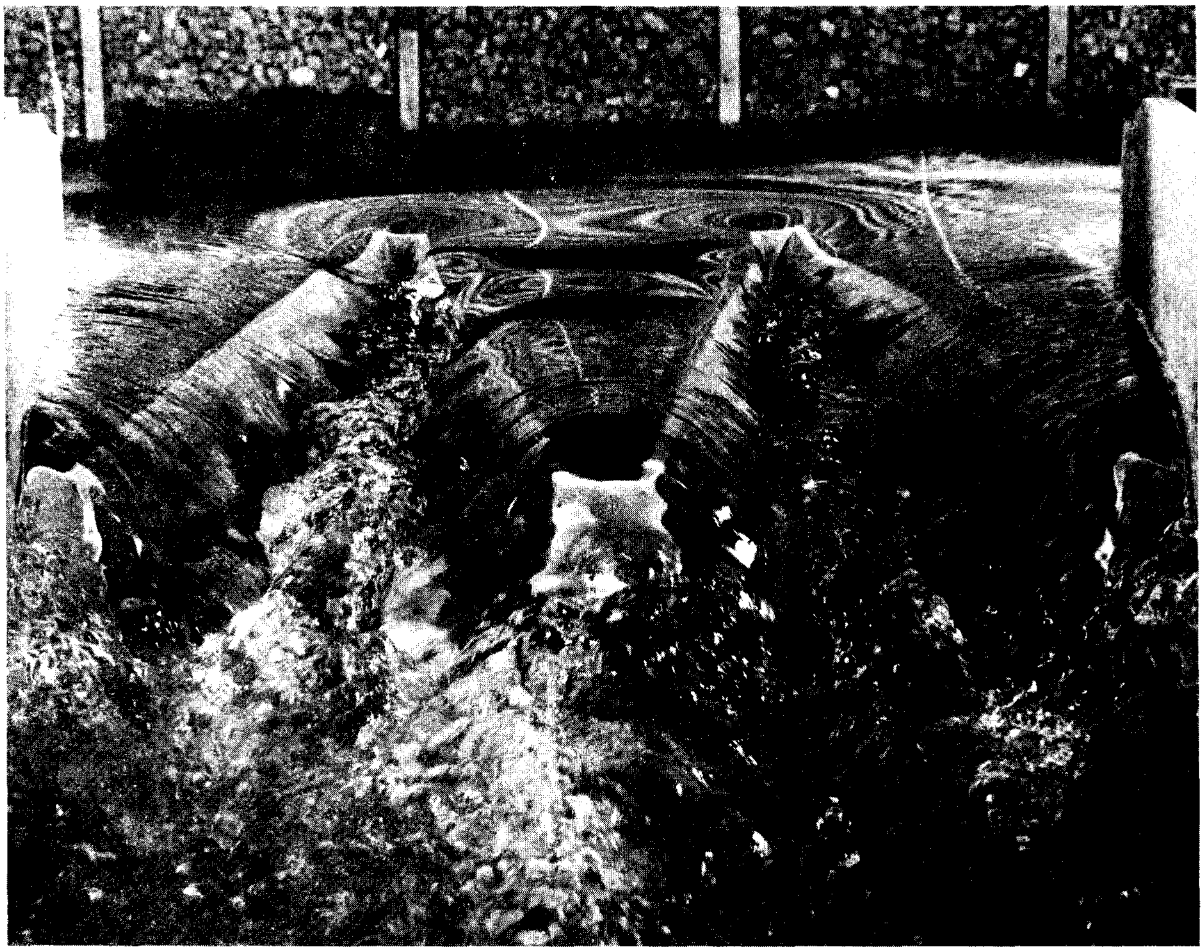


Figure 12.—View of the recommended labyrinth spillway (test 4), $Q = 9050\ ft^3/s$, reservoir El. 4677.5. P801-D-80195

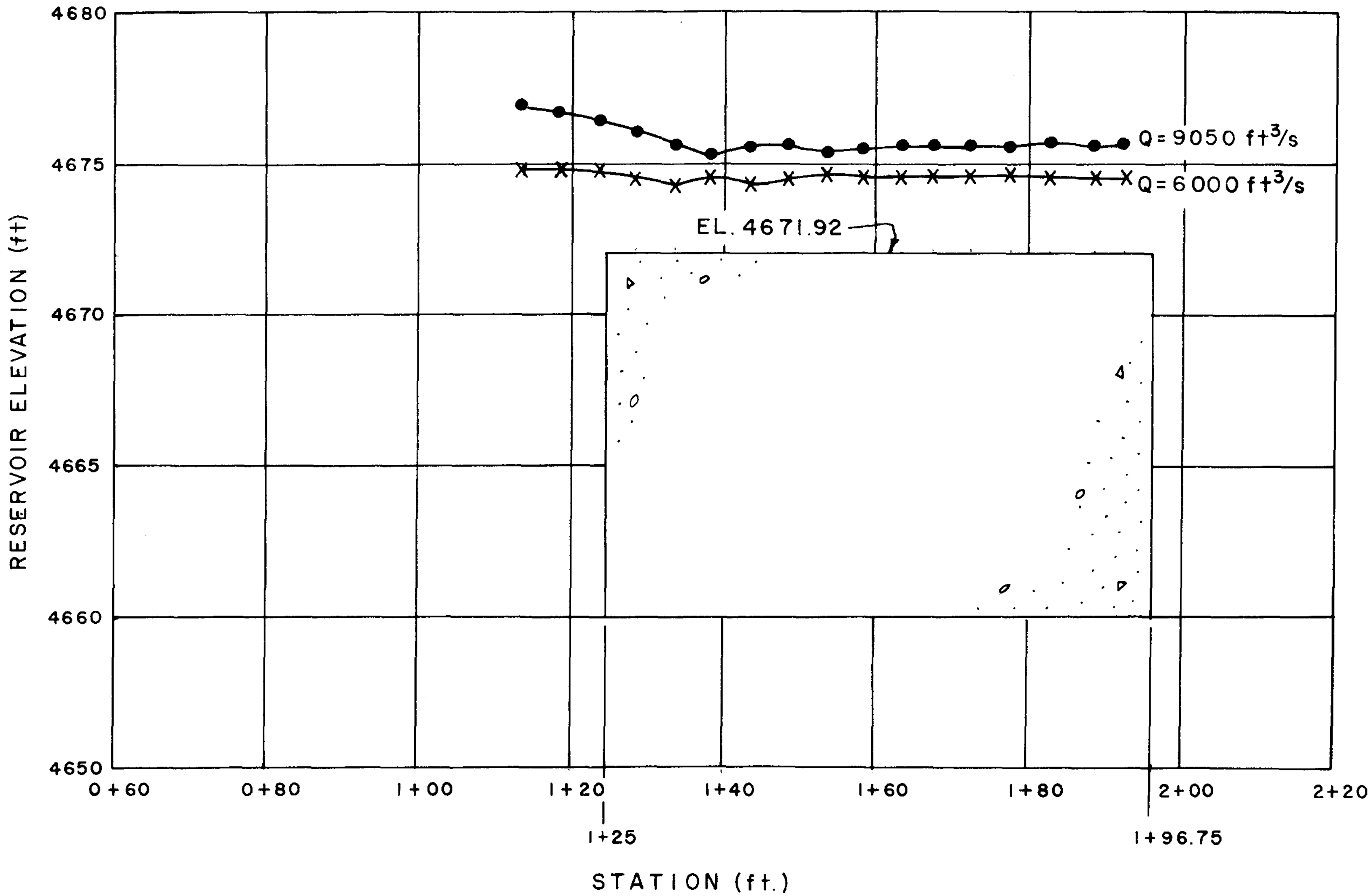


Figure 13.—Upstream water surface profiles for recommended labyrinth spillway (test 4).

Figure 14.—Cross waves in spillway chute, facing downstream, (test 4), Q = 9050 ft^3/s, reservoir El. 4677.5. P801-D-80196

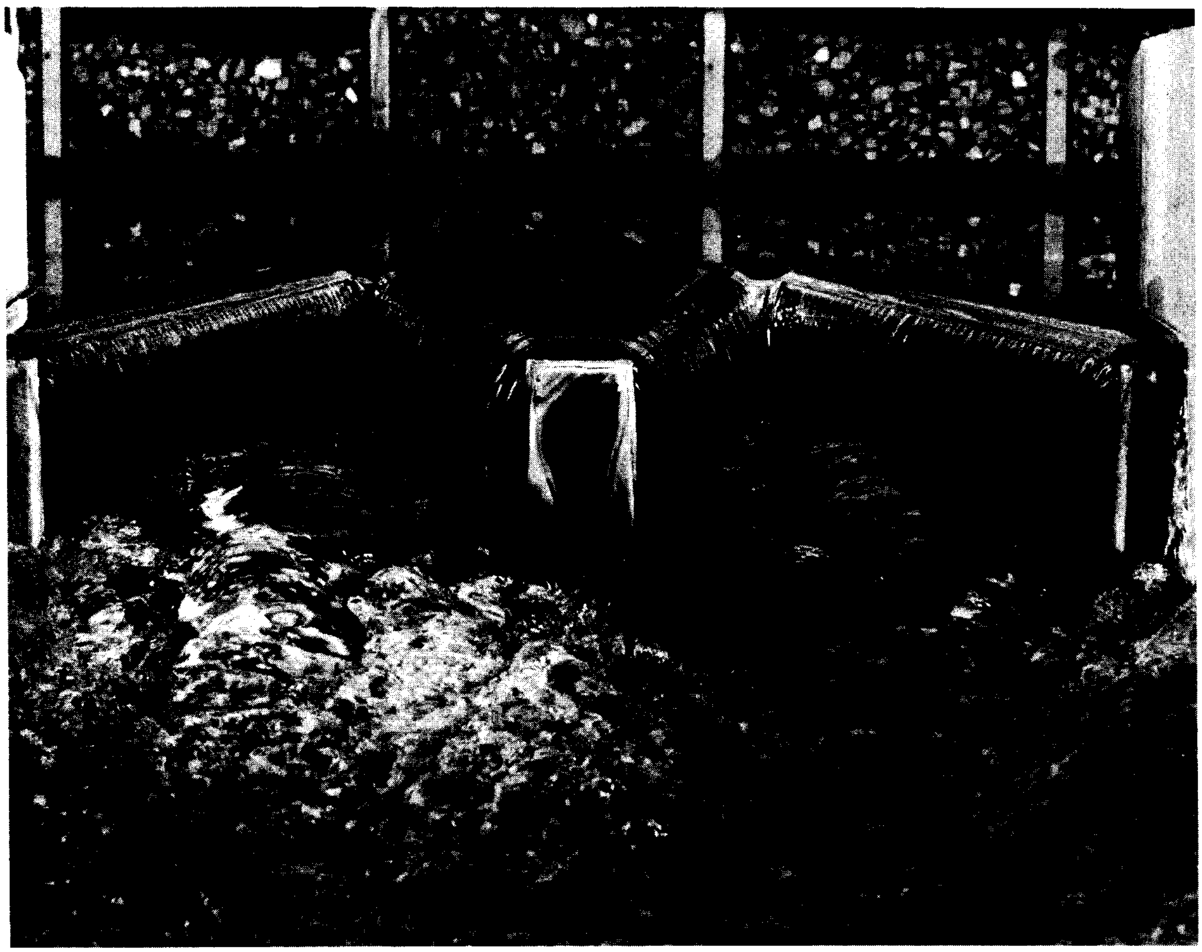


Figure 15.—View of labyrinth spillway (test 4), nonaerated low flow, Q = 2000 ft³/s, reservoir El. 4673.5. P801-D-80197

Figure 16.—View of labyrinth spillway (test 4) with splitter pier, Q = 2000 ft^3/s, reservoir El. 4673.5. P801-D-80198

Mission of the Bureau of Reclamation

The Bureau of Reclamation of the U.S. Department of the Interior is responsible for the development and conservation of the Nation's water resources in the Western United States.

The Bureau's original purpose "to provide for the reclamation of arid and semiarid lands in the West" today covers a wide range of interrelated functions. These include providing municipal and industrial water supplies; hydroelectric power generation; irrigation water for agriculture; water quality improvement; flood control; river navigation; river regulation and control; fish and wildlife enhancement; outdoor recreation; and research on water-related design, construction, materials, atmospheric management, and wind and solar power.

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.

A free pamphlet is available from the Bureau entitled "Publications for Sale." It describes some of the technical publications currently available, their cost, and how to order them. The pamphlet can be obtained upon request from the Bureau of Reclamation, Attn D-922, P O Box 25007, Denver Federal Center, Denver CO 80225-0007.