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SPILLWAY MODEL STUDY FOR TWO LICK CREEK DAM

Project Report No. 54

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Prepared for

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ABSTRACT

A hydraulic model study of the spillway for Two Lick Creek Dam was conducted in the Hydraulics Laboratory, Fritz Engineering Laboratory, for the design engineers, Gilbert Associates, Inc. The dam is being built in Indiana County, Pennsylvania for the Pennsylvania Electric Company and New York State Electric and Gas Company.

The 40 to 1 scale model was used to check the hydraulic performance of the spillway, including erosion downstream of the spillway. Gate piers and spillway abutment corners were redesigned to improve flow equalization in the three bays. Gated and free flow discharge ratings were obtained.

The erosion studies were carried out using a material with one part of high early strength cement to approximately 200 parts of sand to model the sandstone in the prototype. These tests showed no dangerous erosion patterns and verified the original design of the flip bucket and sloping apron discharge from the spillway.

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1. INTRODUCTION

A dam is being built across Two Lick Creek in Indiana County, Pennsylvania to provide year-round flows sufficient for the cooling water requirements at the Homer City Power Station. This is a mine entrance conventional power plant owned jointly by the Pennsylvania Electric Company and New York State Electric and Gas Company. Gilbert Associates, Inc. of Reading, Pennsylvania are the design engineers. After preliminary design of the spillway portion of the dam,(1)* Gilbert Associates requested that a model study be conducted in the hydraulics facility of Fritz Engineering Laboratory. The model study encompassed three areas of concern to the design and operation of the spillway.

1. Flow equalization in the three bays.
2. Discharge characteristics.
    a. Ungated
    b. Gated
3. Erosion on downstream rock apron.

Items 1 and 3 were expected to result in design modification to equalize the flow and to assure structural safety against erosion. The flow characteristics will be useful in operation of the gates during floods.

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*Numbers in parentheses refer to items in the list of references.

The model scale and extent had to be determined before work could proceed on model construction. Two areas were available. One was an existing tank 10 ft. wide, 35 ft. long, and 2 ft. deep. Considerable work was needed to prepare this tank for a spillway test. It also had a maximum flow rate of approximately 4 cfs. The other space measured 16 ft. by 35 ft. but required construction of a new tank and piping system.

The laboratory supply system has a capacity of nearly 8 cfs. For Froude number modeling the discharge ratio varies as the 2.5 power of the length ratio. To model the prototype spillway design flow of 62,500 cfs a scale ratio of 1 to 35 or less is fixed by the system capacity. A ratio of 1 to 40 was selected and required a flow rate of about 6.3 cfs in the model. At this scale the vertical height of the model is about 2.5 ft. which is too great for the existing tank.

A choice was now possible. A 1 to 40 scale model could be built in a new tank or a 1 to 60 scale model could be built in the existing tank. The latter choice would require a second, large scale model for discharge calibration of the gates. However, the only facility for the large scale tests was committed to July 1, 1967. It also had a maximum discharge of 2 cfs. A full bay could be modeled at a scale ratio of 1:40. A half bay could be modeled at a scale of 1:30. As these scales are the same or only slightly larger than the comprehensive model, a single model at a scale of 1:40 was selected. The area modeled is 640 ft. wide by 925 ft. long as shown in Fig. 1.

2. MODEL CONSTRUCTION

The tank built specifically for this model test has a test section 16 ft. wide, 25 ft. long, and 3 ft. deep. The flow enters the main tank through a single perforated steel plate, from a 4 ft. deep by 3 ft. long head tank. The water is distributed in the head tank by a 10 inch diameter manifold. Water is obtained from the laboratory constant pressure tank and flows to the head tank manifold through a 12 inch pipe. Flow rate is controlled by a 12 inch gate valve just before the manifold. The tail tank is 3 ft. long by 3 ft. deep and drains through an existing splitter into a volumetric tank or the main sump.

Orifice plates are inserted at the upstream flange of the 12 inch elbow visible in Fig. 5. An orifice with an 8 inch diameter opening is used for flow rates above 1 cfs. A 4 inch orifice is used for flow rates between 0.1 cfs and 1.5 cfs. The pressure differential across the flange taps was measured on 100 inch U-tube manometers using gage fluids of 1.75 or 2.95 specific gravity as required for accuracy. As the location of the orifice at an elbow is not accepted practice, both orifices were calibrated in place by means of the volumetric tank. The rating curves determined by least squares fit to the calibration data are:

$$Q = 1.887\ H^{0.498} \qquad \text{8'' orifice}$$

$$Q = 0.424\ H^{0.508} \qquad \text{4'' orifice}$$

where

Q = volume flow rate, cfs

H = metering differential, ft. of water

The model construction proper was divided into 4 units. The spillway proper including gates and flip bucket were made of hard mahogany and plexiglas by the Bethlehem Model Shop. Figure 4 shows this unit before installation. The relatively uniform portions of the rock-fill dam and topography were constructed of marine plywood by the university carpenter shop. They also made the support structure for the more varied areas of the model.

Fritz Laboratory personnel did the final work on modeling the complex areas. Templates were prepared from topography maps and attached to the supporting wood structure as shown in Fig. 2, which shows the upstream area at this stage. The space between the templates is filled with lightweight concrete. Figure 3 shows the final model in the same area as Fig. 2.

The area immediately downstream of the flip bucket was to be used in the erosion studies. Consequently an area approximately 6 ft. square was not modeled in rigid material. Pea gravel was placed in this area for a few preliminary tests. Figure 5 is an overall view with pea gravel in the erodible bed region. The erodible bed material is described in detail in Chapter 4.

3. FLOW CHARACTERISTICS

3.1 Original Design

Free discharge performance was determined for the original design. At higher flow rates three problems became apparent. All discharges and distances are in prototype units.

At flow rates above about 40,000 cfs, entrance conditions and flow distribution between bays deteriorated. At the spillway design flood of 62,500 cfs the flow separated at the left side of bay 3. This reduced the flow through bay 3 and caused bays 1 and 2 (especially 2) to carry the displaced flow. This condition also caused poor flow conditions in the flip bucket. Unfortunately no photographs were obtained of this condition. The dashed lines in Fig. 6 show the surface profiles parallel to the spillway axis at the crest. The pile-up on pier noses was also higher than is desirable.

Downstream of the piers, the flow in adjacent bays converged to produce large rooster tails. This could cause excessive erosion as well as spray in the prototype.

The operation of the flip bucket appeared very poor in these early tests. However, no apron was installed at this time. Later addition of a temporary wood apron between the training walls indicated that the flip bucket did work as planned. The effectiveness of the flip bucket is illustrated by Fig. 7 which shows the spillway design flow after the pier modifications described below.

The modifications to the piers and abutments are superimposed on the original spillway design in Fig.6. The left abutment was improved by extending the 14 ft. radius in a "bulb-nose" to guide the flow into bay 3. The longer, more tapered pier noses were added to help distribute the flow and to reduce pile-up. The downstream pier extensions reduced the rooster tail to a considerable extent as shown in Fig. 7.

After these modifications were completed on the model, free and gated discharge tests were repeated. The flow separation in bay 3 was eliminated. The improvement may be seen in Fig. 9 which shows the surface profile along the spillway crest before and after the modification. Figure 8 is a photograph of the surface at 62,500 cfs. discharge after modification.

The results of the head-discharge measurements for free and gated flow are presented in Fig. 13.

The modifications produced no significant change in the free flow rating. For free discharge conditions the changes eliminated separation and distributed the discharge more evenly between the three bays. The longer pier noses reduced gated flow slightly. The reduction amounts to about 5 percent at a 3 foot gate opening. The flood of record (9,600 cfs) will still be passed with all gates open 3 ft. and the water surface at elevation 1185.

Vortex formation was evident at most gate openings before and after approach conditions were modified. In most cases strong, hollow core vortices formed on the left side of bay 3 and the right side of bay 1. Though circulation was apparent in the surface flows in the other corners of bays 1 and 3 as well as in bay 2, no stable surface depression formed at these locations. The vortex strength increased with gate opening. The surface flow pattern is shown by a time exposure of confetti in Fig. 14. All gates are open 2 ft. and the discharge is 7000 cfs with the water surface elevation 1183. Figure 15 shows the vortices for a gate opening of 10 ft. and discharge of 26,000 cfs.

Various schemes were tried to inhibit vortex formation but none seemed effective -- especially when prototype size and cost were considered. When vortices action was eliminated no noticeable change in flow rate or upstream surface elevation occurred. Apparently the vortexes have very little or no effect on discharge conditions though they could result in gate vibration problems.

4. EROSION STUDIES

4.1 General

The apron below the flip bucket is to be cut in the good sandstone formation at the site. The structural safety of the spillway, training walls, and outlet tunnel could be endangered by erosion of the sandstone during flood flows. The designers furnished an estimate that the prototype rock would be eroded by a velocity of 20 fps. The model studies of erosion were to simulate this behavior and to determine any areas of erosion which might endanger the structure.

A literature survey produced a small amount of information. The TVA (2, 3) and U. S. Bureau of Reclamation (4) were the only organizations with experience. Both recommended a mix of approximately 1 part aluminous cement to 100 parts sand with additives to aid uniform mixing. However, the nature of the material requires trial and error selection of mix proportions. A basic mix was selected which had proportions by weight of:

Cement	1/100 - 1/250	of sand
Water	4/25	of sand
Sodium Metaphosphate	1/25	water
Bentonite	1/5	cement

Consultation with a local cement chemist led to the substitution of high early strength portland cement for the aluminous cement. The aluminous cement is not regularly available and is quite variable from batch to batch. No further changes in mix proportions were made as the high early strength cement gave good results.

Flume tests on trial sample mixes resulted in the following relation of sand-cement ratio to the velocity at which erosion begins.

TABLE 1

RANGE OF MIX AND EROSION VELOCITY

Mix Ration (sand/cement)	Model Velocity	Prototype Velocity
170/1	6.5 fps	41 fps
210/1	3.0 fps	19 fps
225/1	2.0 fps	13 fps
250/1	1.2 fps	8 fps

Two preliminary tests were conducted on the model while this information on the erodible material was being acquired. A bed of 1/4 inch pea gravel was laid down and the standard flood was run with all gates open 3 ft. The first of these runs had the apron formed in pea gravel. Figures 16 and 17 show the before and after conditions. For the second run the false floor was used in the apron area with the results as shown in Fig. 18. These two tests served to acquaint the personnel with flood operatinn (following prototype operation instructions for gate control) and to indicate the areas in which erosion could be expected.

The values in Table 1 cover the range of erodible bed material actually tested in the model. The 170/1 bed was placed after only limited flume data was available, but still indicated the trend of the beds with strengths more nearly correct. Table 2 presents a summary of all tests on the sand-cement material. The standard flood has a peak flow of 10,000 cfs and a duration of 12 hours. For erosion studies this was simulated by opening the gates in the specified sequence as rapidly as allowed (20 minutes between any two gate movements) and then continuing flow at the 10,000 cfs value until sudden gate closing at the end of the time period. In the model, the gate opening sequence (1) took 33 minutes and the flood duration was 1 hour 54 minutes. The spillway design flood has a peak of 62,500 cfs and the same time duration as the standard flood. For erosion tests using the spillway design flood the gates were opened 3 ft. following the standard operating procedure, then rapidly opened all the way. In all tests the reservoir elevation was maintained at elevation 1183. Due to uncertainty about prototype tailwater conditions the given value of elevation 1135 was used for spillway design floods, while the tailwater was arbitrarily set at elevation 1115 for the standard flood.

TABLE 2

SUMMARY OF EROSION TESTS

Test	Bed	Flood	Mix	Photographs	Contour Map
1	1	Standard	170/1	Fig. 21	
2	2	Standard	225/1	22	Fig. 30
3		Spillway Design		23	31
4	3	Standard	250/1	24	
5		Spillway Design		25	
6	4	Spillway Design*	225/1	26	32
7	5	Standard	210/1	27	33
8		Spillway Design			34
9	6	Standard	225/1	28	35
10		Spillway Design		29	36

*No previous standard flood.

Several general observations apply to all erosion tests. The material in all beds suffered severe erosion by small trickles leaking under the closed gates before and after the test run proper. The erosion pattern developed rapidly after each gate adjustment. The last hour of each test run produced only minor increases in scour depth. Similarly, the spillway design flood tended to even out the bed pattern resulting from the preceding standard flood, but did not cause a significant increase in the depth or extent of scour.

Results are presented in a pair of photographs for each test run and by contour maps for the more important runs. Table 2 lists the figures pertaining to each test. Figure 20a shows the bed before testing in a general view. Figure 20b is a closeup picture of the apron, training walls, and berm as the material appears before testing. Figure 19 shows the "sounding board" system, with arbitrary stationing, used to make the contour maps. This proved to be a quick and convenient technique. Figures 21 through 29 show the results of each test run. The views correspond with those in Fig. 20 showing the before testing bed condition. Figures 30 through 36 are contour maps of the apron and the area immediately downstream of the apron. Horizontal scale in prototype feet is given by the arbitrary stationing selected for the erosion study.

4.2 Description of Erosion Tests

Test No. 1 - Fig. 21 -- The bed material for test No. 1 had more cement than the others. After the standard flood, the bed had eroded generally with moderate depressions in the center of each bay where the jet struck the apron. As may be seen in the photographs, the dike and right-hand hillside suffered considerable erosion by wave action.

Test No. 2 - Figures 22, 31 -- With more information on the erosion resistance of the sand-cement mixture, the second erosion bed was placed with 1 part cement to 225 parts sand. Erosion between the training walls was similar to that in Test No. 1. More severe erosion was apparent beyond the training walls and the berm on the right side was subject to moderate erosion on its vertical face. Wave action again ate away at the dike and hillsides.

Test No. 3 - Figures 23, 31 -- This test passed the spillway design flood over the second bed 3 days after Test No. 2 had been run. Some increase in erosion occurred on the apron itself. Wave action, as well as the main flow, reduced the berm to about one half its original height and erosion extended up the berm behind the training wall.

Test No. 4 - Fig. 24 -- The weakest material tested was used in Test Nos. 4 and 5. Severe erosion holes were worn by each jet. Surface wave action actually undercut the dike and hillside. The entire moveable bed was penetrated in the three scour holes. This represents about 25 ft. of erosion.

Test No. 5 - Fig. 25 -- The weak bed was further eroded by the spillway design flood, run 5 days after Test No. 4. The bottom of the tank was exposed for a distance of about 50 ft. extending across the apron and around the outlet structure. The berm was nearly destroyed and deep erosion occurred at the end of the right training wall.

Test No. 6 - Figures 26, 32 -- The material was a duplicate of the second bed, and was expected to erode at the prototype velocity of 15 fps. To accommodate visitors, the spillway design flood was run over a fresh bed. The gates were opened to 3 ft. according to schedule, and then opened in larger steps to free flow. The one new feature is the extensive scour hole in front of bay 1. An unusual scour hole also developed far downstream near the berm. This was attributed to the placement of the bed. This may also explain the erosion on the apron, but another bed of this strength was to be made to check this condition. The 10 ft. deep erosion would undermine the toe of the flip bucket.

<u>Test No. 7</u> - Figures 27, 33 -- The material for this test models most nearly the specified prototype material which erodes at 20 fps. To eliminate wave damage and more nearly model the rock-fill dike and the corner of the dam in the erosion area, these features were built of material with twice the cement in it. The performance is much improved as the photographs show. Scour holes approximately 10 ft. deep developed in front of bays 1 and 2 though there was no scour at the flip bucket. A local scour hole developed at the base of the berm about 50 ft. beyond the end of the training wall.

<u>Test No. 8</u> - Fig. 34 -- The spillway design flood increased the extent of erosion below bays 1 and 2 with only slight increases in depth of scour.

<u>Test No. 9</u> - Figures 28, 35 -- The strong dike was retained for this, the sixth bed. The material used had a sand-cement ratio of 225/1, repeating the material of beds 2 and 4. The gate operation was modified to check the influence of the out of scale side seals used in the model. The gates were set 3 ft. open and sealed with a sealing compound and plastic tape to give negligible seal effect. Temporary gates were installed upstream of the radial gates to allow the reservoir to be filled to elevation 1183 before initiating flow. Water was admitted to one bay at a time following the standard sequence of 2, 1, 3 in opening the gates. However, the step by step raising of the gates to the 3 ft. point could not be followed.

The jets over the flip bucket were more evenly distributed without the overturning and converging fins caused by the gate seals. Thus the jets were not as concentrated, but impinged on the apron as a continuous sheet when all gates were open equal amounts. Vortex formation above the gates was pronounced. A pair of vortices formed, one in each of the outer corners formed by a pier and gate, regardless of the number of gates open.

Erosion was similar to Test No. 2, as expected. A severe scour hole formed at the end of the right training wall as soon as flow began in bay 1. It is believed that a poor finish at the joint between the wall and the molded berm caused this.

Test No. 10 - Figures 29, 36 -- The spillway design flood passed over bed 6, 3 days after Test No. 9, resulted in minor changes in erosion patterns already evident. The severe condition at the toe of the flip bucket apparent in Figs. 28 and 29 was entirely the result of an operating condition caused by the fixed gate position. At the end of Test No. 9, the reservoir emptied by discharging through the partially open gates, with the tailwater well below the flip bucket elevation. This flow running slowly over the bucket lip plunged and dug deeply directly in front of the bucket. The deposition visible in Fig. 28 is the material removed by this action. The prototype would never be subjected to this condition and the actual rock would not be susceptible to this type of flow.

5. CONCLUSIONS

The model tests resulted in several design changes to better equalize flow between the three bays. These modifications are shown in Fig. 9 and included:

1. Increased radius on right hand abutment to 5 ft.
2. Extended pier noses in a 10 ft. radius with a 1.5 ft. tip radius.
3. Extended 14 ft. radius on left abutment to form a bulb nose.
4. Extended piers downstream to the end of the flip bucket, tapering to a 2 ft. thickness.
5. Raised training walls to elevation 1145.

The spillway rating curve for gated and ungated flows was generated and may be used as a guide for flood operation.

The erosion studies verified the effectiveness of the flip bucket in smoothly dropping the flow onto the sandstone apron. As no serious erosion patterns developed, these studies indicate that the spillway is safe against damage by any erosion which does occur. Problems are not likely except at weak areas in the rock itself.

ACKNOWLEDGMENTS

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Mr. W. Miller and Staff
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Lehigh University

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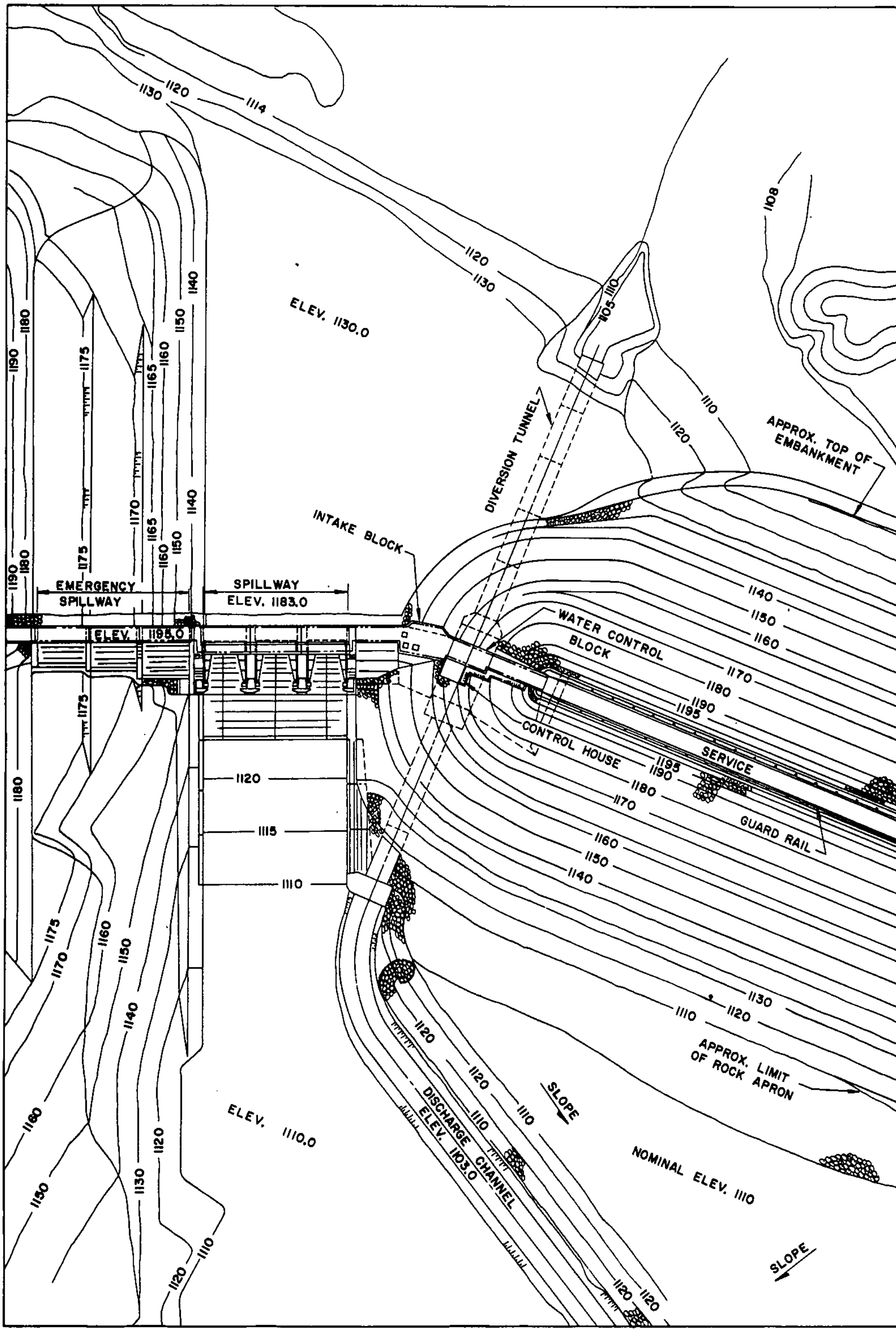


Fig. 1 Plan of Modeled Area

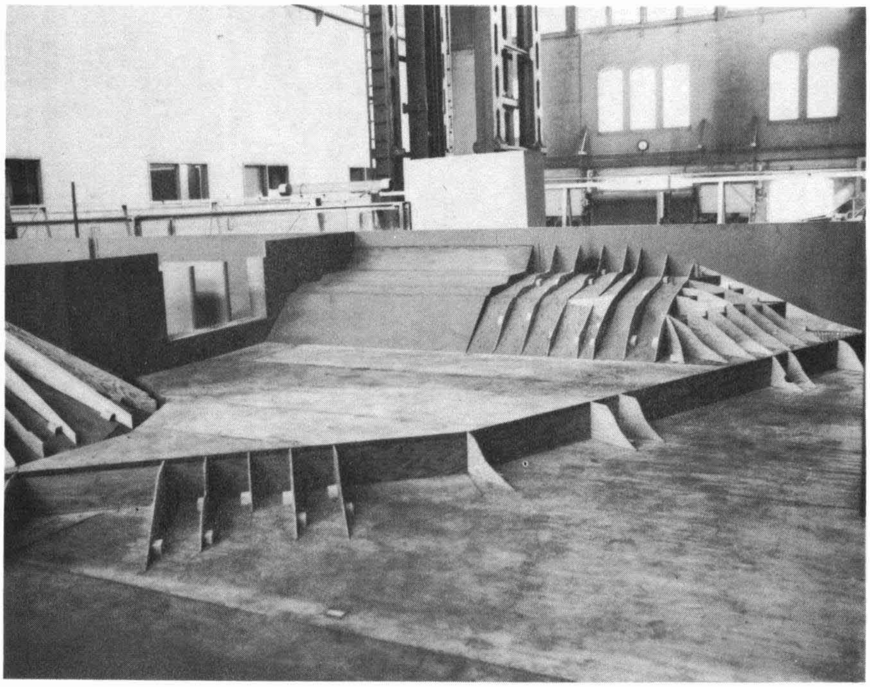


Fig. 2 Upstream Templates

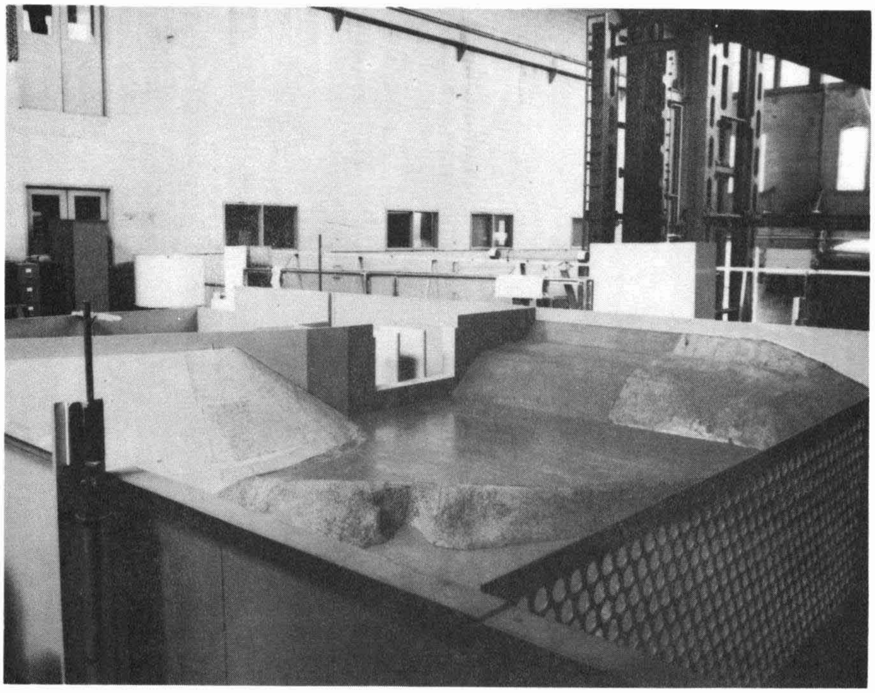


Fig. 3 Upstream Topography

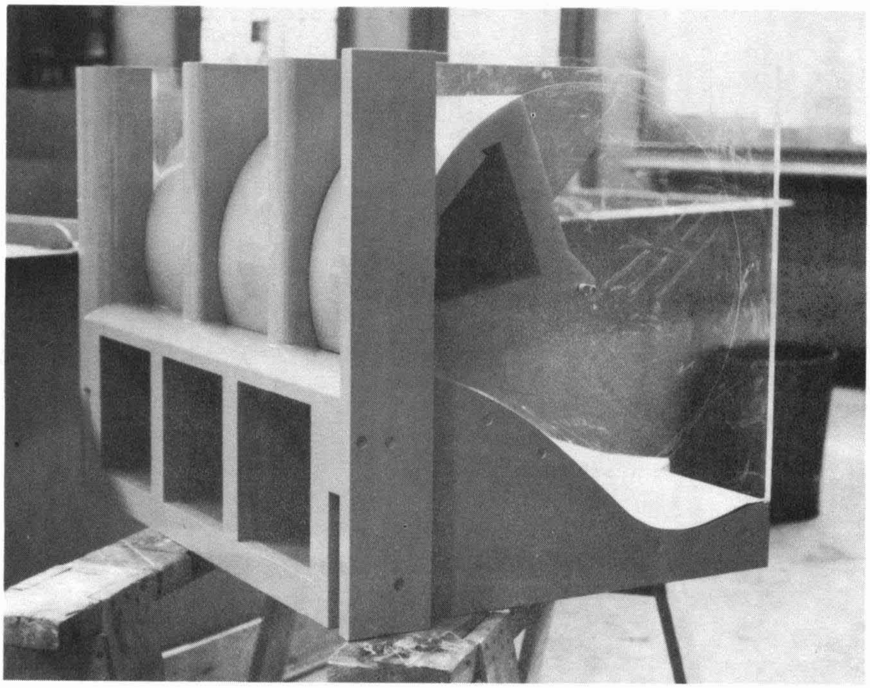


Fig. 4 Spillway Model

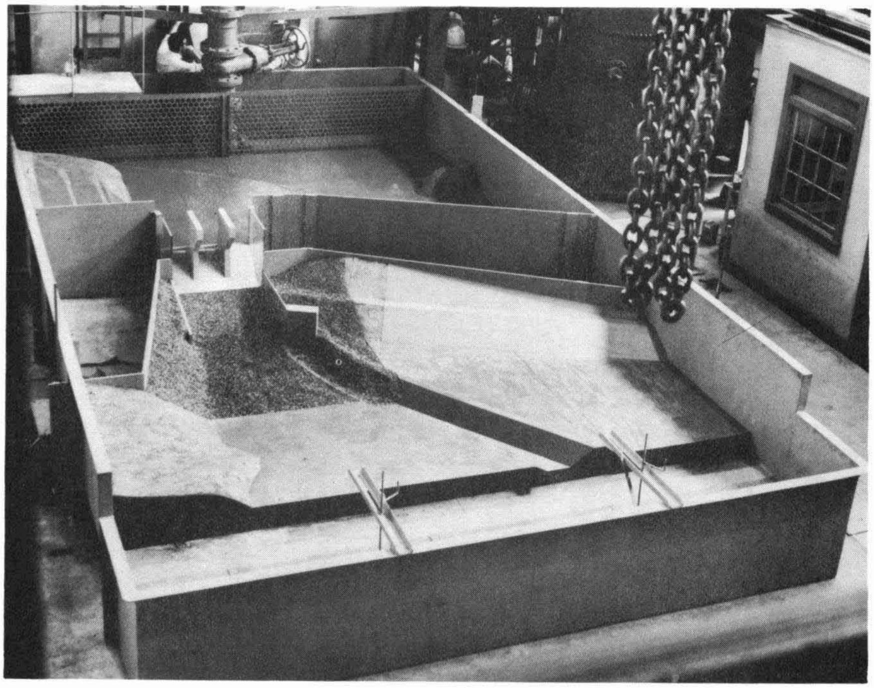


Fig. 5 Completed Model

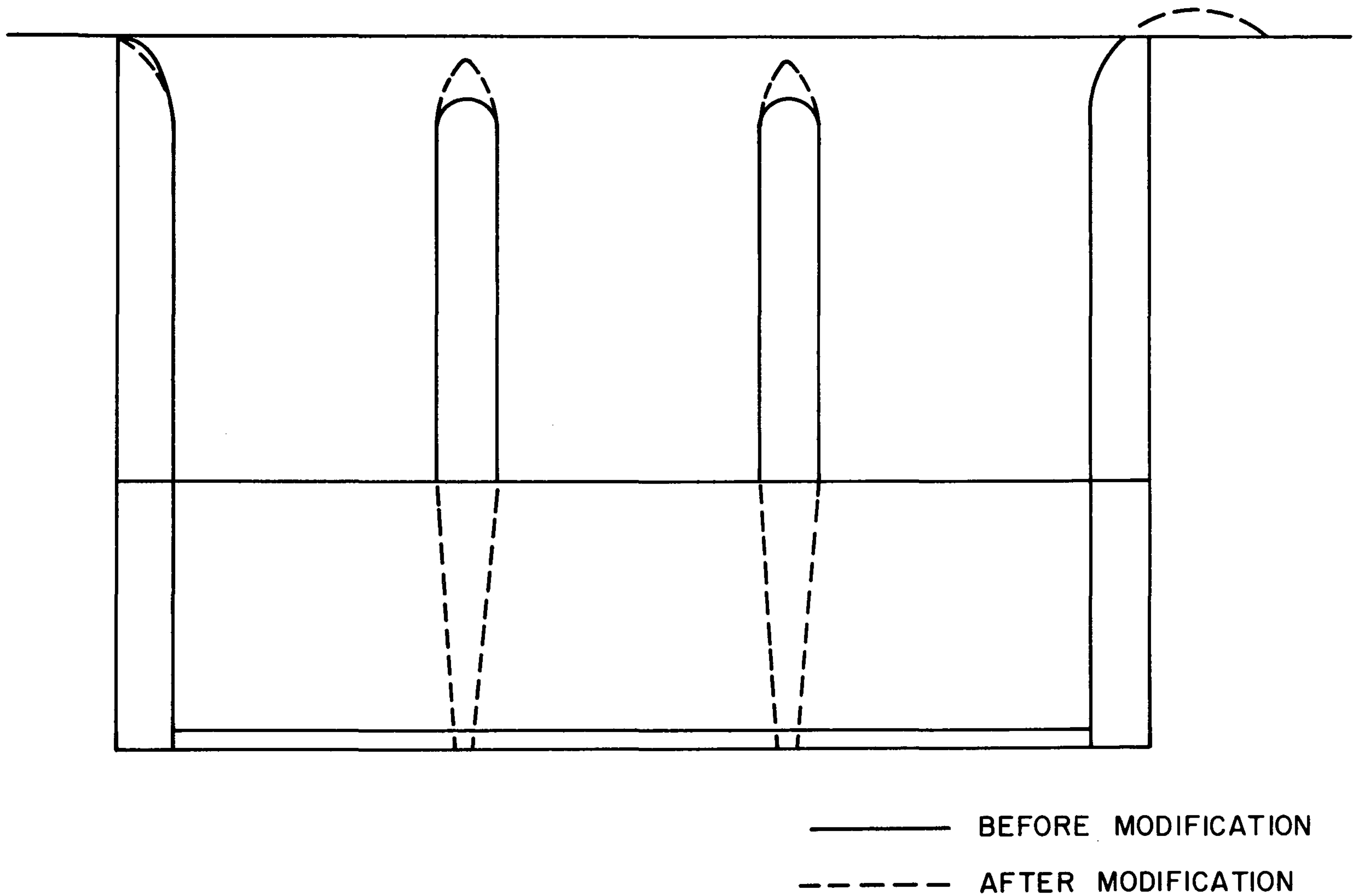


Fig. 6 Plan of Modified Spillway

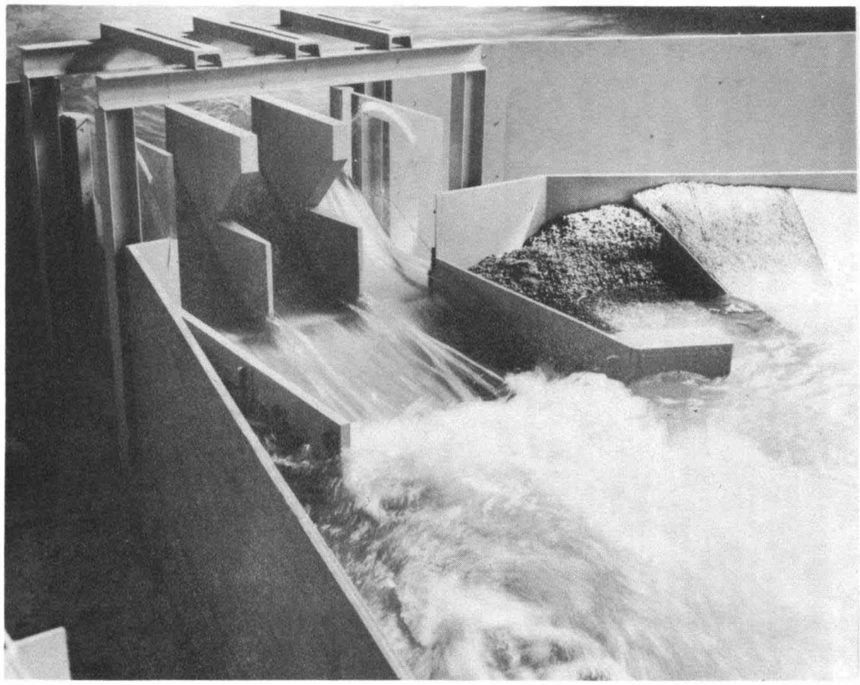


Fig. 7 Flip Bucket Operation
(Q = 62,500 cfs, Tailwater = 1135)

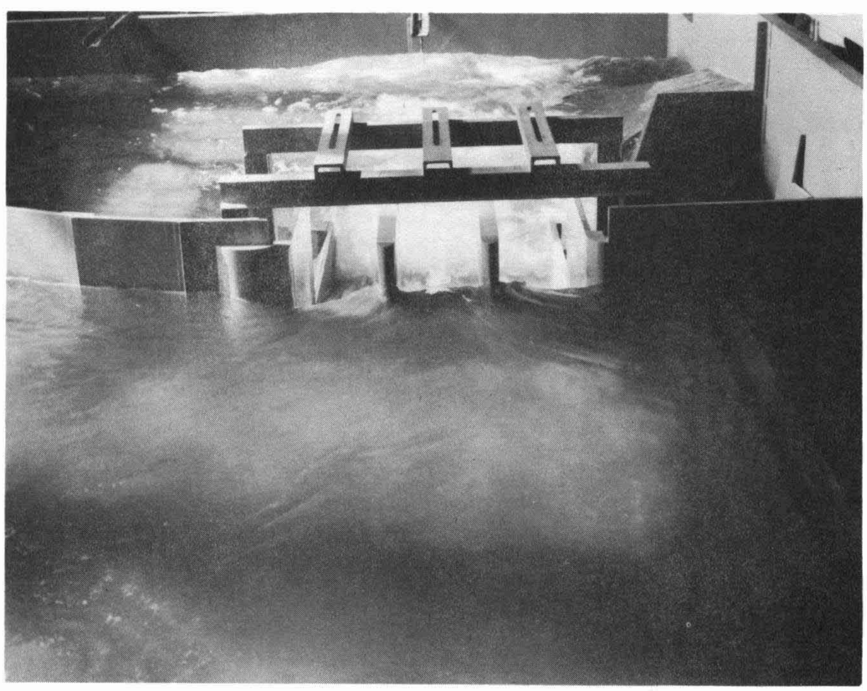


Fig. 8 Flow Distribution
(Q = 62,500 cfs)

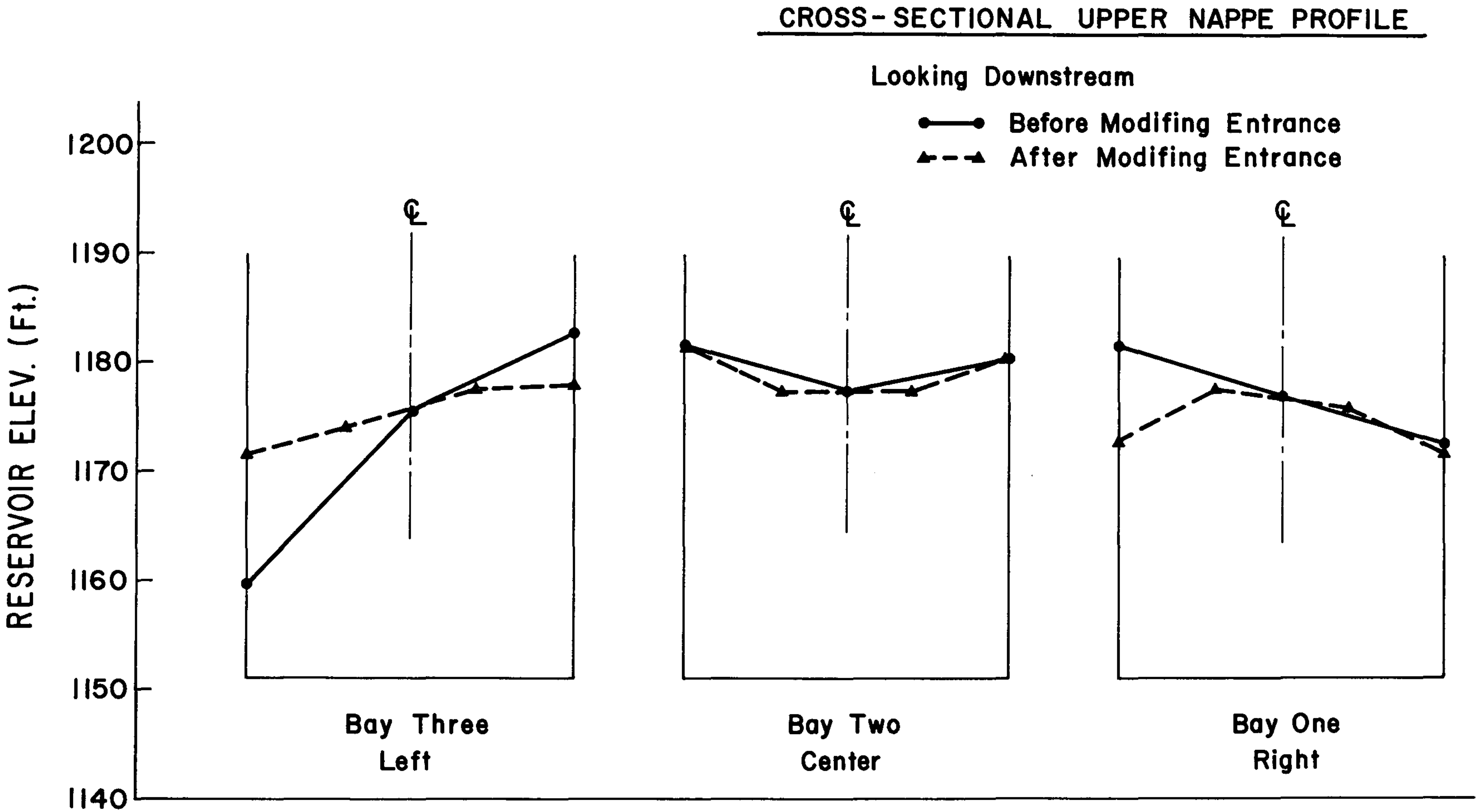


Fig. 9 Surface Profiles Along Crest

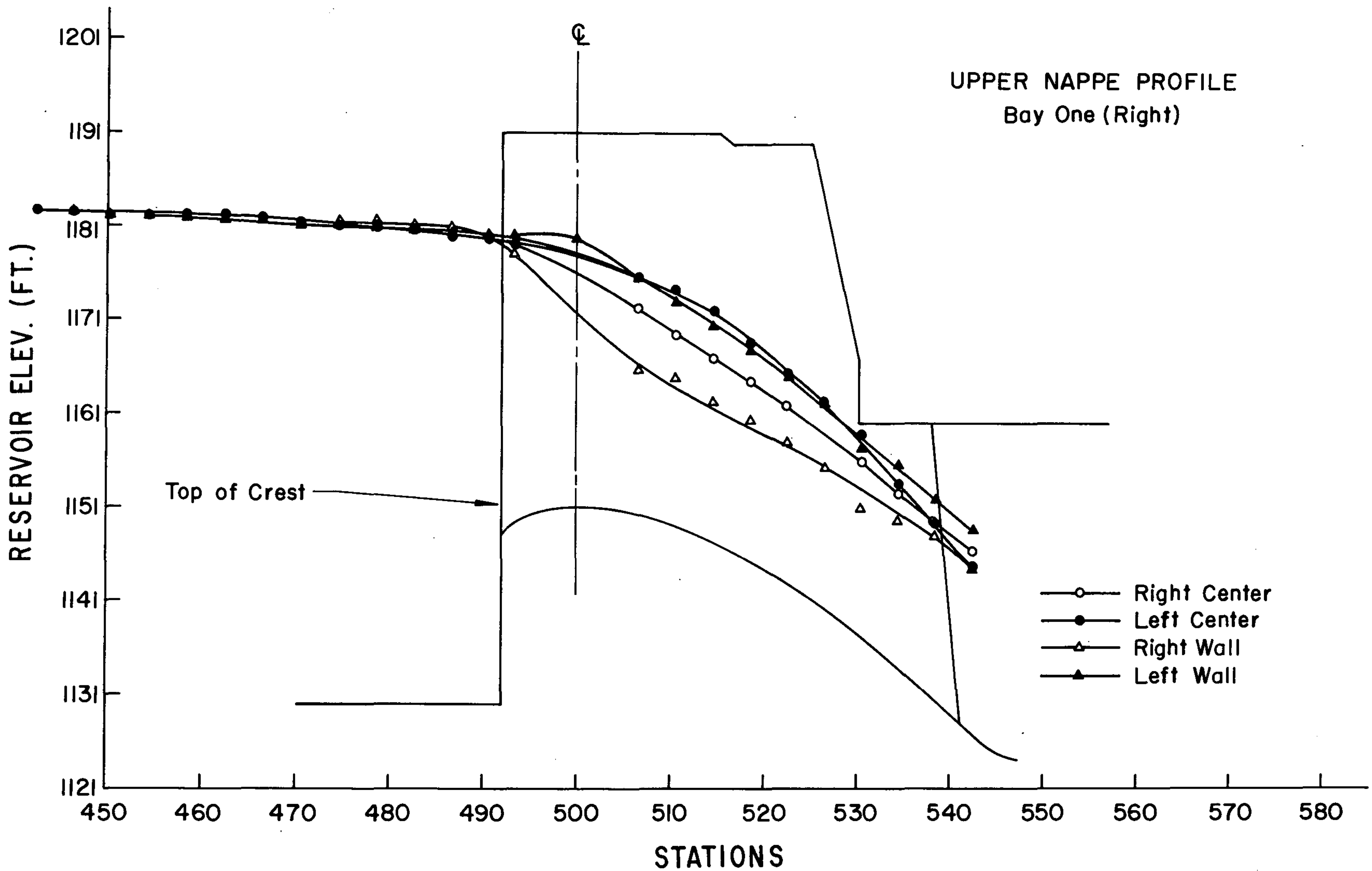


Fig. 10 Surface Profiles - Bay 1

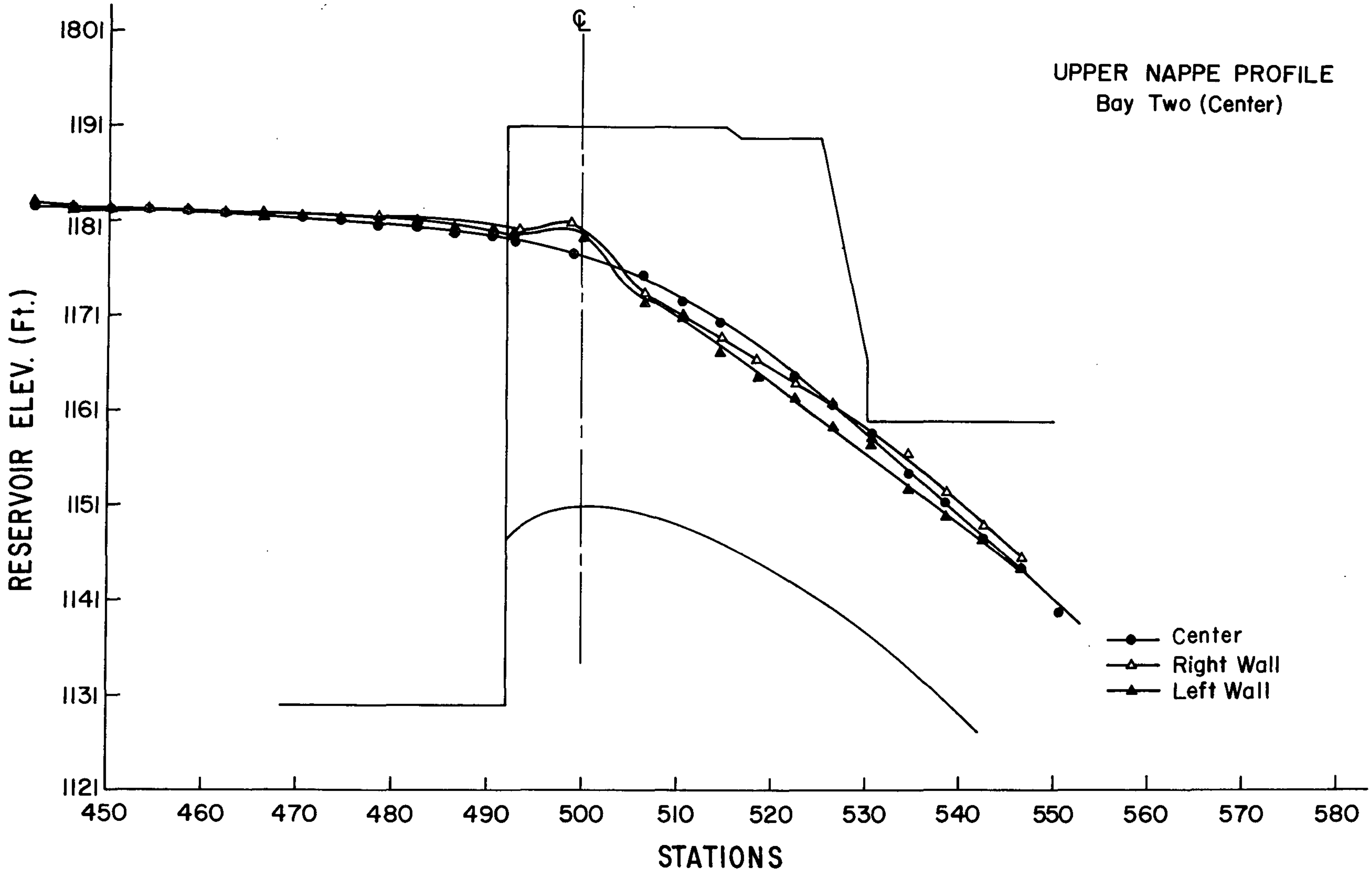


Fig. 11 Surface Profiles - Bay 2

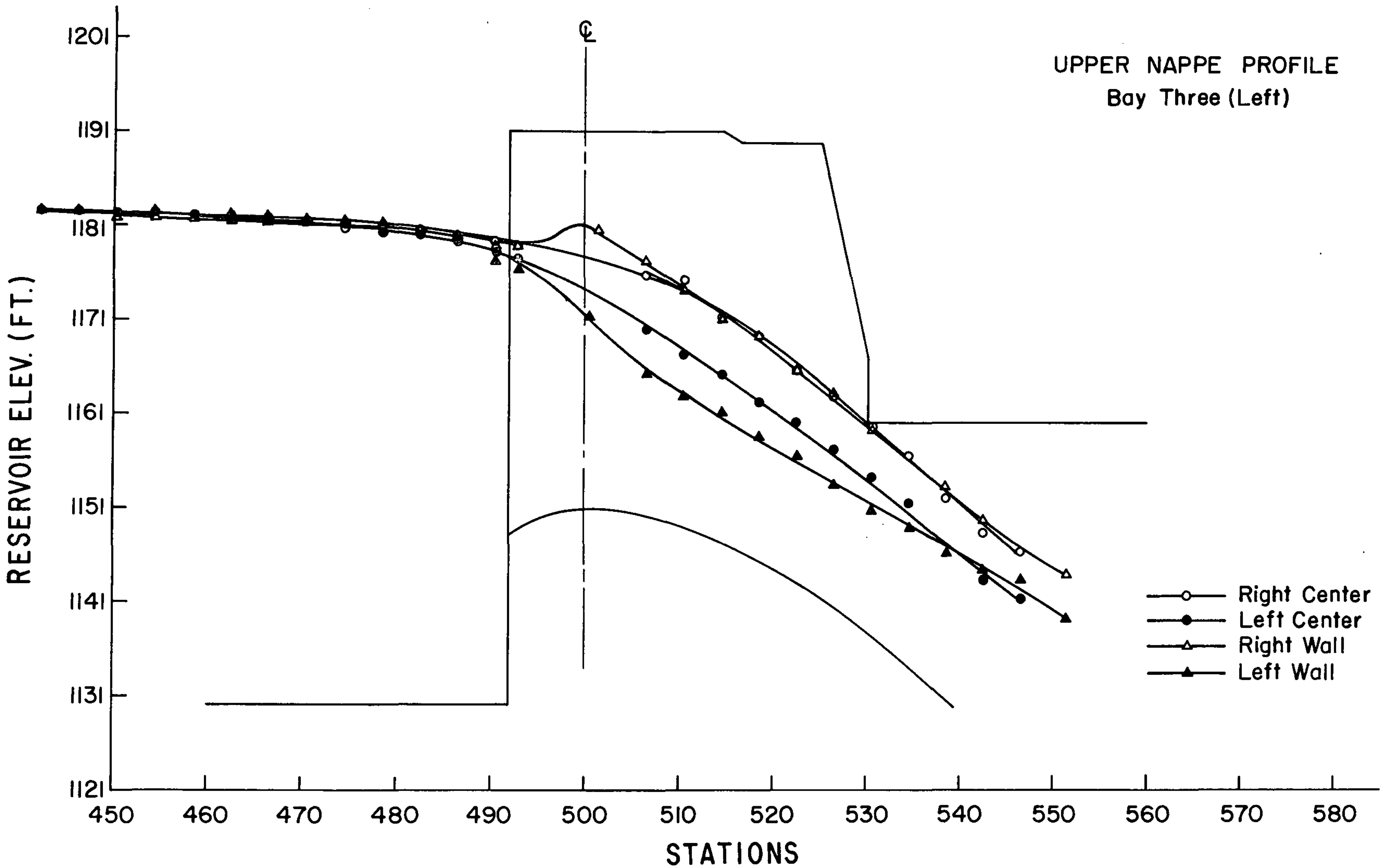


Fig. 12 Surface Profiles - Bay 3

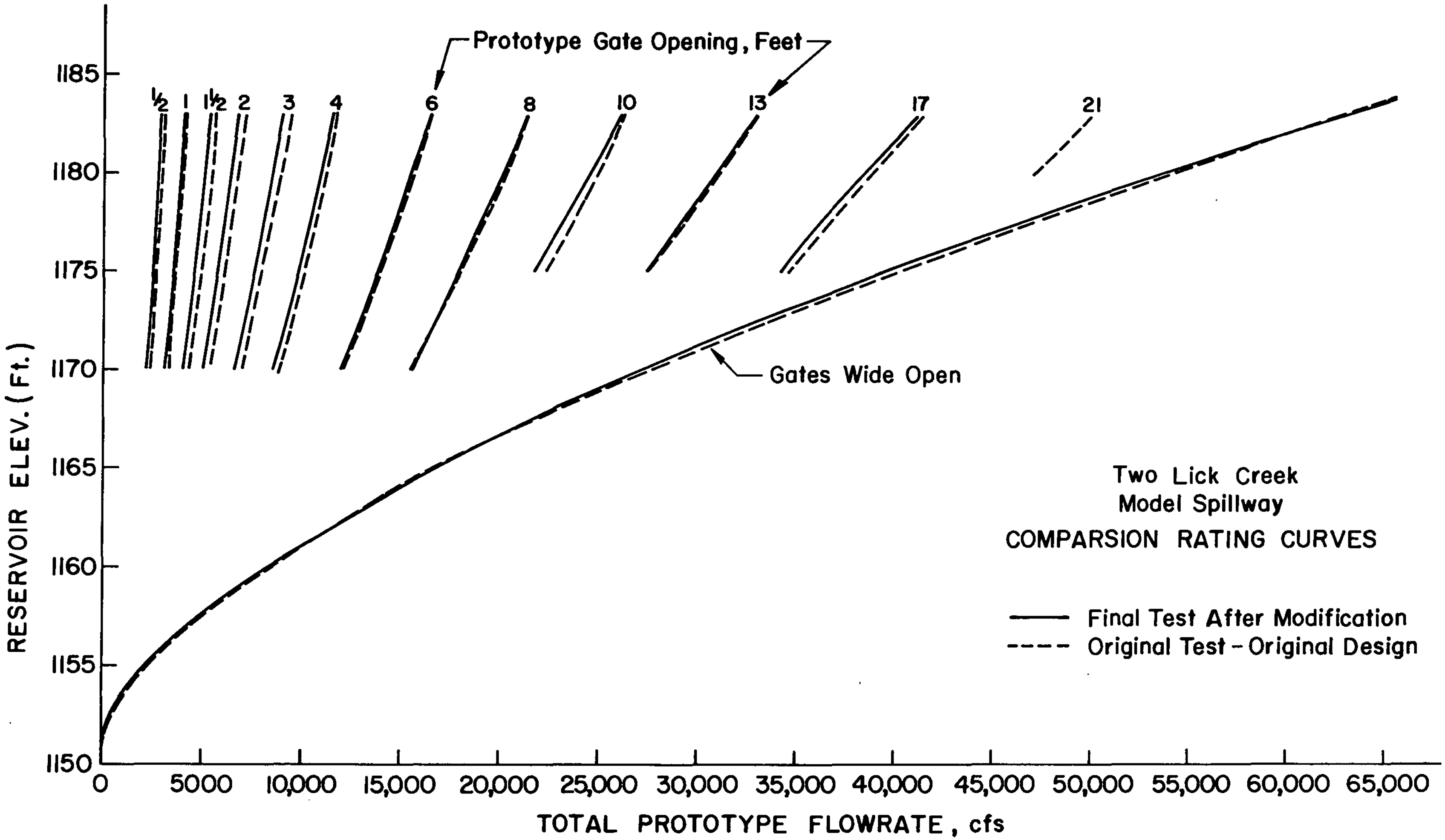


Fig. 13 Spillway Rating Curve

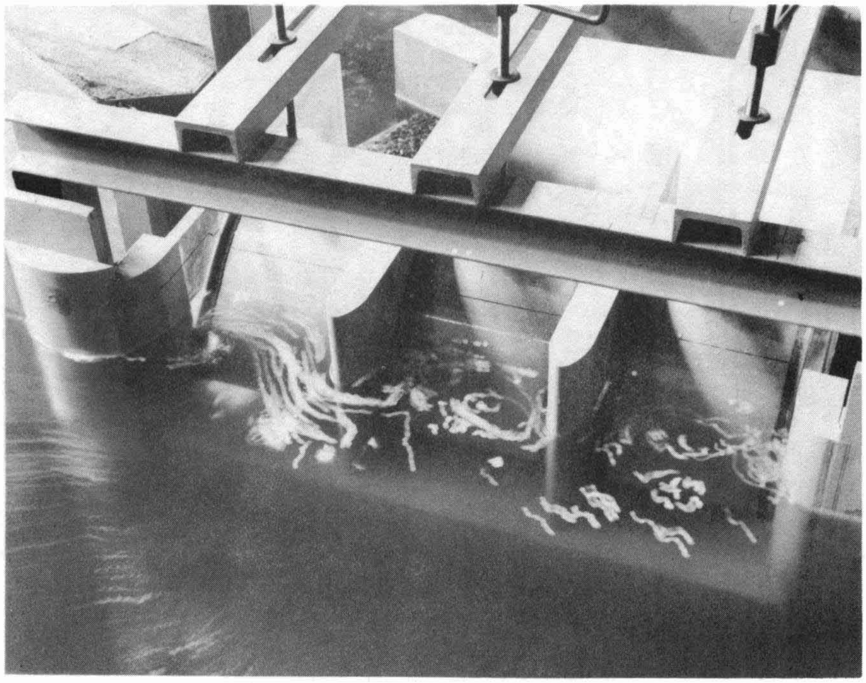


Fig. 14 Path Lines Show Circulation

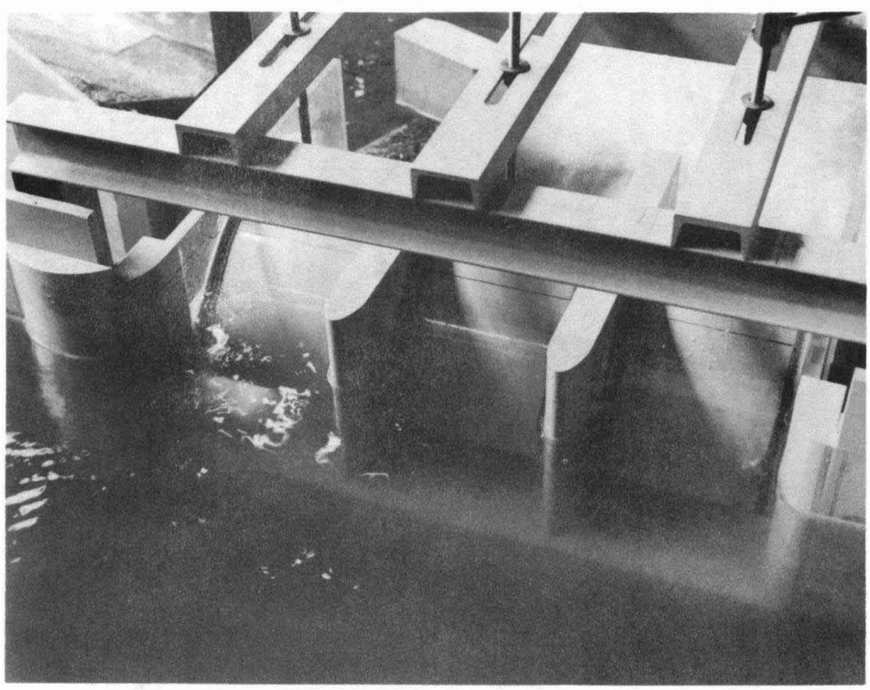


Fig. 15 Vortex Formation - Gates Open 10 Feet

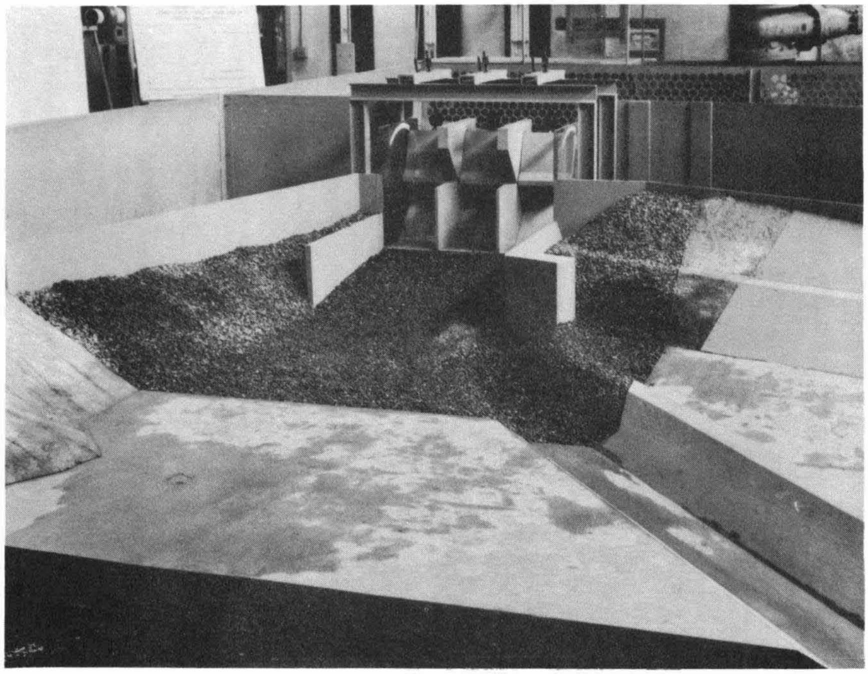


Fig. 16 Pea Gravel - Before Test

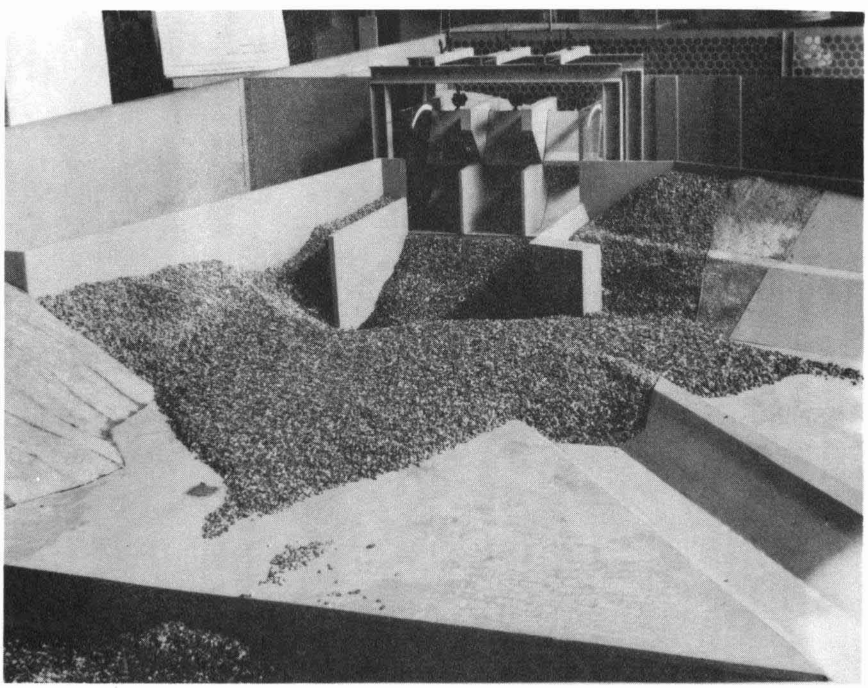


Fig. 17 Pea Gravel - After Standard Flood

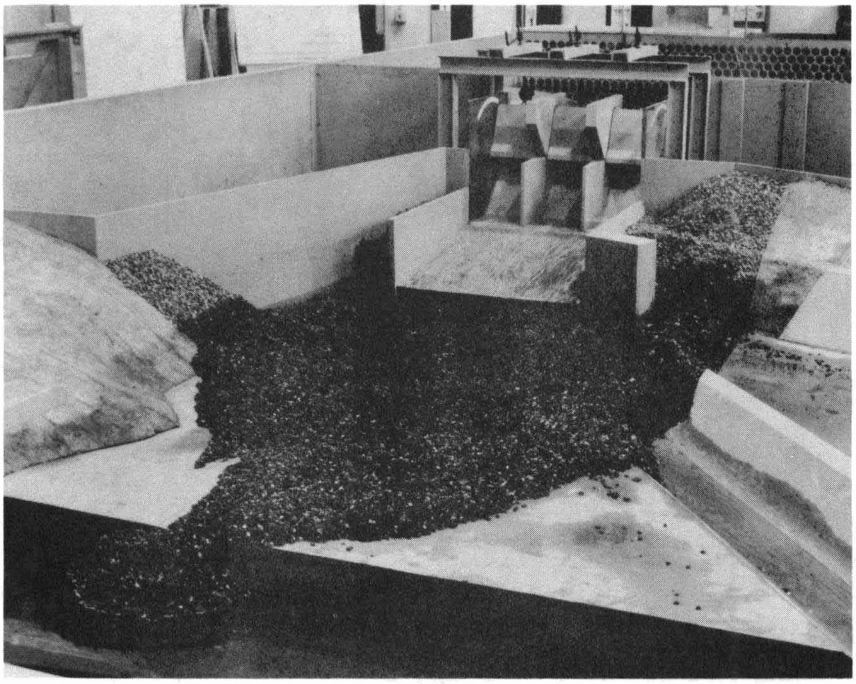


Fig. 18 Pea Gravel with Plywood Apron
- After Standard Flood

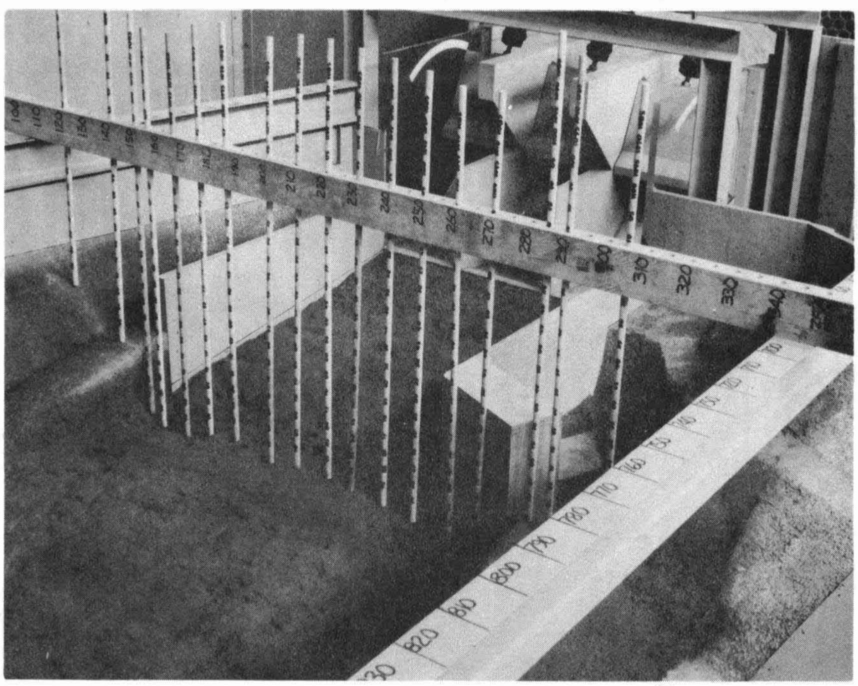
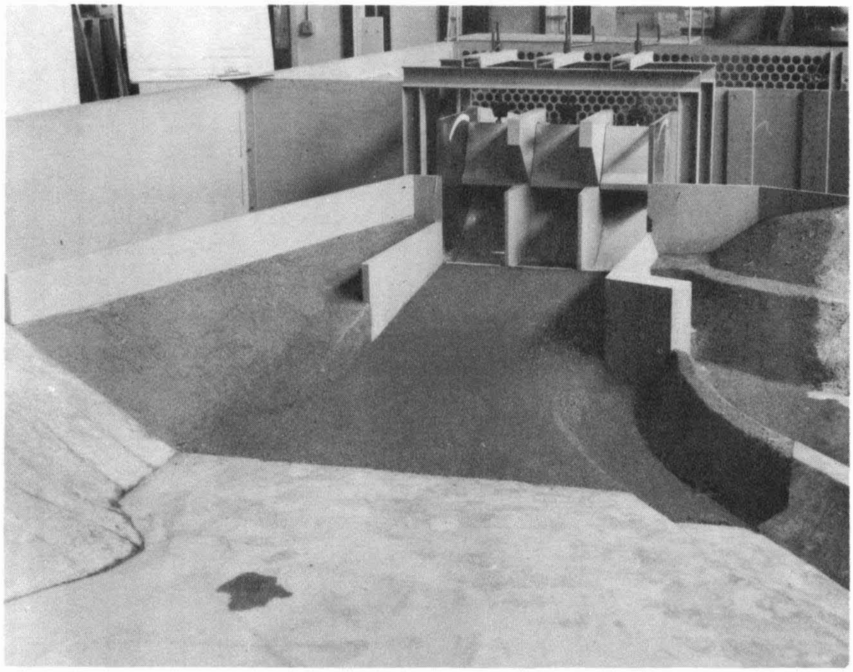
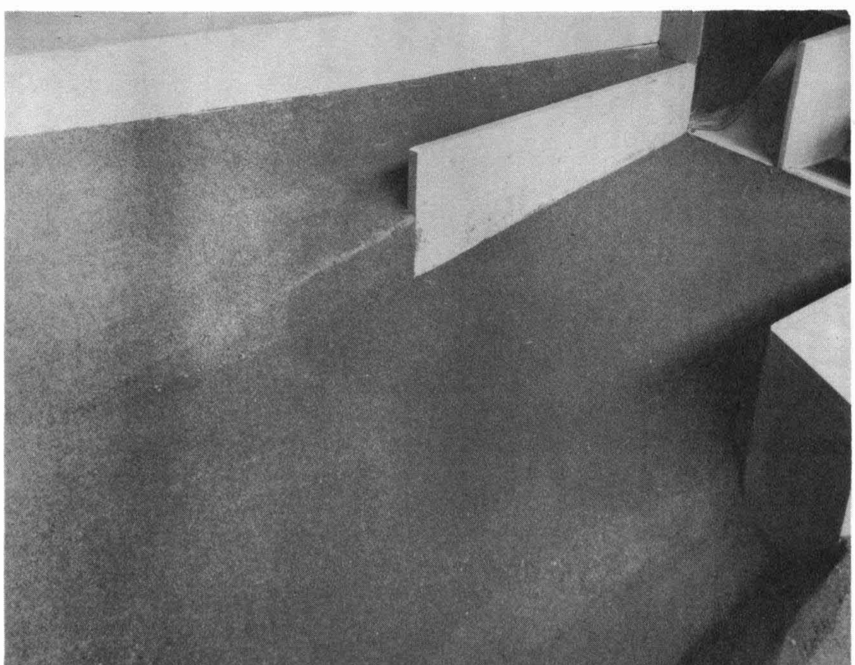


Fig. 19 Erosion Measurement

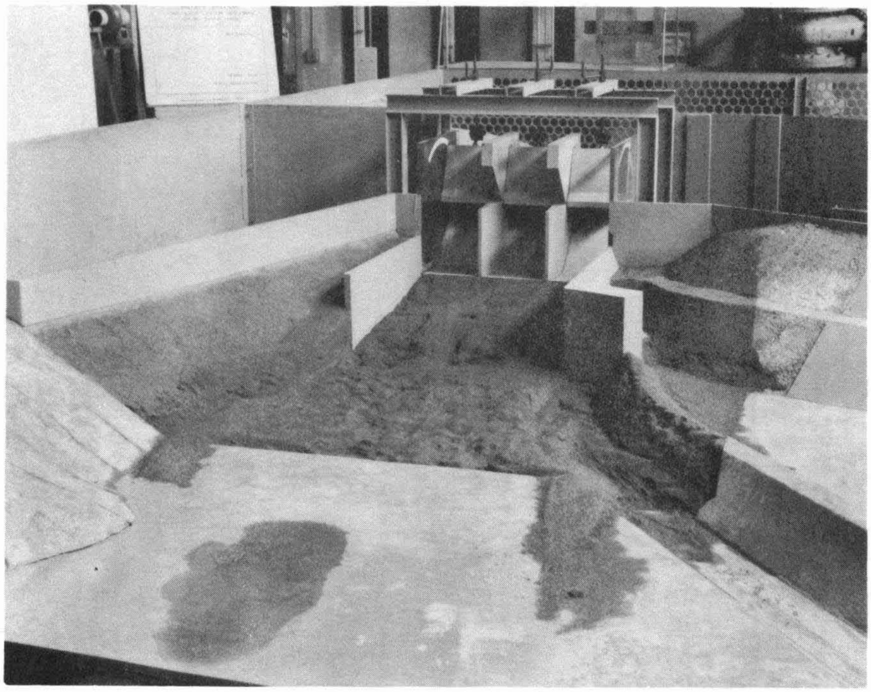


(a) Overall View

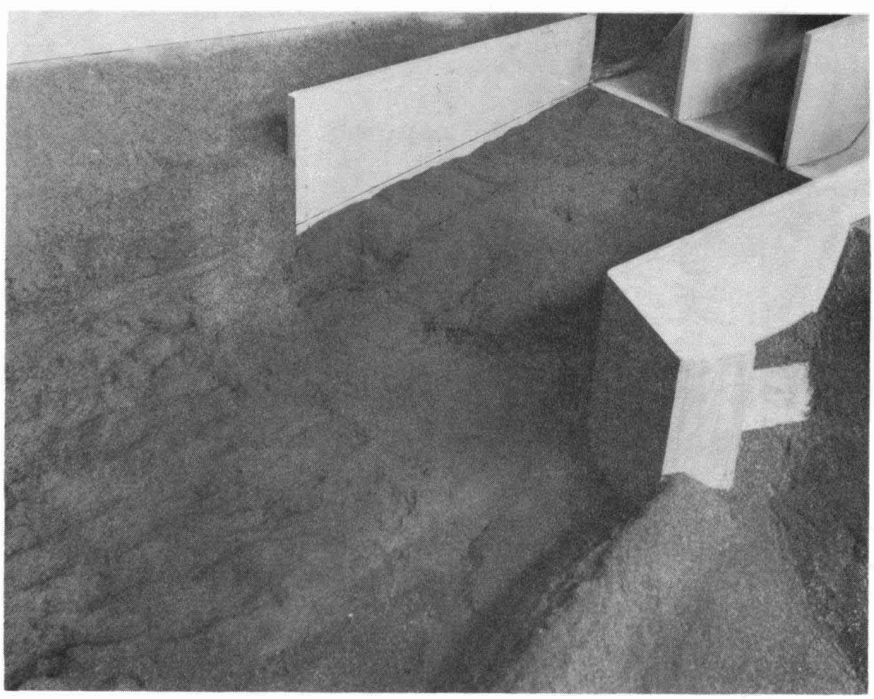


(b) Closeup View

Fig. 20 Erosion Bed Ready for Test

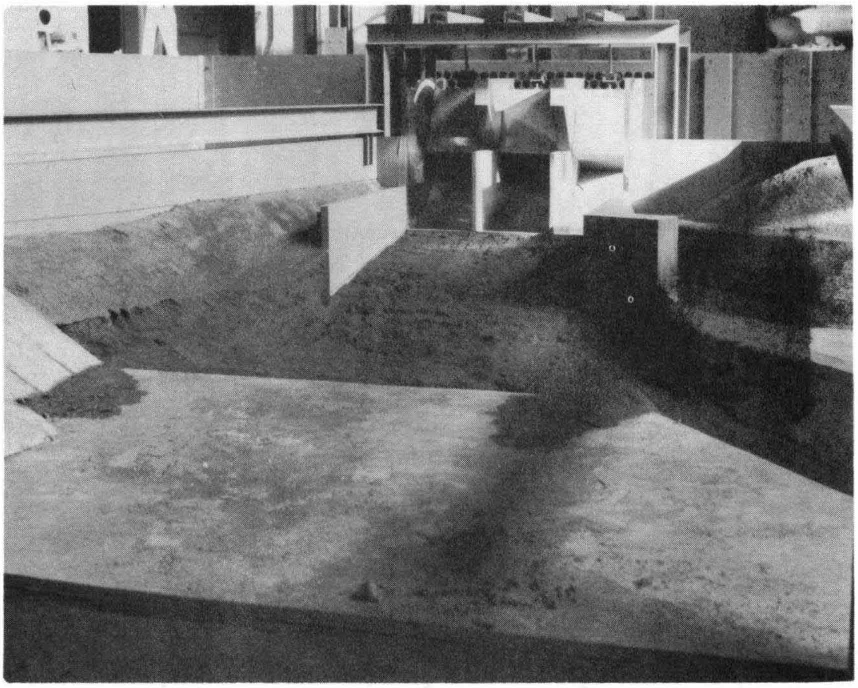


(a) Overall View

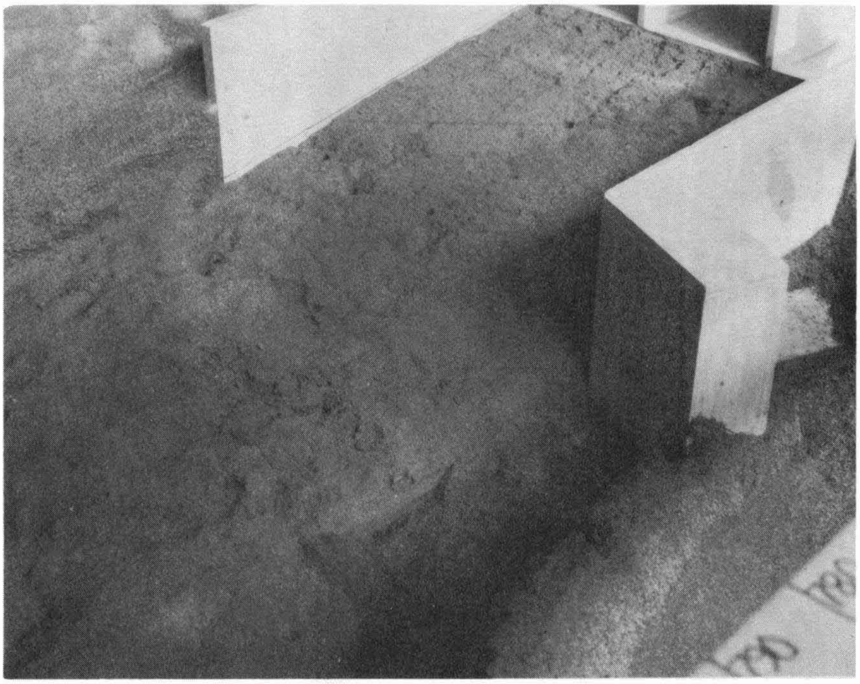


(b) Closeup View

Fig. 21 Erosion Test No. 1 - Mix 170:1, Standard Flood

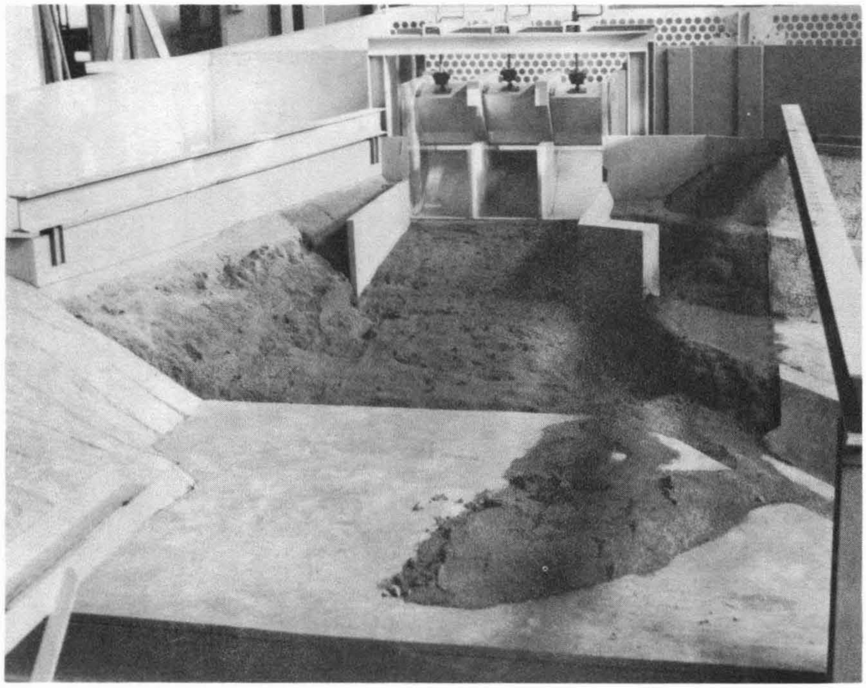


(a) Overall View

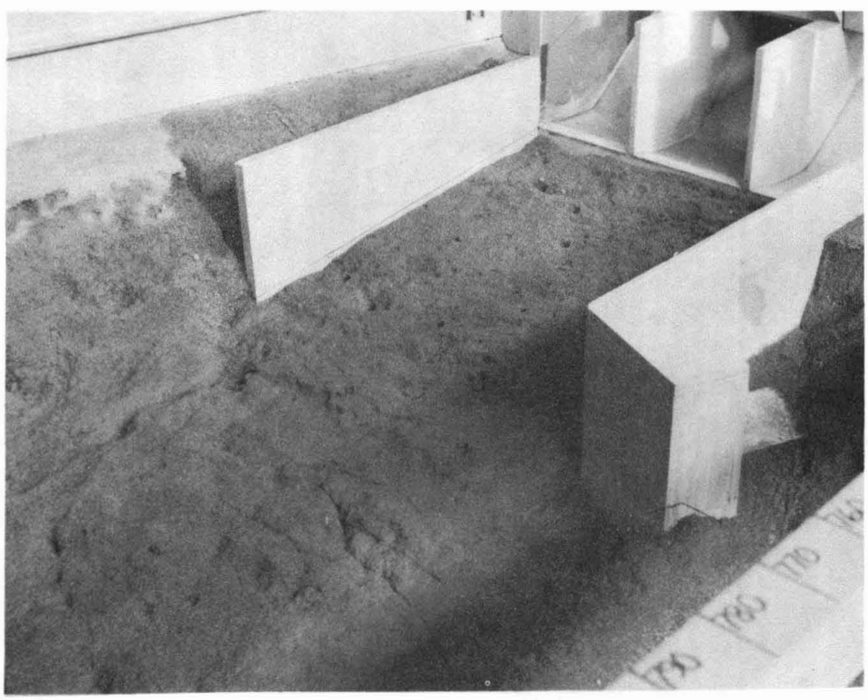


(b) Closeup View

Fig. 22 Erosion Test No. 2 - Mix 225:1, Standard Flood

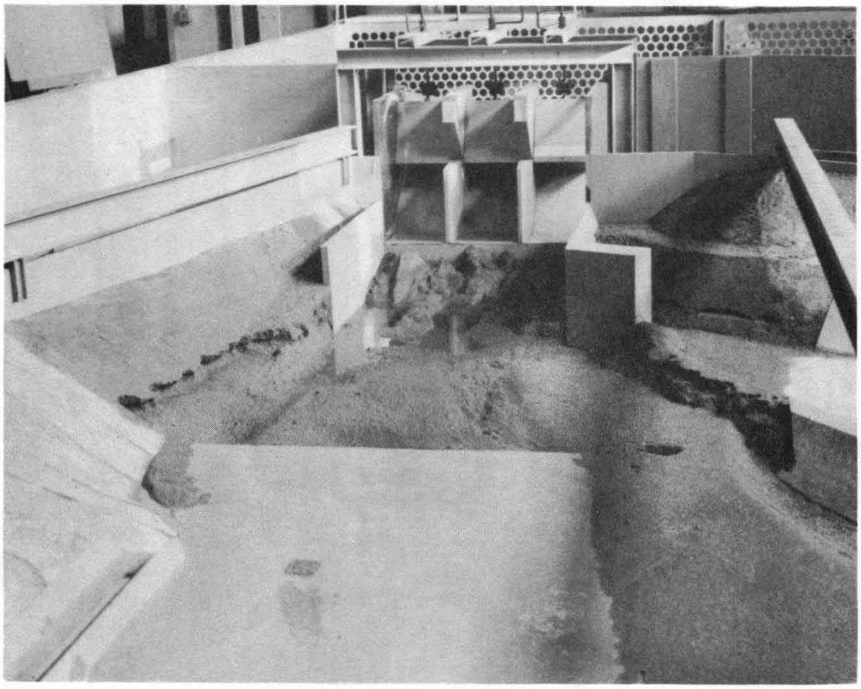


(a) Overall View

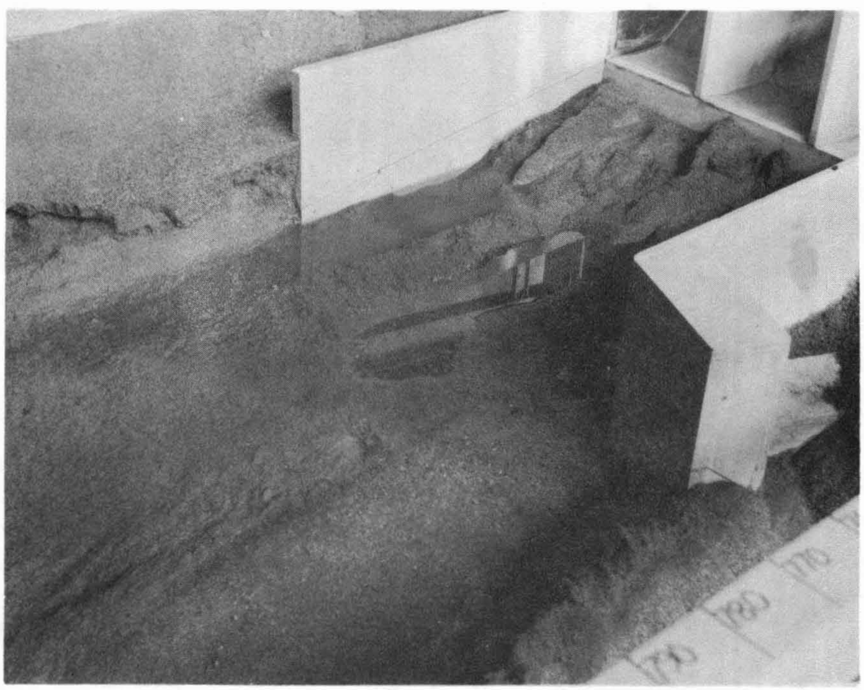


(b) Closeup View

Fig. 23 Erosion Test No. 3 - Mix 225:1, Spillway Design Flood

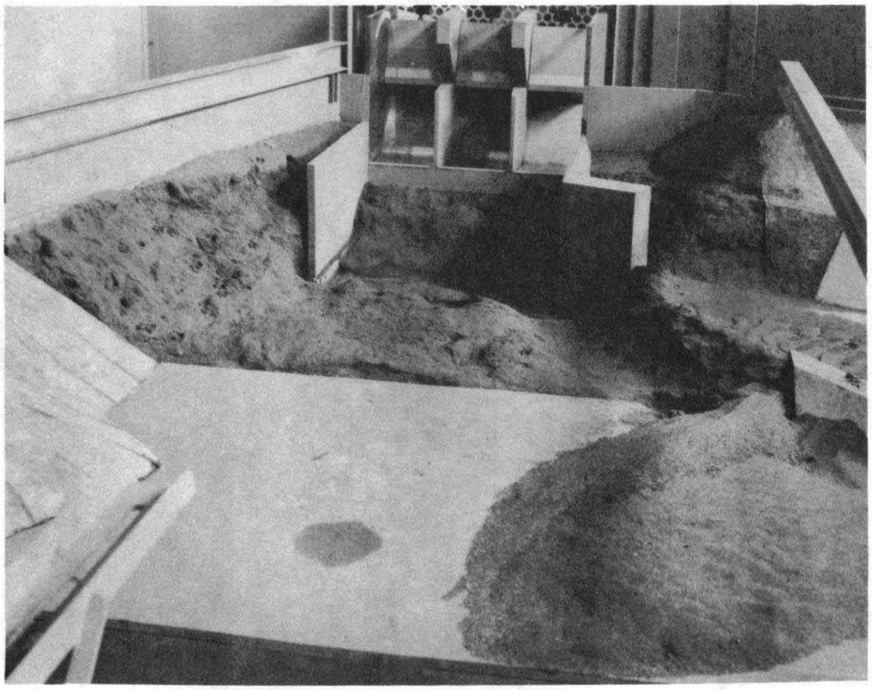


(a) Overall View

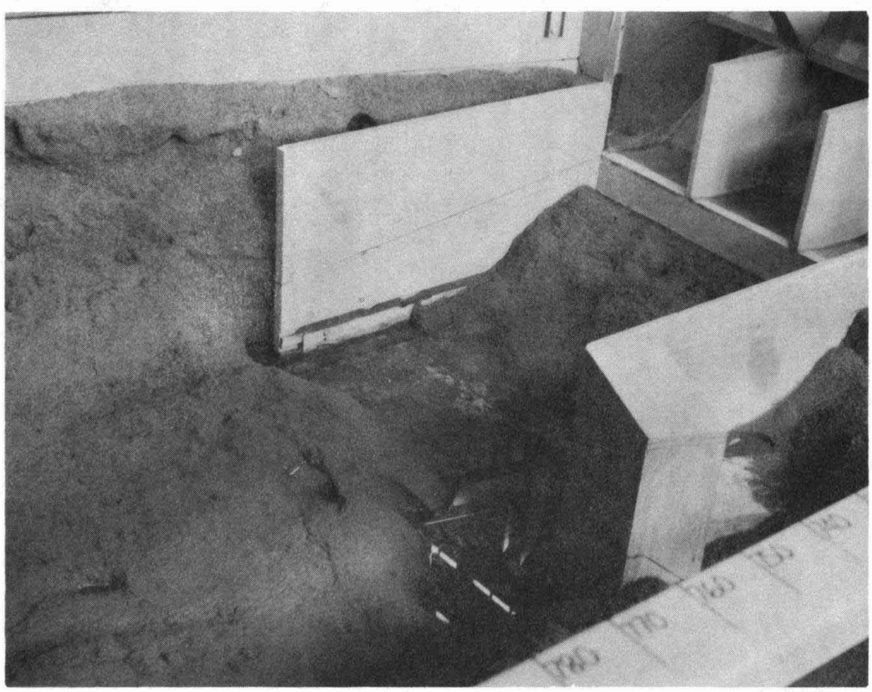


(b) Closeup View

Fig. 24 Erosion Test No. 4 - Mix 250:1, Standard Flood

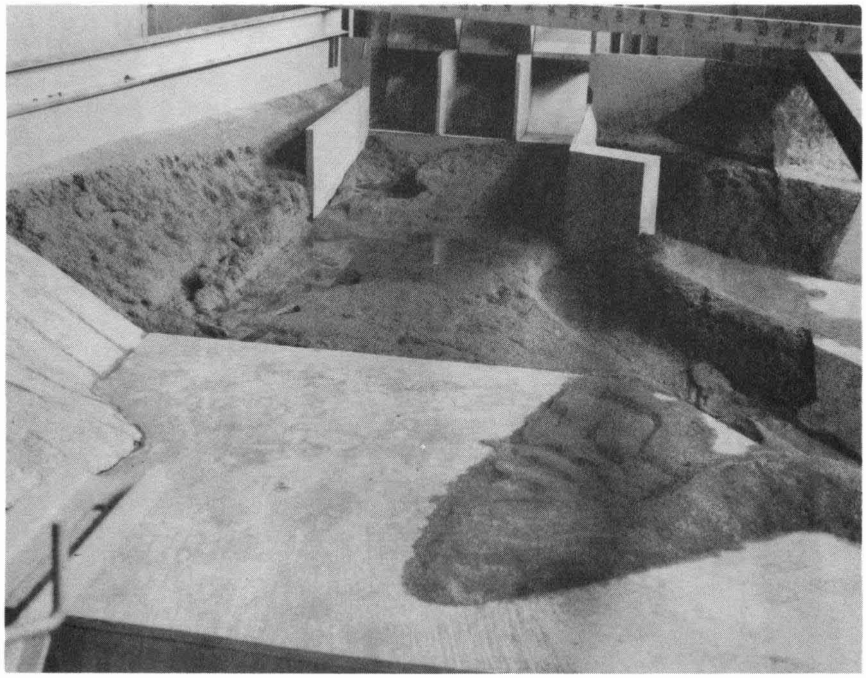


(a) Overall View

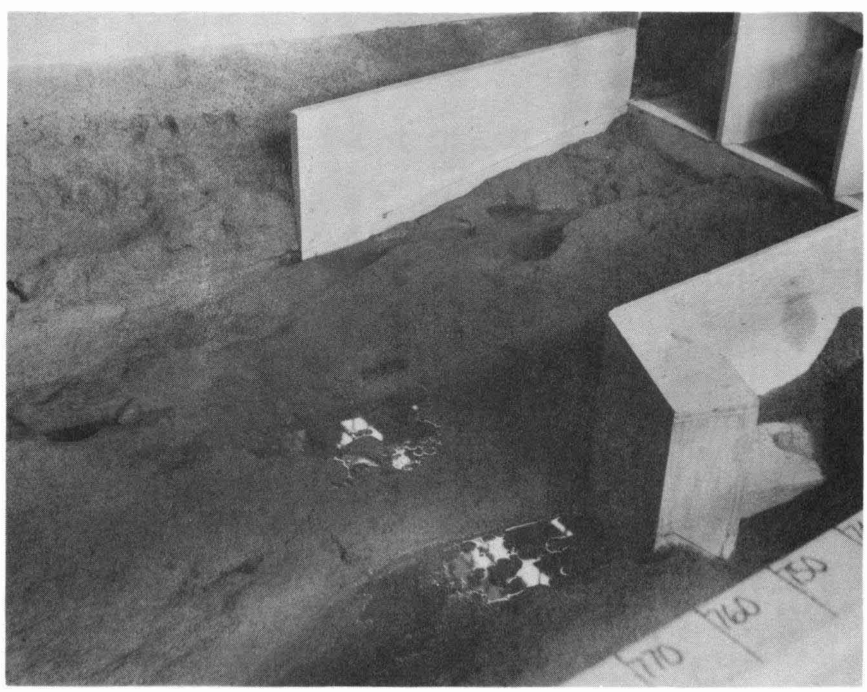


(b) Closeup View

Fig. 25 Erosion Test No. 5 - Mix 250:1, Spillway Design Flood

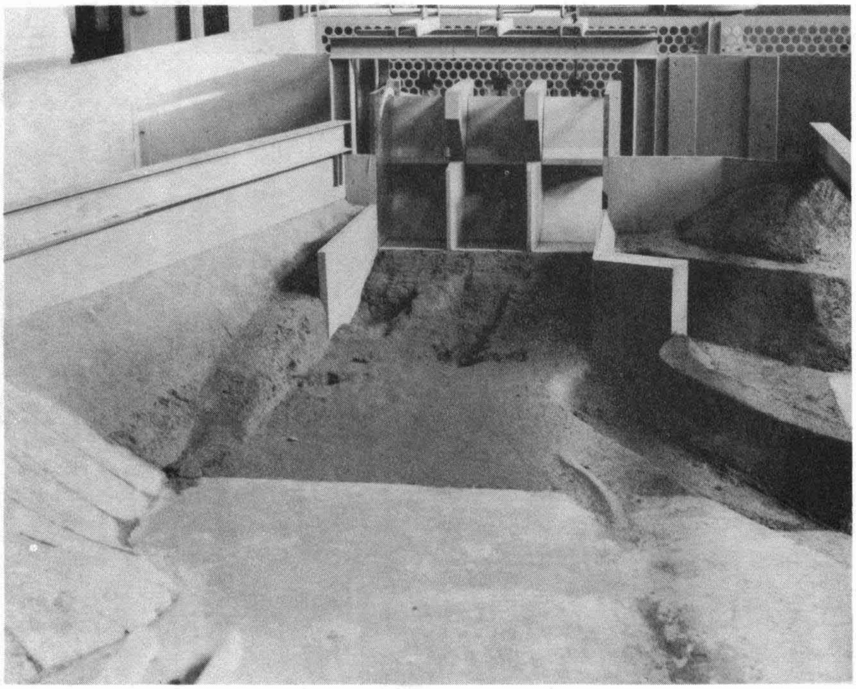


(a) Overall View

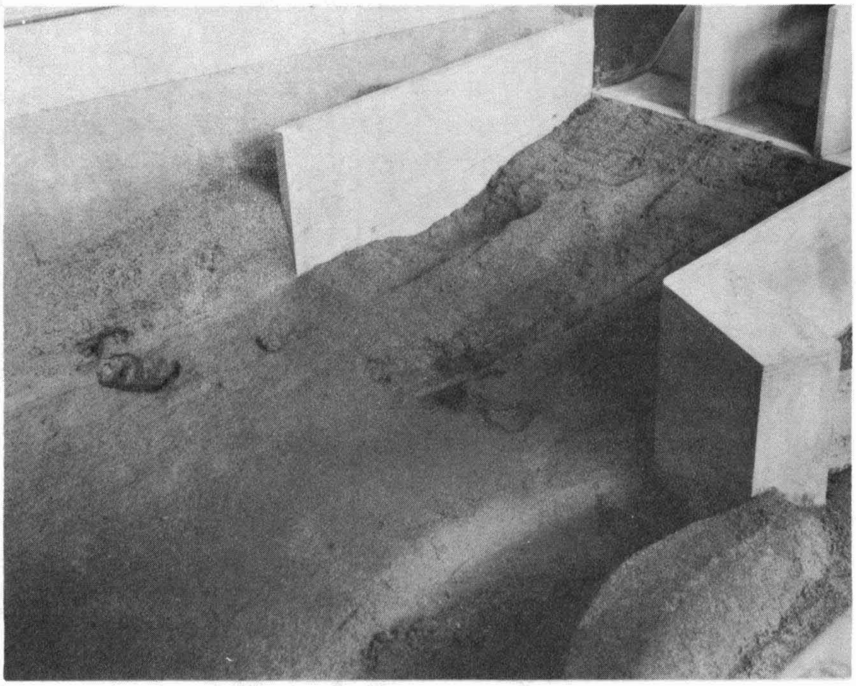


(b) Closeup View

Fig. 26 Erosion Test No. 6 - Mix 225:1, Spillway Design Flood

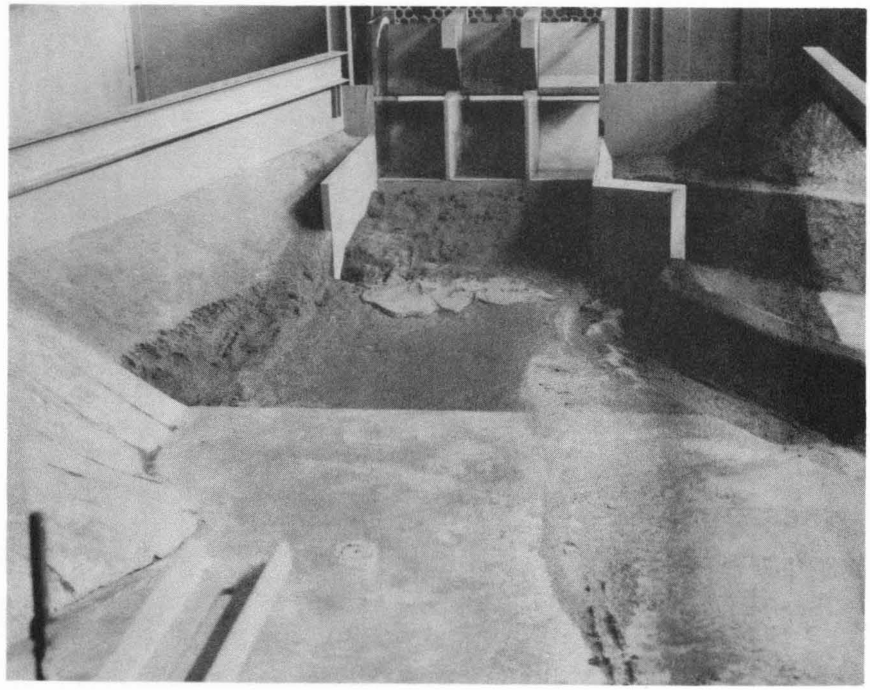


(a) Overall View

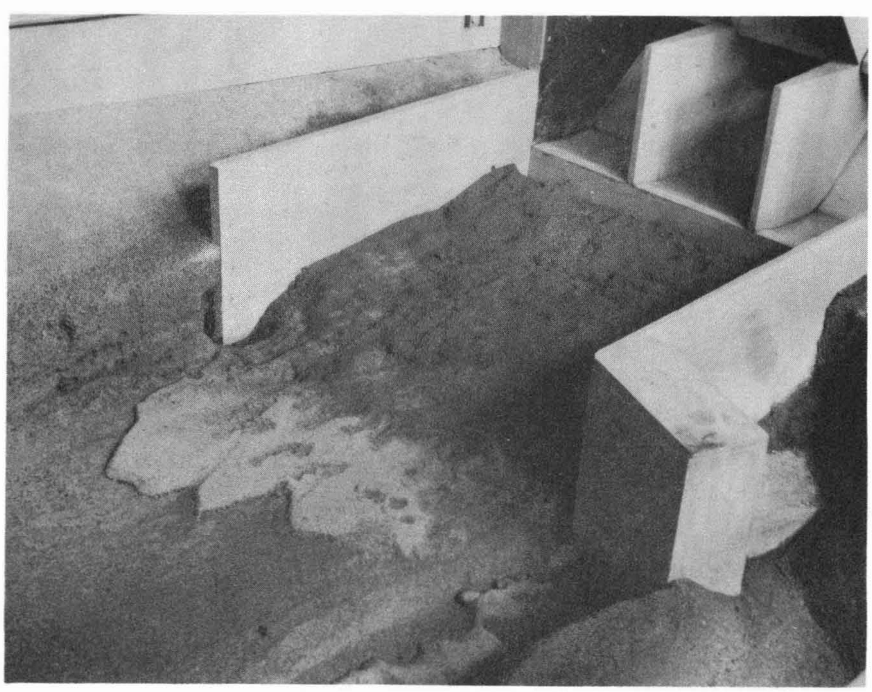


(b) Closeup View

Fig. 27 Erosion Test No. 7 - Mix 210:1, Standard Flood

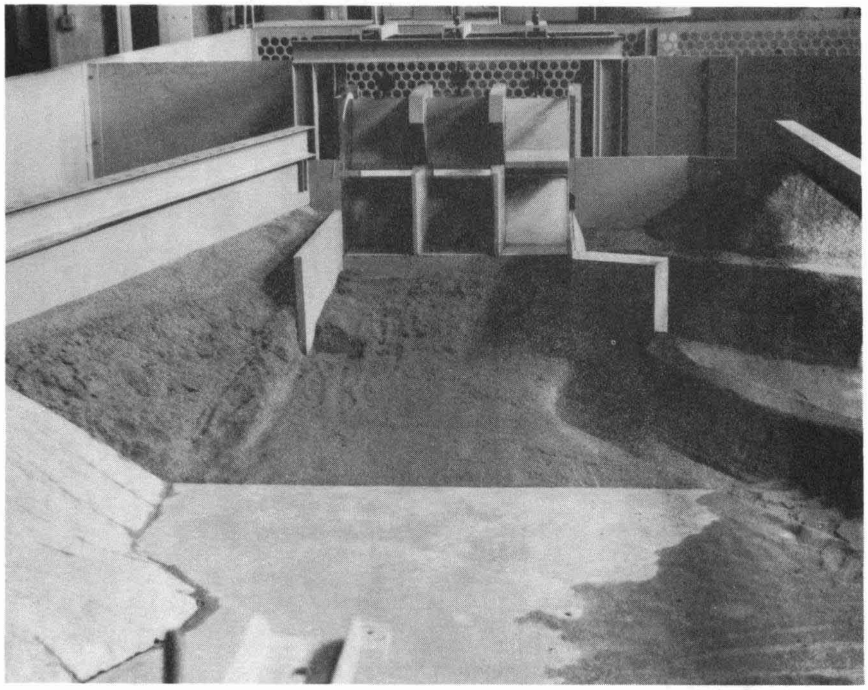


(a) Overall View

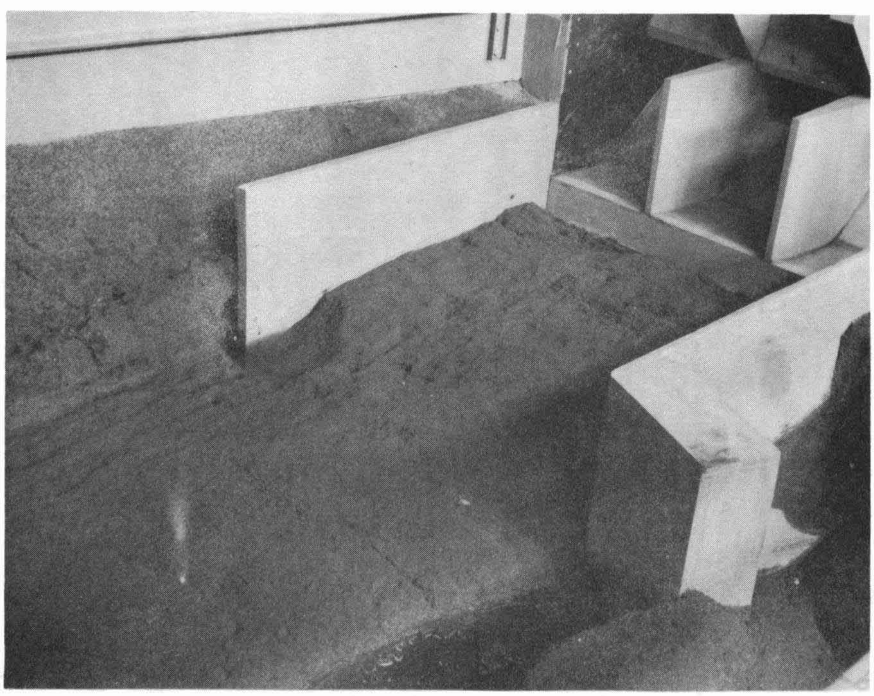


(b) Closeup View

Fig. 28 Erosion Test No 9 - Mix 225:1, Standard Flood



(a) Overall View



(b) Closeup View

Fig. 29 Erosion Test No. 10 - Mix 225:1, Standard Flood

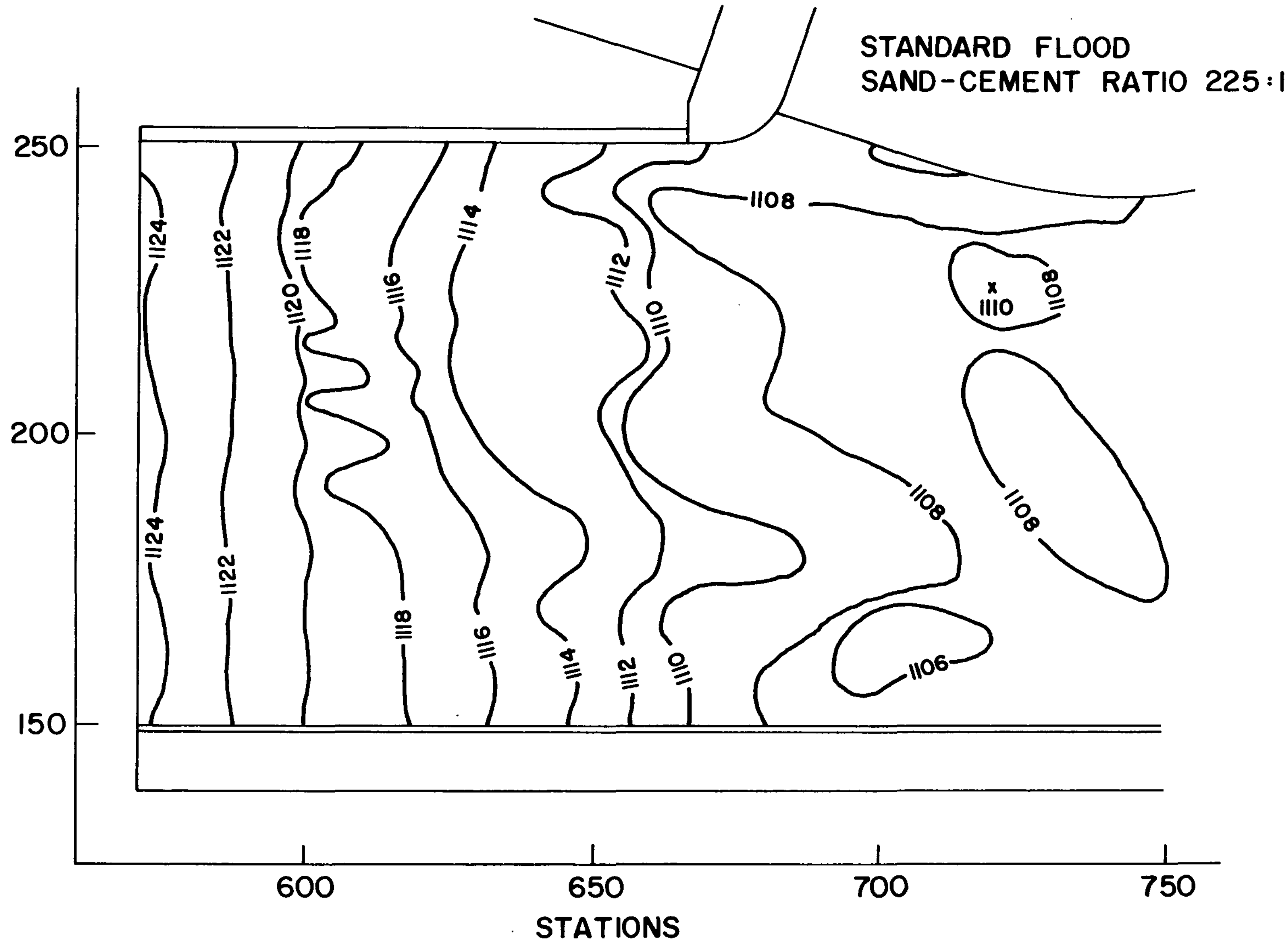


Fig. 30 Contour Map - Test No. 2

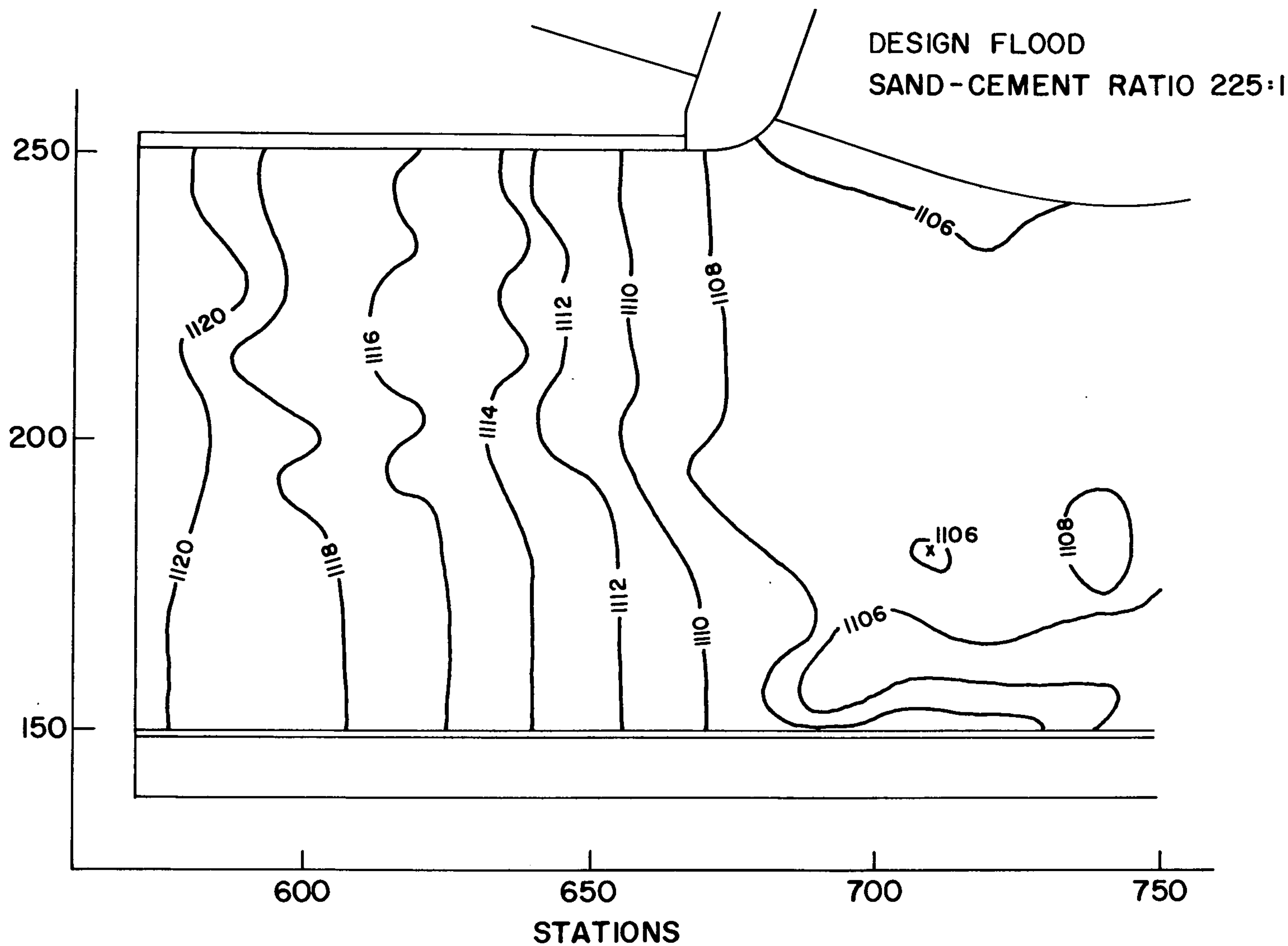


Fig. 31 Contour Map - Test No. 3

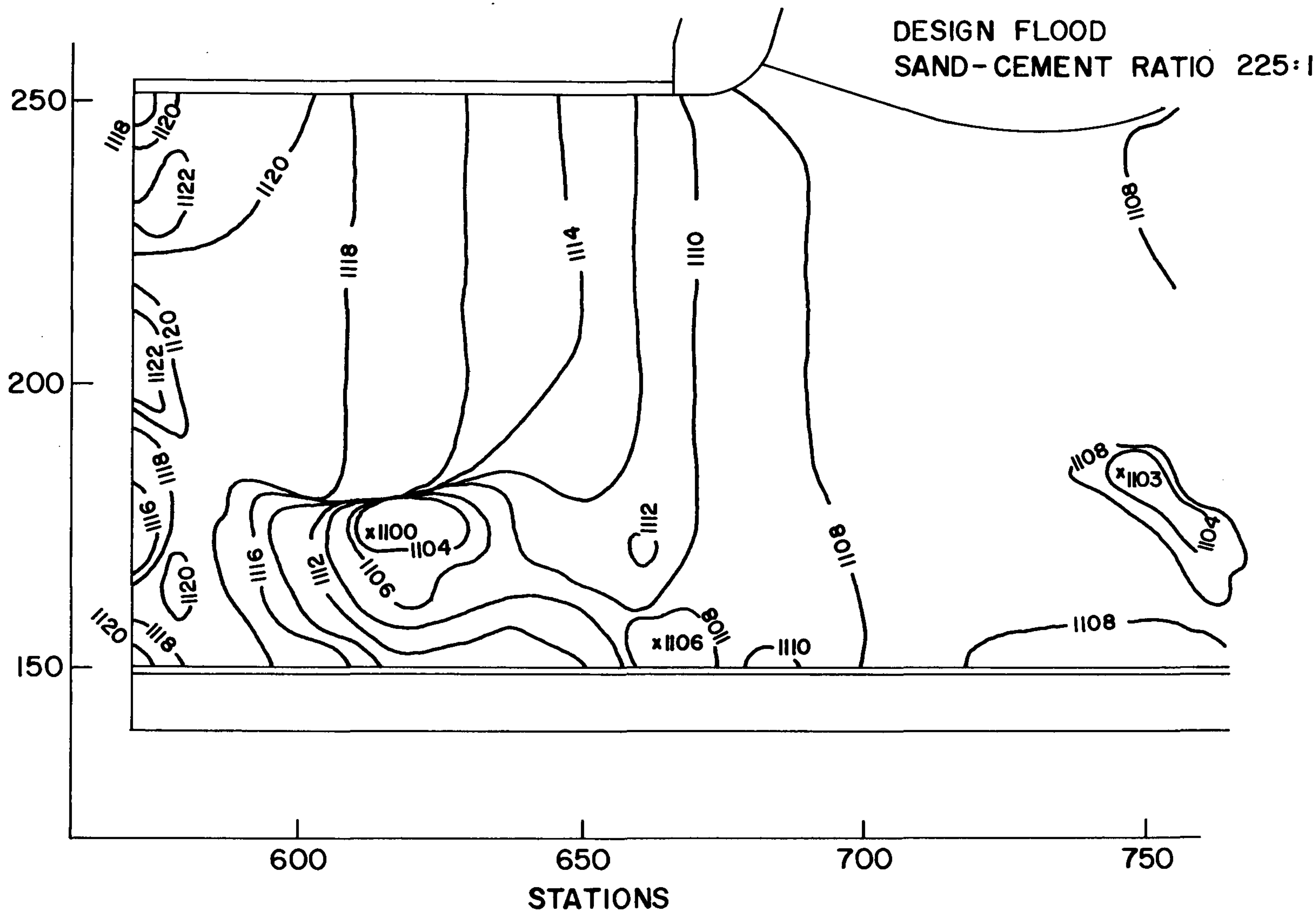


Fig. 32 Contour Map - Test No. 6

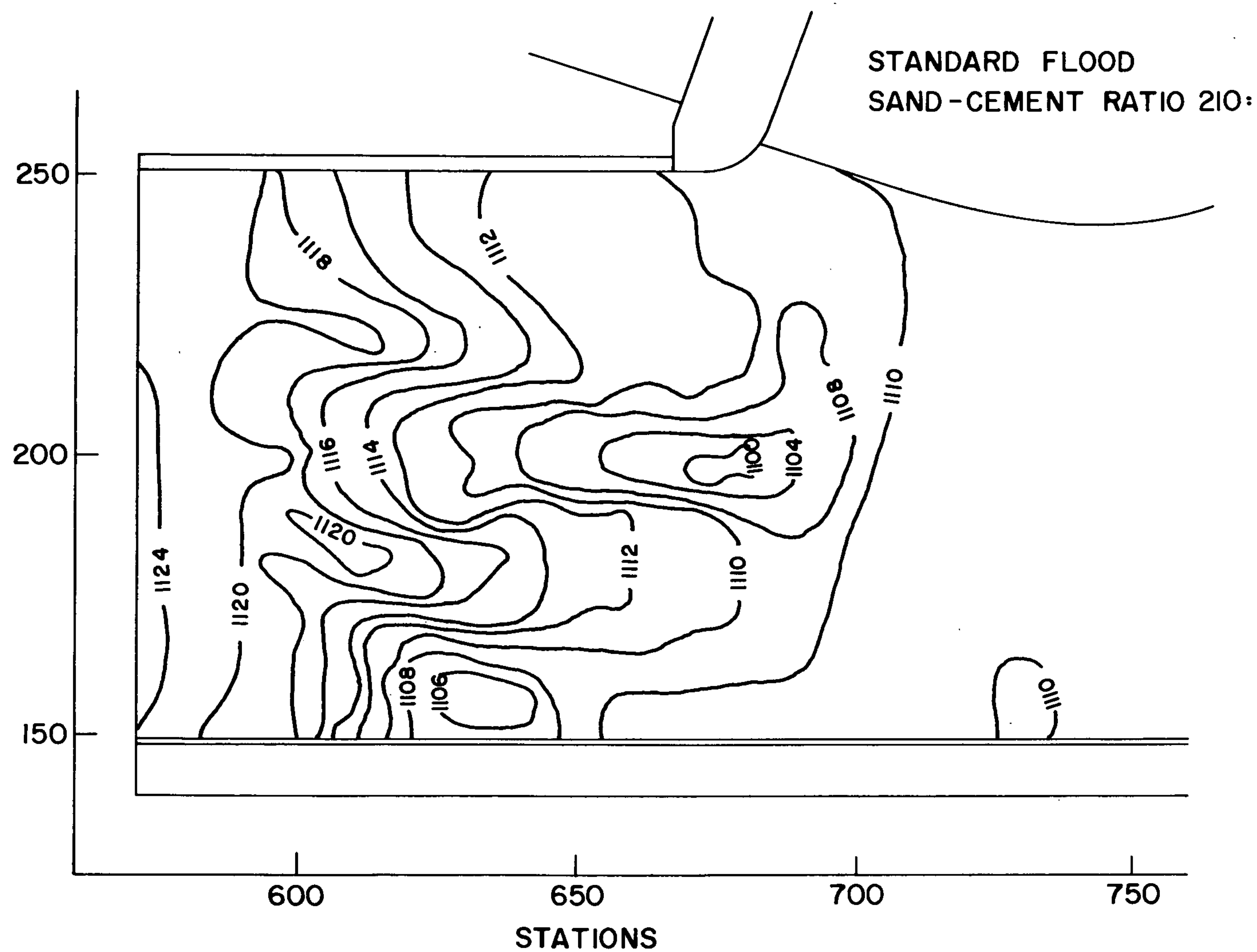


Fig. 33 Contour Map - Test No. 7

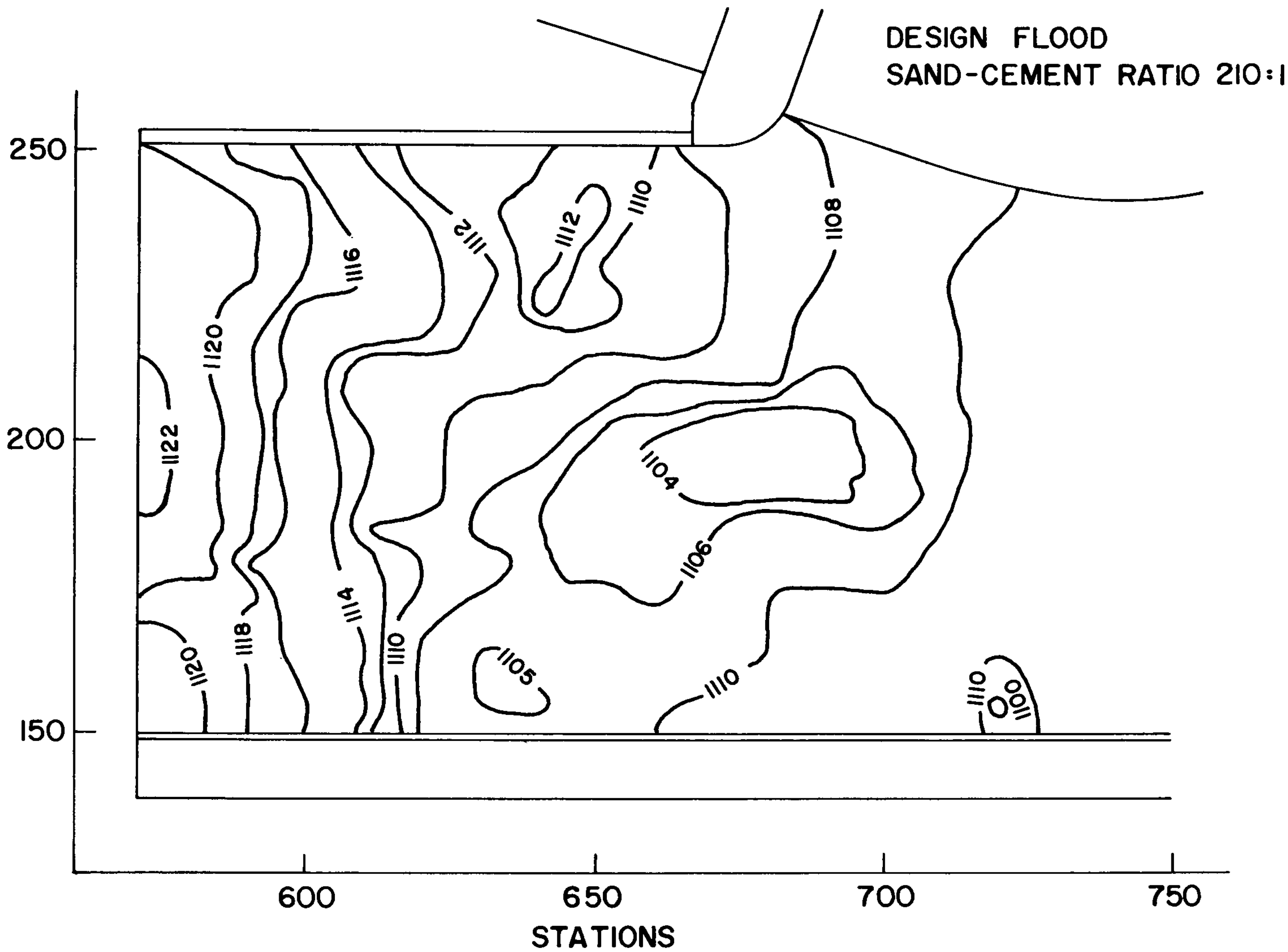


Fig. 34 Contour Map - Test No. 8

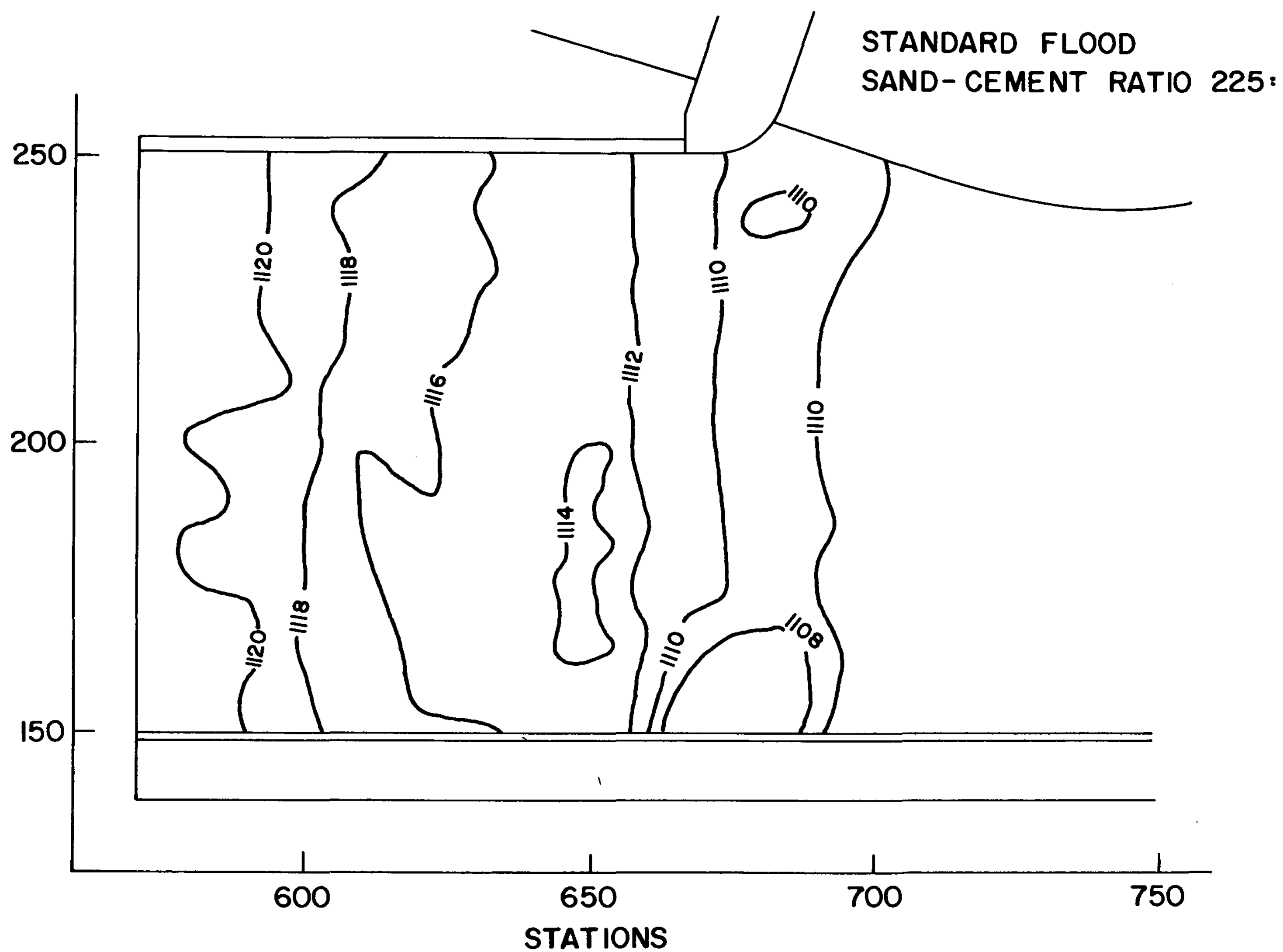


Fig. 35 Contour Map - Test No. 9

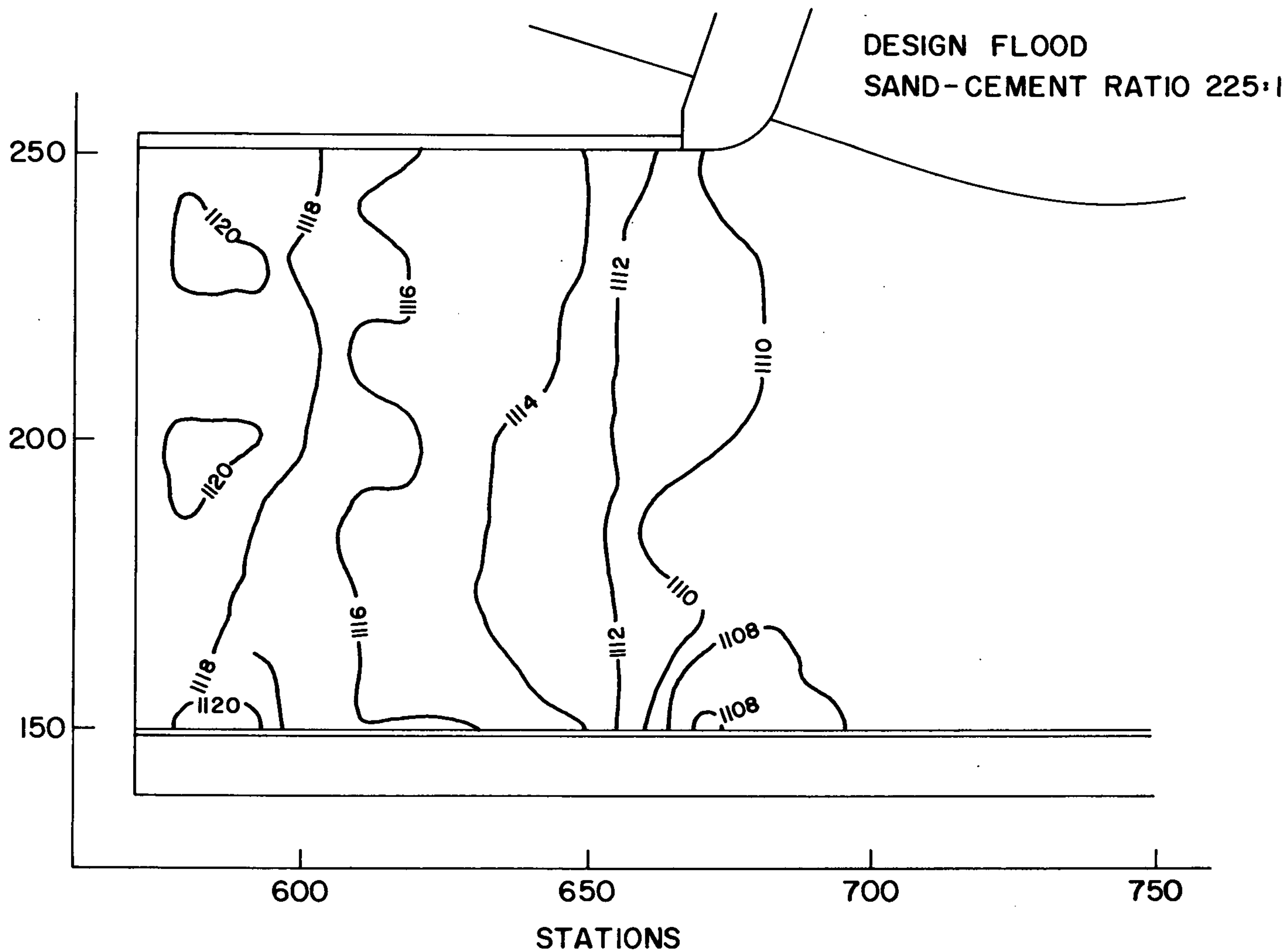


Fig. 36 Contour Map - Test No. 10

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