Ouachita River Project
Arkansas
Remmel Dam
Arkansas Power and Light Company
Little Rock, Arkansas

Report
Hydraulic Model Studies
Remmel Dam Auxiliary Spillway

for
Fargo Engineering Company
Jackson, Michigan

by
James W. Ball
Colorado State University
Fort Collins, Colorado

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ARKANSAS
REMMEL DAM SPILLWAY

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PURPOSE

The purposes of the hydraulic model studies discussed in this report were to determine optimum methods for increasing the spillway capacity at Remmel Dam and to ascertain the effect of the future installation of an additional power unit on the performance of the spillway during flood flow conditions. Methods considered for increasing the spillway capacity included:

1. passing flood flows over the north (and south) abutment sections of the dam;
2. raising the spillway radial gates to a higher elevation, and;
3. providing protective stilling basins and riprap protection where necessary downstream from the abutments.

DEFINITION OF TERMS

In this report the term <u>auxiliary spillway</u> refers to the north abutment overflow section in particular, but does in some cases include the south abutment overflow section which was not tested on the model.

The term <u>trough</u> refers to the concrete lined channel into which the flow from the auxiliary spillway plunges.

The term <u>plunge pool</u> refers to the boundaries of the area into which the auxiliary spillway flow plunges into the tailwater. The plunge pool includes such items as the trough, the upstream bench along the trough, the walls between the abutment buttresses, the wall along the downstream side of the trough, the training wall at the north end of the trough, and any riprap protection or pavement.

CONCLUSIONS

1. The abutment sections of Remmel Dam can be used as auxiliary spillways if adequate protective measures are provided.

2. Initial tests on the model established that protection would be needed downstream from the north abutment to prevent undermining of the downstream toes of the buttresses when over topping flows plunge from the crest of the abutment section into the tailwater onto the river bank below, (figure 14C and D).

3. Riprap-lined plunge pools alone will not provide adequate protection. Undermining of the buttresses and excessive scour at the abutment and of the river bank takes place, (figures 15 and 16).

4. The plunge pool design developed for the north abutment can handle floods with reservoir levels up to elevation 325 without scour damage to the existing structure or objectionable flow conditions in the river channel if an adequate concrete-lined trough and adjacent paved and riprapped areas are provided. The recommended arrangement, (figures 21, 26, and 27) will easily handle flows with a 7-foot head on the auxiliary crest (Res. El. 322) and will adequately handle flows at a head of 10 feet on the crest (Res. El. 325), (figure 27B and D). The concrete-lined plunge pool trough of this design varies in width, depth, and elevation and extends from the north wall of the powerhouse to Buttress No. 2, (figure 21).

5. The bench or ledge along the toes of the buttresses upstream from the trough should vary from 5 feet wide at the north end at Buttress No. 2 (section AA, figure 21), to 7 1/2 feet at Buttress No. 8, (section BB, figure 21), because of the variable trajectory distance of the overtopping jet between these points. The width should be 7 1/2 feet from Buttress No. 8 to the powerhouse, (section C-C, figure 21).

6. Because of the minor turbulence observed at the downstream toes of the buttresses under the overtopping jet, it was concluded that some protection of the surfaces between the buttresses would be desirable. Riprap, perhaps equivalent to two layers of 12-inch stones, placed behind 5-foot high walls between buttresses at their downstream toes is considered adequate, (figure 21).

7. Riprap should be sorted or graded when taken from the quarry for placement along the downstream side of the plunge pool trough. The largest stones should be placed downstream from the paved area north of the present powerhouse, (section C-C, figure 21); the next largest downstream from the sloping trough wall, (section C-C to section B-B) and the smallest (6-inch to 12-inch) behind the capped sea wall.

8. Flow conditions at the north end of the abutment were measurably improved by placing a wing wall upstream from the crest and opposite Buttress No. 2, some 270 feet from the north wall of the powerhouse, (figure 21).

The wing wall should be vertical and end at the upstream face of the abutment. The wall should extend 5 feet upstream perpendicular to the axis of the crest, curve on a 5-foot radius for 90 degrees and then extend into the hillside, (detail X, figure 21). The height of the wall should be sufficient to prevent overtopping at the design flood reservoir level, (El. 322, or El. 325).

The downstream edge of the wing wall at the crest should be a sharp 90 degree corner, to assure that the flow breaks away cleanly at this location. The corner could be formed of an angle iron to assure a sharp edge.

9. The area north and downstream from Buttress No. 2, should be protected from the overtopping flow by a vertical training wall placed downstream from Buttress No. 2, and offsett 3 1/2 feet to the north of the wing wall, (figure 21). The top of the training wall should be at El. 321 as shown on Figure 28. The offset allows the spillway jet to pass over the crest without impingement on the training wall until the flow has passed over the crest.

10. A parapet wall with its top at El. 318, should extend north of the training wall along the downstream edge of the abutment walkway, ramp and steps, into the natural topography.

The parapet wall prevents water from eroding the topography near the steps and obviates the need for an obstruction (gate or bulkhead) across the top of the abutment crest.

11. The 1-foot overhanging cap or sea wall on the downstream trough wall at the north end of the recommended trough, (figure 21) turns the flow back upstream onto the overtopping jet where it is conveyed southward into the tailwater and flows towards the river channel, (figures 27A and B). The overhang is needed only on the portion of the wall that is above tailwater (approximately El. 292). A 50-foot length of the sea wall is considered adequate. The total height of the wall should be approximately 10 feet. The remainder of the wall need not be capped. The height from Buttress No. 2, to Buttress No. 8, should be 9 feet and that from Buttress No. 8, to Buttress No. 16, should vary from 9 feet to 0, (figure 21).

12. The best treatment of the area downstream and between Buttress No. 16, and the north wall of the powerhouse is to pave it with concrete as shown on Figures 21, 26, and 27. This area on the model was at El. 252, Figure 21. However the paved area should be kept low enough to allow construction of the penstock for the future outdoor unit without undue excavation. Riprap consisting of stones ranging from 2 to 4 feet should be placed for about 20 feet downstream from the end of the pavement. A few 6-foot stones at the edge of the concrete would be desirable.

13. Although some improvement in scour conditions downstream and to the south of the concrete-lined trough was accomplished by placing a wall across the south end of the trough, (figures 24 and 25), the

improvement was not considered sufficient to justify its use, particularly when larger riprap stones (4 to 6 feet) placed at strategic locations minimized the scour, (figure 25).

14. There will be tendency for material to be carried upstream into the tailrace of the present powerhouse when the auxiliary spillway operates at substantial discharges. It may be necessary to dredge the tailrace following a large flood.

15. The capacity of the radial gate controlled spillway can be increased by increasing the maximum height to which the bottom lips of the gates can be raised when large floods are anticipated, (figures 10 and 11). An increase of 4.6 percent can be realized at reservoir elevation 322 by raising the gates from El. 305 to El. 307, and an increase of 26.1 percent can be realized by raising the gates to El. 310. The increase in capacity with no obstruction from the gates would be 33.1 percent, (table I, and figure 10).

16. The capacity of the auxiliary spillway (abutment sections) can be increased by streamlining the upstream edge of the crest or shaping the crest to the ideal profile for a 7-foot head, (figures 7, 8, and 9). Streamlining (rounding) the upstream edge, (figure 7B and C), gives only a small increase while shaping the crest to an ideal basic profile, (figure 7D), gives a substantial increase. The increase for the shapes with rounded edges was approximately 6 percent while that for the

ideal profile was 15.9 percent. Capacity curves for the present crest and the ideal profile are given on Figure 9.

17. The total flood capacity of the gated and auxiliary spillways combined, with the present abutment crest shape, the radial gates set at El. 310, and reservoir elevation 322 will be approximately 241,000 cfs, (figure 11). The capacity at reservoir elevation 325 will be approximately 291,000 cfs.

18. Because of the long length of the north abutment it is important to provide some means of aerating the under nappe of the overtopping flow at intermediate points. Ventilation is required to prevent structural vibrations that might be induced by fluctuating pressures under the nappe. Aeration can be accomplished by constructing sharp edged piers at the downstream faces of intermediate buttresses. The model indicated that piers 3.5 feet high and 16 inches wide placed at Buttresses No. 8, and No. 14, (figure 19), would split the jet and provide the necessary aeration for reservoir levels up to and including El. 322. The height should be increased to 4 1/2 to 5 feet if the reservoir level is likely to reach El. 325. The piers will not decrease the capacity of the auxiliary spillway because they are located downstream from the critical depth section.

19. When water flows over the tops of radial gates, as it will at Remmel Dam during high floods, air is prevented from entering the

space between the downstream side of the gate and the nappe of the overtopping flow and a region of sub-atmospheric pressure forms unless some method of venting is provided. This is true of all except Gate No. 1, adjacent to the powerhouse (and possibly Gate No. 12, at the south abutment). A sub-atmospheric pressure of approximately 2 feet of water (prototype) was indicated by the model.

The nature of the flow around the southwest corner of the powerhouse and just downstream from Gate No. 1, provides aeration to this gate, (figure 12B). This same condition may or may not occur at Gate No. 12, depending on the nature of the flow over the pier at the south end of the spillway.

20. The water flowing around the northwest corner of the powerhouse forms a contraction and only the edge of the jet contacts the powerhouse wall as it plunges downward into the tailwater, thus the north wall of the powerhouse will be subjected to a mimimum of flowing water during operation of the auxiliary spillway.

21. The piers extending upstream from the abutment area to be occupied by a future power unit will cause some disturbance and turbulence in the flow as it passes over the crest. This condition is not considered to be detrimental.

22. Flow releases through the main spillway should not be concentrated through a single gate or a small number of gates at large openings when the river flow downstream is very small and the tailwater is low.

Under such operation the tailwater elevation is usually below normal and the spillway jet produces high impact forces that could damage or tear out parts of the dentated sill. Such operation may have contributed to the damage to the sill experienced in the past. Generally flow should be distributed through as many gates as possible. Also, operation of individual gates should be scheduled to reduce visible eddy action in the downstream river channel to a minimum.

23. Although eddies form in the river channel below the dam for the main gated spillway operating with or without flow over the abutment, (figure 13), their velocities are low when flow is uniformly distributed across the river and scour attributed to them is minimal.

24. The future installation of an outdoor generating unit such as that represented on the model, (figures 29 and 30), will not materially affect the performance of the auxiliary spillway, (figure 31). Riprap adjacent to the north side of this structure should be in the range of 4 to 6 feet in size or concrete, otherwise wasted during construction of the power unit enclosure could be used to protect this area either in the form of slabs or as a binder between riprap stones.

25. Flow overtopping the abutment above the future unit will impinge on the top of the penstock passage leading to the unit, (figure 31B). The top of the passage should be of sufficient strength and rigidity to withstand this force and any vibration. Also, there is an increased

force against the generator enclosure, (figure 29), caused by water rising to the top of this shelter on the upstream side and tapering to about 8.3 feet below the top at the downstream side. This occurs for reservoir elevation 322, (figure 32). For a superflood with the reservoir at elevation 325, the water at the upstream side of the shelter rises to a height of about 5 feet above the shelter. The water surface recedes to about 7 feet below the top of the shelter at the downstream side of the shelter, (figure 32).

26. Flow conditions on top of the future unit and along the remainder of the abutment to the north are quite satisfactory for Reservoir El. 322 and acceptable for Reservoir El. 325.

27. Scour and flow conditions with the future power unit in place are quite acceptable, (figure 33).

INTRODUCTION AND ACKNOWLEDGEMENTS

The Remmel Dam of the Arkansas Power and Light Company is located in the Ouachita River, 6 miles northwest of Malvern, Arkansas, and 70 miles southwest of Little Rock, (figure 1). The dam is a hollow reinforced concrete slab and buttress, or Ambersen type, structure 935 feet in length at its top which is at El. 315, (figure 2). The top or crest of the dam is approximately 70 feet above the river bed.

The dam has a radial gate controlled spillway with a gross length of 357.5 feet. There are twelve radial gates 27.5 feet long and 15 feet high. The spillway crest is at El. 290 and normal maximum reservoir level is El. 305. The spillway capacity at this elevation is approximately 72,000 cfs.

A powerhouse 110 feet long with three units is located to the north of the main spillway. Two abutment sections extend, one from the north end of the powerhouse to the north canyon wall, and the other from the south end of the spillway to the south canyon wall. The north abutment is approximately 333 feet long and the south abutment 135 feet. The tops of the abutment sections are level at El. 315, for 285 feet and 107.6 feet respectively.

The Federal Power Commission in the past few years has required licensees to provide safety inspections of hydroelectric-project works by independent consultants. In connection with this inspection the spillway capacity of Remmel Dam was found to be inadequate with respect to present day design parameters.

At a meeting attended by F.P.C., licensee and consultant personnel, an inflow design flood was agreed on for which a discharge capacity of 198,000 cfs was calculated to be required. The Arkansas Power and Light Company engaged Fargo Engineering Company of Jackson, Michigan to make cost studies of alternative methods for the most feasible plan. The least expensive scheme was that which would allow flood flows to overtop the north and south abutment sections of the dam. This could be done, however, only if adequate protective energy dissipators could be provided at the bases of the abutment section at a reasonable cost.

The Fargo Engineering Company engaged Mr. Alvin J. Peterka, P. E., Consulting Engineer of Lakewood, Colorado to advise on the need for and scope of a hydraulic model study. Since a model is the only known means of solving the problems inherent in a rehabilitation study of this type, Mr. Peterka arranged to have a hydraulic model constructed and tested at Colorado State University under contract to Fargo Engineering. Mr. James Ball, P. E., was in charge of the model study, and worked with Mr. A. J. Peterka in developing the recommended designs. Mr. Larry Everett, P. E., of Fargo Engineering visited the CSU Engineering Research Center on several occasions to pass on the practicability of the designs being developed. Mr. John White, Manager, Production Dept., Arkansas Power and Light Company, also visited the laboratory to observe the performance of the model. Dr. Hamidur Rahman Khan and Dr. Giuseppe Palumbo, graduate students at CSU, assisted in planning the model, conducting the tests and preparing material for the final report.

It was for the primary purpose of determining the capability of the abutment sections, particularly the north one shown on Figure 2A, to withstand overtopping and to develop methods of protecting them against failure due to downstream scour, that hydraulic model studies were initiated. Since the north abutment was geologically vulnerable from the overtopping, the studies were concentrated on the north abutment area, with the thought that protective measures developed for this abutment could be applied to the south abutment if deemed desirable. The model was constructed, however, to provide other hydraulic data useful in solving the current problem, including operation of the prototype structure in the future. The model was also modified to determine the feasibility of adding an outdoor power unit without disrupting the construction of the new plunge pool.

Because of the large quantities of water required for a complete model of adequate size and the substantially higher cost of such a model, the model was limited to 3 radial gates of the main spillway, the powerhouse and the north abutment area as shown on Figure 4. Moreover, the model constructed on a 1:42 geometric scale was considered quite adequate for the studies. At this scale the model is large enough to provide dependable results which may be used to accurately predict the performance of the prototype structure. The partial model was physically arranged so that with careful interpretation of the results, the performance of the hydraulic features of Remmel Dam could be predicted. The results obtained from the model study are the subject of this report.

MODEL

Description of Model

The 1:42 model used for the study consisted of three components, the abutment section, the powerhouse and a spillway section containing three radial gates, (figure 3). These components were placed in a plywood-lined box 27 feet long and 13 feet wide, (figures 4 and 5), having different depths upstream and downstream from the dam. The box was 3.25 feet deep in the upstream or reservoir portion and 2.75 feet deep in the downstream or tailwater portion. The box was supported by steel I-beams placed longitudinally on the laboratory floor and by 2 x 4 timbers placed laterally across the beams. The box was made waterproof by taping the joints and treating the box and taped joints with fiberglass. The floor of the plywood-lined box represented elevation 229. The model represented a reservoir area of 4 acres extending 300 feet upstream from the dam and a tailrace area of 10.5 acres which included a 750-foot length of river channel downstream.

The abutment and powerhouse components were constructed of fiberglass treated plywood and the spillway was made up of a fiberglass treated plywood framework covered and shaped from auto body putty. The spillway crest was formed by attaching two metal profiles, one at each end of the framework, placing the body putty between them and screeding to the profiles. The piers were cut from wood and treated with fiberglass and painted. The piers were fixed to the spillway crest with fiberglass and screws. The gates and their radial support arms were cut from transparent plexiglass sheet. The gates were hinged at the piers with 3/16" diameter brass pins. Holes were made in the side arms of the gates to suspend them at different elevations during the tests. The holes were placed so that the bottoms of the gates corresponded to elevations 305, 307, and 310.

The abutment, powerhouse, and spillway section were fixed to the floor with aluminum angles and screws. Water was supplied to the model through a 14-inch valve-controlled pipeline containing a vertical mixed-flow pump. A rock baffle was placed in the upstream end of the head box to provide tranquil reservoir-type flow to the model. The discharge to the model was measured by a calibrated orifice meter placed in the 14-inch supply line.

The reservoir elevation was measured by a point gage installed upstream of the model dam. The elevation of a bench mark on the reservoir bottom below the point gage was determined precisely with respect to the model crest by using an engineers level. The water surface elevation was determined by adding the difference in point gage readings, expressed in prototype terms to the elevation of the bench mark.

The reservoir topography between the rock baffle and model dam was molded in compacted sand which was stabilized by a layer of weak sand-cement mortar. The ground and rock surface contours used in the model tests are shown on Figure 4.

The downstream rock topography in the main river channel and in the area downstream from the north abutment was formed of compacted sand topped with a layer of weak concrete. The remainder of the model river bank, including the overburden was formed of a special mortar sand with a gradation that represented prototype material varying from 1/16-inch to 4 inches. Although the gradation of the prototype material was unknown, the representation appeared quite good when compared with photographic records of the area. Scour conditions obtained with this material in the model are expected to be quite representative of those to be expected in the prototype. The gradation of the mortar sand is shown on Figure 6.

Small rock particles, geometrically representing 6-inch to 48-inch prototype stones were used for riprap studies on the model.

Model Test Procedure

The model was tested by starting the pump, bleeding air from the pipeline, particularly from the upstream and downstream sides of the orifice and then adjusting the valves in the line to give the desired reservoir elevation, usually El. 322. The tailbox was filled slowly to the desired elevation before the flow was increased to give the desired headwater elevation. The tailwater level was controlled by a hinged weir located at the downstream end of the tailwater box. The weir or tailgate could be elevated to give the tailwater elevation corresponding to the discharge for the particular reservoir elevation, usually El. 292.

A model test consisted of operating the model for an initial test period of 1 hour, with the reservoir at El. 322. This was sufficient time to stabilize the scour pattern, record data, and take photographs and movies. The model was sometimes operated a half-hour longer at a larger flow.

Calibration of the main spillway gates for reservoir elevations above the crest of the abutment sections was made by blocking off the abutment section. Calibration of the abutment section was made by blocking the main spillway. A 10-inch orifice meter in the supply line was used to measure discharges on the model.

In shutting down the model the tailwater elevation was kept high until overtopping of the crest ceased to preserve the scour pattern developed in the sand during the test. The tailwater was then drained by lowering the hinged tailwater weir. The tailwater at the site will still be high when overtopping of the abutments ceases, therefore the procedure adopted was quite representative of the field operation.

Usually the flow and scour conditions observed during a test were recorded by still and motion pictures. Six-hundred feet of 16 mm movie film were taken during the model studies.

STUDIES RELATED TO SPILLWAY CAPACITY

Calibration of Auxiliary Spillway (Abutment)

Effect of Shape of Crest on Spillway Capacity -- Because of the flat crest and the square upstream corner of the walkway along the top of the abutment section or auxiliary spillway, it was considered possible that some increase in capacity could be realized by streamlining the upstream corner of the crest or shaping the crest to a basic profile based on data obtained by several experimenters. Four shapes were tested to evaluate the effect of crest shape on capacity, (figure 7). The effect of rounding the upstream corner of the crest was first investigated. However, since only a small increase was obtained the crest shape was modified to conform to a nappe profile for a 7-foot head, (figure 7D).

Calibrations of the existing flat crest and the crests with rounded upstream corners were carried out on the model as soon as the model components were installed in the test box while plans were being made to include the upstream and downstream topography. The rating curve for the existing flat crest is shown on Figure 8. The curve indicates a capacity of 18,000 cfs at reservoir Elevation 322, (7-foot head). These test results include the effects of the sloping ramp and the stepped section at the north end of the abutment.

Since the objective was to increase the discharge for a given head, the shape of the crest was examined in relation to several crest shapes studied by the Bureau of Reclamation.(1) It was reasoned that rounding the upstream edge of the crest or extending the crest upstream with a shape that would decrease or eliminate the vertical contraction would increase the capacity. Since the effect of streamlining the upstream edge of the crest would be evaluated staisfactorily by using streamlined upstream extensions, a small extension with a compound radii rounding, (figure 7B), was first tested. The two radii were 0.7 and 3.2 feet.

As expected the streamlining decreased the contraction over that for the sharp-edged original crest and increased the flood capacity slightly, to 18,900 cfs at Res. El. 322, (figure 8).

Using the discharge data and assuming an effective crest length of 302 feet, allowing for the sloping ramp and steps, the discharge coefficient C in $Q = CLH^{3/2}$ for the original crest, was 3.22 while that for the crest with the streamlined extension was 3.39. In this equation, Q is the discharge quantity in cfs, L is length of crest in feet, and H is the head on the crest in feet. Since the increase was somewhat less than anticipated, a further study was made of the crest shapes studied by the Bureau of Reclamation and one with a higher efficiency than the first was selected for the next test. The size of the upstream extension was approximately the same as the first but the streamlining consisted of a single radius of 4 feet, (figure 7C). The

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(1) Bureau of Reclamation Engineering Monograph No. 9, "Discharge Coefficients for Irregular Overfall Spillways."

rating curve for this shape was essentially the same as that for the first modification.

From the tests on the two modifications, it was concluded that the streamlining of the upstream corner of the walkway would increase the capacity slightly, about 5.3 percent, and that the reservoir elevation for 18,000 cfs with the streamlined crest would be 321.8 or 0.2 of a foot lower than with the original crest. Also, it was believed that the flat top of the original crest formed a control and that any significant increase in capacity could be obtained only be reshaping the original crest to correspond to a basic ideal crest shape,[(2)] shown in Figure 7D. This ideal shape has a coefficient of approximately 3.9, which would give an increase in discharge of 21.7 percent at Res. El. 322, based on 302 feet of crest length. This crest shape was not tested during the early part of the test program because of the urgency of obtaining other significant data. The rating curve for this crest was obtained in later tests and compared with that for the original crest which had then been shortened to 270 feet by placing the vertical curved wing wall upstream near Buttress No. 2.

The coefficient of discharge for the 270-foot long original crest and reservoir elevation 322 was 3.40 and the discharge was 17,000 cfs, (figure 9). The ideal crest shape had a discharge coefficient of 3.94 at Reservoir El. 322, and a discharge of 19,600 cfs, (figure 9). The

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(2) Boulder Canyon Project Final Reports, Bulletin 3, "Studies of Crests for Overfall Dams."

increase in discharge was 15.9 percent over that of the original crest. With the ideal crest shape on both abutments the increase in flood capacity of the dam with the spillway gates raised to El. 310 would be only 2 percent, or the reservoir elevation for the same discharge with the ideal crest would be approximately 0.4 of a foot lower. The rating curve for the north abutment with a vertical upstream wing wall placed 270 feet from the powerhouse, is shown on Figure 9, which also shows the rating curve for the ideal crest.

Calibration of Main Gate-Controlled Spillway

Effect of Maximum Gate Opening on Spillway Capacity -- In preliminary studies to evaluate the flood capacity of the main and auxiliary spillways of Remmel Dam, the model was operated near the proposed maximum reservoir El. 322. These tests showed a significant change in reservoir level when the gates were first raised to clear the flowing water and then lowered to the position with the gate lips at El. 305. With the model discharge constant, the reservoir elevation was recorded with the gate lips free of the water, at El. 307 and at El. 305. The reservoir elevations for these three gate positions were 320.9, 322.8, and 323.4 respectively.

From the tests it was concluded that the reservoir level for the same discharge could be lowered by approximately 0.6 of a foot by raising the gates from El. 305 to 307 and that an additional reservoir lowering of 1.9 feet could be realized by raising the gates free of the flow. The results were discussed with the representatives of the Fargo

Engineering Company, and the Arkansas Power and Light Company who decided it would be possible to raise the gate lips to Elevation 310 in anticipation of a large flood. The model was then calibrated for various reservoir and gate elevations to evaluate the effect of gate opening on the flood capacity of the dam.

The model was calibrated by blocking off all flow over the abutment section north of the powerhouse. The gates of the regular spillway were set at elevations 305, 307, and 310 and were then removed from the model for four separate calibrations. The results of these tests are shown on Figure 10. From the calibrations it was concluded that the decrease in discharge caused by the gates was small when they were placed at elevation 310; that the discharge was decreased materially by lowering the gates to elevation 307, and that a further but minor decrease in discharge resulted from lowering the gates to elevation 305. Also, that much would be gained if the gates could be raised to El. 310, for large floods. The results were essentially in agreement with the preliminary tests.

Flood Capacity at Remmel Dam

From the calibrations of the gated section of the spillway and the north abutment it was possible to estimate the total flood capacity of the dam for various reservoir levels. The data for the gated spillway and north abutment given in the following table were taken directly from Figures 9 and 10, while the discharge for the south abutment was estimated by using the length of that abutment compared to the length of the north abutment. The data from Table I was then plotted on Figure 11.

Table I
Flood Capacity at Remmel Dam

Res.	Gate Position				Abutment Discharge	
El.	Out.	El. 310	El. 307	El. 305	North**	South***
322	229,000	217,000	180,000	172,000	17,000	9,000
321	218,000	208,000	171,000	164,000	13,000	7,000
320	207,000	198,000	163,000	156,000	9,000	5,000
319	197,000	189,000	155,000	148,000	6,000	3,500
318	187,000	180,000	148,000	141,000	3,800	2,100
317	177,000	171,000	140,000	134,000	2,000	1,100

* Gated Spillway Discharge taken from Figure 10
** Length of overflow section North Abutment = 270 ft., Discharge from Figure 9
*** Length of overflow section South Abutment = Variable (107' 7" to 135' 4"), depending on head

Considering the total flood capacity to be 198,000 cfs with the spillway gates at El. 305 and the reservoir at El. 322, the capacity can be increased to 206,000 cfs (or 4 percent) by raising the gates to El. 307. However, by raising the gates to El. 310, the capacity can be increased to 243,000 cfs or about 23 percent greater than with the gates at El. 305. A further raising of the gates (their removal from the model) increased the capacity to 255,000 cfs or about 6 percent above that with the gates at El. 310. It is highly recommended that provision be made to allow raising the gates to El. 310 when large floods are anticipated.

STUDIES RELATED TO FLOW CONDITIONS IN RESERVOIR AND OVER SPILLWAY CRESTS AND GATES

Flow Conditions Upstream from Trashway

Because of the long upstream pier at Buttress No. 26, near the south end of the powerhouse and the much shorter pier nose at the north end of Spillway Gate No. 1, an eddy formed between the two piers and adjacent to the trashway entrance. As the reservoir level rose the eddy became more pronounced and the disturbance at the trashway entrance increased to give very poor flow conditions into the wasteway. Although the flow conditions were rough and unsightly, they are not considered to be of a critical nature.

Flow Conditions at Spillway Radial Gates

With the gates set with their lips or bottom edges at elevation 307, the model flow was increased gradually to investigate flow conditions at the gates during a rising flood. A large contraction and flow disturbance occurred in the approach area upstream from the north end of Gate No. 1. This condition was attributed to the pier arrangement upstream from the trashway, a long pier nose extending upstream from the powerhouse (Buttress 26), and the short pier nose at the north end of Gate 1, Buttress 27. A large ridge or pile-up of water occurred near the south end of Gate 1 as a result. This ridge contacted the gate lip when the reservoir reached El. 307. As the flow was further increased the water came in contact with the lips of the other gates as the reservoir level reached El. 309. As the flow was increased further, orifice flow occurred at the gates and eventually the parapet and the tops of the gates were flooded with a substantial depth overtopping the gates. The flow conditions behind the gates will be discussed in a subsequent section of this report.

Aeration of Flow Overtopping Spillway Gates

During operation of the model in the early tests before the top of the trashway bay was closed, flow passing from the upstream side of the powerhouse onto the spillway flow prevented aeration of the jet overtopping Gate No. 1. Also, the jets overtopping the remaining gates prevented aeration of the spaces bounded by the downstream sides of these gates and the nappes of the jets overtopping them. Air was pumped from these spaces until a condition of equilibrium existed with a sub-atmospheric pressure. The pressures were measured with the gates at El. 307. The sub-atmospheric pressure under these conditions was 0.45 feet or approximately two feet of water prototype. The pressure in the prototype may be somewhat greater or less due to a difference in air entrainment characteristics and physical arrangement of the structural members of the model and prototype. This sub-atmospheric pressure adds to the forces on the gates and it would seem wise to ascertain its effect on the structural soundness of the gates with the reservoir at high elevations. If necessary, special venting should be provided to minimize the forces.

Flow Conditions at Southwest Corner of Powerhouse

During the first part of the test program the top of the trashway passage upstream from the powerhouse was not covered as it is in the field. As a result there was a considerable flow through this opening into the trashway which passes under the south end of the powerhouse. The flow over the wall onto Gate No. 1 was therefore relatively small and the downstream side of the gate was not always aerated. The flow conditions for this case are shown on Figure 12A.

Later in the test program the top of the trashway bay upstream from the powerhouse was closed forming a covering surface with its top at El. 315. This change allowed a greater quantity of water to flow southward and plunge over the wall downstream from Gate No. 1. This action forced the jet overflowing the gate to be deflected southward and provided the large aeration space shown on Figure 12B. The flow from this source plunged onto the jet overtopping the parapet above the gate, split it and then continued downward onto the jet issuing under the gate. From the tests it was concluded that Gate No. 1, would always be adequately aerated but that the remaining gates, except possibly Gate No. 12, would not be aerated. The aeration conditions for the other gates were described previously.

Flow Conditions at Northwest Corner of Powerhouse

Because of the square northwest corner of the powerhouse there was a contraction of the water as it overtopped the abutment at this point. The contraction to the north deflected the main flow away from the north wall of the building so that only a thin jet flowed along this wall as the jet plunged into the tailwater. This contraction thus minimizes the water that will be in contact with the powerhouse as it overtops the abutment and plunges into the tailwater.

Flow Conditions Induced by Long Piers Extending Upstream and to North of Present Powerhouse

The presence of the long piers extending upstream from the abutment north of the powerhouse, where provision is made for future expansion of the powerplant introduced turbulent eddies in the water that flows over the abutment crest. The effect of the eddies, although it continued into the tailwater downstream, was not objectionable.

Aeration of Flow over Abutment Section

When the model was first placed in operation with no topography in place and with thin sheets of water overtopping the abutment there was a definite tendency for the jet to flutter in a manner similar to that observed by many experimenters on long weirs and drum gates with thin overflows. Because of the tendency to flutter, two piers equivalent to 3.5 feet high, 10 inches thick, and 16 inches wide were placed downstream from, but in contact with Buttresses 8 and 14, (figure 19). The 16-inch dimension was parallel to the abutment axis. With the piers in place, the fluttering action subsided and was not noticeable as the head over the abutment increased. Piers located in this manner are downstream from the section of critical depth and thus have no influence on the capacity of the crest. Their purpose is to split the flowing water sheet and permit air to enter the underside of the nappe to prevent pressure fluctuations of sufficient magnitude to induce fluttering. Rapid pressure fluctuations could affect the stability of the abutment section. Also, the piers provide assurance against complaints that might originate from the vibration of such things as doors, dishes, windows and other objects in homes located in the vicinity of the dam.

It is recommended that two aeration piers be provided on the prototype. Actually the shape of the aeration piers is not important so long as they split the flow and provide aeration at the low heads. The piers could be square, or of I or H beam section.

Near the end of the test program when an ideal crest shape was installed in place of the original square-edged walkway, Figure 7D, to determine how much increase in flood capacity could be realized, the overtopping jet was examined over a wide range of head. With two aeration piers at Buttresses 8 and 14, the jet had no tendency to flutter.

Flow Conditions at North End of North Abutment and Desirability of Shortening the Overflow Section

The ramp section and steps above elevation 315 at the north end of the abutment were not conducive to good flow conditions at this location. Also, it would have been necessary to provide a longer trough downstream to receive the overflow from this section. Because of these considerations it was decided to limit flow only to the abutment section south of Buttress No. 2. An upstream wing wall was placed in the reservoir at a point 270 feet north of the present powerhouse. The vertical wall was perpendicular to the abutment axis for 10 feet, curved on a 17.5-foot radius for 90 degrees, and extended parallel to the abutment into the hillside, (figure 19). This positioning of the wing wall isolated a flat section of the abutment about 18 feet long as well as the ramp and steps to the north of the wing wall. A vertical training wall, perpendicular to the abutment axis and 3.5 feet farther north than the upstream wing wall, was placed downstream from the abutment. This arrangement gave an offset of 3.5 feet from the upstream wing wall to the training wall and made it unnecessary to block the walkway from the abutment to the ramp and stairs.

Flow conditions near the upstream wing wall were very good, and although a tranquil pool of water covered part of the ramp section to the north there was no objectionable flow action. Flow from the crest spread and contacted the downstream training wall as it plunged into the trough downstream. The flow conditions were very good.

From the tests it was concluded that the upstream wing wall placed at Buttress No. 2 would give improved conditions over those without the wall, that the decrease in discharge due to shortening the overtopping length would be minor, but that the wing wall used was much larger than necessary. It was proposed that the wing wall alignment be changed such that it extended perpendicular to the abutment axis for 5 feet upstream, curved on a 5-foot radius for 90 degrees and then extended parallel to the abutment axis into the hillside, (figure 21). The height of this wall should be such that it there would be adequate freeboard during large floods with reservoir elevation 322 or 325. This wall was installed on the model near the end of the regular test program. The wall gave very satisfactory flow conditions and is recommended for the final design. The wall will be discussed further in a subsequent section of this report.

STUDIES RELATED TO FLOW AND SCOUR CONDITIONS DOWNSTREAM FROM DAM

Flow Conditions Below Spillway Radial Gates

As each test on the model was begun, with low tailwater downstream, the jet from the spillway gates impinged on the upstream faces of the dentates of the sill. The flow striking the dentates shot vertically upward to a height somewhat above El. 290. This condition exerts a very large force in the downstream direction and is one that would exist with low tailwater and a large opening of a single or small number of gates. This action may have contributed to the removal of some of the dentates in the past. As the tailwater is increased a hydraulic jump forms upstream from the dentates and much of the energy in the jet is dissipated before reaching the dentates and the forces in the downstream direction are reduced. Possible damage to the dentates could be prevented by limiting the gate operation to small openings when the tailwater is low.

Flow Conditions Downstream From Abutment

The topography upstream and downstream from the model dam was not contained in the model for the initial test which was made primarily as a preliminary pilot test to assess the problems that might arise due to passing water over the north abutment of the dam. In this test the floor downstream from the abutment was at El. 229, thus a plunge pool about 63 feet deep was present for the maximum estimated flood with the reservoir at El. 322.

The flow conditions in the area and downstream from where the overtopping jet plunged into the tailwater, were quite tranquil, indicating that the problem of dissipating the energy in the jet would not be too difficult if a deep pool were used to dissipate the flow energy.

Flow Conditions Downstream From Dam

In general the large flows from the main spillway, because of their higher velocities, will have a lower watersurface elevation than to the sides of this flow; such as in the areas downstream from the powerhouse and north abutment. This lower water surface, the topography configuration and the fluid friction produce flow toward the spillway discharge which forms a reverse or counterclockwise eddy downstream from the powerhouse and abutment. When the auxiliary spillway is operating, flow moving downstream tends to nullify the eddy. This action is very beneficial and prevents strong eddy currents and undue erosion of the bank. This was clearly illustrated in calibration tests where the main spillway gates were blocked off and flow was confined to the auxiliary spillway. A fast moving counterclockwise eddy formed adjacent to the north bank near the end of the structure and caused a significant increase in scouring action. This action will not occur on the prototype since the auxiliary spillway will never operate without large discharges through the main spillway.

The surface flow pattern below the dam in most cases when the auxiliary spillway was operating consisted of a system of streams and eddies; one stream moving straight down the river below the spillway gates, another moving from the auxiliary spillway diagonally to the river channel to join flow from the main spillway, and three eddy areas, (figure 13A).

The largest of the eddies formed downstream and to the north of the main river channel. The motion in the eddy was counterclockwise. The eddy was bounded by flow in the river channel, the north river bank, the flow from the auxiliary spillway and smaller clockwise and counterclockwise slow moving eddies. These smaller eddies were located just downstream from the north end of the abutment. The third eddy area formed just downstream from the powerhouse. The area was bounded by the main spillway flow, the face of the powerhouse and flow from the auxiliary spillway. Two slow moving surface eddies, one clockwise and the other counterclockwise, occupied the area. It appeared that these eddies contributed to the deposit of sand observed in the tailrace after many of the model tests.

The eddy conditions and intensity changed somewhat with the design of the plunge pool for the auxiliary spillway flow. All conditions were more tranquil for the recommended design than any design tested.

When the auxiliary spillway was not in operation the pattern consisted of two large eddies and the stream from the main spillway, (figure 13B). The larger of the two eddies formed to the north of the spillway flow and was bounded by this flow, the north river bank and the second eddy which formed downstream from the north abutment. The larger eddy moved in a counterclockwise direction while the smaller and slower one moved in a clockwise direction.

Although operation of the auxiliary spillway alone will not occur on the prototype, flow conditions downstream were observed during calibration tests when the main spillway on the model was blocked off. The main flow from the abutment continued downstream and toward the river channel as before but with different actions at the sides. A rather fast moving counterclockwise eddy formed adjacent to the main flow and just downstream from the north end of the abutment. This eddy attacked the bank and caused excessive scour. There was also a single eddy downstream from the powerhouse which was significantly different from that with both spillways operating.

Flow and Scour Conditions, Model Topography of Compacted Sand (Test 4)

For the initial preliminary test related to using the north abutment as an overflow spillway the model topography was molded entirely of compacted sand, (figure 14A and B), having the gradation shown on Figure 6. The test also provided some indication as to what scour severity might be expected in subsequent testing of the model.

The upstream and downstream topography was shaped in accordance with information contained on several drawings of the prototype and extrapolation of these data. The downstream contours later revised to conform to two recently surveyed sections are shown on Figure 4.

Water was supplied to the model at a very slow rate until the tailwater reached approximately El. 292 which was the estimated elevation for the combined flow of the main and auxiliary spillways. Once this tailwater elevation was established on the model the flow was increased until the depth over the abutment was about 5.9 feet prototype. At this point the erosion of the river bank downstream from the abutment had become so severe that the model supply was shut off. The scouring action was so rapid that nothing would be gained by prolonging the operation to raise the head on the abutment to 7 feet, (Reservoir El. 322). Photographs were taken before and after but not during this test, (figure 14).

The flow had eroded a deep hole along the downstream side of the abutment and had piled the material in a high ridge downstream, (figures 14C and 14D). The bottom of the pool opposite Buttress No. 3 was in the range of El. 265 to El. 270. It was concluded from the test that some form of protection would be required near the dam and in the area of impact of the overtopping jet, particularly near the north end of the abutment section.

The test with compacted sand representing the topography pointed out the importance of having the rock surfaces as well as the overburden in the model for subsequent tests, particularly in the area of the downstream river channel and for some distance downstream from the abutment. Rock contours from project drawings and drill hole records were used to establish rock surfaces in these areas on the model, (figure 4). All subsequent tests were made with the rock surfaces in the model.

Flow and Scour Conditions, Plunge Pool Formed of Armored Slopes (Test 5)

In the first test with the rock surfaces in place the plunge pool was formed entirely of excavated surfaces armored with riprap, the sizes of which ranged from 10 inches to 4 feet, (figures 15, 16A and 16B). This material was well graded. After placing an armor coat of about 3 to 4 feet thickness the surfaces were then covered with one layer of 30 to 42-inch size stones.

The finished surface cross section downstream from Buttress No. 1 was at El. 300 for 14 feet downstream from the buttress. The surface then rose on a slope of 2:1 until it daylighted at the natural ground surface downstream. The surface to the north of Buttress No. 1 sloped upward at 2:1. The surface to the south sloped downward to El. 280 at Buttress No. 8 where the level portion continued downstream for 28 feet then sloped upward at 2:1. The surface south of Buttress No. 8 sloped downward to El. 275 at Buttress No. 11 where the horizontal width downstream from the buttress was 40 feet. From Buttress No. 11 to the north wall of the powerhouse the surface sloped downward to El. 255 at the powerhouse. The armor in this area extended 85 feet downstream from the buttresses.

Erosion at the north end became quite severe almost immediately upon setting the model in operation with flow over the abutment. The riprap surfaces were damaged severely and the topography eroded rapidly until the bedrock surface north of Buttress No. 10 was exposed for some distance downstream, (figure 16C and 16D). It was concluded that armored surfaces excavated in the overburden would not be a solution to the problem, particularly near the north end of the abutment where the plunge pool was shallow. It was thought that a channel excavated in the rock in this area would form a plunge pool that would provide dissipation of energy in the flow and thus prevent excessive scour downstream.

Flow and Scour Conditions, Plunge Pool of Armored Slopes and Rock Channel (Test 6)

Based on the results of the previous test the model area just downstream from the abutment section was altered to include an excavated channel (in rock) with a bottom elevation of 275 between Buttresses 1 and 10 and a width that varied from 20 feet at Buttress No. 1 to 35 feet at Buttress No. 10, (figure 17). The rock surface of the previous test was retained between Buttress No. 10 and the powerhouse. The downstream channel wall and the wall next to the buttresses were placed on 1/4 to 1 slopes in the assumed rock surfaces. The overburden downstream from the channel was cut on a 2 to 1 slope and the slope armored with riprap ranging up to 4 feet. A vertical end (or training) wall was placed downstream opposite Buttress No. 1 and the warped surface from vertical at the end of the wall to a 2:1 slope downstream from the channel, was protected with rock stabilized with concrete. The channel, filled with riprap to approximately El. 280, as it existed before the test, is shown on Figure 18A.

As the reservoir approached El. 322, the "boil" or roll of the water downstream from where the overflowing stream plunged into the channel began to attack and dislodge riprap along its downstream edge, (figure 18B). As the test continued the roll and currents downstream from it continued to dislodge and move the riprap and wash away the overburden. The movement and washing was at a much slower rate than in Test 5 where only rock and riprap protected surfaces were used. The conditions in the area downstream from the abutment section after an hour's operation of the model at Reservoir El. 322.9 are shown on Figure 18C. Essentially all of the riprap was removed from the channel and moved downhill to the south and east. The channel bottom and rock surface north of Buttress No. 15 were essentially swept free of all riprap.

The scour near the north end of the abutment section was considered too severe. Also, the deep excavation immediately downstream from the buttresses to form the channel was not desirable. It was concluded that a concrete-lined channel placed near the rock surface, as interpreted from the drill holes, would be required. These conclusions led to the arrangement used in the next test.

Flow and Scour Conditions, Plunge Pool with Concrete-lined Trough No. 1 (Test 7)

After testing the variable width trough, with bottom at El. 275 between Buttresses 1 and 10, it was decided that the trough was too low with respect to the toes of many of the buttresses. With a full reservoir it was not considered safe to excavate below and adjacent to the downstream toes of the buttresses. It was therefore decided that the trough should be placed with its invert at approximately the same level as the bottoms of the buttresses, (figure 19).

The north end of the trough started at Buttress No. 2 and had a constant slope from this point to Buttress No. 8. At Buttress No. 8 the slope flattened and continued to Buttress No. 16 where it ended approximately where the north wall of a possible future outdoor power unit would be located. The surface between the end of the trough and the present powerhouse was protected with riprap ranging in size from 6 inches to approximately 4 feet. A vertical training wall was placed at the north end of the trough at 90 degrees with the axis of the dam to prevent scour of the abutment area. The arrangement is shown on Figure 20A.

The cross section of the trough was the same between Buttresses 2 and 8. The elevation of the bottom of the trough at Buttress No. 2 was 300 while that at Buttress No. 8 was 265. The upstream edge of the trough was placed a constant distance of 7 1/2 feet from the toes of the buttresses forming a bench with a 1:1 slope downward to the trough floor 5 feet below. This was done to further insure against possible undermining of the buttresses. The trough width was 40 feet and the downstream wall of the trough between Buttress 2 and 8 was vertical and rose 10 feet above the bottom, (figure 19).

The upstream wall of the trough was the same between Buttresses 2 and 16, however the vertical downstream wall sloped from 10 to 0 feet high between Buttresses 8 and 16. The bottom elevation of the trough at Buttress No. 16 was 255. Riprap backfill was placed downstream from the wall as shown on Figure 19.

A vertical curved wing wall was placed in the reservoir upstream from Buttress No. 2. The wall was offset 3.5 feet to the south of the training wall. This offset was for the purpose of allowing the flow to be confined by the downstream wall without the need for an obstruction across the walkway on top of the abutment section.

The performance was better than the previous arrangements. However, as the flow over the abutment increased, the jet near the north end of the abutment struck the floor of the trough, continued across it and produced excessive erosion downstream, (figure 20B). However, there was a significant improvement in the scour pattern over previous tests, (figure 20C). There was some movement of the rock placed between the south end of the trough and the powerhouse. It appeared that the width of the trough could be narrowed a great deal without destroying its effectiveness. This change was included for the next test.

Flow and Scour Conditions, Plunge Pool with Concrete-lined Trough No. 2 (Test 8)

The trough was narrowed throughout its full length and a vertical wall 9 feet high was placed along the entire length at its downstream edge. The upstream 1:1 sloped wall was kept 7 1/2 feet from the toes of the buttresses. The bottom elevation at Buttress No. 2 was set at 300, while at Buttresses 8 and 16 the bottom elevation was 265 and 255 respectively. The bottom width of the trough was 15, 23, and 33 feet respectively at Buttresses 2, 8 and 16.

During initial operation of this design the water jet near the north end of the north abutment was quite turbulent with some overtopping of the downstream wall of the trough and movement of the riprap. An upstream overhang of about 1 foot placed along the top of the wall for 56 feet at its north end served to deflect the jet back into the trough onto the incoming flow. Very quiet conditions were present downstream from the trough. It appeared that the width of the trough could be further reduced near the north end, and the model was altered accordingly for the next test.

Flow and Scour Conditions, Plunge Pool with Concrete-lined Trough No. 3 (Test 9)

For Test 9 the trough along the downstream side of the north abutment was narrowed and minor changes in bottom elevation were made, (figure 21). The upstream edge of the trough, however, was kept 7 1/2 feet from the toes of the buttresses and 5 feet above the trough bottom. The width of the bottom of the trough opposite Buttress No. 2 was 10.5 feet and its bottom elevation was 300. At Buttress No. 8 the width was 21.0 feet and the bottom elevation was 268. The trough terminated near Buttress No. 16 where the width was 22.5 feet and the elevation was 255. The downstream wall of the trough was vertical and the height was kept constant at 9 feet between Buttresses 2 and 8 and was then sloped from 9 feet to zero between Buttresses 8 and 16. The north 50 feet of the downstream wall was capped with a 1-foot overhang to deflect the flow back upstream. The model ready for testing is shown on Figures 22A and 22B.

Flow conditions in the tailwater above the trough were excellent, (figures 22C and 22D), and the erosion downstream from it was mild with essentially no movement of riprap along the downstream side of the downstream wall, (figure 23). However, the riprap armor which contained some rock up to 4.0 feet was washed from an area just to the south off the end of the trough, (figure 23B). It was considered desirable to prevent or minimize this relatively small area of excessive scour. It appeared that the overspilling jet was deflected downstream and to the south as it passed through the trough causing a cross flow over the edge of the pavement at the end of the trough.

<u>Effect of Cross Walls on Scour at South End of Trough</u>

The effects of placing various types of walls at the south end of the trough were investigated. The wall designs and positions used are shown on Figure 24. A wall 2 feet high with a 1:1 slope on its north side was placed across the end of the trough for the first modification, (Wall No. 1, figure 24).

This first modification produced little if any improvement in scour conditions and the wall height was increased to 5 feet but at the same time retaining the 1:1 slope on its north side, (Wall No. 2, figure 24). Slight improvement was realized but further improvement seemed desirable, (figure 25C). The 1:1 slope on the north side of the wall was removed to determine whether or not it was more effective than a vertical wall of the same height, (Wall No. 3, figure 24). There was no discernible difference for the two walls, (figure 25D).

A 1-foot overhang was added to the north side of the 5-foot high vertical wall to determine whether or not the cross flow at the end of the trough was a major factor contributing to the scouring action, (Wall No. 4, figure 24E). There appeared to be a decided improvement particularly along the south side near the downstream end of the wall, (figure 25E). The maximum scour occurred near the upstream end of the wall, apparently where the jet overspilling the abutment struck the wall and riprap. It appeared that a few heavier stones from 5 to 6 feet in size, strategically placed, would essentially eliminate the scour. Actually there was very little movement of any of the riprap when the model was operated for a half hour period with the larger stones in place. This would be one means of minimizing the scour and would be entirely adequate for discharges in the range of 17,000 to 20,000 cfs.

After examining and photographing the scour conditions the model was again placed in operation and the reservoir level increased to elevation 325.2, or a head of 10.2 feet over the abutment. The estimated discharge for the north abutment at this reservoir elevation is approximately 30,000 cfs. Although erosion of the left river bank downstream was not materially changed for the greater discharge over the abutment, the riprap between the powerhouse and the end of the protective trough was all moved downstream off the rock surface. The riprap was deposited downstream beyond a line extending from the downstream powerhouse wall northward. The boiling and turbulence at the surface were noticeably increased over that for the lesser discharge of 17,000 and a reservoir elevation of 322.

After photographing the results of this test the 5-foot high wall at the south end of the trough was removed and the riprap between the powerhouse and trough replaced with stones varying from 1 to 4 feet prototype, except for an area about 20 feet wide adjacent to the end of the trough where the scour had persisted in previous tests, (figure 25A). The size of the surface riprap in this area was increased to 4 and 5-foot stones. Operation of the model at reservoir El. 322 for a half hour resulted in only a slight rearrangement of some of the smaller stones, (figure 25B). After examining the scour conditions the model was again set in operation with the gates at El. 307 and the reservoir at El. 323 (1 foot higher than usual).

After a half hour of operation it was found that the riprap in the area off the end of the trough had undergone some further movement. It was concluded from this test that the riprap size and arrangement was adequate for this flow which was approximately 21,000 cfs but that it would become unstable with any appreciable increase in flow.

Although the flow conditions for floods of up to 30,000 cfs over the north abutment were considered entirely acceptable it was questionable whether or not riprap sizes from 1 to 6 feet placed off the end of the trough would be adequate for flows greater than 21,000 cfs. It was believed that paving the area between the powerhouse and south end of the trough or extending the trough to the powerhouse would be necessary to eliminate objectionable scour for flows in the range of 25,000 cfs to 30,000 cfs over the North Abutment. Such an arrangement was not considered for testing at the time because of the possible future installation of an outdoor generating unit in this area.

Since there was little or no movement of 1 to 3-foot riprap in the backfill downstream from the downstream vertical wall of the trough, it is believed that the riprap beyond the wall need not be larger than 2 feet. The riprap strip need not be more than 20 feet in width. The width tested on the model was equivalent to 20 feet.

In a further attempt to eliminate the scour off the end of the trough, a dentated wall 10 feet high was placed along the end of the trough, (Wall No. 5, figure 24). The base of the wall was 6 feet wide with a 1:1 slope between dentates that were 10 feet high, 3 feet wide and 6 feet long. The 3-foot wide dentates were placed on 6-foot centers leaving a 3-foot wide space between them. Graded riprap from 1 to 4 feet was placed to elevation 255 in the area between the trough and the powerhouse and the model was operated for a half hour with the reservoir at El. 322.

At the end of the run it was found that there was very little scour adjacent to the south side of the wall but that the scour was to bed rock a short distance south of the wall in the area where the overtopping jet from the abutment plunged to the riprap surface, (figure 25F). It seemed that the jet striking the top of the wall produced a turbulent zone to the south side of the wall and that this turbulence disturbed and moved the stones from this relatively small area.

To evaluate conditions at higher discharges the model was placed in operation with the reservoir at El. 323 without disturbing the scour pattern obtained at Reservoir El. 322. At the end of a half hour of operation a significant movement of the riprap was noted with the area of exposed bedrock much greater than for the lower reservoir elevation. It was again concluded from the two tests that walls at the end of the trough were relatively ineffective in preventing scour. However, a 5-foot high vertical wall was placed across the trough behind Buttress No. 15, Wall No. 6, (figure 24). This arrangement provided some pavement adjacent to the wall and to the south. The purpose was to confine any turbulence from the wall to the paved surface.

After a half hour operation the scour conditions were again examined. The scour was minimal and was less than for any of the walls placed at the end of the trough, (figure 25H). It appeared that with this wall and a discharge with the reservoir at El. 322 there would not be a need for riprap larger than 1 to 4 feet. The model was again placed in operation for half an hour with the reservoir at El. 323 to determine the effect of the larger flow. A relatively small area of the bedrock just south of the end of the trough was exposed, again indicating that it will be necessary to place larger rock in the area or to pave the area between the powerhouse and the south end of the trough. It was concluded that paving the area would make the abutment safe for any flows up to and including reservoir elevation 325 (29,000 cfs over the north abutment). The model was then altered to include the pavement.

Flow and Scour Conditions, Recommended Plunge Pool with Concrete-lined Trough, Pavement and Riprap (Test 10)

After it was decided to pave the area between the powerhouse and the end of the trough near Buttress No. 16, the model trough dimensions were changed slightly, (figures 21 and 26A and B). The cross section at Buttress No. 2 was maintained and the floor was kept at El. 300, the same as for the previous test. However, a slight change in this section is recommended for the prototype as discussed below.

During a test with the abutment crest shaped to an ideal profile the nappe of the overflow was steeper than with the present flat crest and the jet impinged on the downstream portion of the bench upstream from the trough. Although this action did not impose any critical conditions it was evident that narrowing the bench in this region to allow the jet to strike the floor within the trough would be more desirable. It is believed that making the bench 5 feet wide instead of 7 1/2 feet wide at the toe of Buttress No. 2 (as tested on the model) and maintaining the width to 7 1/2 feet at the toe of Buttress No. 8 would accomplish this. Since such an alteration will not change the effectiveness of the plunge pool with flow over the present crest, it is recommended for the final design and shown on Figure 21. The cross section at Buttress No. 8 was retained. The cross section at Buttress No. 16 was kept the same shape and length. The downstream wall of the trough between Buttresses 8 and 16 was kept vertical and tapered from a height of 9 feet at Buttress 8 to zero at Buttress No. 16. The paved area downstream from the abutment and between Buttress No. 16 and the north powerhouse wall was set at elevation 252, (figures 21 and 26C). This gave a 3-foot vertical offset in the floor of the trough at Buttress No. 16. The treatment at the toes of the buttress between the powerhouse and Buttress No. 16 was the same as at the toes of all buttresses north of No. 16. (7 1/2-foot bench at El. 257 and 1:1 slope down to El. 252.) The surfaces downstream from the trough and pavement were sloped where necessary and a riprap strip approximately 20 feet wide was placed along the downstream edge of the trough and pavement. The riprap size varied from approximately 6 inches to 4 feet with only a small number of the latter. The general maximum size was in the range of 2 1/2 to 3 1/2 feet. When placing riprap on the prototype the largest stones should be placed adjacent to the paved area, the next largest between section C-C and B-B, and the smallest between sections B-B and A-A, (figure 21).

Flow conditions were very satisfactory with the reservoir at El. 322 and the tailwater at approximately elevation 292, (figure 27A).

Examination of the scour conditions after an hour of testing showed very minor movement of riprap opposite the paved area between Buttress No. 16 and the powerhouse, (figure 27C). Actually there was some deposition of sand on the riprap downstream from Buttresses 10 through 13. The conditions were considered entirely satisfactory and it appeared that only the smallest riprap would be needed above the tailwater level at the north end of the abutment. The trough, paving, excavation and riprap surfaces used in this test with slight alterations shown on Figure 21, are recommended for the final design.

Since this design was considered to be final it was desired to determine how well it would handle a superflood with the reservoir at elevation 325. The model was operated for one half hour at this reservoir level and then re-examined.

Flow conditions on the surface downstream from the abutment were more turbulent than with the reservoir at El. 322, but they were considered entirely acceptable, (figure 27B).

Examination of the scour conditions after the test showed no significant movement of riprap north of Buttress 16, (figure 27D). However, except for some large stones along the downstream edge of the pavement between the powerhouse and Buttress 16, the riprap material had been moved downstream, exposing a strip of bedrock about 25 feet wide. Since there was no tendency for undermining the pavement the arrangement was considered adequate to handle floods in which the reservoir rises to elevation 325 (approximately 29,000 cfs over the north abutment). It would seem wise to place a row or two of large riprap (4 to 6 feet) adjacent to the downstream edge of the paved area between the powerhouse and Buttress No. 16. The width of the riprap strip below this section should be 10 to 12 feet wide.

Flow Conditions at North Training Wall

Flow conditions at the recommended wing wall and training wall at the north end of the Auxiliary Spillway were examined in detail during the tests with the reservoir level at elevations 322 and 325. The water depth on the abutment walkway to the north of the walls was recorded as was the elevation of the water surface along the training wall, (figure 28). The edge of the jet as it flowed along the face of the training wall reached a maximum elevation of 317 when the reservoir was at El. 322. The maximum elevation at the upstream end of the training wall was 320 when the reservoir was at El. 325. In both cases the depth of water on the walkway to the north of the training wall was approximately 2 feet. It is possible that a parapet wall along the downstream edge of the walkway north of the training wall need not be higher than 2 feet. However, it would seem desirable because of uncertain conditions to supply freeboard to prevent overflow behind the wall which could undermine and cause the wall to fail. A 3-foot high parapet is suggested. The information on Figure 28 can be used to establish the height and extent of the training wall.

During the test it was noted that better flow conditions with a lower watersurface profile along the training wall occurred when the downstream south corner of the wing wall was a sharp 90-degree corner. The corner of the wing wall on the model was shaped as near 90 degrees as possible using plasticene. Surface tension of the water on this material prevented it from springing completely free from the downstream face of the wall, thus increasing the elevation along the wall over that for a sharper corner of a material less affected by surface tension.

It is recommended that this corner of the prototype be made smooth and square, perhaps by armoring it with an angle iron.

Deposit of Material in Powerhouse Tailrace

During the tests made on the model for a study of the adequacy of the riprap at the north end of the powerhouse, it was noted that there was always a tendency for material scoured from the river banks to deposit in the tailrace area. This indicates that dredging of sand and gravel from this area may be required after each flood in which the abutments are overtopped.

Flow and Scour Conditions, Recommended Plunge Pool With Future Power Unit Enclosure in Place

The recommended final design arrangement for protection downstream from the north abutment with the powerhouse as it presently exists was modified to include a possible future outdoor power unit, (figure 29). The unit structure with its main top elevation 278.25 was 60 feet wide and extended 111 feet downstream from the downstream face of the present powerhouse. The cylindrical enclosure for the unit that extended upward from the main roof surface was 23 feet to the north and 22 feet downstream from the present powerhouse structure. The top of the cylindrical enclosure was at El. 300. The draft tube with the top of its downstream opening at El. 259 extended to a position 111 feet downstream from the downstream face of the present powerhouse structure. The tailrace was excavated on a 6:1 slope from the end of the draft tube until the slope daylighted downstream. The side slopes of the excavation were approximately 1/2 to 1. The model arrangement before tests were made is shown on Figure 30.

The trough of the plunge pool to the north of the future unit ended at the north wall of this unit. The shape of the trough was kept the same as developed in previous tests for the recommended design without the future unit. A band of riprap (6 to 42-inch rock) 20 feet wide was place along the downstream edge of the trough.

Water overflowing the abutment for 60 feet north of the present powerhouse would plunge into approximately 15 feet of water on top of the penstock section of the future unit.

Flow Conditions at Future Power Unit Enclosure

The model was placed in operation slowly so as not to disturb the downstream topography with an unnatural condition such as the tailwater being too low or the spillway flow too large. The reservoir was at El. 322 with a discharge of approximately 17,000 cfs over the Auxiliary Spillway.

The flow to the north of the future power unit was essentially the same as without the unit in place, (figure 31A). The jet overtopping the crest above this unit plunged through about 15 feet of tailwater onto the top of the penstock passages, (figure 31B). The flow was turned downstream by the impingment on the roof of the passages to come in contact with the cylindrical enclosure of the power unit. Here the flow was deflected upward and to each side of the enclosure. The water at the extreme upstream point on the cylinder was about the same height as the enclosure which was approximately El. 300. The elevation decreased toward the sides of the cylinder finally reaching the tailwater elevation at the downstream centerline about 58 feet from the upstream point. The elevation of the water surface at the downstream side of the enclosure was 291.7, (figure 32). The pressures that would accompany the above described water elevations should be taken into consideration in the design of the enclosure for the future unit. Also, the impact of the overtopping jet on the roof of the penstock passages and the possibility of vibrations that could cause structural damage should be considered. Flow conditions adjacent to the old powerhouse and underneath the jet upstream from the line of impingement were quite tranquil and acceptable.

A final test was made with the reservoir level increased to El. 325 with a discharge of approximately 29,000 cfs over the auxiliary spillway. The flow conditions were generaly similar to those for the lower reservoir elevation, however, all eddy currents and flow actions were more intense but acceptable. The water surface elevations on the wall of the circular enclosure are shown on Figure 32.

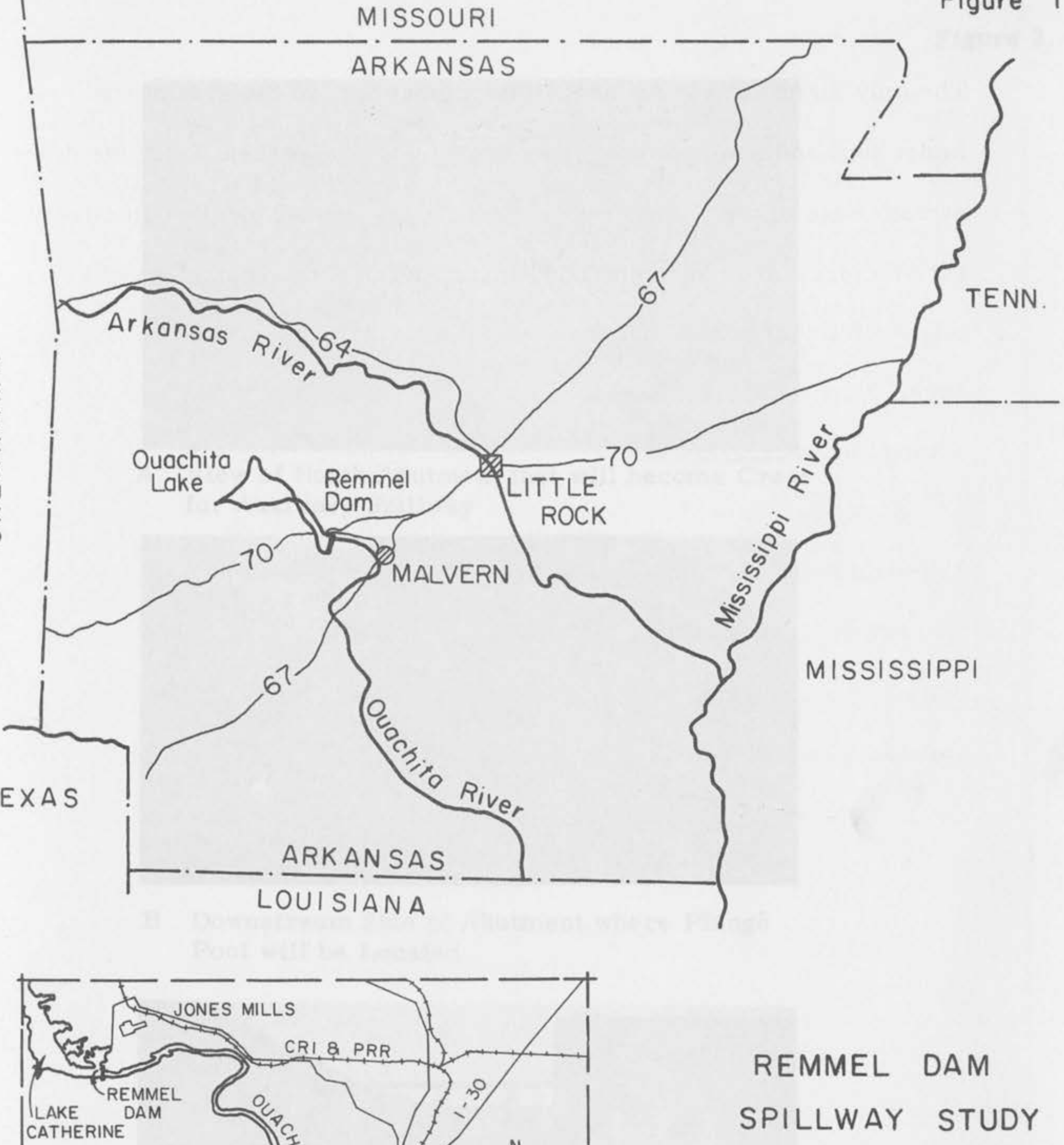
Scour Conditions With Future Power Unit Enclosure in Place

After operation of the model for an hour with the reservoir at El. 322, there had been no significant scouring of the river bank materials. There was definite movement of the overburden but the riprap downstream from the trough was mostly intact, (figures 33A, 33B and 33C).

The only significant movement in the riprap was to the side of the future unit and some distance (about 25 feet) downstream from the downstream edge of the trough. These conditions were considered entirely satisfactory. It is recommended that some of the larger stones be placed in the area and that these stones not protrude above the concrete edge. Waste concrete during construction of the future unit could be dumped on rock in the area to make it more scour resistant.

Scour conditions after operating the model at reservoir elevation 325 for a half hour were more severe than at the lower reservoir elevation but the larger riprap remained in place just downstream from the edge of the concrete trough, (figure 33D). The scour conditions were considered acceptable for the flow conditions imposed.

Figure 1



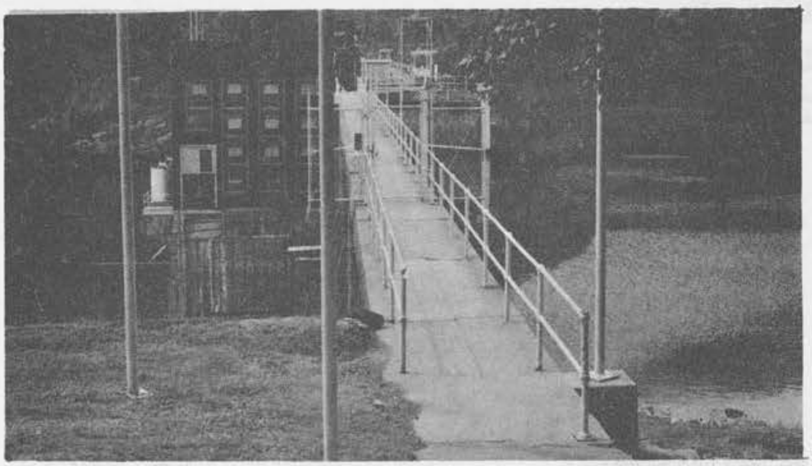
REMMEL DAM

SPILLWAY STUDY

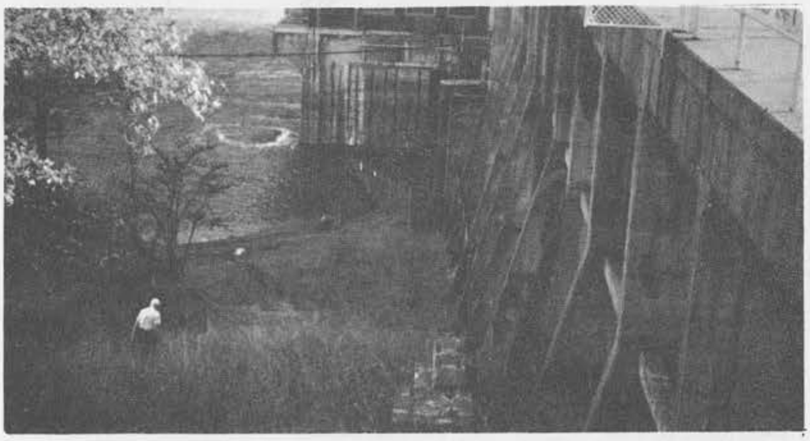
1:42 Scale Model

Location Map

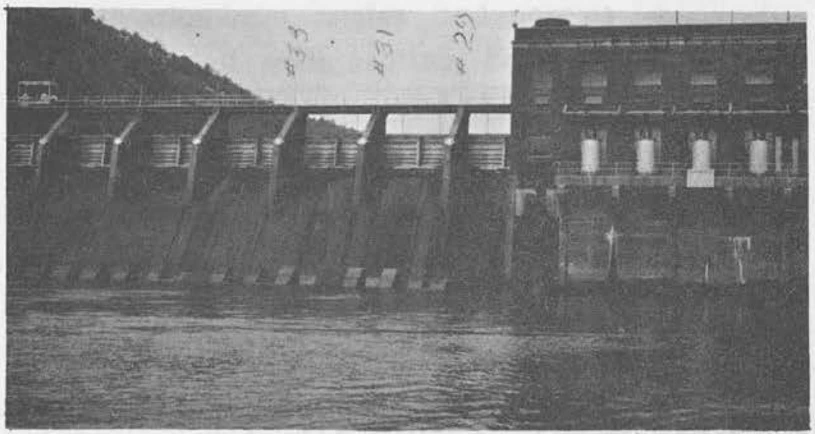
Figure 2



A View of North Abutment that will become Crest for Auxiliary Spillway



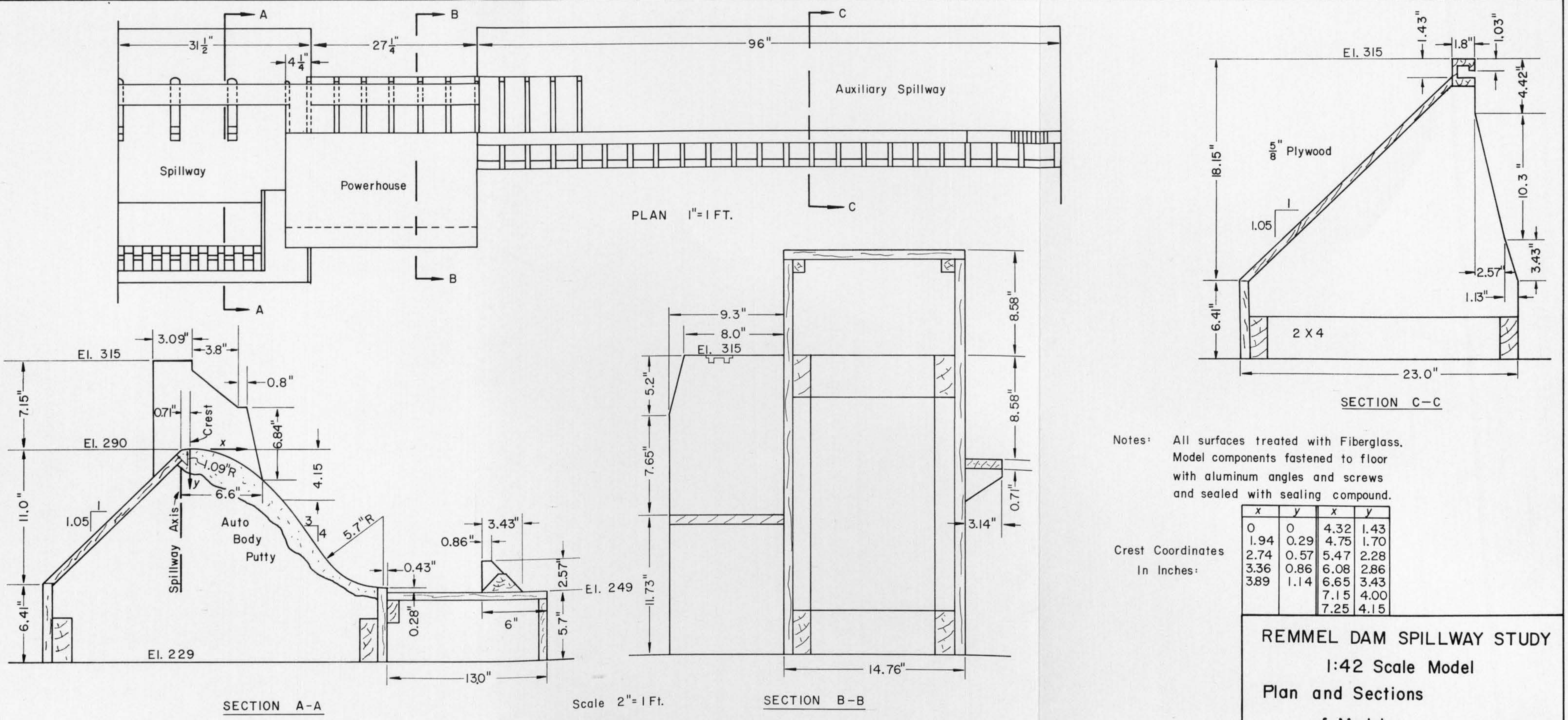
B Downstream Side of Abutment where Plunge Pool will be Located



C Downstream Side of Main Spillway and Powerhouse

Remmel Dam Spillway Study
1:42 Scale Model
Views of Existing Prototype Spillway
Powerhouse and North Abutment

FIG. 3



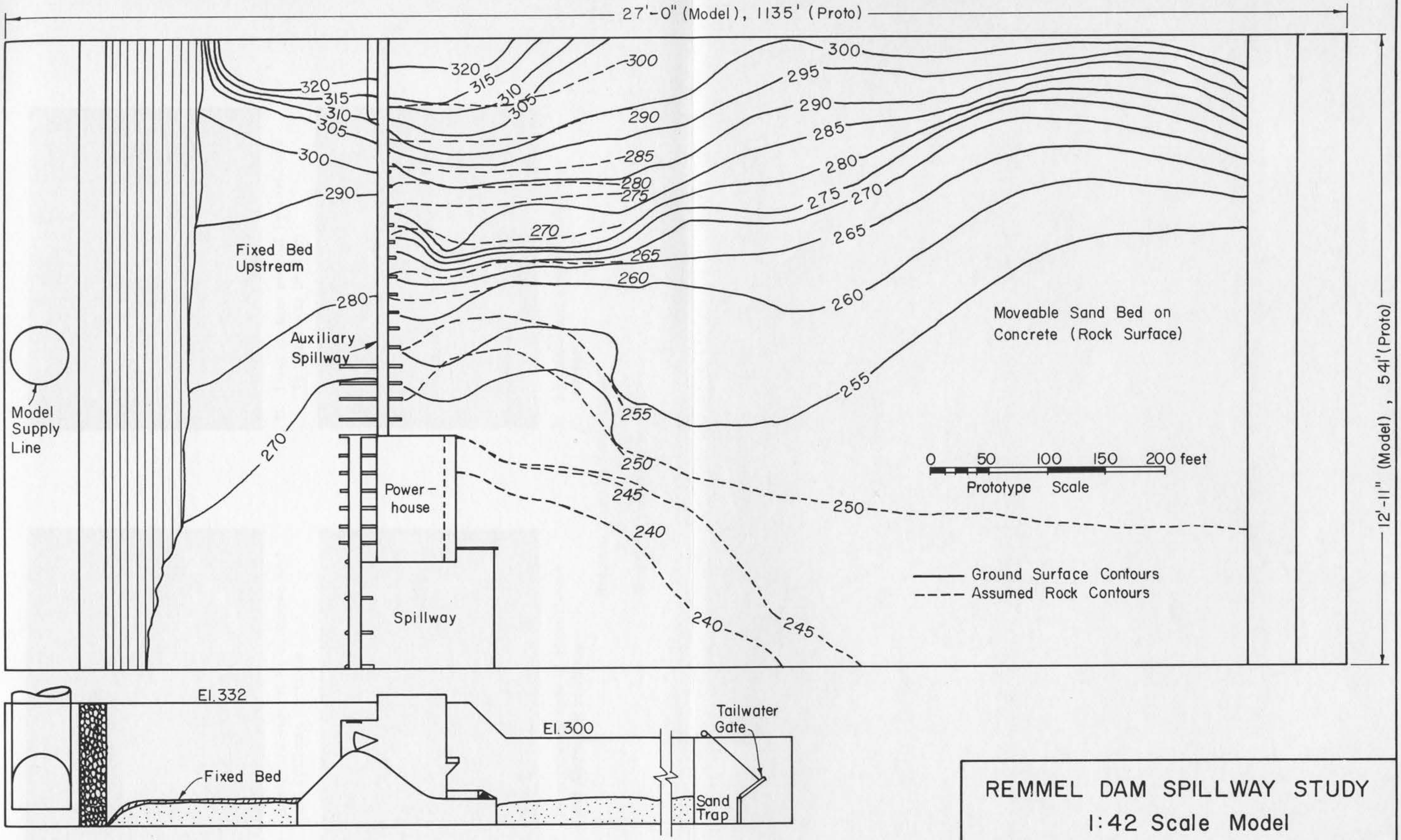
Notes: All surfaces treated with Fiberglass.
Model components fastened to floor
with aluminum angles and screws
and sealed with sealing compound.

Crest Coordinates
In Inches:

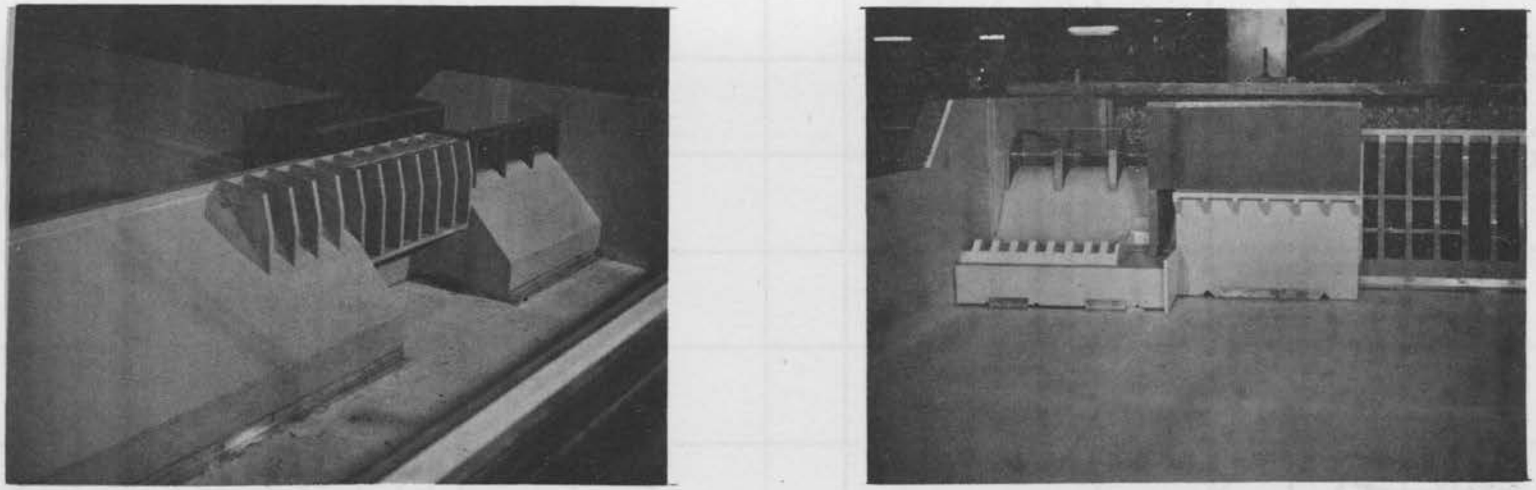
x	y	x	y
0	0	4.32	1.43
1.94	0.29	4.75	1.70
2.74	0.57	5.47	2.28
3.36	0.86	6.08	2.86
3.89	1.14	6.65	3.43
		7.15	4.00
		7.25	4.15

REMMEL DAM SPILLWAY STUDY
1:42 Scale Model
Plan and Sections
of Model

FIG. 4

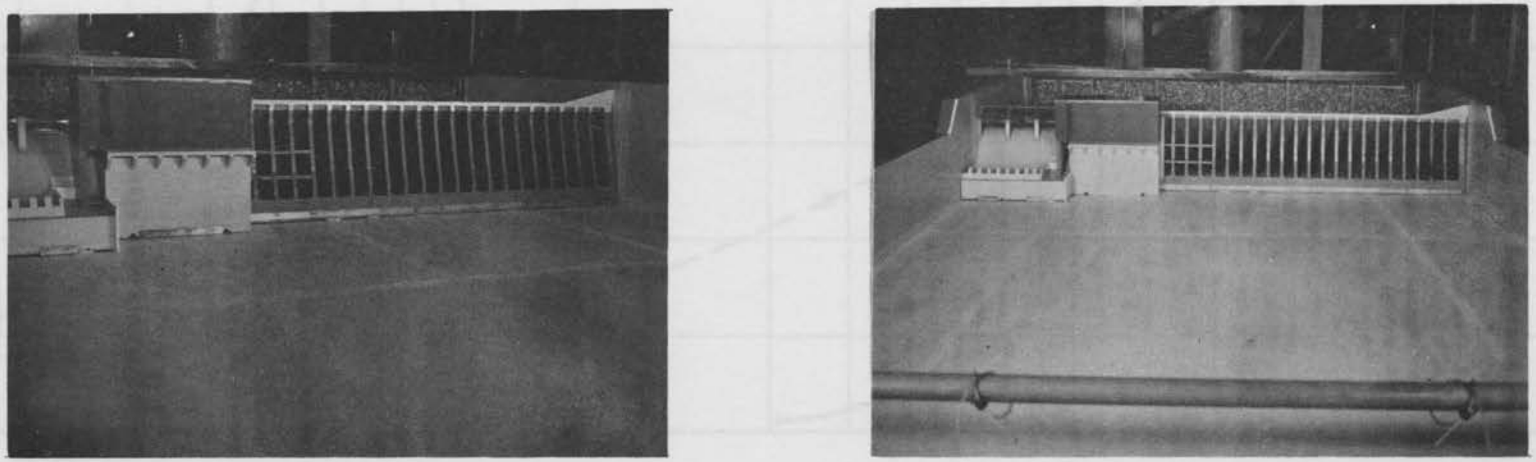


REMMEL DAM SPILLWAY STUDY
1:42 Scale Model
Ground and Rock Contours Used
For Model



A Upstream side of Powerhouse and and Spillway sections

B Downstream side of Powerhouse and Spillway section



C Downstream side of Powerhouse and Abutment sections

D Downstream side of completed Model

Remmel Dam Spillway Study
1:42 Scale Model
Model Components in Place
Without Topography

Figure 5

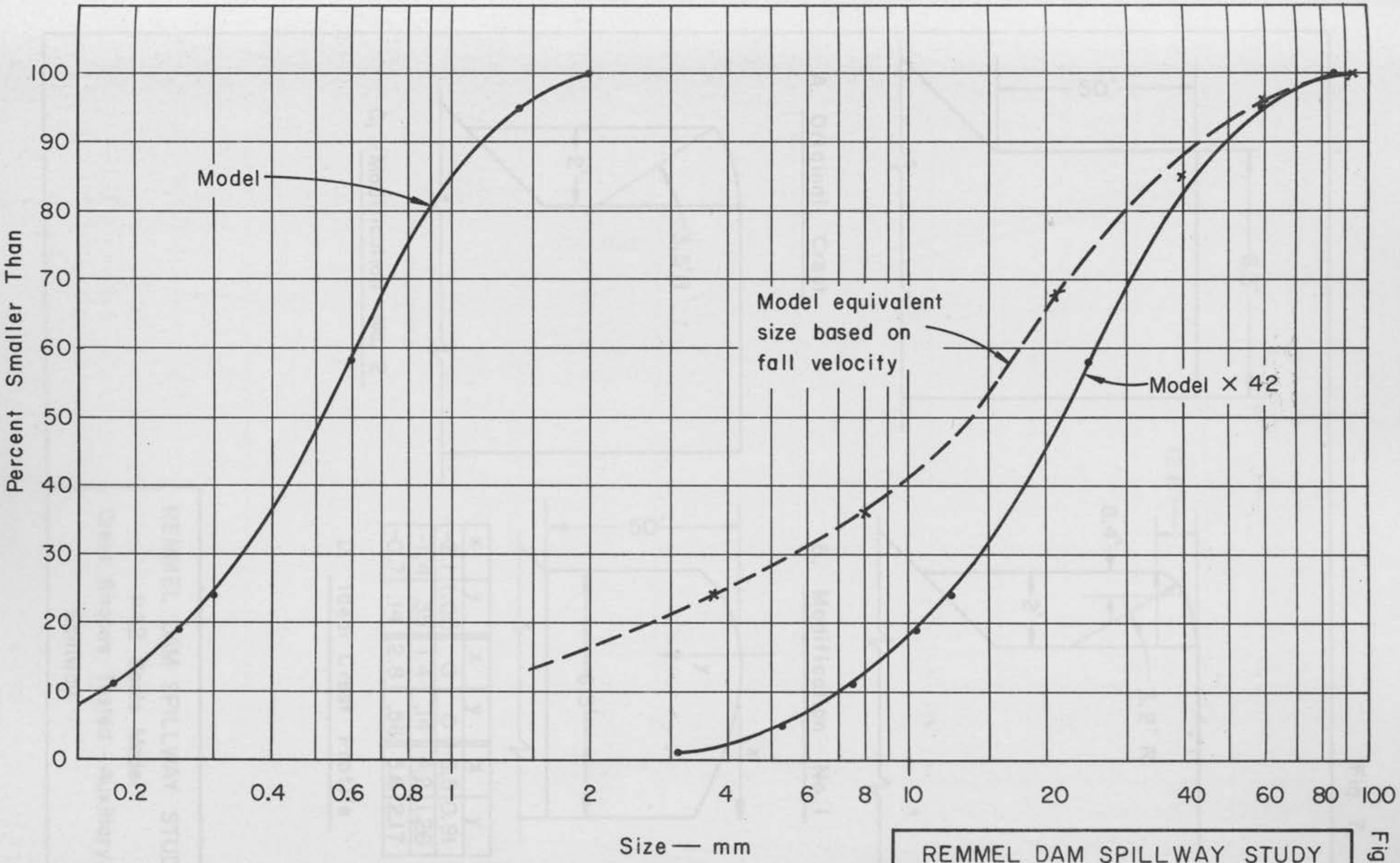
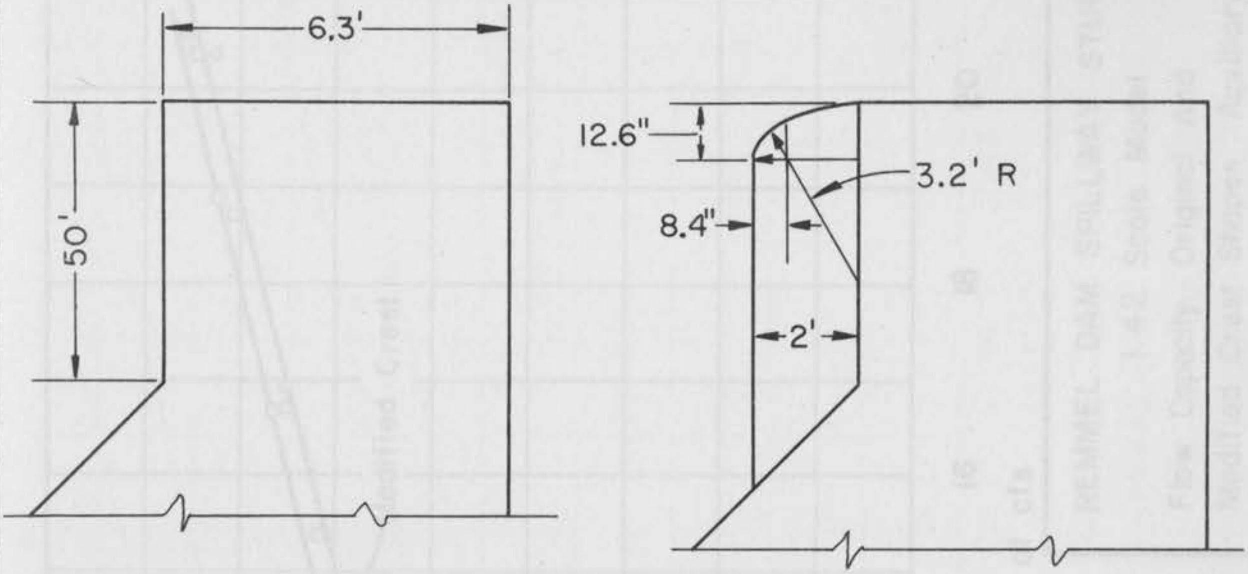


Fig. 6

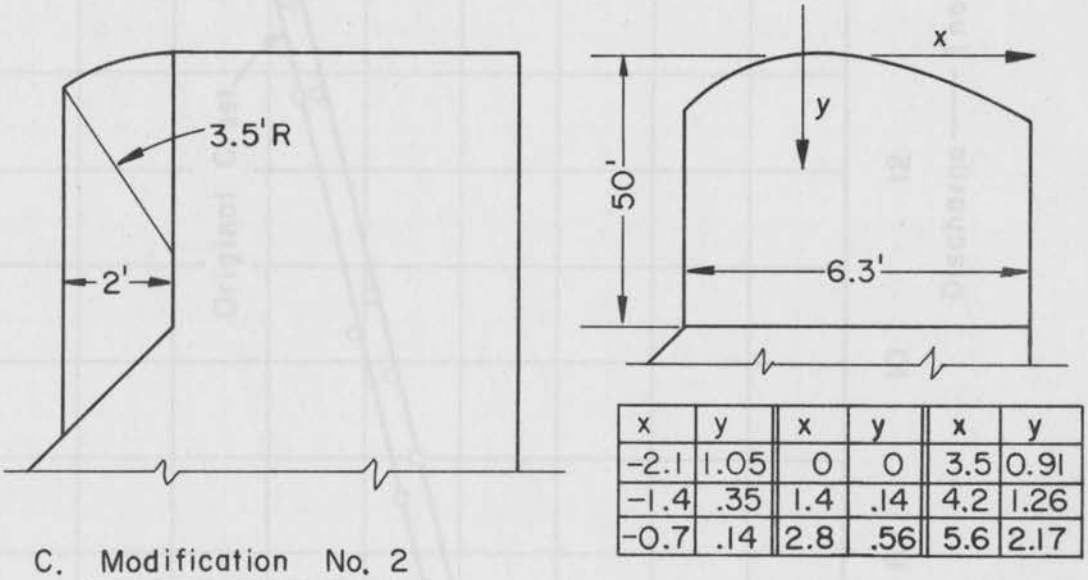
REMMEL DAM SPILLWAY STUDY
1:42 Scale Model
Graduation of Sand Used For Model
Moveable Bed

Fig. 7



A. Original Crest

B. Modification No. I

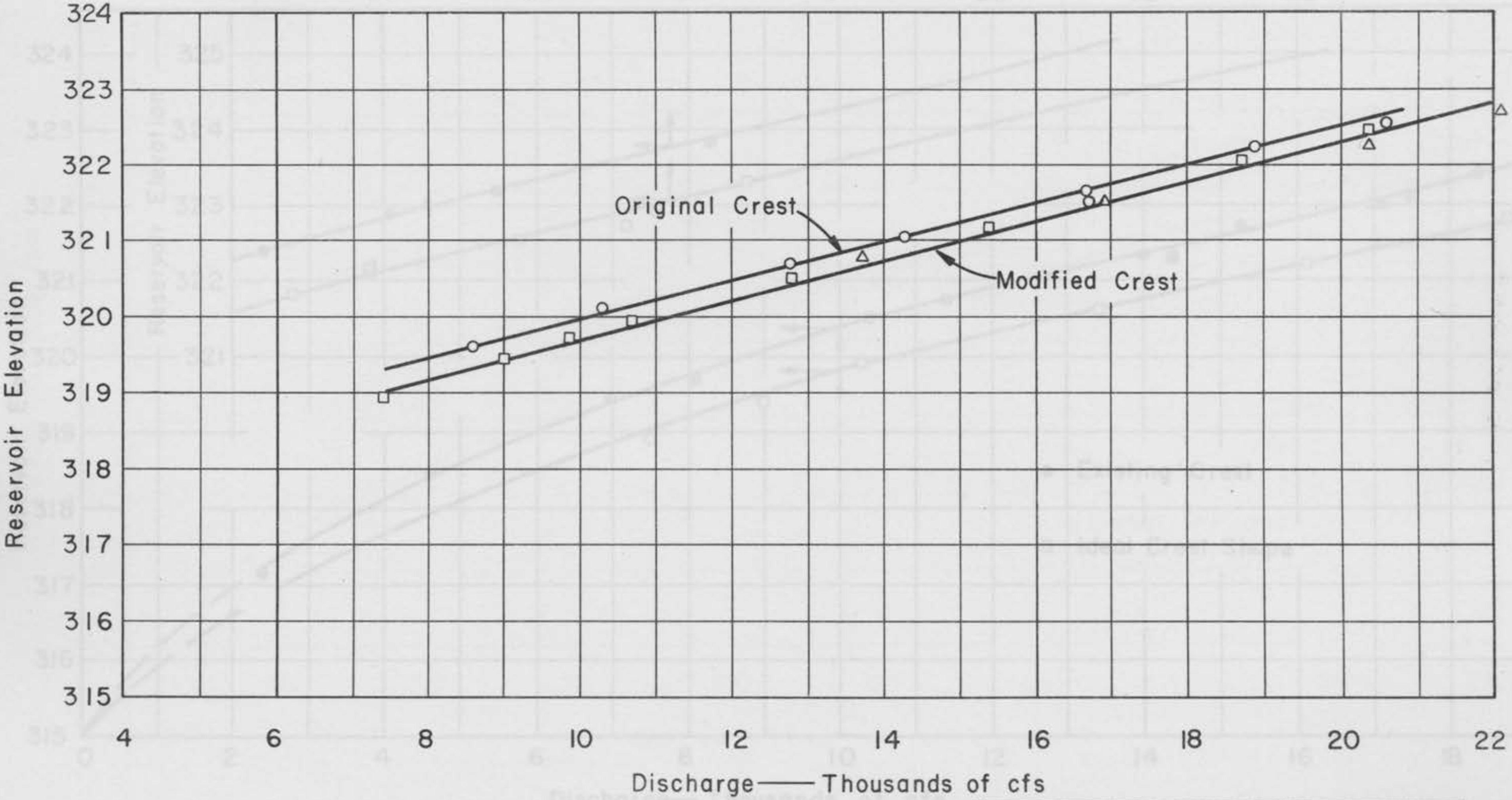


x	y	x	y	x	y
-2.1	1.05	0	0	3.5	0.91
-1.4	.35	1.4	.14	4.2	1.26
-0.7	.14	2.8	.56	5.6	2.17

C. Modification No. 2

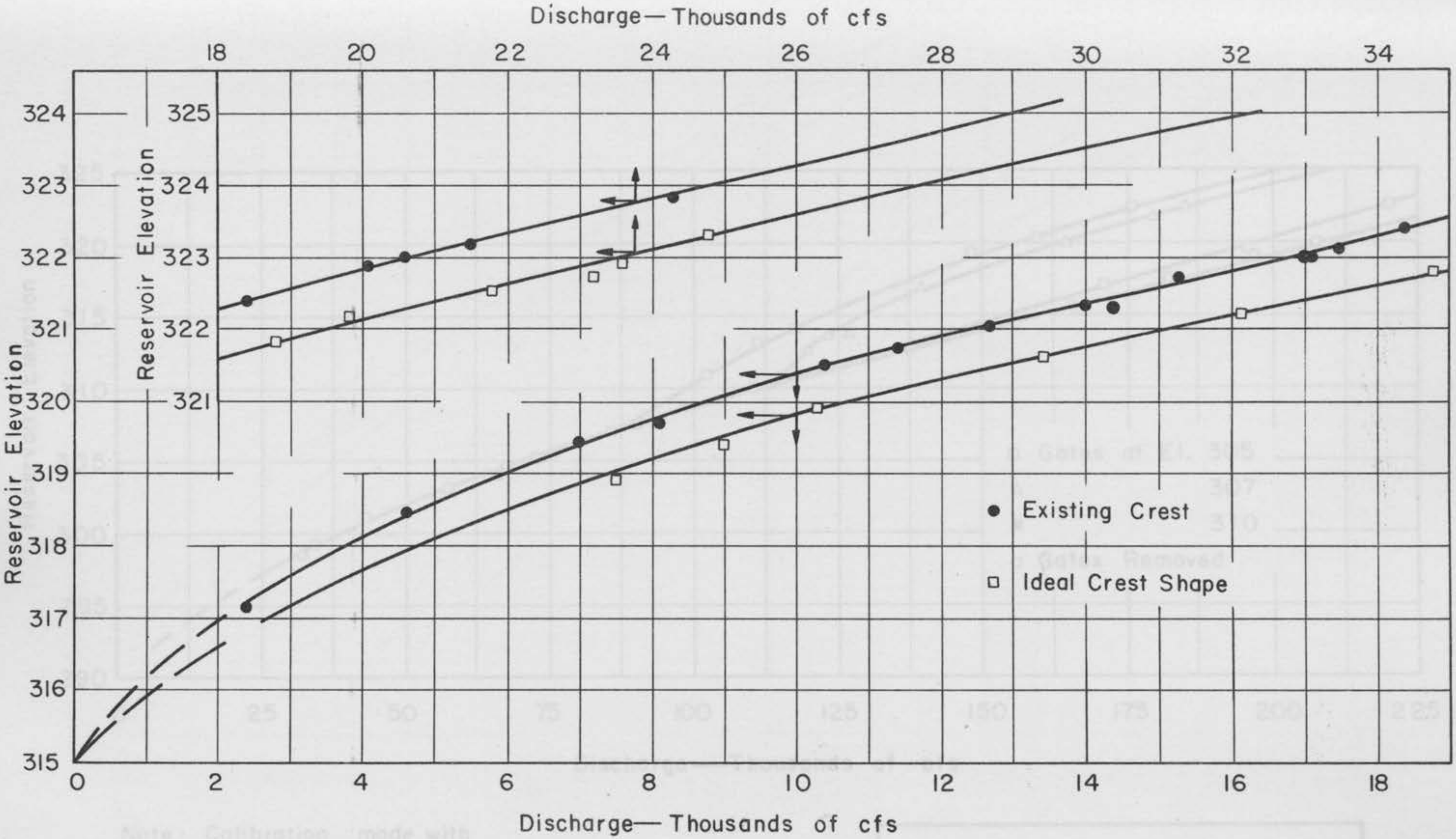
D. Ideal Crest Profile

REMMEL DAM SPILLWAY STUDY
1:42 Scale Model
Crest Shapes Tested – Auxiliary
Spillway



REMMEL DAM SPILLWAY STUDY
1:42 Scale Model
Flow Capacity - Original And Modified Crest Shapes. Auxiliary Spillway

Fig. 8

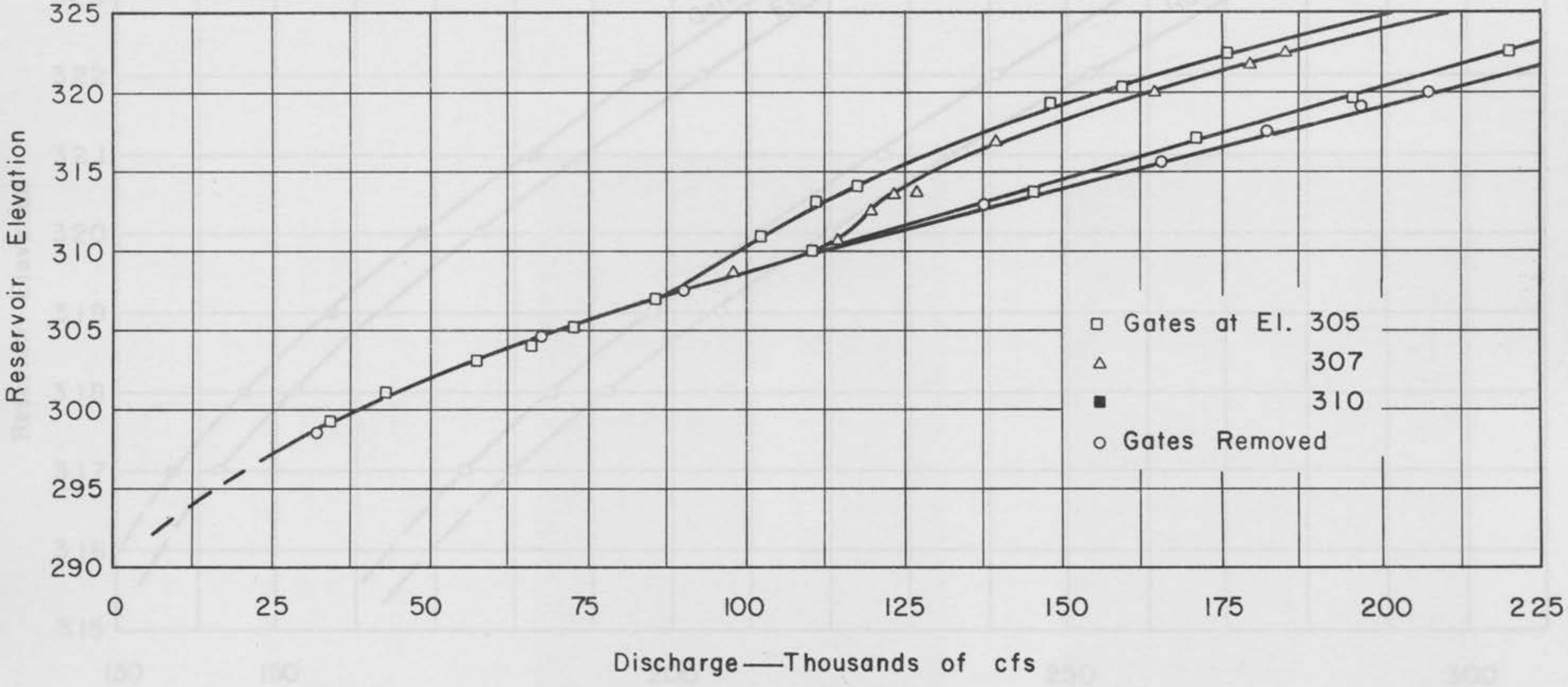


Notes: North abutment overflow section length = 270 ft.

Calibration made with gated spillway blocked off.

REMMEL DAM SPILLWAY STUDY
1:42 Scale Model
Flow Capacity - North Abutment Only

Fig. 9



Note: Calibration made with
Auxiliary Spillway blocked off.

REMMEL DAM SPILLWAY STUDY
1:42 Scale Model
Flow Capacity – Gated Spillway
Only Discharging

Fig. 10

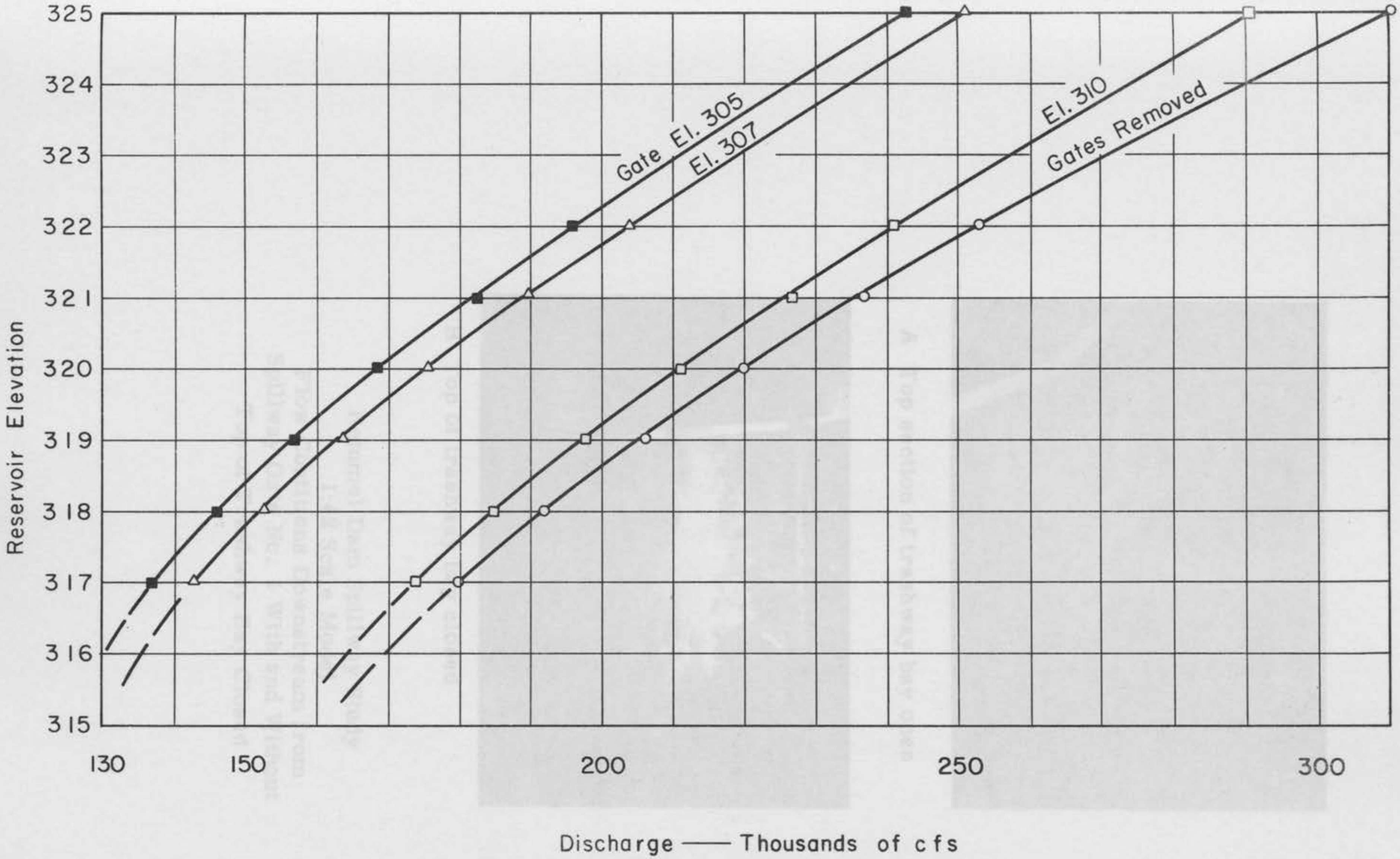
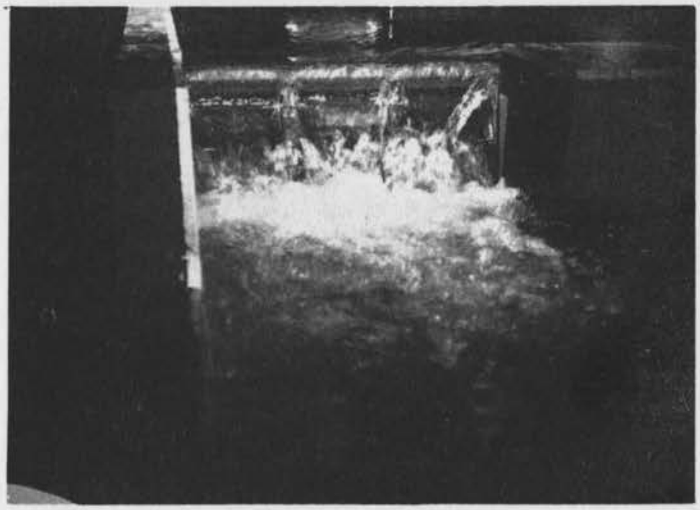


Fig. 11

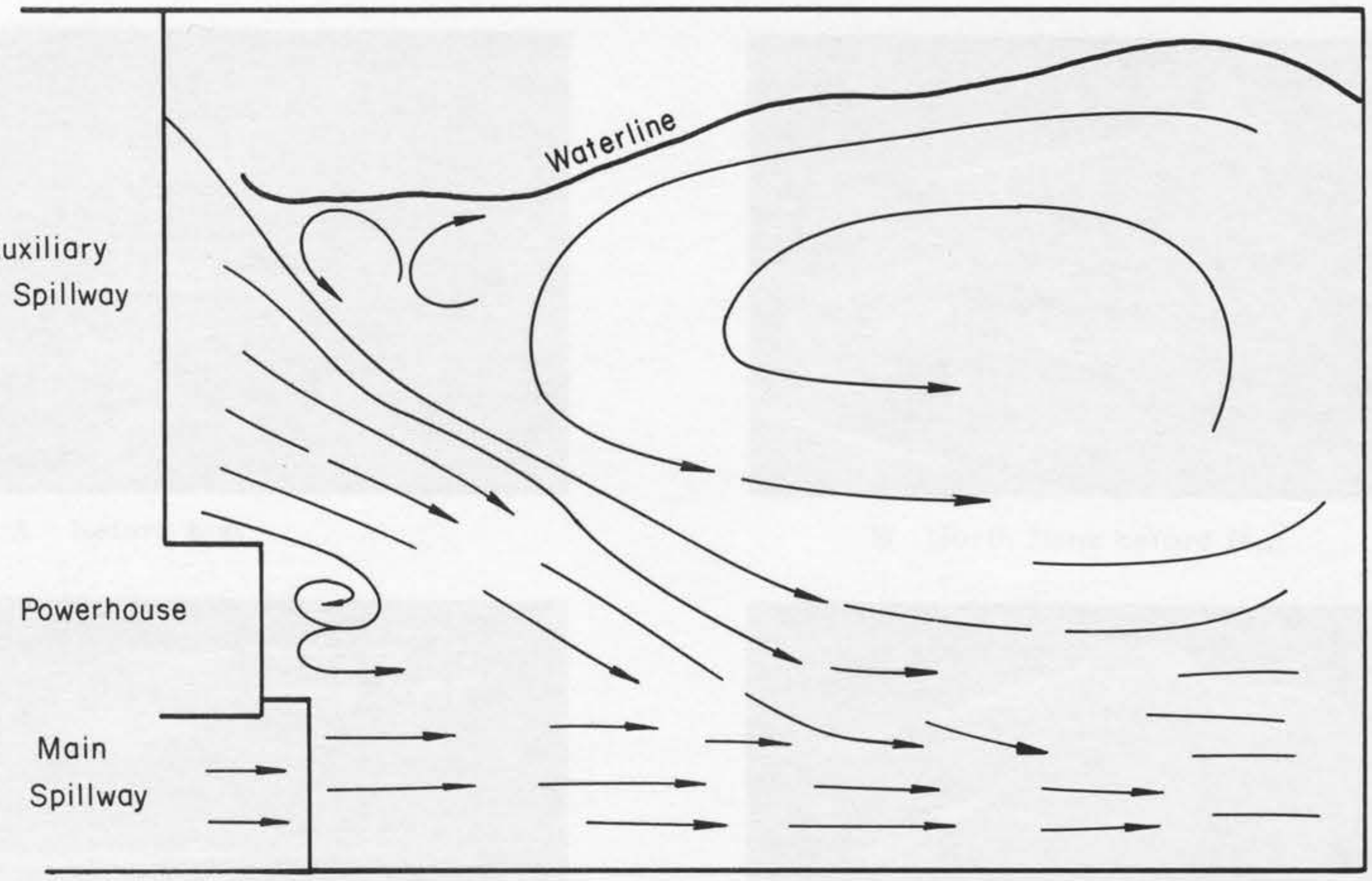
REMMEL DAM SPILLWAY STUDY
1:42 Scale Model
Flood Capacity at Dam Spillway
Gates at Various Elevations

A Top section of trashway bay open

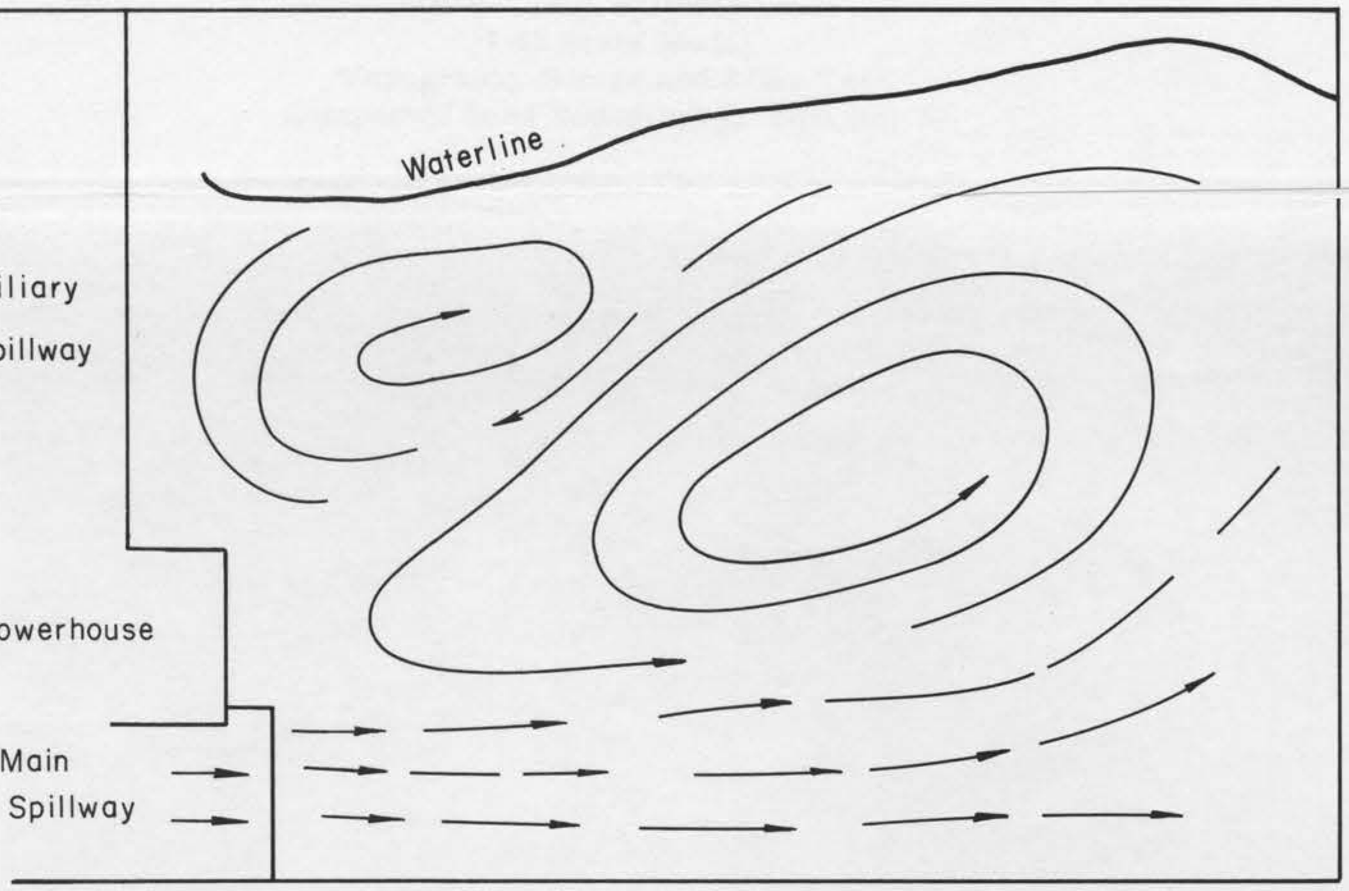


B Top of trashway bay closed

Remmel Dam Spillway Study
1:42 Scale Model
Flow Contitions Downstream from
Spillway Gate No. 1 With and Without
Top of Trashway Bay Closed



A. MAIN AND AUXILIARY SPILLWAY DISCHARGING



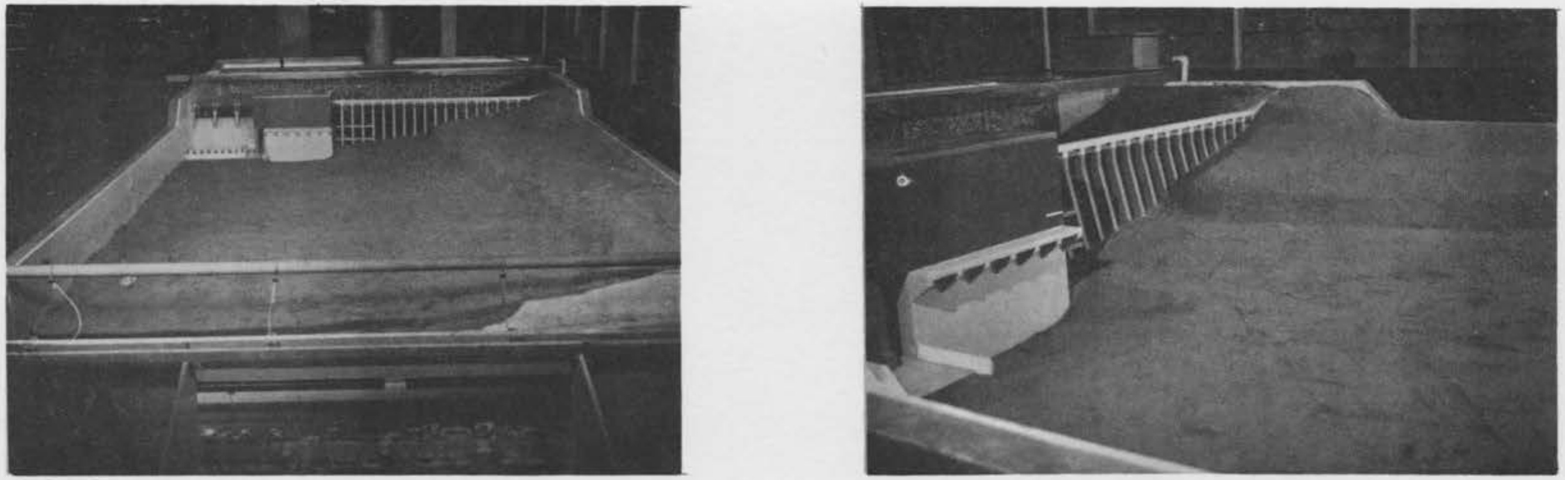
B. MAIN SPILLWAY ONLY DISCHARGING

Scale 1 inch = 120' Prototype

REMMEL DAM SPILLWAY STUDY
1:42 Scale Model
Flow Currents Downstream
From Dam

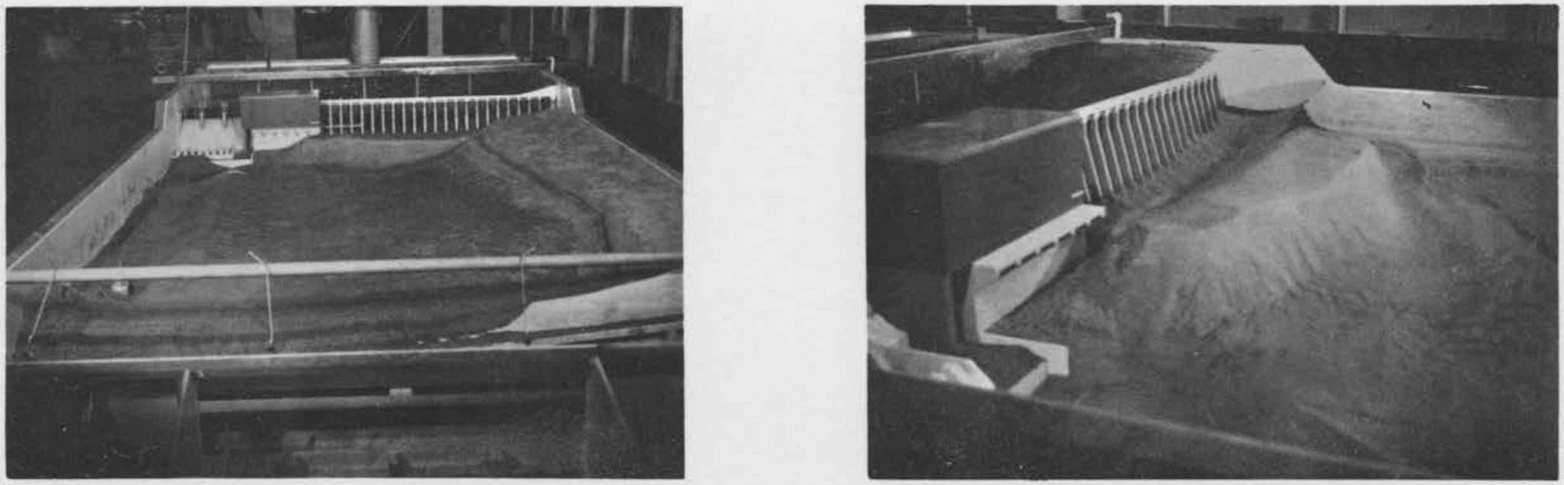
FIG. 13

Figure 14



A Before test

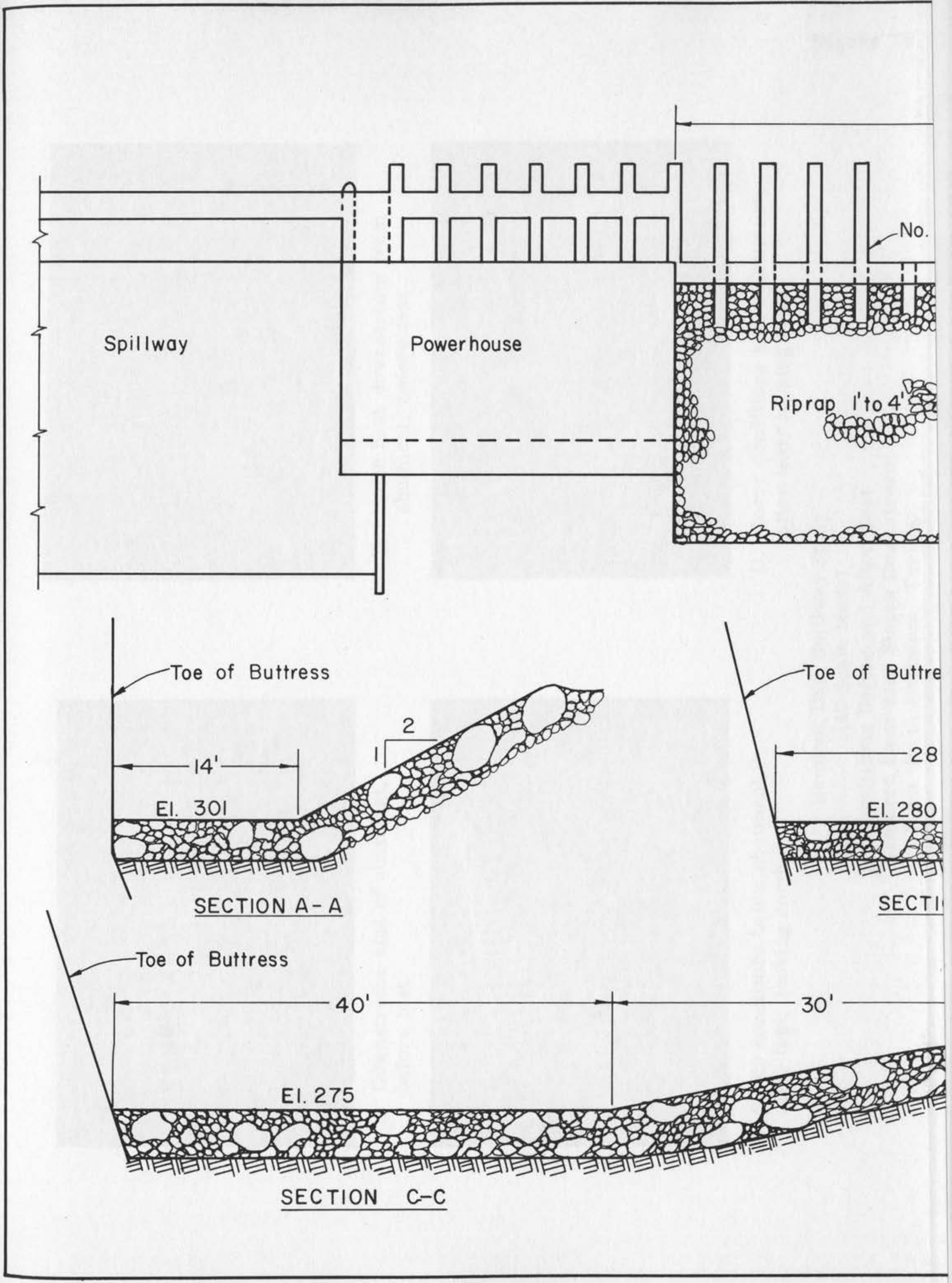
B North Bank before test



C Scour Conditions at end of test

D Scour Conditions Downstream from North Abutment

Remmel Dam Spillway Study
1:42 Scale Model
Topography Before and After Test
Compacted Sand Topography, Test No. 4

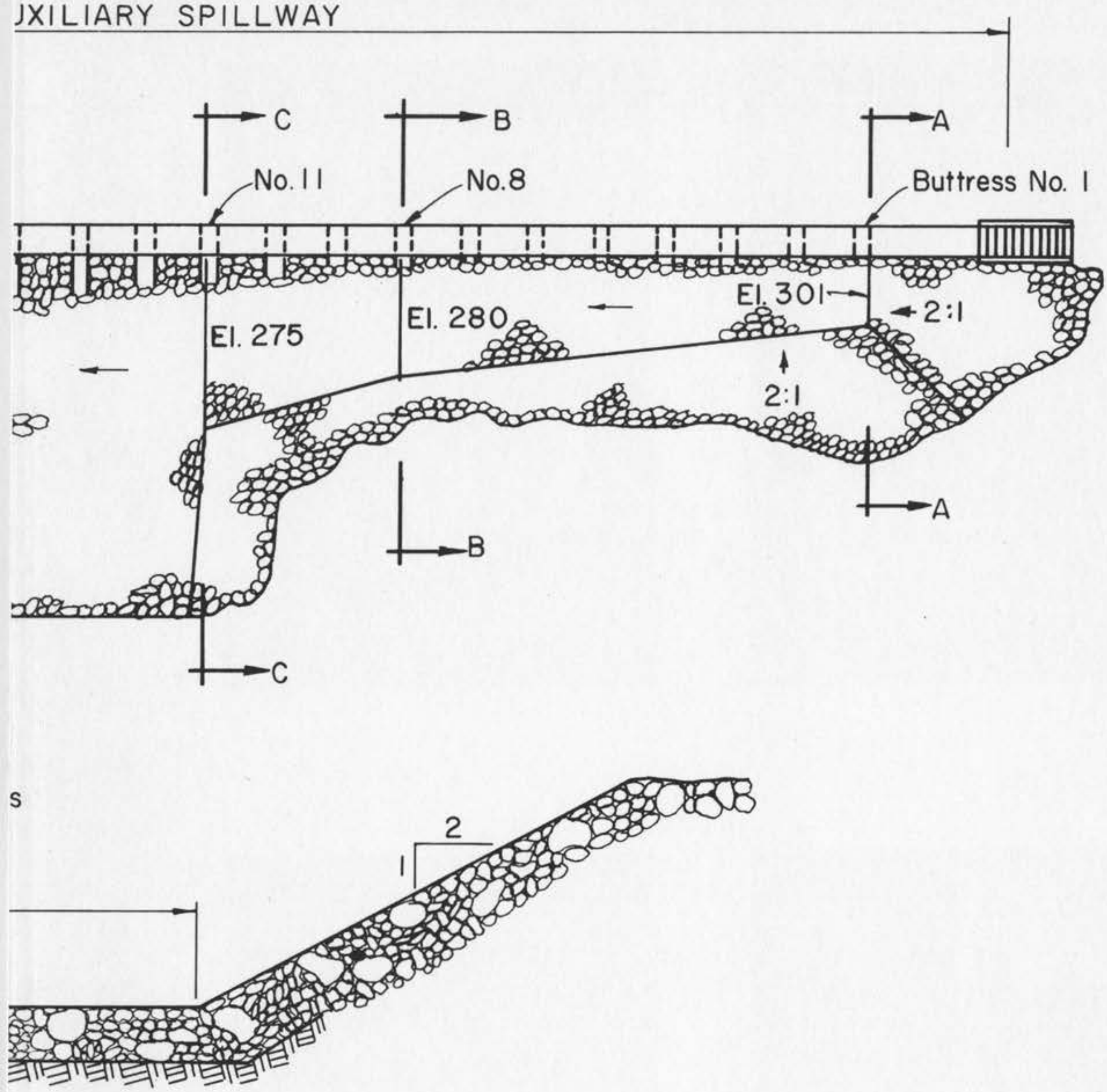


SECTION A-A

SECTI

SECTION C-C

FIG. 15

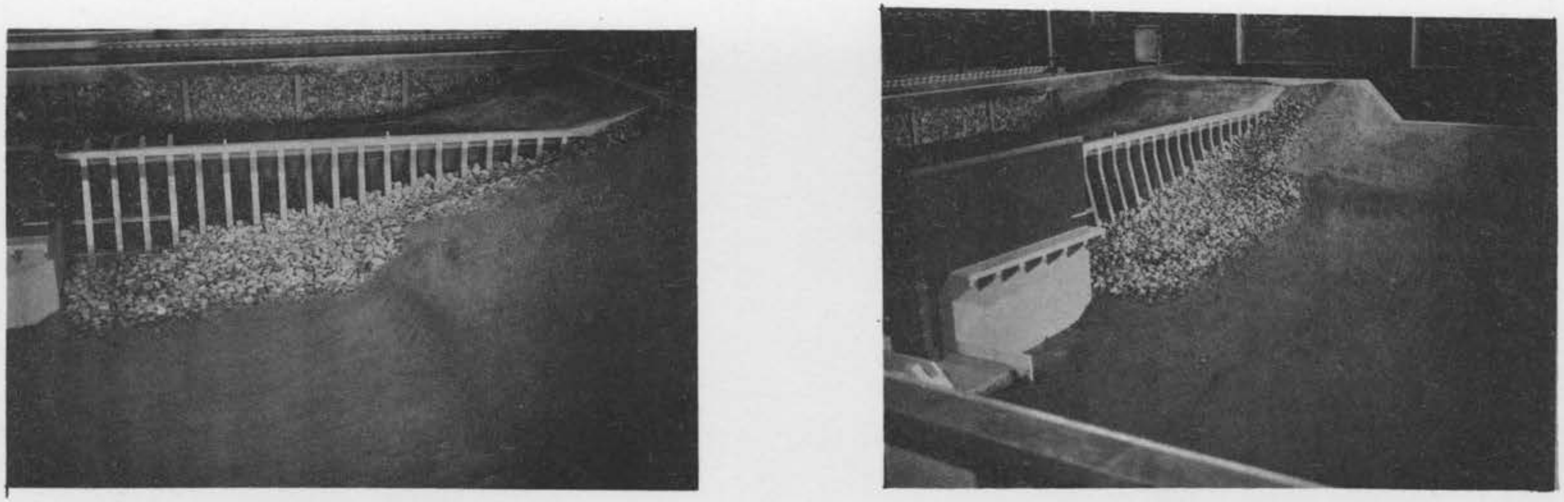


REMMEL DAM SPILLWAY STUDY

1:42 Scale Model

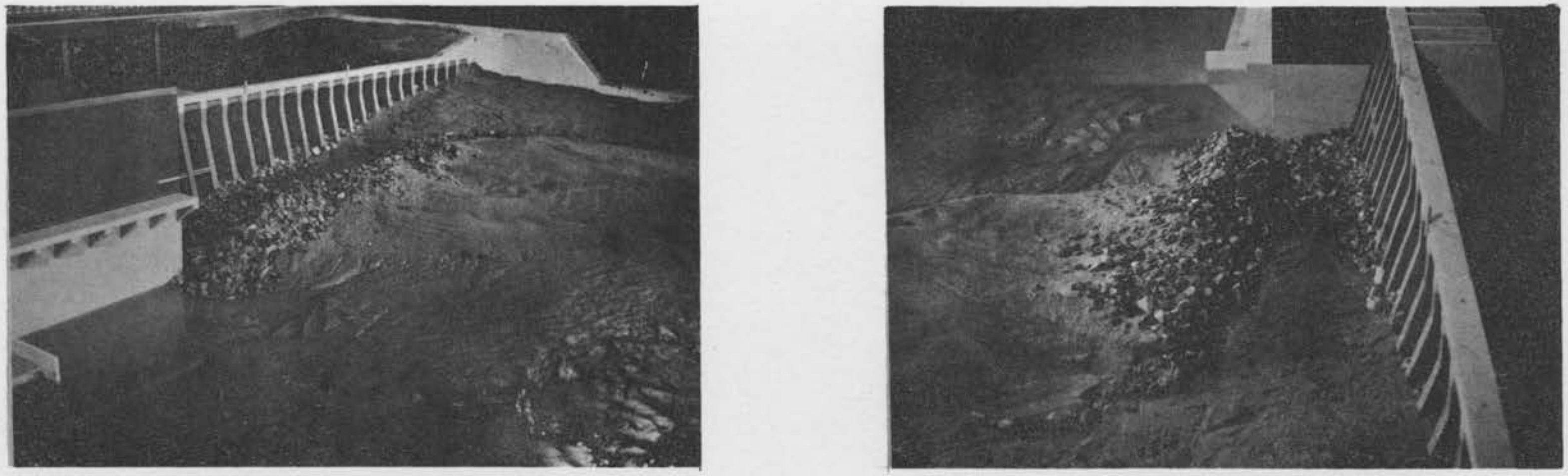
Plunge Pool Formed by Excavated Slopes Armored With Riprap-Auxiliary Spillway

Test No. 5



A Downstream side of abutment, before test

B. North bank downstream from abutment, before test



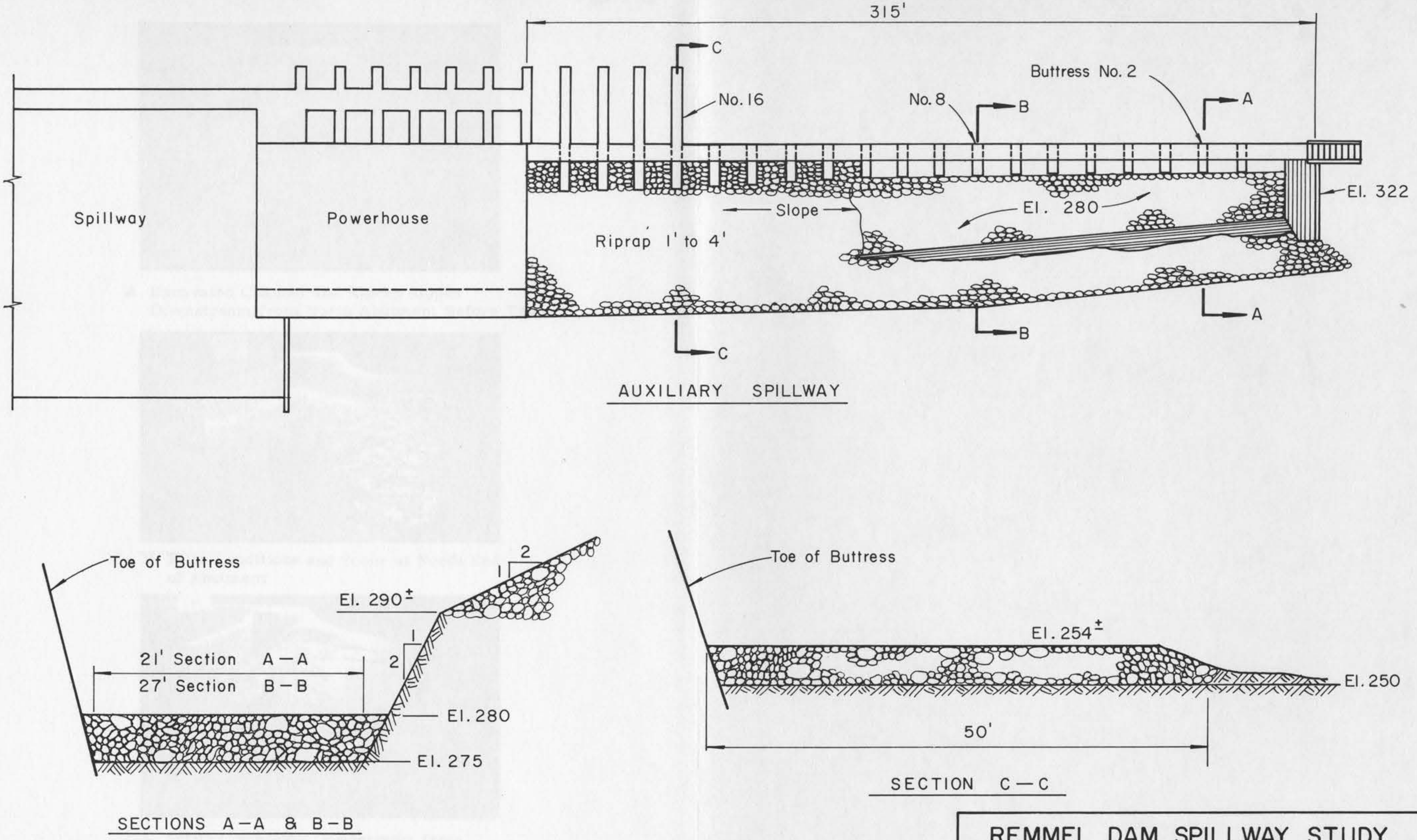
C Scour conditions below abutment, after test, looking north

D. Scour conditions below abutment, after test, looking south

Remmel Dam Spillway Study
1:42 Scale Model
Conditions Before and After Test
Armored Excavated Slopes Downstream
from North Abutment, Test No. 5

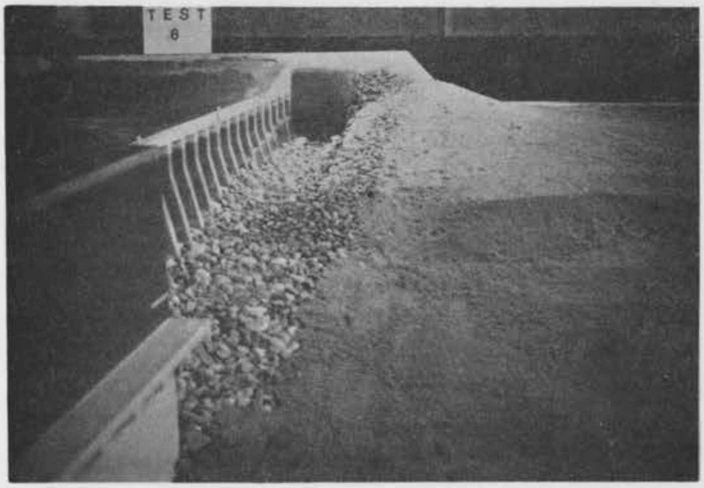
Figure 16

FIG. 17

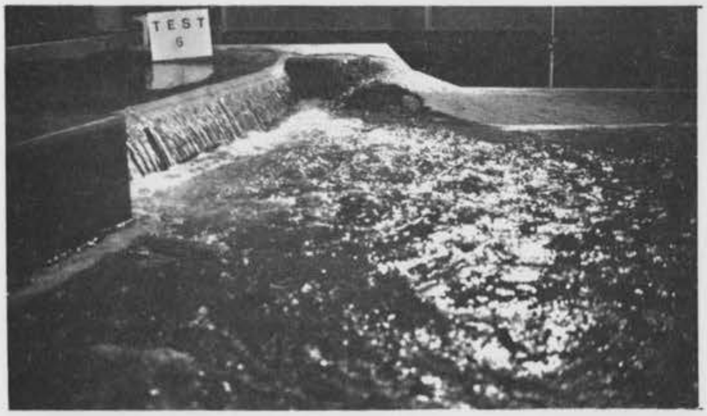


REMMEL DAM SPILLWAY STUDY
1:42 Scale Model
Details of Plunge Pool With Excavated Channel and Riprapped Slopes— Auxiliary Spillway. Test No. 6

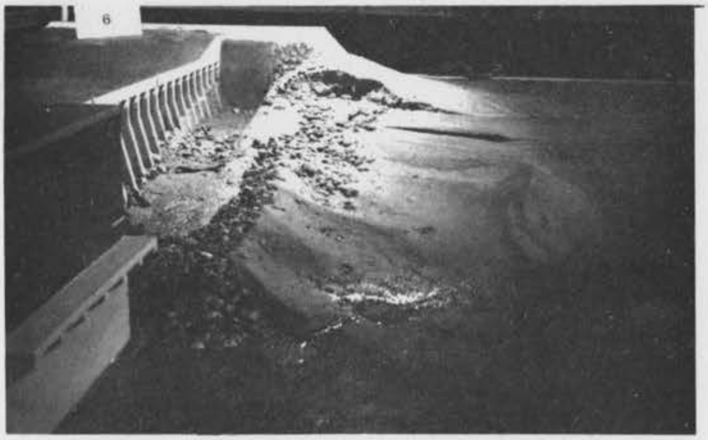
Figure 18



A Excavated Channel and Riprap Slopes Downstream from North Abutment Before Test



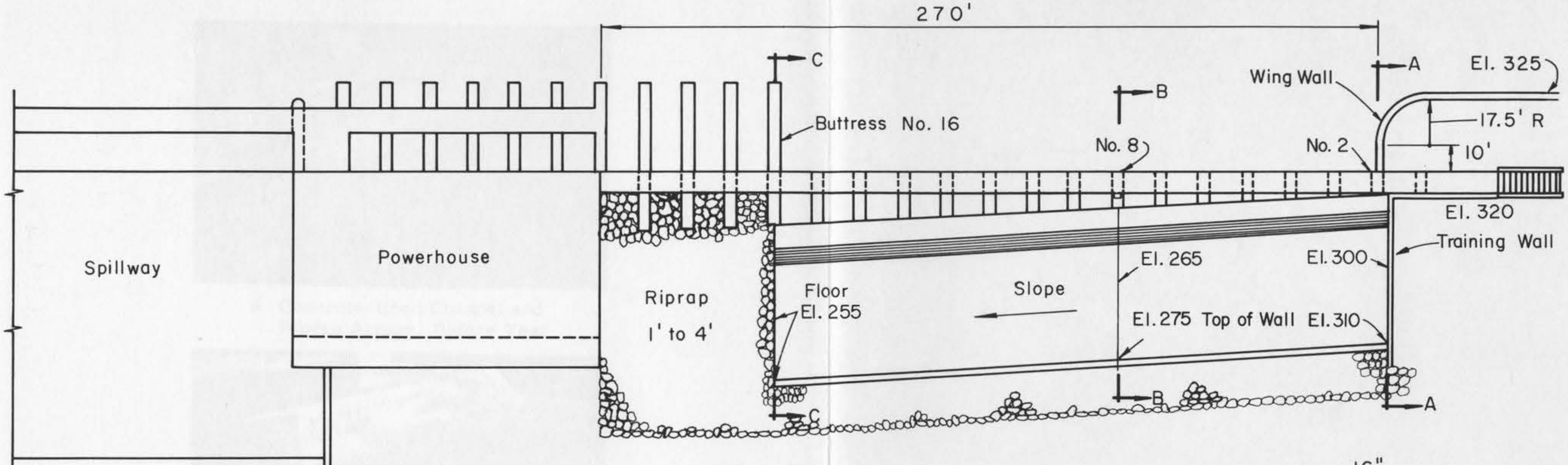
B Flow Conditions and Scour at North End of Abutment



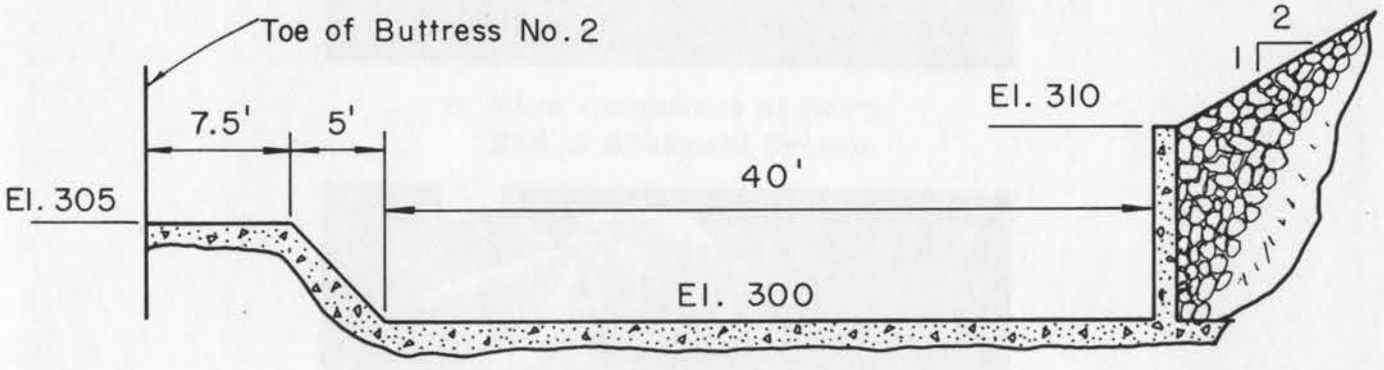
C Scour Conditions Downstream from Abutment

Remmel Dam Spillway Study
1:42 Scale Model
Conditions Before and After Test - Rock Excavation
and Riprap Slopes Downstream from
North Abutment, Test No. 6

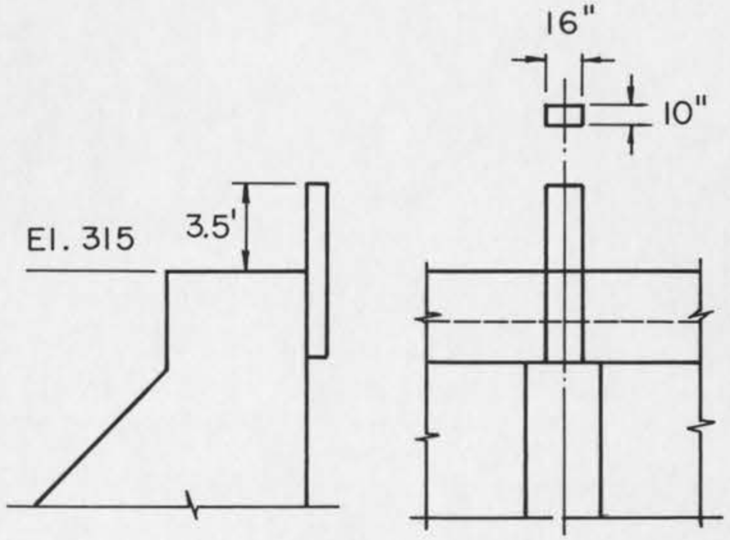
FIG. 19



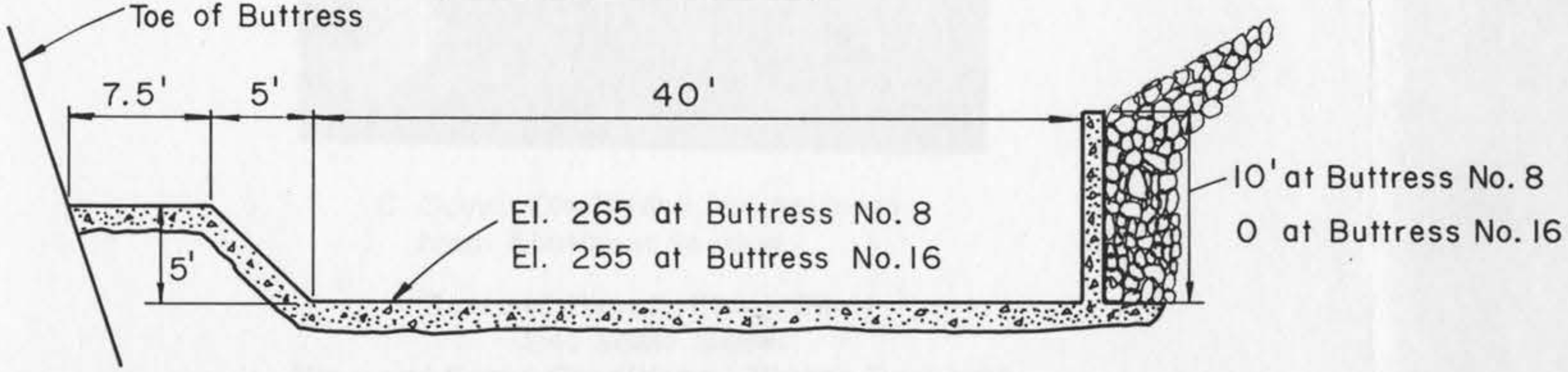
AUXILIARY SPILLWAY



SECTION A-A



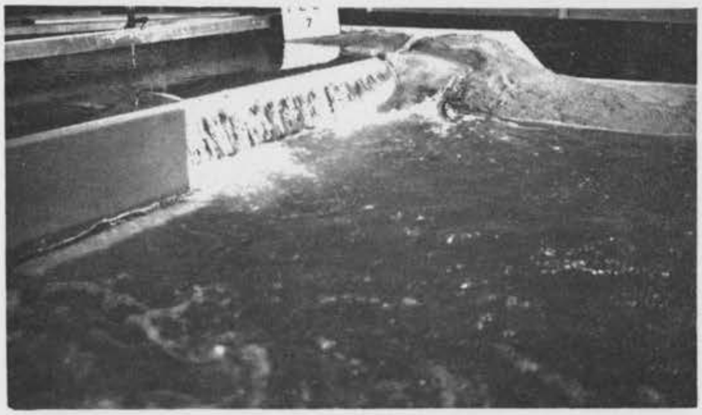
AERATION PIERS
BUTTRESSES No's. 8 & 14



SECTIONS B-B & C-C

REMMEL DAM SPILLWAY STUDY
1:42 Scale Model
Details of Plunge Pool and
Protection - Auxiliary Spillway
Test No. 7

A Concrete-lined Channel and
Riprap Armor, Before Test

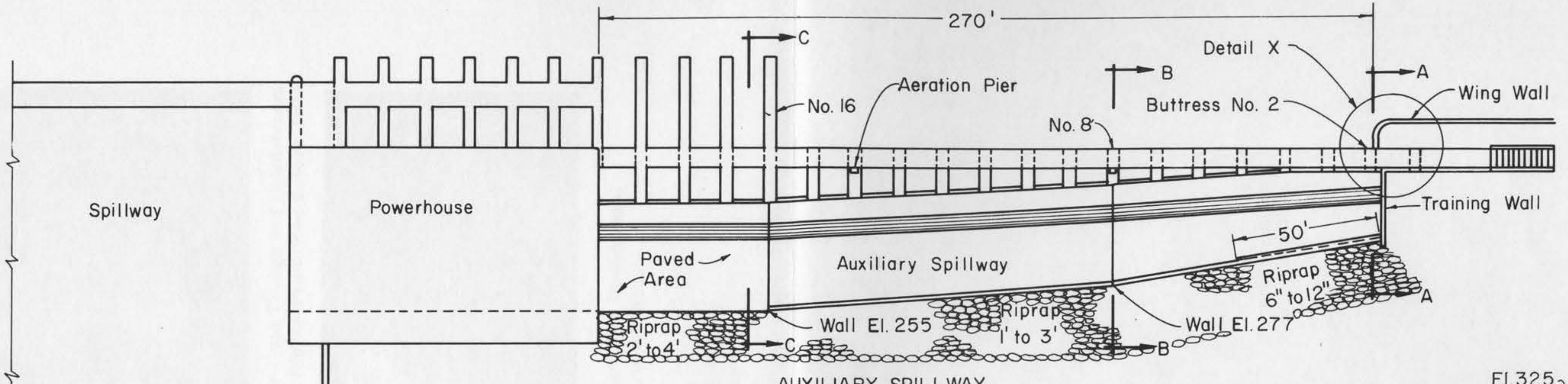


B Flow Conditions at North
End of Abutment Section

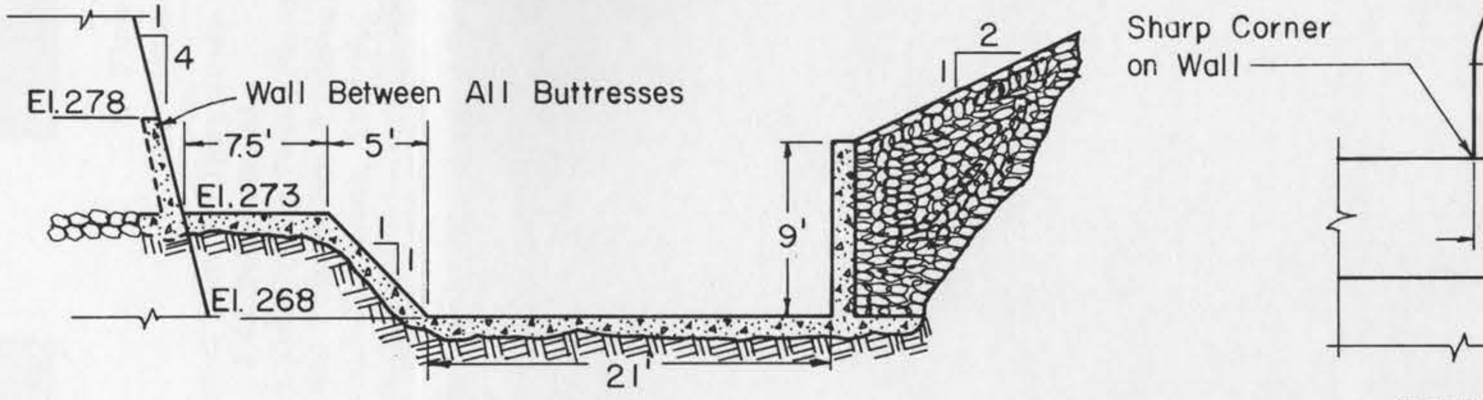
C Scour Conditions Downstream
from Abutment Section

Remmel Dam Spillway Study
1:42 Scale Model
Flow and Scour Conditions, Plunge Pool with
Concrete-lined Trough No. 1 and
Riprap Protection, Test No. 7

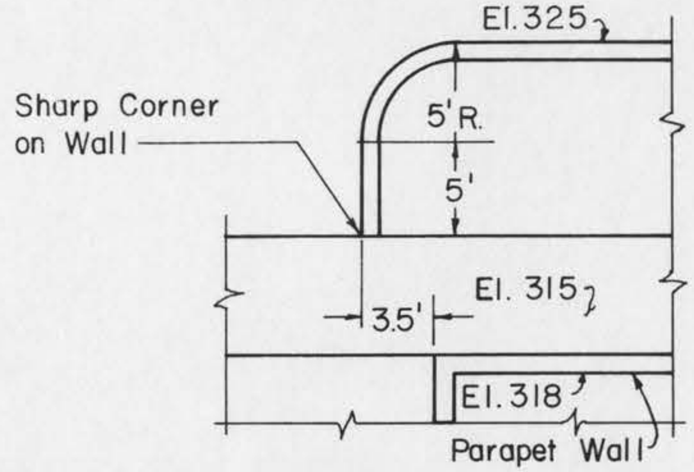
FIG. 21



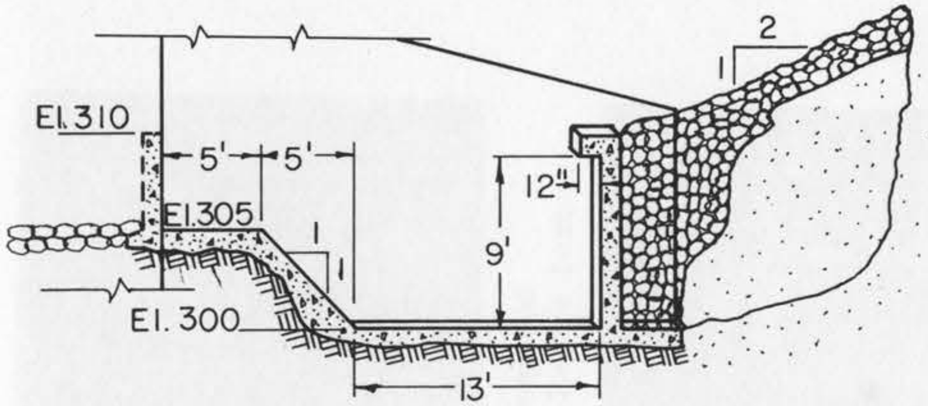
AUXILIARY SPILLWAY



SECTION B-B

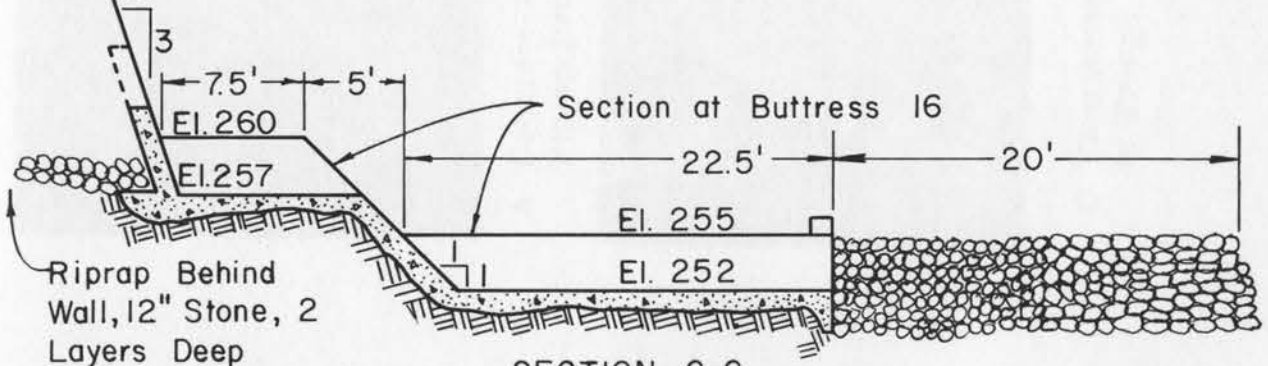


DETAIL X



SECTION A-A

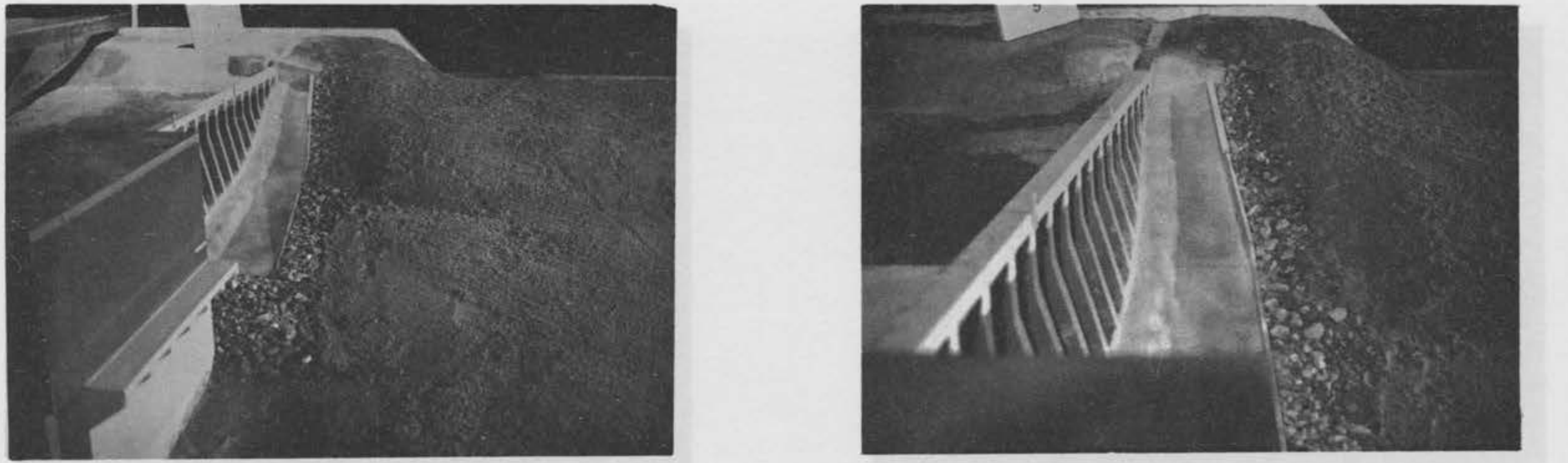
NOTE: Area between buttresses 16 and 20, armored with riprap for test No. 9, was paved for test No. 10



SECTION C-C

REMMEL DAM SPILLWAY STUDY
1:42 Scale Model
Recommended Plunge Pool And
Protection- Auxiliary Spillway
Test No. 10

Figure 22



A Concrete-lined Trough and Riprap Protection, Before Test

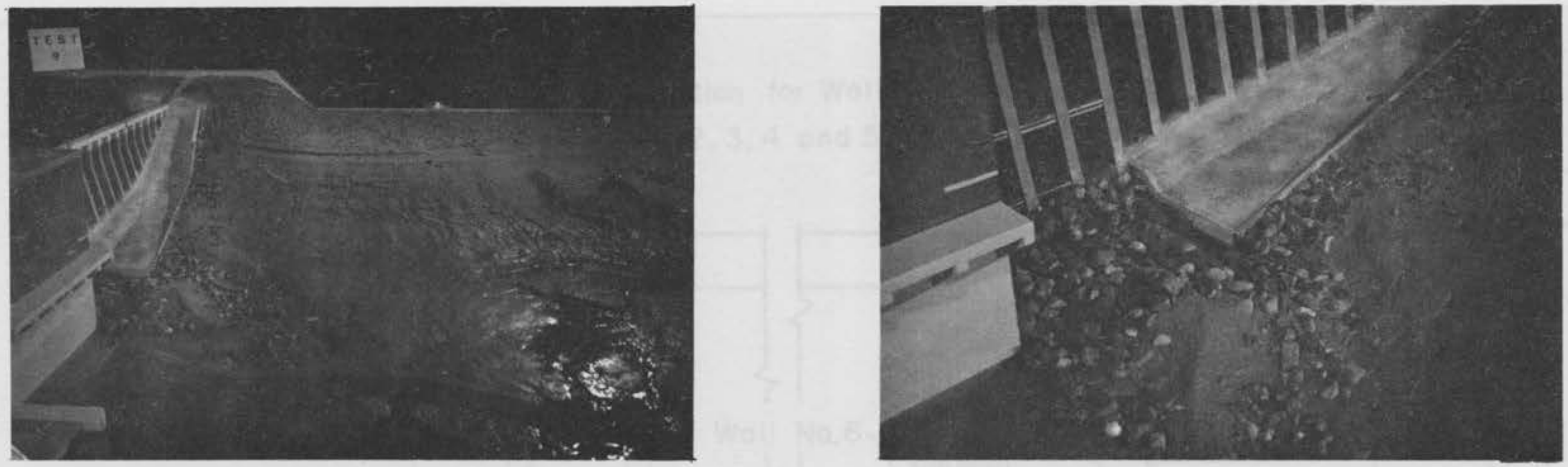
B Concrete-lined Trough and Downstream Wall and Riprap Protection Before Test

C Flow Action at North End of Trough

D Flow Conditions Downstream from North Abutment

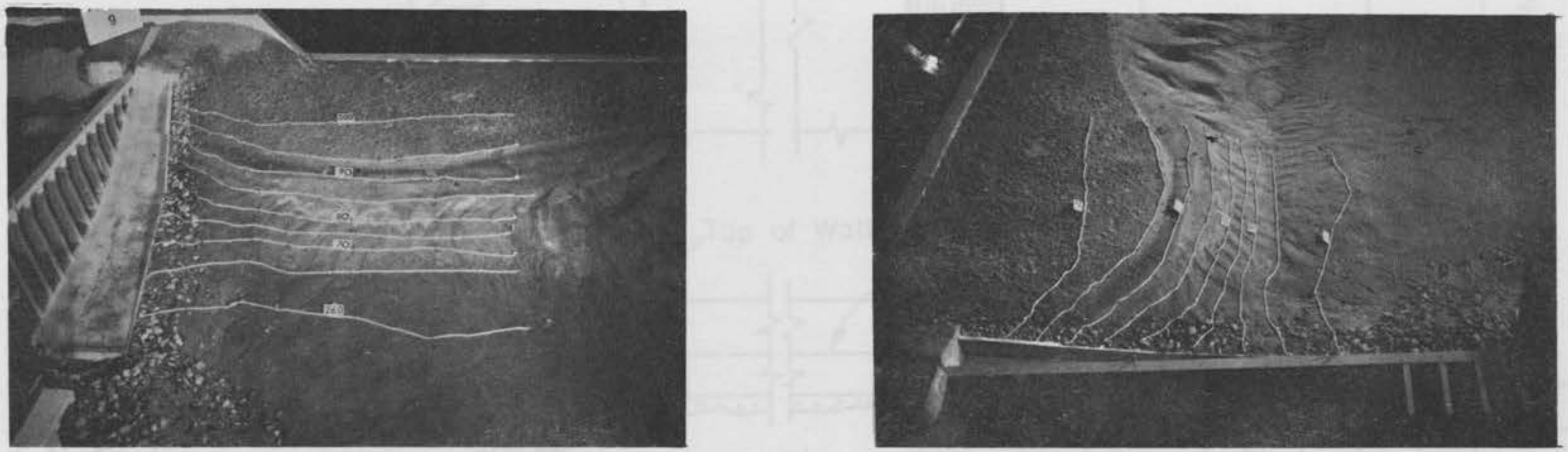
Remmel Dam Spillway Study
1:42 Scale Model
Conditions Before and During Test, Plunge Pool with
Concrete-lined Trough No. 2 Downstream
from North Abutment, Test No. 9

Figure 23



A Scour Downstream from Concrete-lined Trough

B Scour at South End of Trough

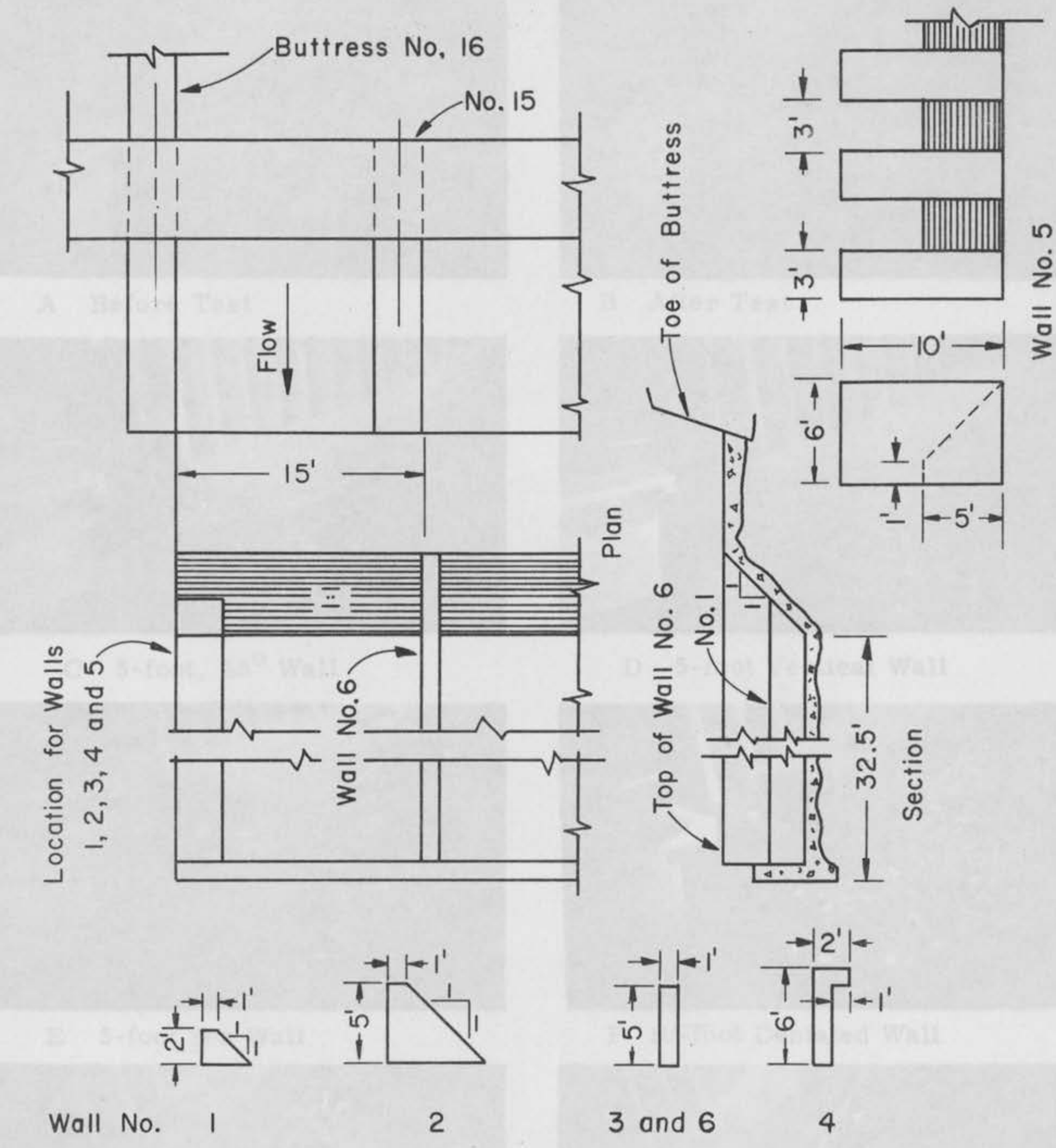


C Contour Lines Showing Condition at End of Test

D Contour Lines Showing Condition at End of Test

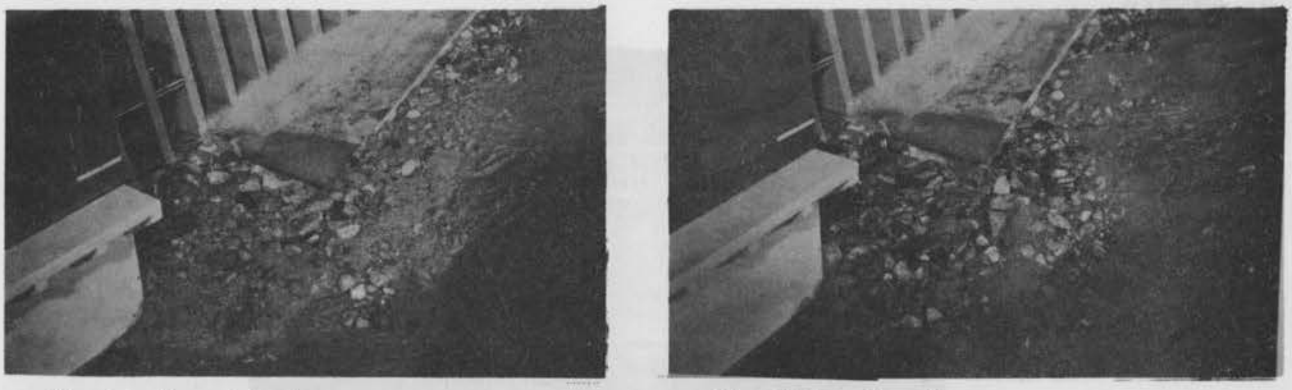
Remmel Dam Spillway Study
1:42 Scale Model
Scour Conditions After Test of 1 Hour (Model) with Reservoir at El. 322, Plunge Pool with Concrete-lined Trough and Sea Wall at North End, Test 9

Fig. 24

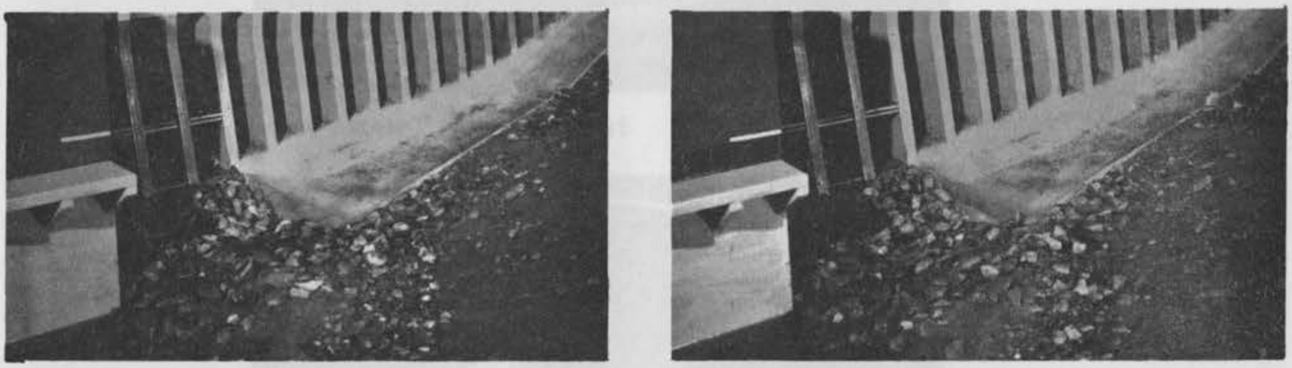


REMMEL DAM SPILLWAY STUDY
1:42 Scale Model
Details of Cross Walls Tested
at South End of Trough

Figure 25

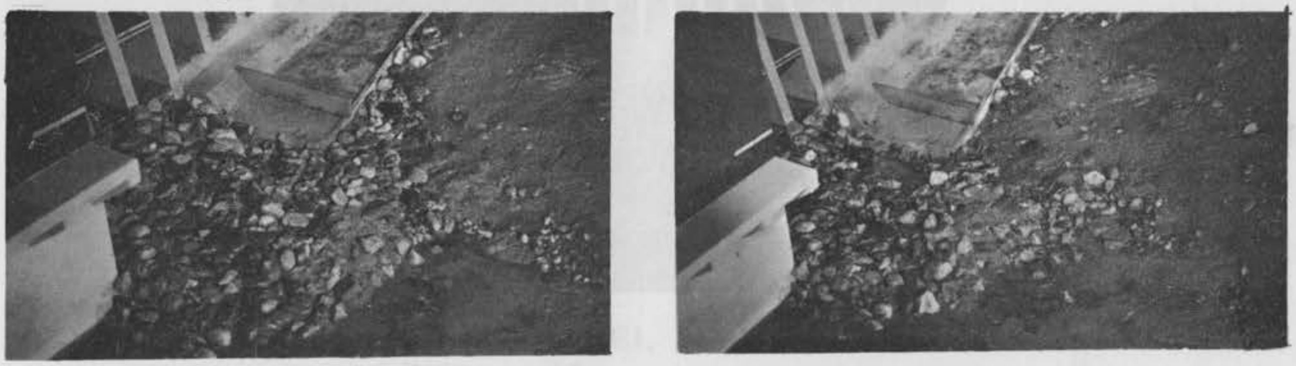


A Before Test B After Test



C 5-foot, 45° Wall D 5-foot Vertical Wall

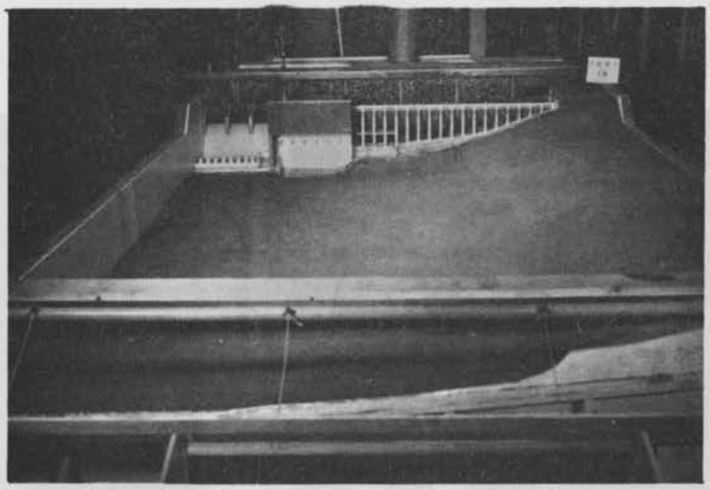
E 5-foot Sea Wall F 10-foot Dentated Wall



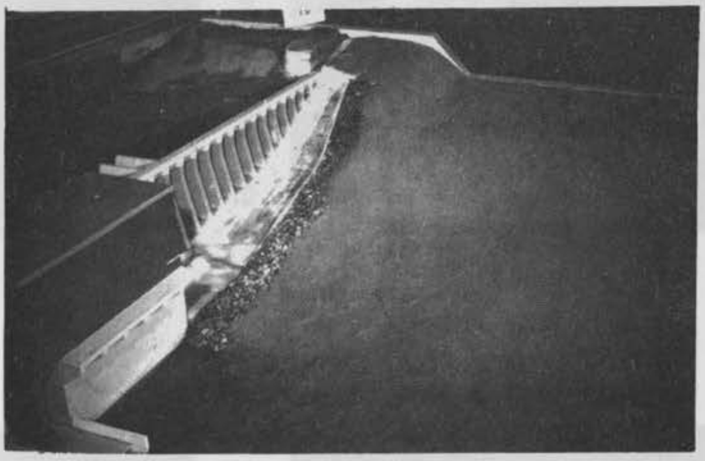
G Reservoir El. 322 H Reservoir El. 323

5-foot Vertical 15 feet from end of Trough
Remmel Dam Spillway Study
1:42 Scale Model
Scour Conditions at South end of Trough
With and Without Various Walls at South End of Trough, Test No. 9

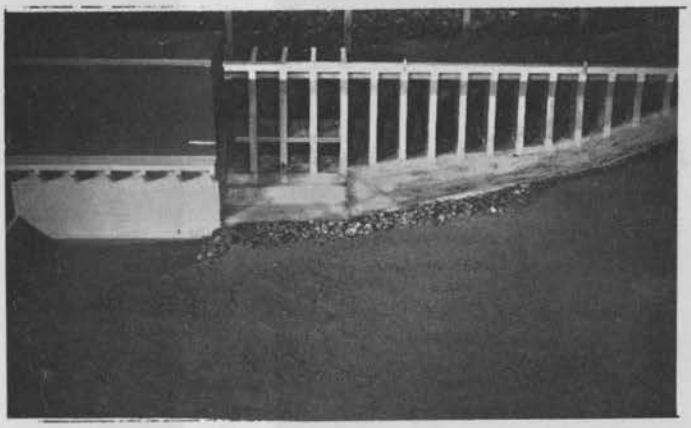
Figure 26



A Model Before Test



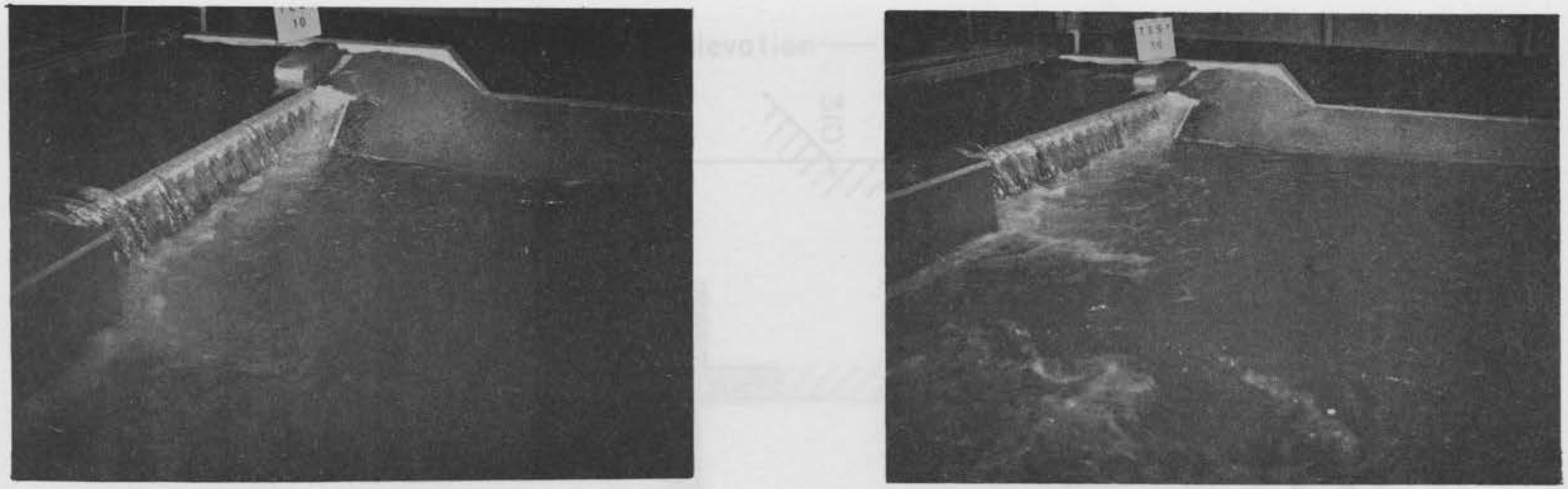
B Trough, Pavement and Riprap Protection
Downstream from North Abutment



C Paved Section at El. 252 between Powerhouse
and Buttress 16

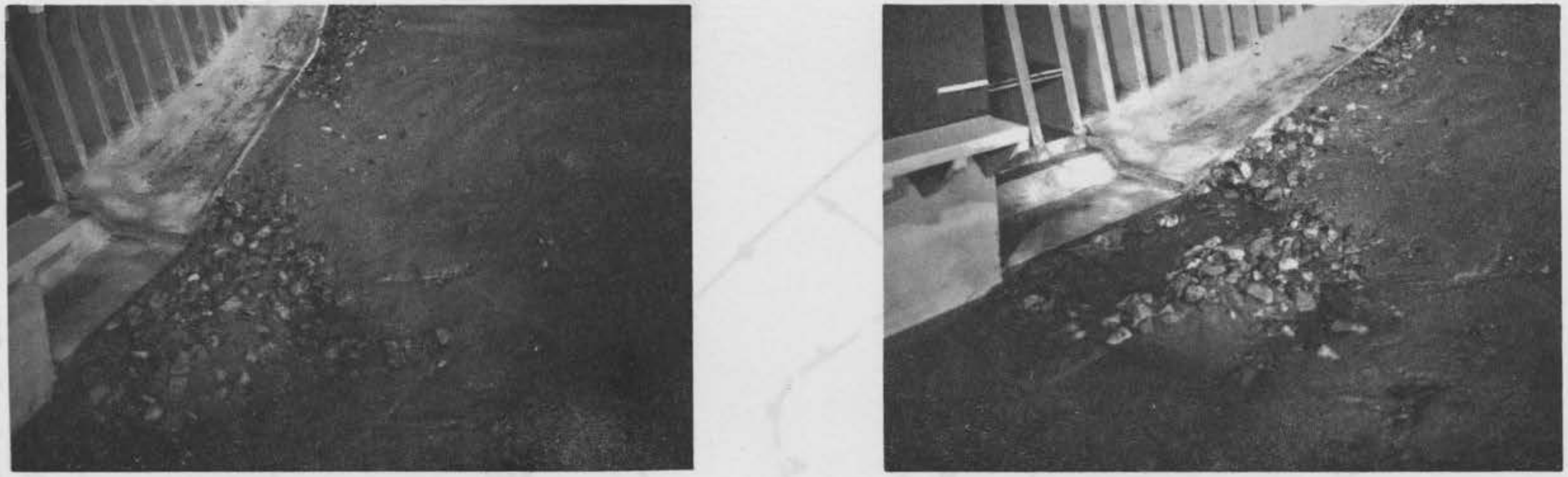
Remmel Dam Spillway Study
1:42 Scale Model
Recommended Plunge Pool with Concrete-lined Trough,
Paved Area, and Riprap Protection Downstream
from North Abutment, Test No. 10

Figure 27



A Reservoir El. 322

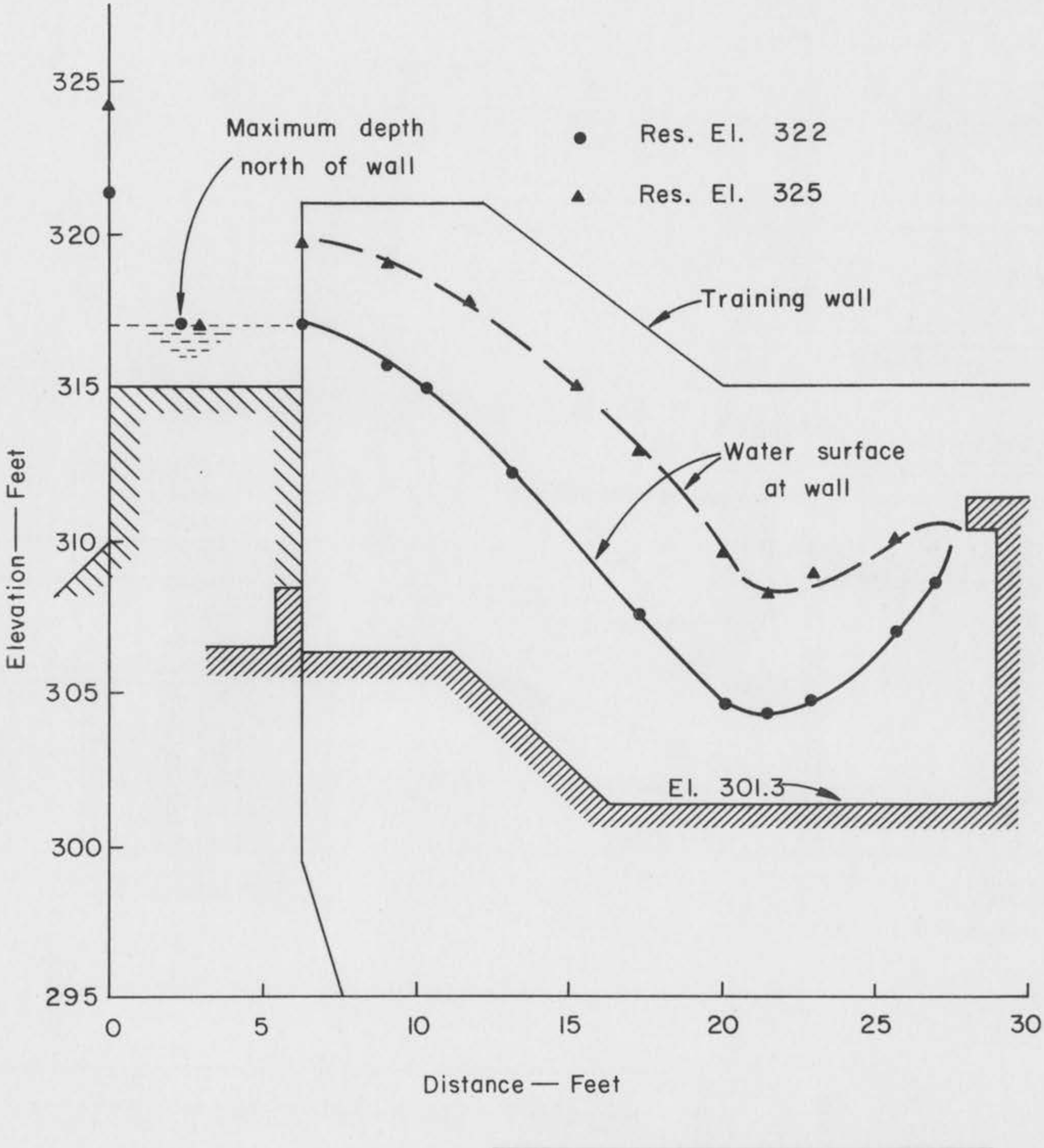
B Reservoir El. 325



C After 1 hr. at Res. El. 322

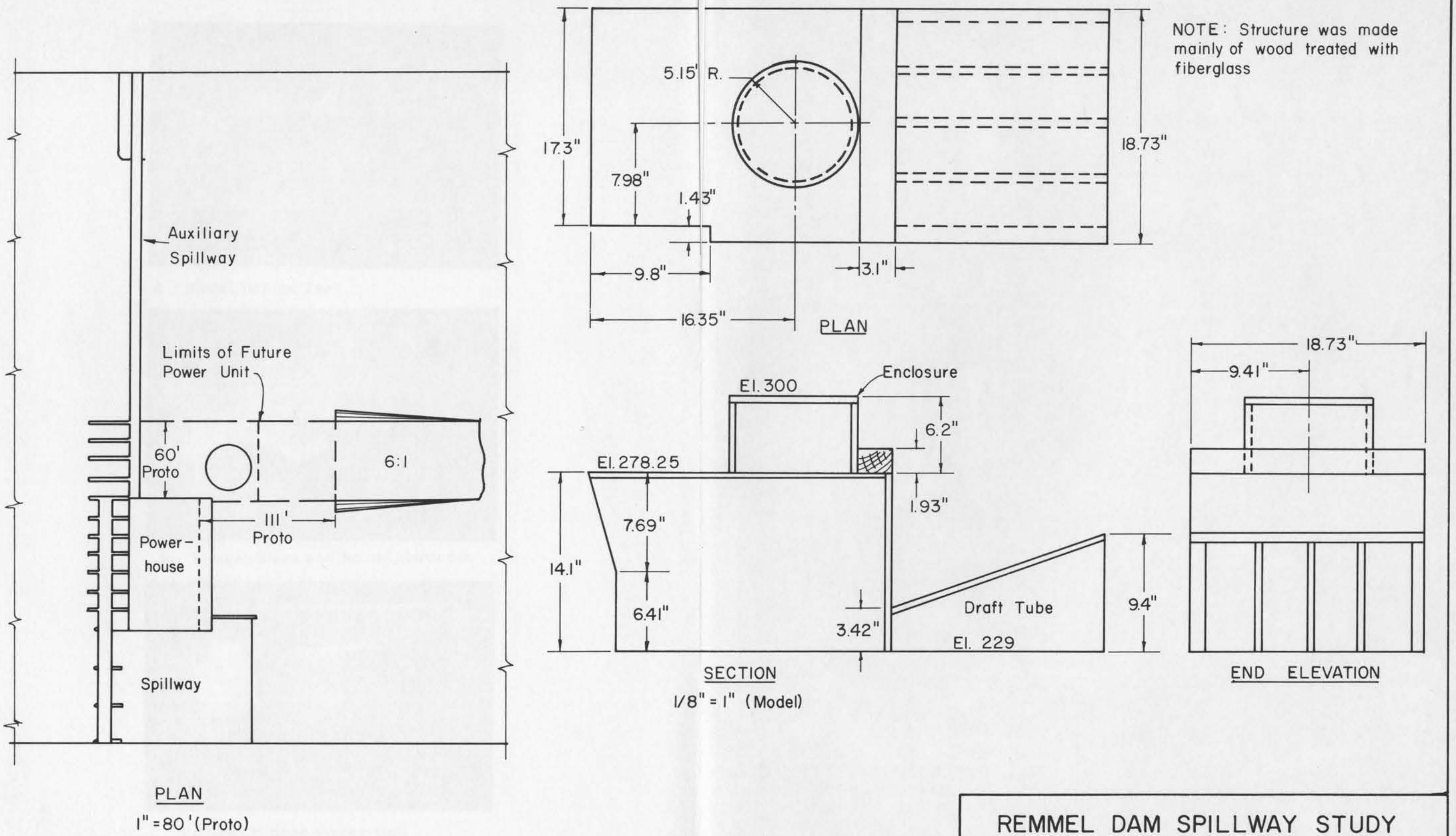
D After Additional 1/2 hr. at Res. El. 325

Remmel Dam Spillway Study
1:42 Scale Model
Flow and Scour Conditions
Recommended Plunge Pool Downstream
from North Abutment, Test No. 10



REMMEL DAM SPILLWAY STUDY
1:42 Scale Model
Water Surface Profile at End
Wall Behind Buttress No. 2

FIG. 29



PLAN
1" = 80' (Proto)

PLAN

SECTION
1/8" = 1" (Model)

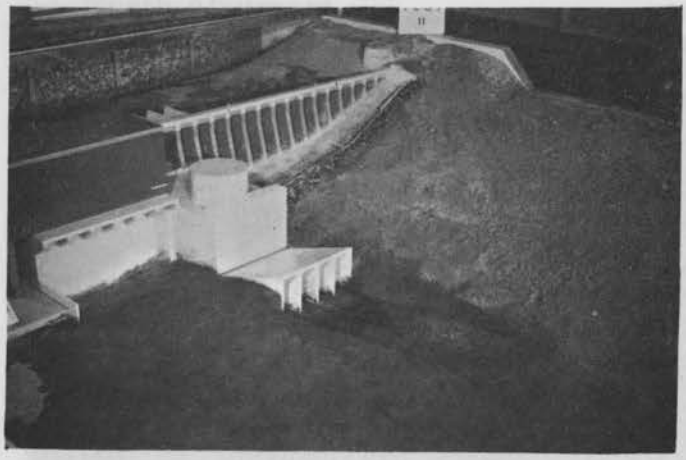
END ELEVATION

NOTE: Structure was made mainly of wood treated with fiberglass

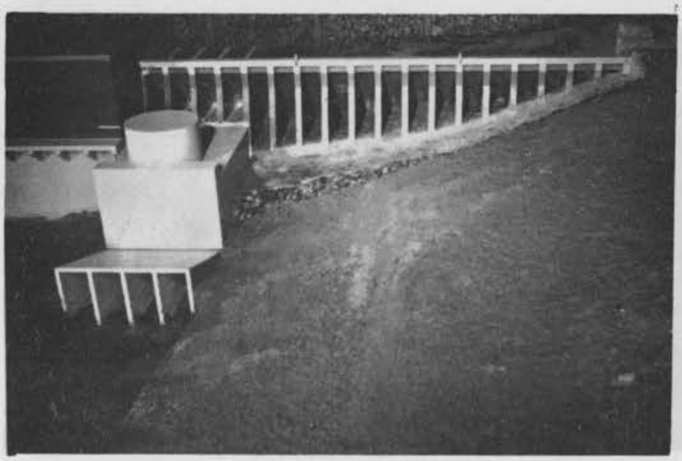
REMMEL DAM SPILLWAY STUDY
1:42 Scale Model
Details of Future Outdoor Power
Unit Enclosure
Test No. 11

Figure 30

A Model Before Test



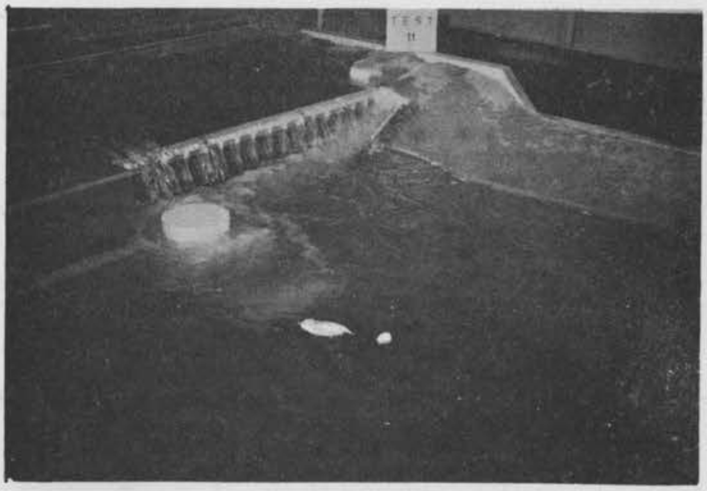
B Powerhouses and North Abutment



C Future Outdoor Power Unit

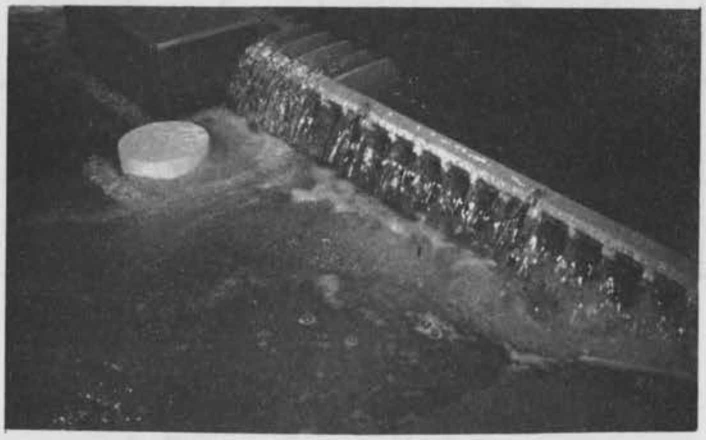
Remmel Dam Spillway Study
1:42 Scale Model
Recommended Auxiliary Spillway Plunge Pool with
Future Outdoor Power Unit Enclosure in Place
Test No. 11

Figure 31



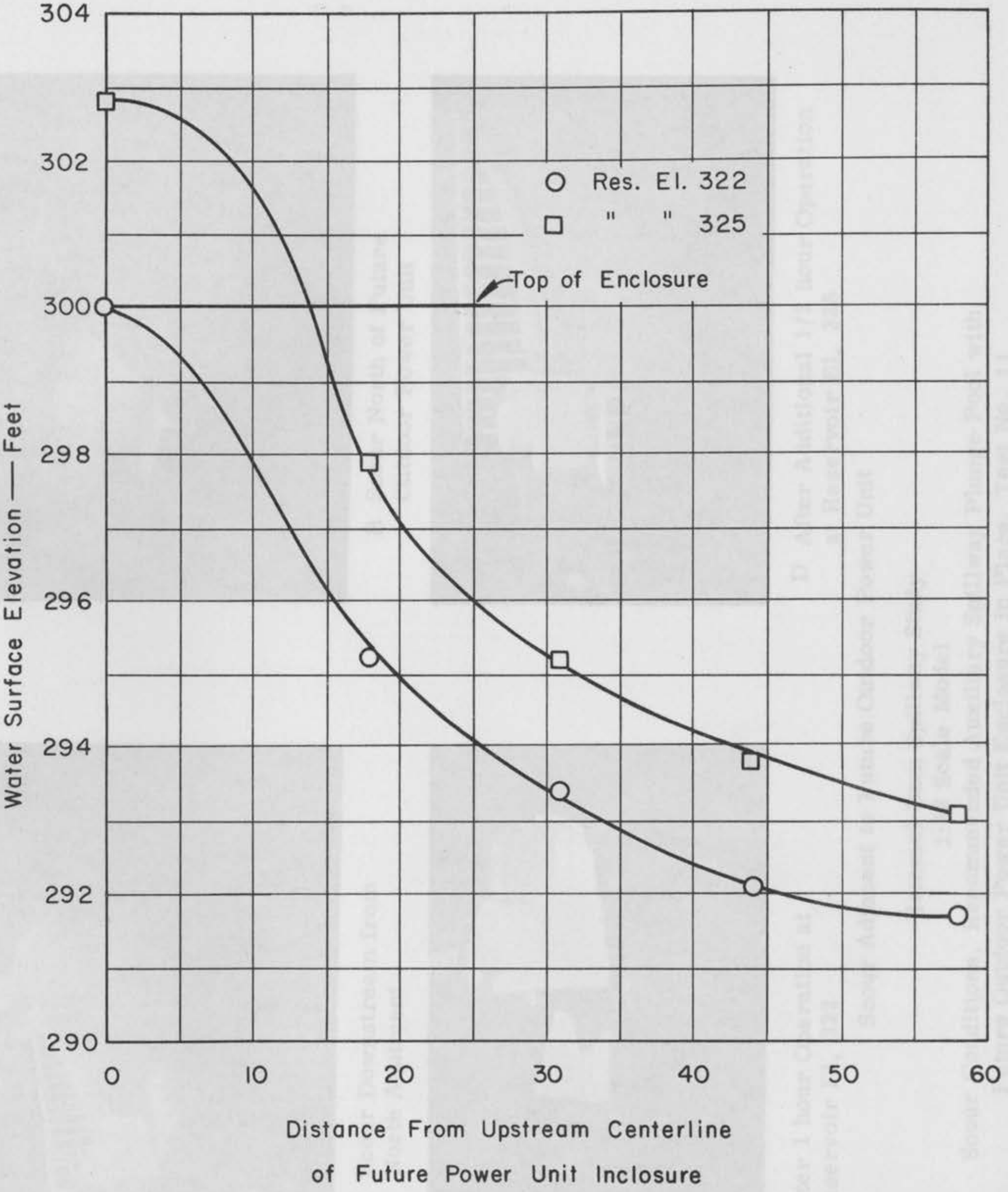
A Flow Conditions Downstream from North Abutment

B Flow Conditions at Future Power Unit



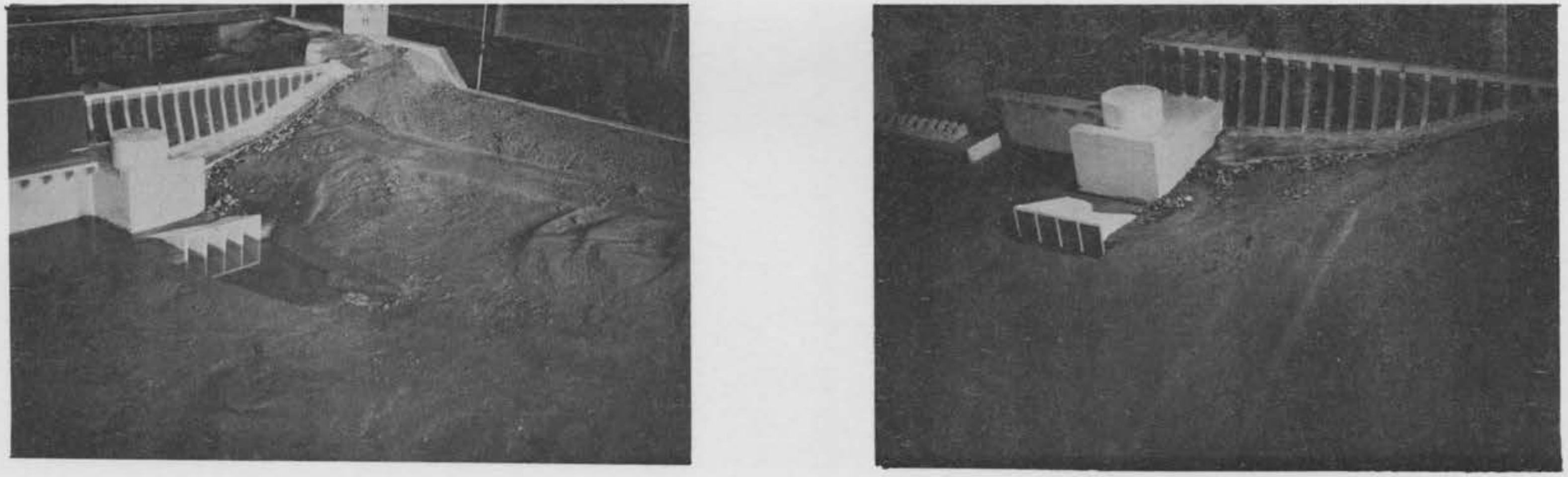
C Flow Conditions at Base of North Abutment

Remmel Dam Spillway Study
1:42 Scale Model
Flow Conditions Downstream from
North Abutment with Recommended Plunge Pool and
Future Outdoor Power Unit Enclosure in Place, Res. El. 322



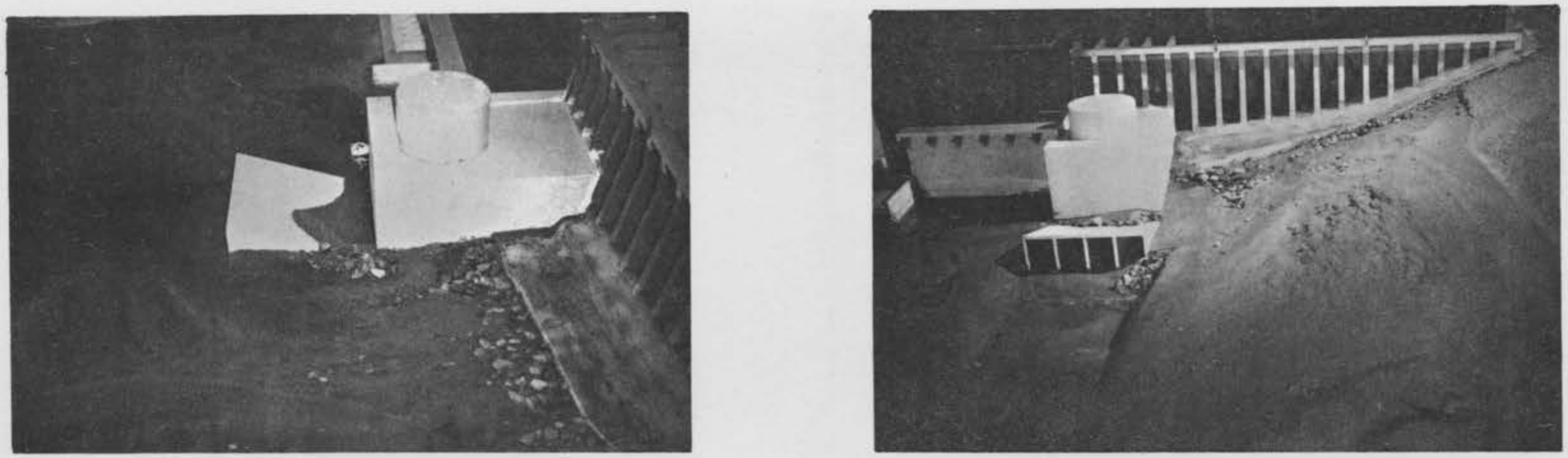
REMMEL DAM SPILLWAY STUDY
1:42 Scale Model
Water Surface Elevations On
Future Unit Enclosure

Figure 33



A Scour Downstream from North Abutment

B Scour North of Future Outdoor Power Unit



C After 1 hour Operation at Reservoir El. 322

D After Additional 1/2 hour Operation at Reservoir El. 325

Scour Adjacent to Future Outdoor Power Unit

Remmel Dam Spillway Study
1:42 Scale Model
Scour Conditions, Recommended Auxiliary Spillway Plunge Pool with Future Outdoor Power Unit Enclosure in Place, Test No. 11