UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

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HYDRAULIC LABORATORY REPORT NO. 136

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HYDRAULIC MODEL STUDIES FOR THE DESIGN OF ROSS DAM SPILLWAYS CITY OF SEATTLE, WASHINGTON

By

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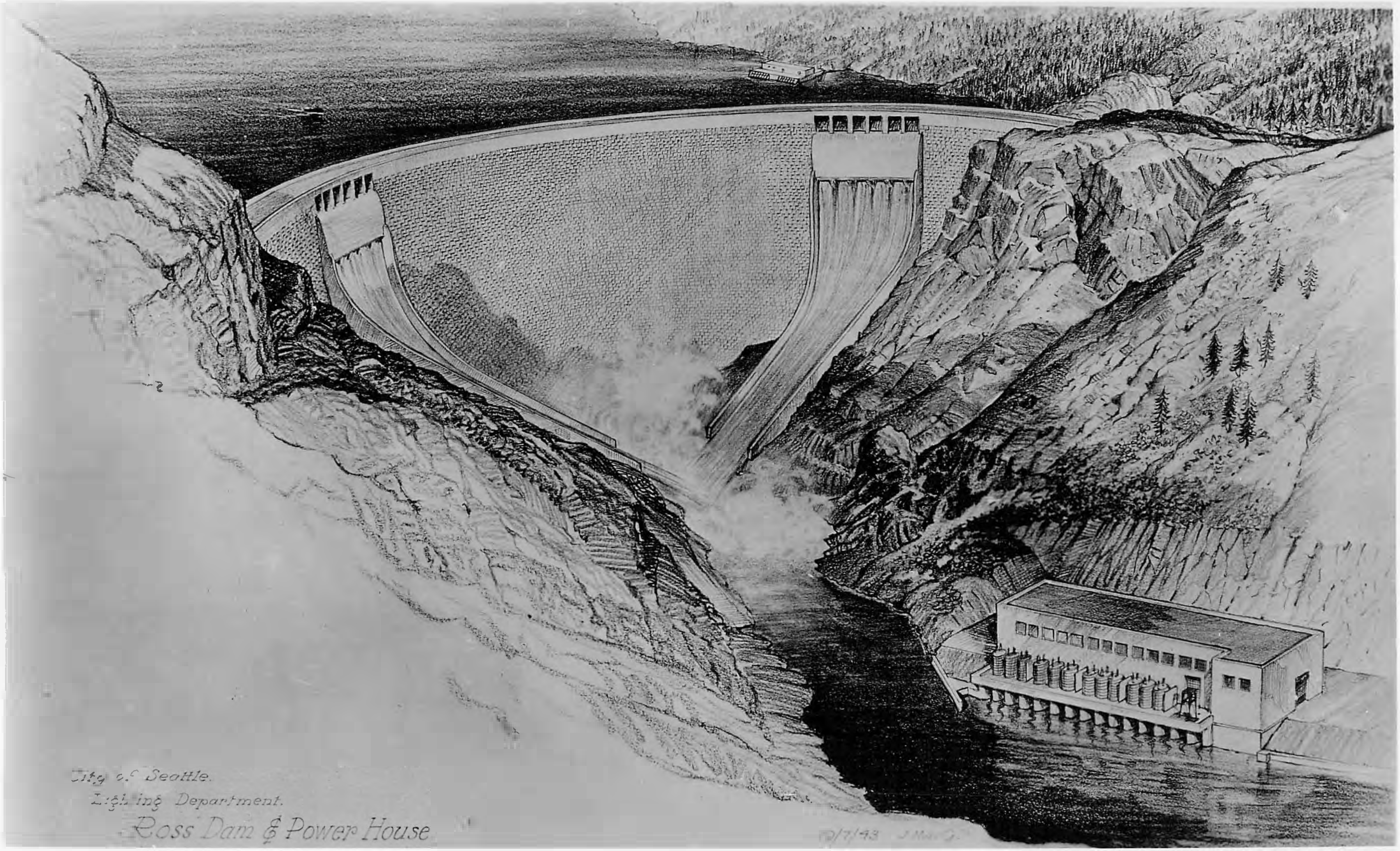
- - -

Denver, Colorado,
Oct. 12, 1943

HYD 136


City of Seattle.
Lighting Department.
Ross Dam & Power House

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UNITED STATES
DEPARTMENT OF THE INTERIOR
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Branch of Design and Construction
Engineering and Geological Control
and Research Division
Denver, Colorado
October 12, 1943

Laboratory Report No. 136
Hydraulic Laboratory

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Subject: Hydraulic model studies for the design of Ross Dam spillway, City of Seattle, Washington.

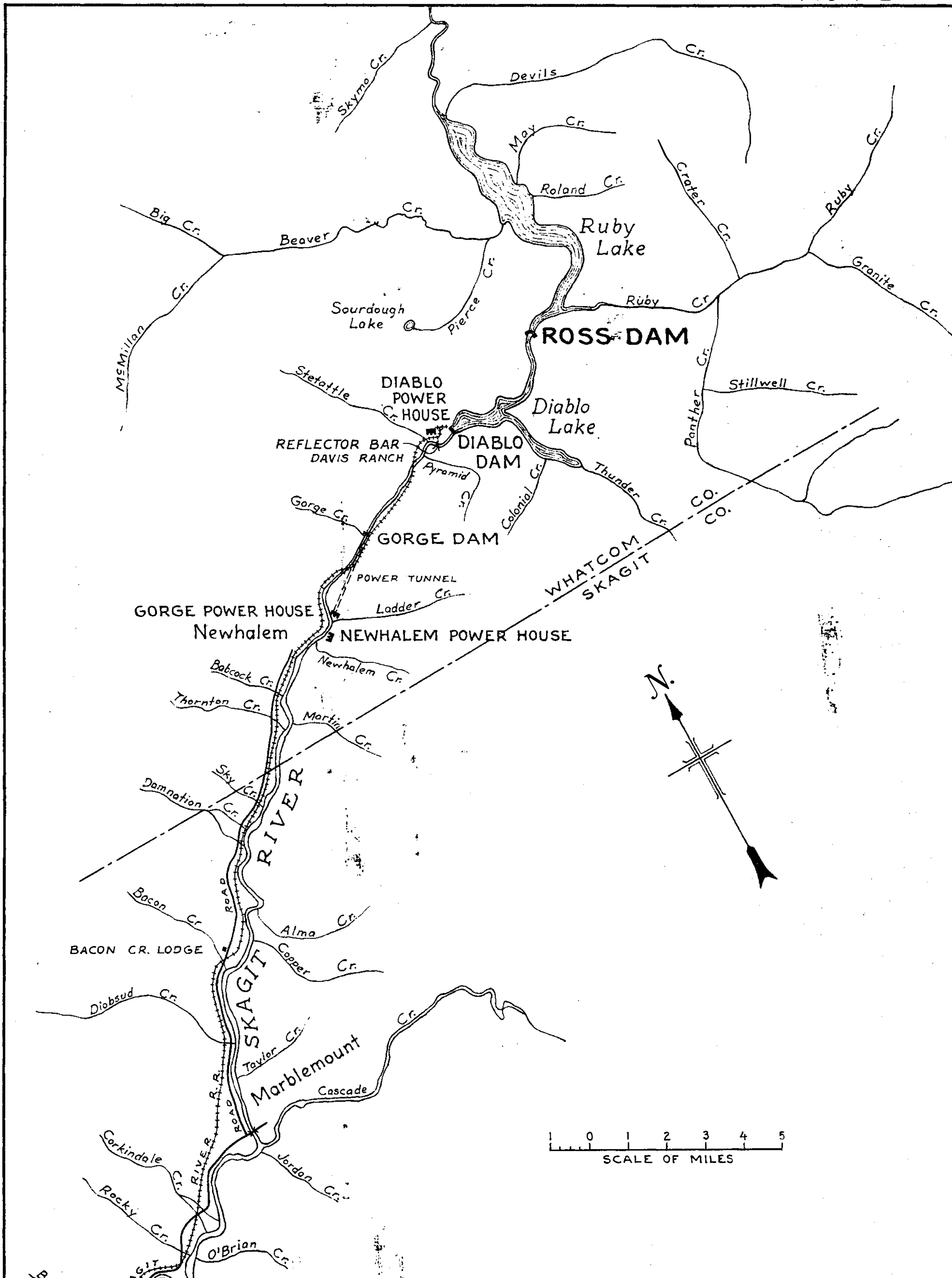
1. Introduction. In an agreement made January 21, 1943, and approved February 25, 1943 (contract I2r-14571), between the United States of America, represented by the Chief Engineer, Bureau of Reclamation, and the City of Seattle, Washington, represented by the Superintendent of Lighting, it was agreed that the Bureau would conduct an investigation pertaining to the design of the spillways to be used in connection with increasing the height of the Ross Dam located on the Skagit River in the State of Washington. This investigation was conducted by the use of hydraulic models by the personnel of the hydraulic laboratory of the Bureau. The scope of this work and the results obtained are reported chronologically herein as the final report.

2. Ross Dam. Ross Dam is located in the Skagit River 5 miles upstream from Diablo Dam and 8 miles upstream from Gorge Dam (figure 1), all of which constitute projects undertaken by the City of Seattle, Washington. Power plants are in operation in connection with the latter two dams, and a power plant is also proposed at Ross Dam.

The first stage of Ross Dam has been completed to elevation 1365 (figure 2), consisting of an arch ring about 1180 feet long at the crest, rising 165 feet above the tailwater, which is the reservoir level for Diablo Dam. The crest of this dam is surmounted by a 15-foot timber dam over which the floodwater spills. The tentative plan is to continue construction in several stages to an ultimate height of about 520 feet above tailwater, or elevation 1725. At this final stage the dam will be constructed as a gravity arch section by adding to the downstream face of the thin arch dam. The immediate plan is to continue the arch ring construction from its present stage in two steps, first to about elevation 1550 and later to elevation 1650. These are referred to in this report as the second and third stages, respectively.

The design of the spillways, with which this report deals primarily, is based upon the above assumptions. In the design of the spillways for the second and third stages of construction, an endeavor was made to match the designs such that a large portion of the spillway channels for the second stage could be utilized, with little alteration, for completion of

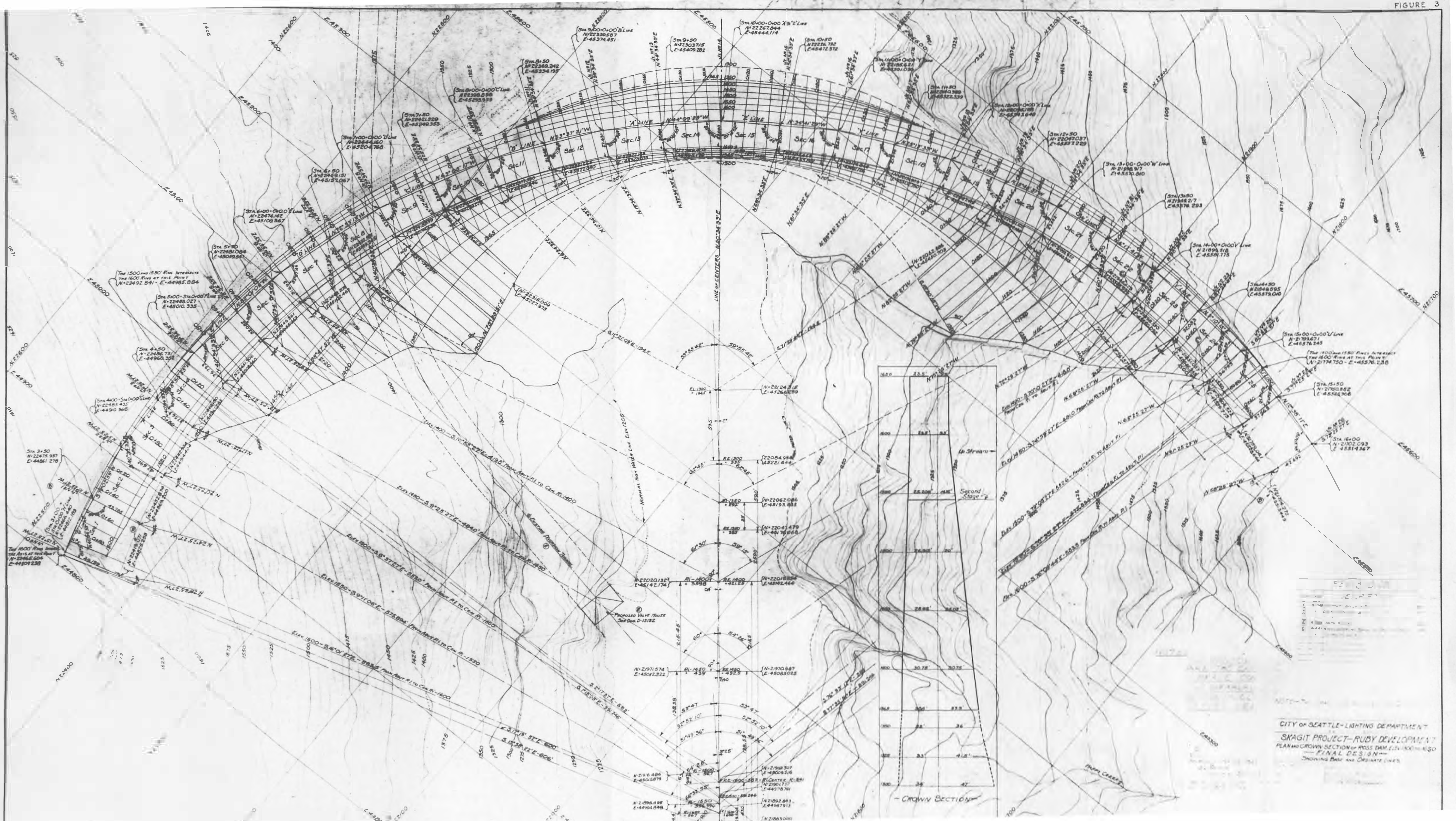
FIGURE 1



SKAGIT PROJECT
RUBY DEVELOPMENT
GENERAL VICINITY MAP
SHOWING LOCATION OF ROSS DAM

ROSS DAM--INITIAL DEVELOPMENT
TO ELEVATION 1365

FIGURE 2


CITY OF SEATTLE - LIGHTING DEPARTMENT
SKAGIT PROJECT - RUBY DEVELOPMENT
PLAN AND CROWN SECTION OF ROSS DAM
— FINAL DESIGN —
SHOWING BASE AND ORDINATE LINES
CROWN SECTION
Up Stream
Second Stage
LINE OF CENTERS
PROPOSED VALVE HOUSE
HAPPY CREEK

the third stage. It was anticipated that this type of planning could result in a considerable saving.

3. The spillways. The maximum expected flood computed from previous records was established as 70,000 second-feet. The final spillway design, however, provides for a capacity of 100,000 second-feet made possible by utilizing an additional five feet of freeboard on the dam.

The plans for the ultimate gravity arch dam rising to elevation 1725 showed a drum-gate-controlled, open-channel spillway skirting the left abutment of the dam. The topography is excellent for that type of spillway in this case, but with the top of the dam limited to elevation 1650, an additional 80 feet of cut would be required, thus making open-channel construction impracticable. Alternate designs could be side-channel or stoney-gate spillways skirting either or both abutments, with tunnels through to the downstream canyon walls. Both schemes are expensive; consequently, in accordance with your instructions, studies were concentrated on spillways which would discharge the water over the dam proper, regardless of construction stage.

4. The models. A model was constructed of the entire arch dam, with top elevation at 1650 to a 1:60 scale. Photographs of the original model showing the canyon-wall topography are presented on figure 4. A drawing indicating the final extent of the model is shown on figure 5. The arch was constructed of wood and lined with sheet metal on the upstream side. The head box was of similar construction, consisting of a wooden box lined with sheet iron and built integral with the upstream face of the dam. The downstream portion of the model dam and the adjacent topography consisted of metal lath construction faced with an inch of cement and sand surface coating. The metal lath was held in place by wooden supports. The box downstream from the dam was lined with sheet metal to a point slightly above maximum tailwater level. The spillway overflow sections were cast in a rich concrete mixture and in such a way that the sand did not penetrate to the outer surfaces. The piers were constructed of wood, and the gates were fabricated of sheet metal.

A second model, to a 1:25 scale, was constructed including one full gate with a half gate on each side to study pressure conditions in the closed portion of the spillways and on the face of the dam. The overflow section was made of concrete, the piers of wood, and the gates of sheet metal. Piezometers were installed throughout for measuring the pressures, and the deflector hood was a transparent plastic sheet through which the action of the water could be observed.

The water supply to both models was measured through accurately calibrated laboratory venturi meters which are systematically checked at regular intervals.

A- Overflow Sections 2 and 3

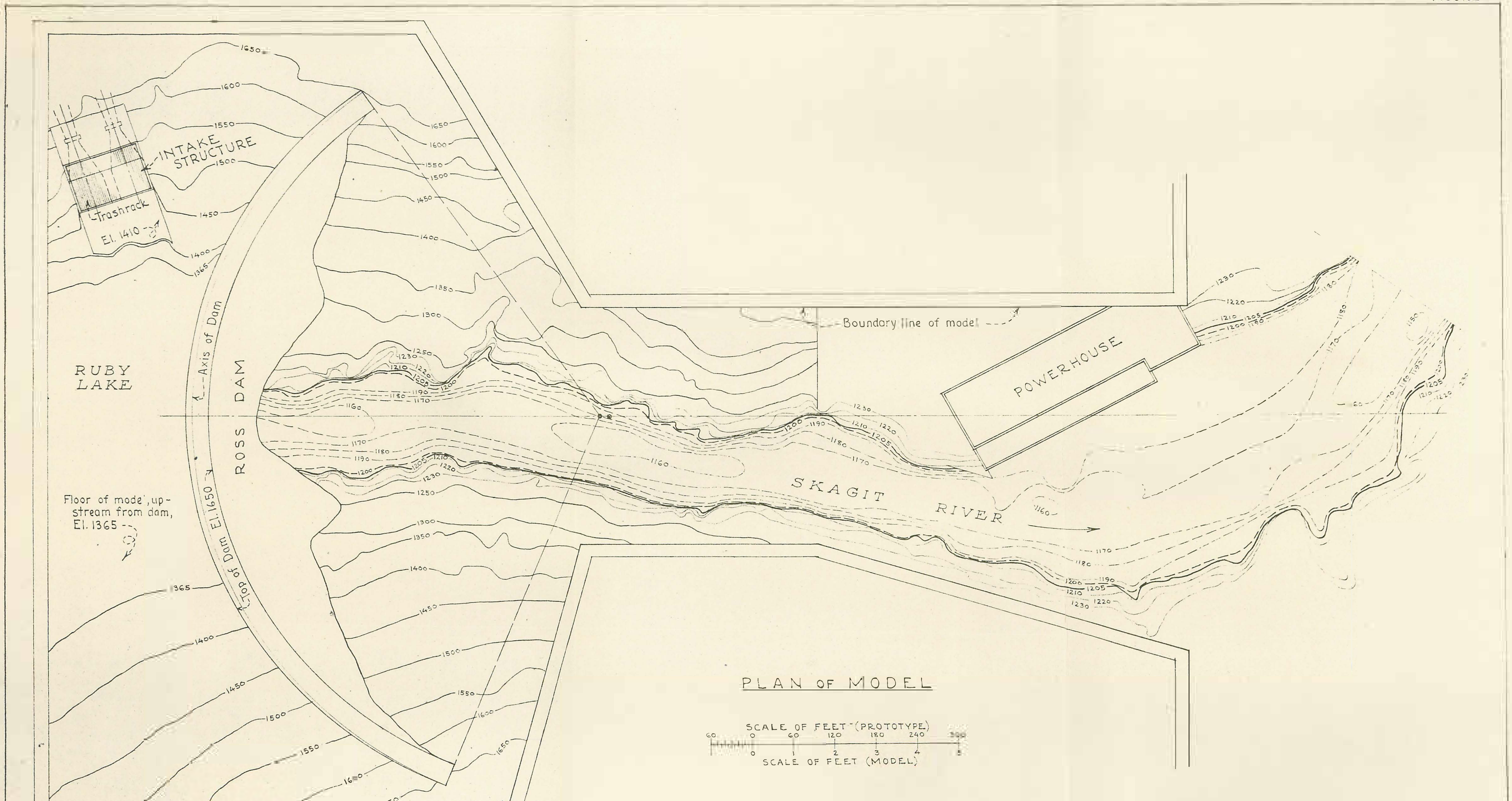
B- Overflow Section 2

C- Overflow Section 3

MODEL AS CONSTRUCTED ON 1:60 SCALE SHOWING ORIGINAL TOPOGRAPHY

FIGURE 4

FIGURE 5



ROSS DAM
EXTENT OF 1:60 SCALE MODEL

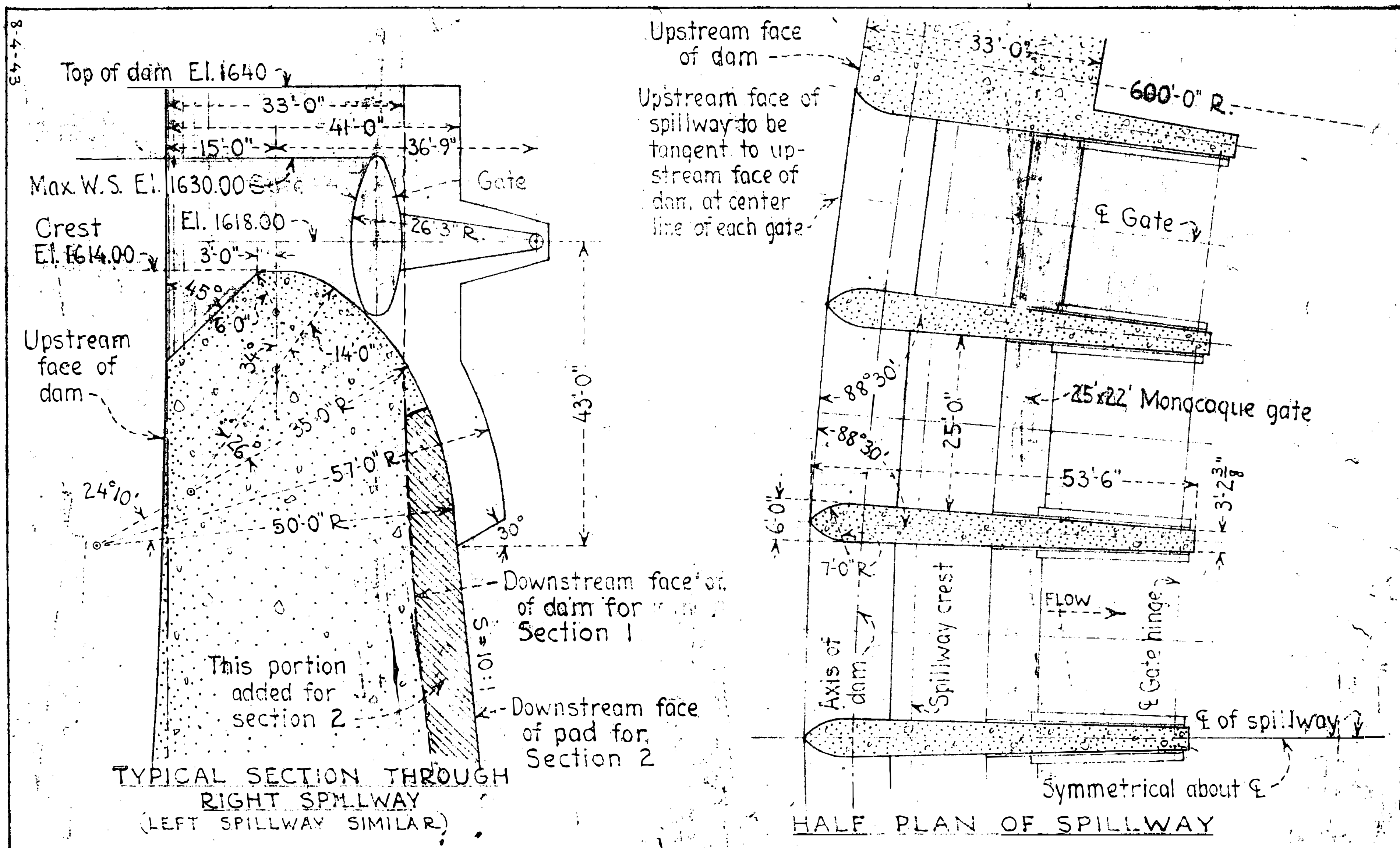
5. Overflow section 1. As a preliminary trial, overflow section 1 (figure 6) was constructed to a 1:60 scale with provision for 35,000 second-feet controlled by six 25- by 22-foot monocoque gates. The center line of the overflow section was midway between construction joints 5 and 6. Although it was intended that similar spillways be located near each abutment, thus making the total spillway capacity 70,000 second-feet, each of the preliminary designs was constructed on one side only. In other words, each succeeding design was alternately constructed on one side and the following design on the opposite side. The crest of overflow section 1 was at elevation 1614.0, making the reservoir elevation approximately 1630 for a discharge of 35,000 second-feet. The gates were set downstream from the crest, as shown on figure 6, to obtain a trajectory with initial velocity directed at a downward angle rather than horizontal as is usually the case. The portion of the overflow section upstream from the gates was designed to produce no subatmospheric pressures in this region for discharges up to 35,000 second-feet.

Two limitations sufficed to make the design of these spillways difficult. First, the dam was of insufficient thickness at the top to construct a true vacuumless overflow shape; and second, cutting into the dam below elevation 1600 was prohibited because of stress considerations in the arch. Overflow section 1 projected past the downstream face of the dam and the jet of water leaving the section fell free of the dam face. From the instigation of the studies it was considered advisable to have the sheet of water in contact with the face of the dam. As a result of this particular test, no other condition was considered acceptable.

6. Overflow section 2. In all subsequent tests the criterion was that the overfalling sheet of water be in direct contact with the downstream face of the dam for all conditions of flow. A heavy concrete bucket was to be provided at the junction of dam and rock surfaces to deflect the jet, and a superelevated channel was to be provided downstream from the bucket to conduct the flow safely to the river. With the jet making definite contact with these surfaces, no damage was expected.

Overflow section 2 (figure 6) was the same as section 1 except for the concrete pad added to the downstream face of the dam. The jet performed as desired for the lower discharges but sprung free of the pad intermittently for discharges greater than 25,000 second-feet. The steepness of the overflow shape encouraged vortices to play back and forth across the section, producing very unstable flow. Photographs of this spillway are shown for a discharge of 35,000 second-feet on figure 7 A and B. The original topography had not been disturbed.

7. Overflow section 3. Overflow section 3 (figure 8) consisted of a combination of two radii used with no intention of simulating the



ROSS DAM SPILLWAY
OVERFLOW SECTIONS 1 AND 2

FIGURE 6

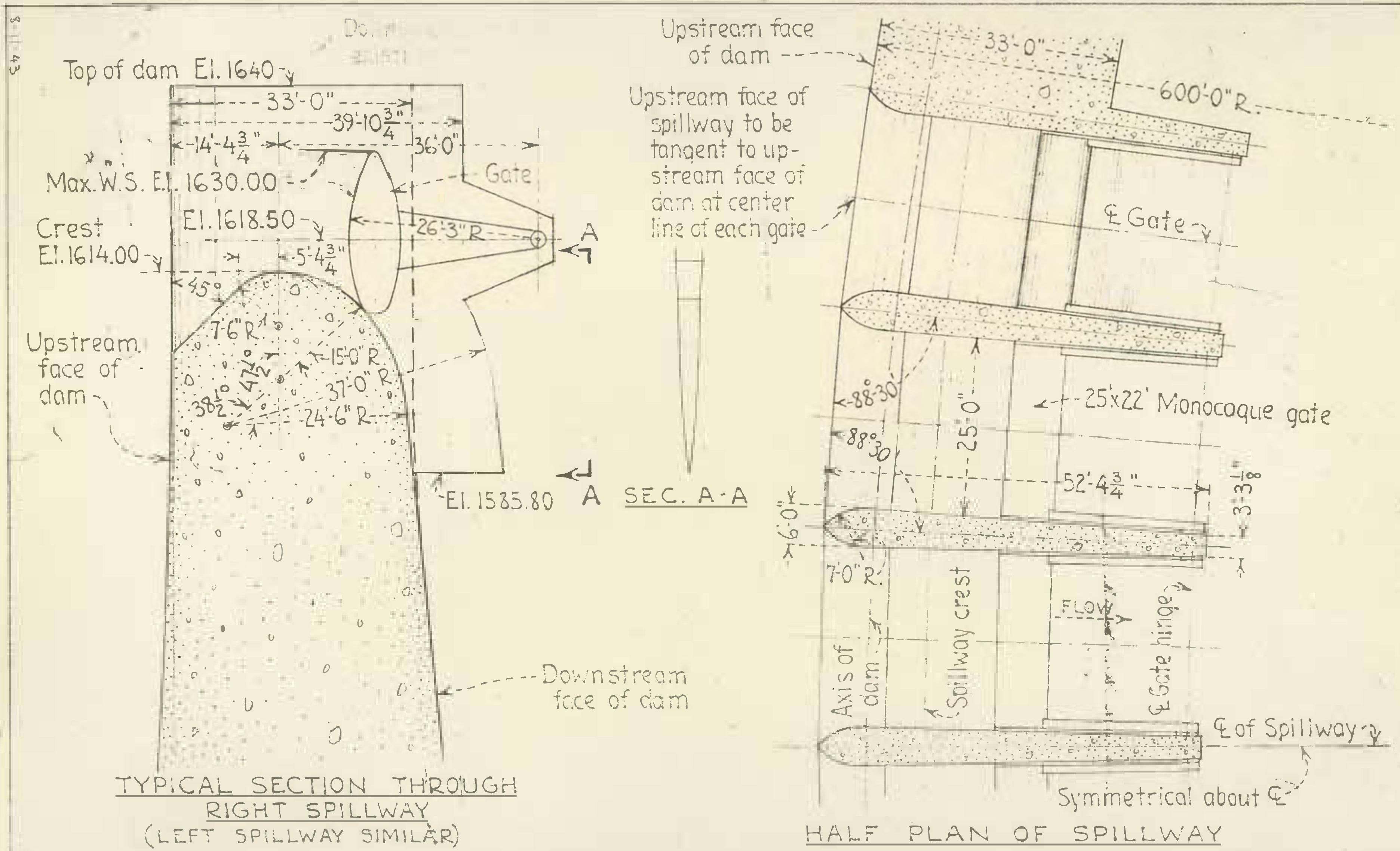
A- Overflow Sections 2 and 3

B- Overflow Section 2

C- Overflow Section 3

DISCHARGE 35,000 SECOND-FEET PER SPILLWAY WITH ORIGINAL CANYON WALL TOPOGRAPHY

FIGURE 7



ROSS DAM SPILLWAY
OVERFLOW SECTION 3

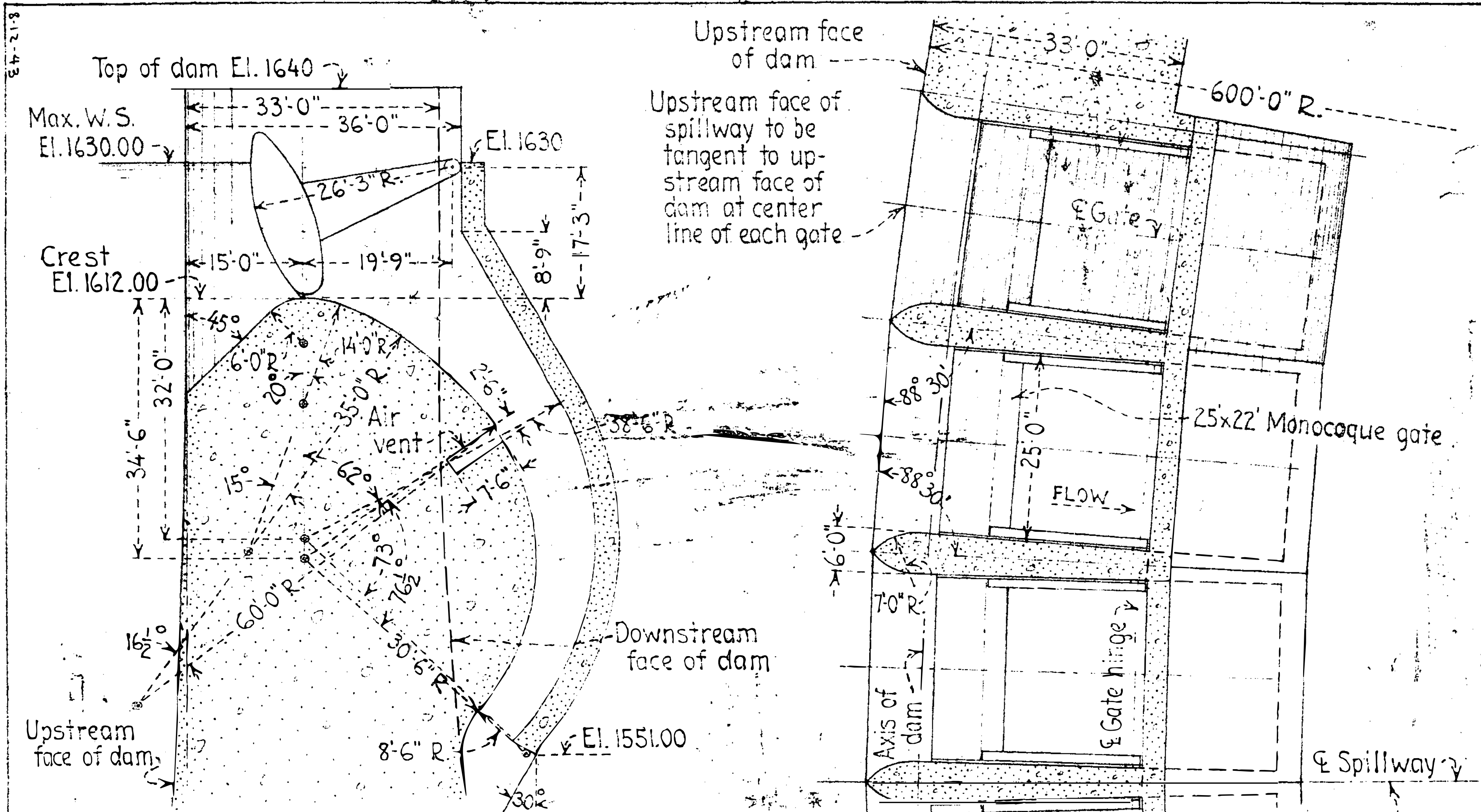
FIGURE 8

actual shape of the lower nappe of the overfalling sheet. The padding on the dam face was removed and the gates again suspended to operate on the curved portion of the overflow section. In this case, the object was to develop negative pressures under the jet in an attempt to virtually force the sheet of water against the overflow face. Results were less desirable than for the preceding case as vortices made it possible for air to enter intermittently under the jet, resulting in an intolerable make-and-break action. Photographs of this spillway are shown for a discharge of 35,000 second-feet on figure 7 A and C. The natural topography was still intact.

8. Overflow section 4. It became apparent that the design of a free overflow shape to perform as desired was a remote possibility, especially with the limitation prohibiting cutting into the dam below elevation 1600. In the design of overflow section 4 (figure 9), an attempt was made to force the overfalling jet against the dam face by a deflector plate. The overflow face projected past the downstream face of the dam on which a vent slot was provided for aeration under the jet. The gates were suspended directly over the crest, thus decreasing their height. The passageway between the overflow face and the deflector wall was proportioned to flow full at the maximum discharge.

The spillway was operated with and without aeration beneath the jet. The flow conditions were improved, but there was a decided pulsation in flow when air was admitted beneath the nappe. A less noticeable fluctuation occurred when the air supply was discontinued. In neither case, however, could pulsation of flow be tolerated. A second disadvantage of this design was the abrupt angle at which the jet impinged against the dam face as it emerged from the conduit. On striking the dam, a portion of the jet was deflected away from the face. Uncertainty existed as to the damage that might result at the zone of impact. Photographs of overflow section 4 discharging at 35,000 second-feet are shown on figure 10 A and B. In this case the canyon wall adjacent to the spillway was excavated on a 1-1/4:1 slope measured normal to the line of centers of the dam.

9. Overflow section 5. Overflow section 5 (figure 11) was the same as overflow section 3 except for the addition of a deflector hood. The spillway consisted of six gates 25 feet in width and was designed for a maximum discharge of 35,000 second-feet with the reservoir at approximately elevation 1630. The resulting performance of this spillway was a decided improvement over the previous one. Flow conditions were excellent for the higher discharges, decreasing in desirability as the discharge was reduced. A dribbling action occurred from the end of the deflector hood at very low flows. The main jet of water maintained contact with the face of the overflow section and dam; thus the dribbling was primarily caused by fins adjacent to the piers. Photographs of overflow section 5 operating at a maximum discharge of 35,000 second-feet are shown on figure 10 A and C.



TYPICAL SECTION THROUGH RIGHT SPILLWAY
(LEFT SPILLWAY SIMILAR)

HALF PLAN OF SPILLWAY

ROSS DAM SPILLWAY
OVERFLOW SECTION 4

FIGURE 9

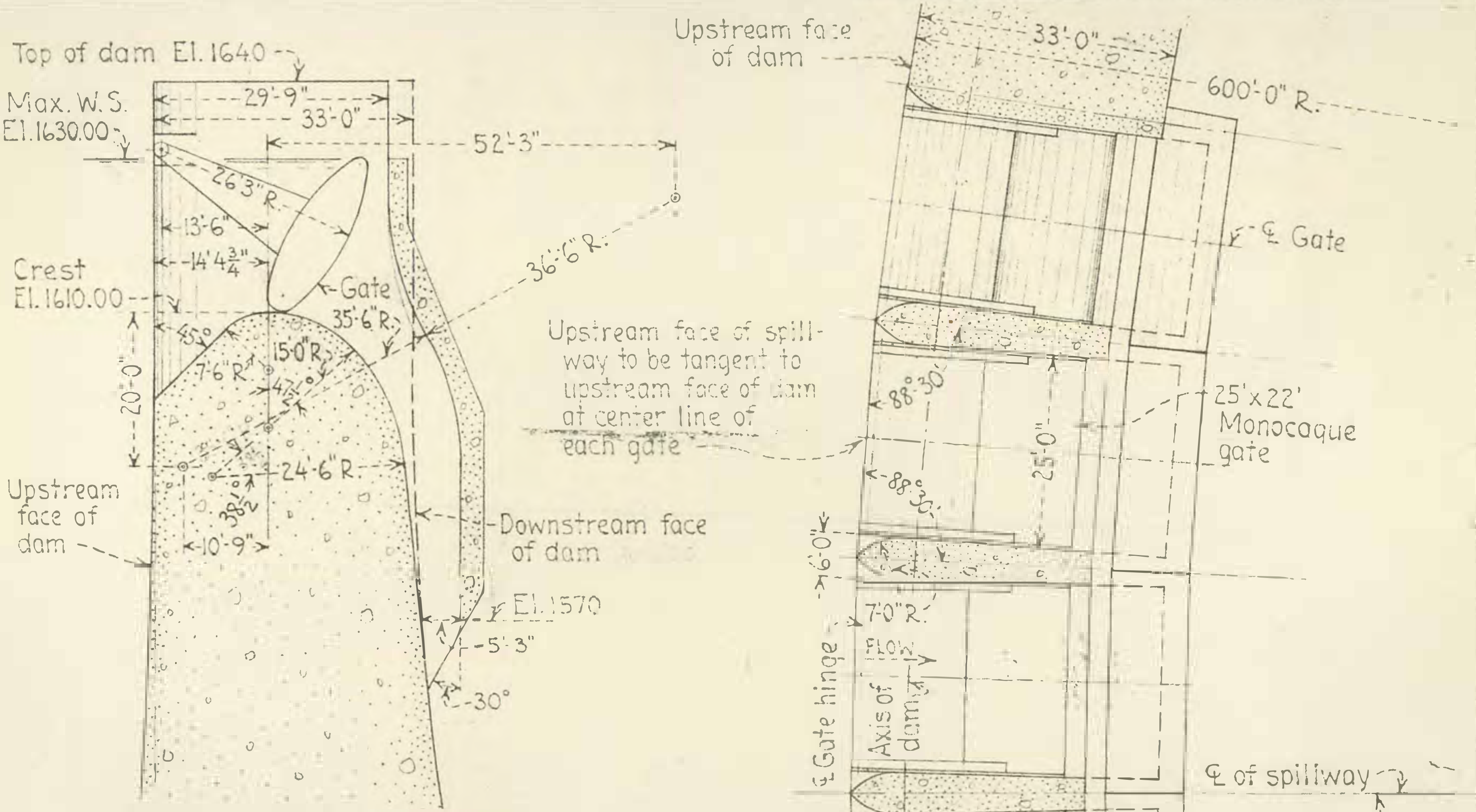
A- Overflow Sections 4 and 5

B- Overflow Section 4

C- Overflow Section 5

DISCHARGE 35,000 SECOND-FEET PER SPILLWAY -CHANNELS EXCAVATED ON 1:1 SLOPE MEASURED TO LINE OF CENTERS.

FIGURE 10



TYPICAL SECTION THROUGH
RIGHT SPILLWAY
(LEFT SPILLWAY SIMILAR)

HALF PLAN OF SPILLWAY

ROSS DAM SPILLWAY
OVERFLOW SECTION 5

FIGURE 11

As this type of design showed promise, a 1:25 model was constructed to investigate pressure conditions.

10. Overflow section 6. Overflow section 6 (figure 12) was constructed to a 1:25 scale equipped with sufficient piezometers to study the pressures on the overflow face and the deflector hood. The deflector hood was extended to elevation 1560, and each spillway was designed to flow full at a discharge of 50,000 second-feet with reservoir at approximately elevation 1635 and overflow crest at elevation 1612. After a digest of stream-flow data by Senior Engineer Ivan E. Houk, the capacity of the two spillways was boosted to a total of 100,000 second-feet as a conservative measure. The gates were reduced in width from 25 to 20 feet (figure 12) without a change in number and were hinged in the opposite direction to those in previous tests. The new spacing permitted the installation of two spillway gates between adjacent construction joints. The split piers at the construction joints were 6.77 feet wide and the intermediate piers 3.83 feet wide, both measurements being at the greatest width.

The 1:25 model consisted of one full gate with a half gate on each side and a short section of dam below them. Model results converted to prototype pressures are shown on figure 13 for overflow section 6 for free crest operation as well as for four gate positions. A maximum negative pressure on the overflow face of 20 feet of water is shown at piezometer 9 with a free discharge of 100,000 second-feet. The discharges as referred to on this model represent the corresponding flow for twelve gates, or the total for both spillways. As the discharge was increased above this value, a back pressure in the conduit produced by additional friction relieved the subatmospheric conditions to some extent. Figure 13 shows that the pressures were more severe for 100,000 second-feet than for discharges at partial gate openings with reservoir at elevation 1630, which was contrary to expectations. It was desired to limit the maximum negative pressure to one-half an atmosphere, or about 15 feet of water.

The maximum positive pressure on the deflector hood, on the other hand, is shown as 20 feet of water or 8.7 pounds per square inch for a flow of 125,000 second-feet. The heavy dash line on figure 13 represents the maximum positive pressure recorded on the hood for all conditions of operation. Discharge at partial gate openings defines this line in the vicinity of piezometers 30, 45, and 60.

The negative pressures are effective in maintaining contact between the jet and the overflow face but are not sufficient to control the entire sheet. There was a tendency for the outer portion to separate from the main jet, thus causing impingement on the deflector hood for all but the very small discharges. The hood therefore served an important purpose at practically all flows.

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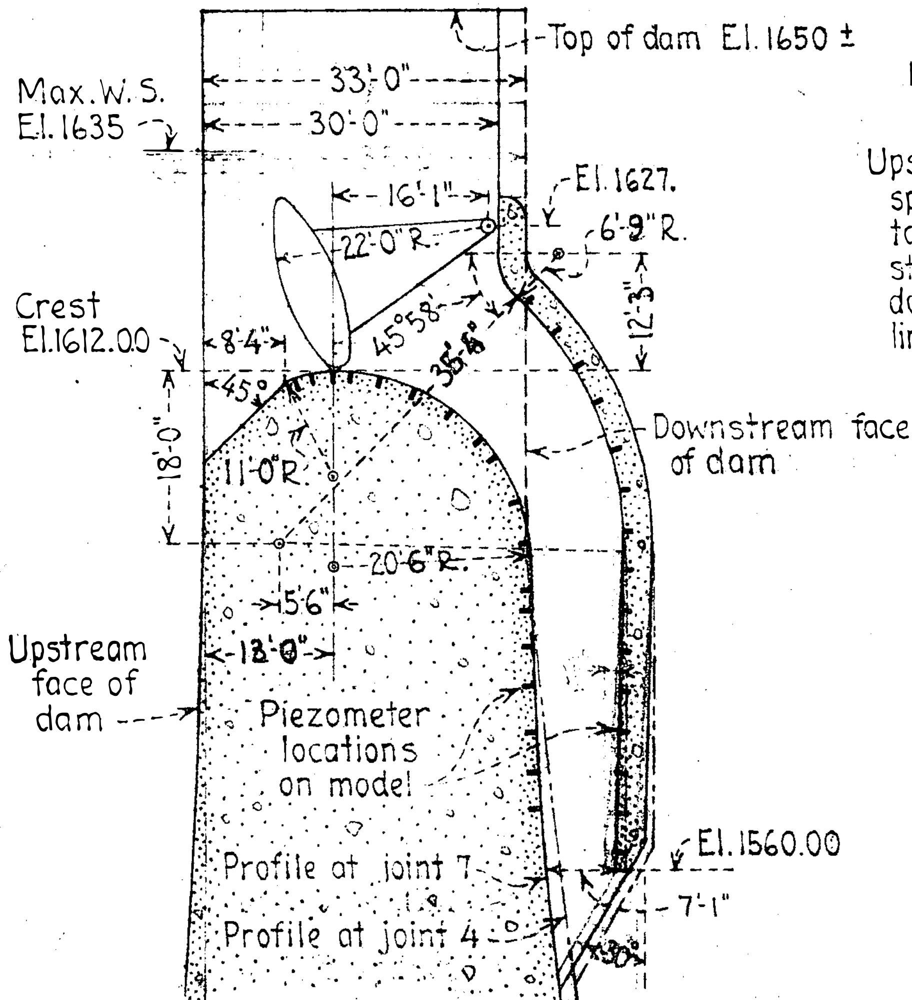
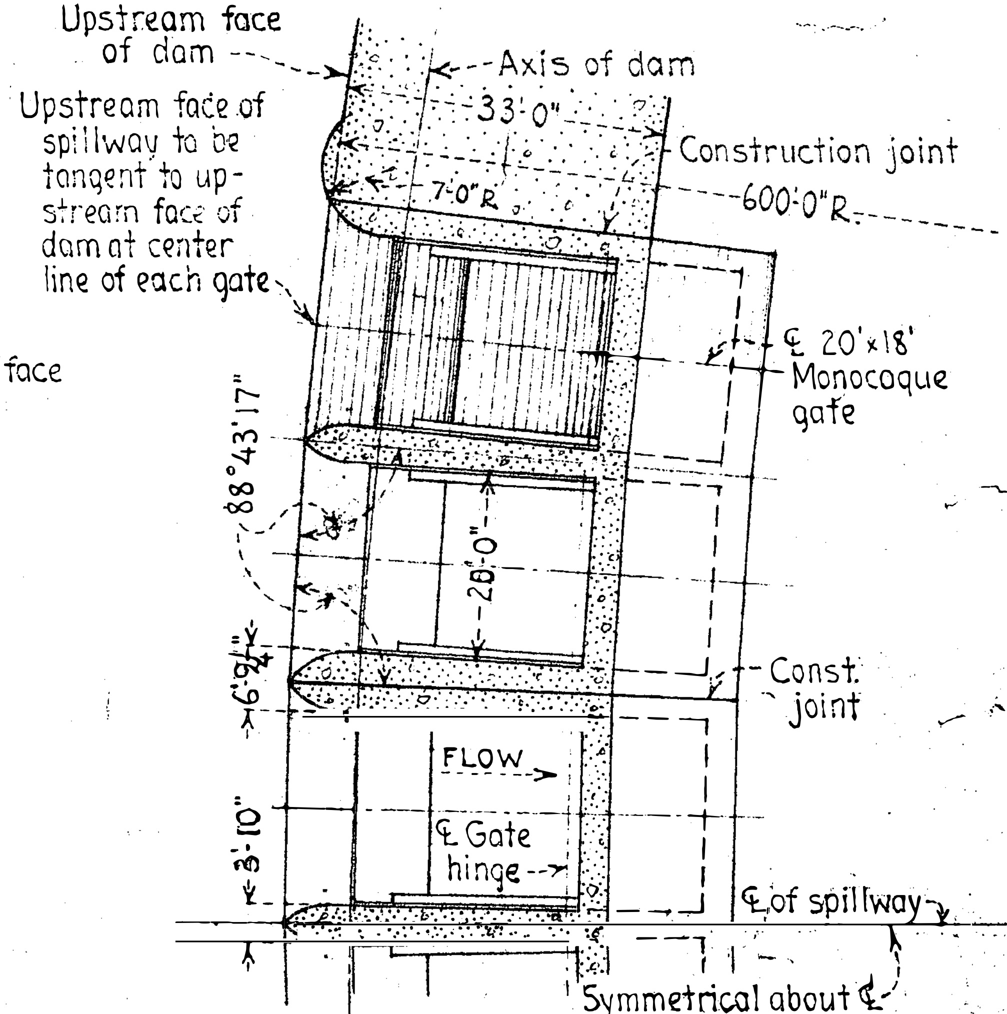


TYPICAL SECTION THROUGH RIGHT SPILLWAY
(LEFT SPILLWAY SIMILAR)

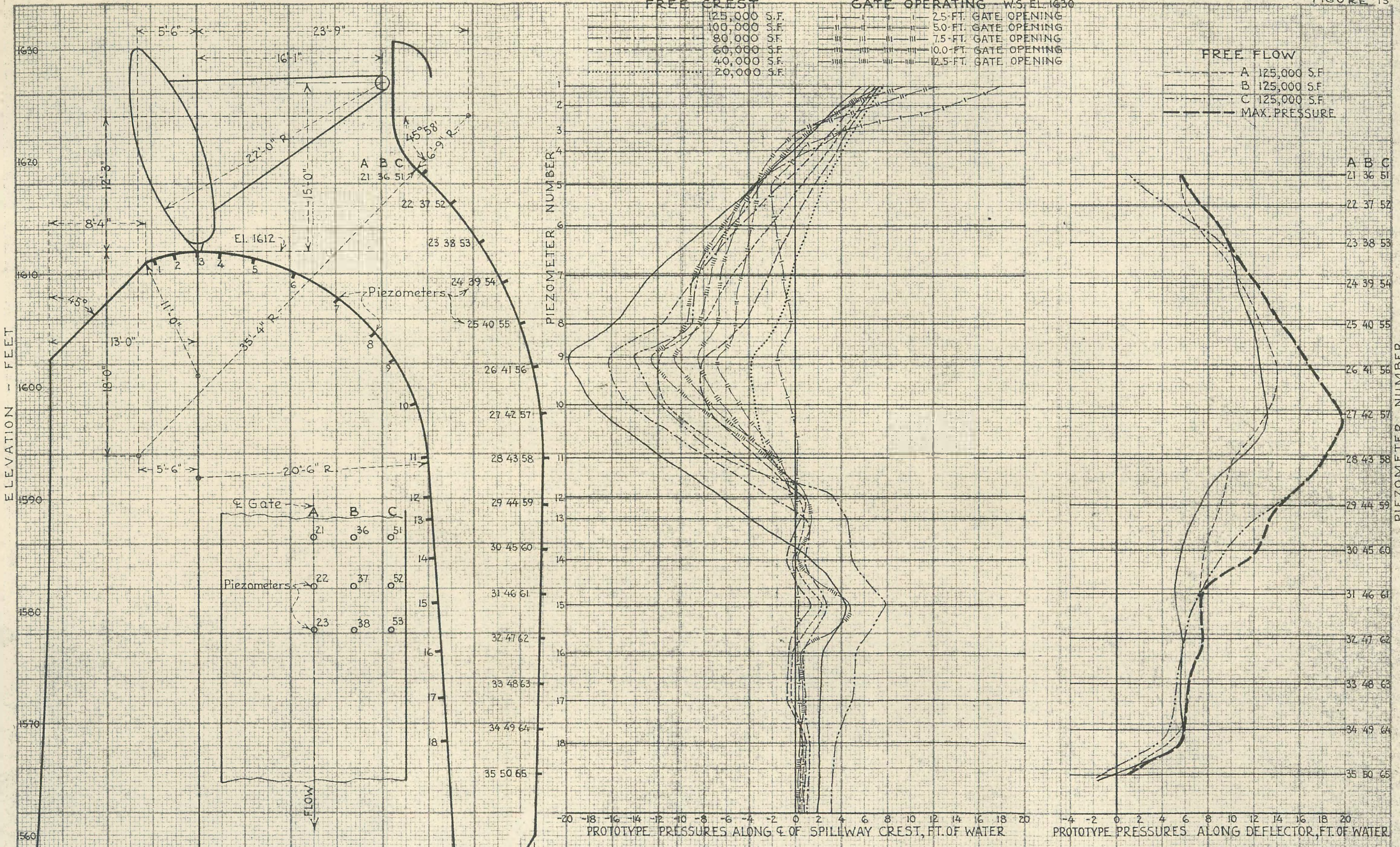


HALF PLAN OF SPILLWAY

ROSS DAM SPILLWAY
OVERFLOW SECTION 6

FIGURE 12

FIGURE 13



1635
1612

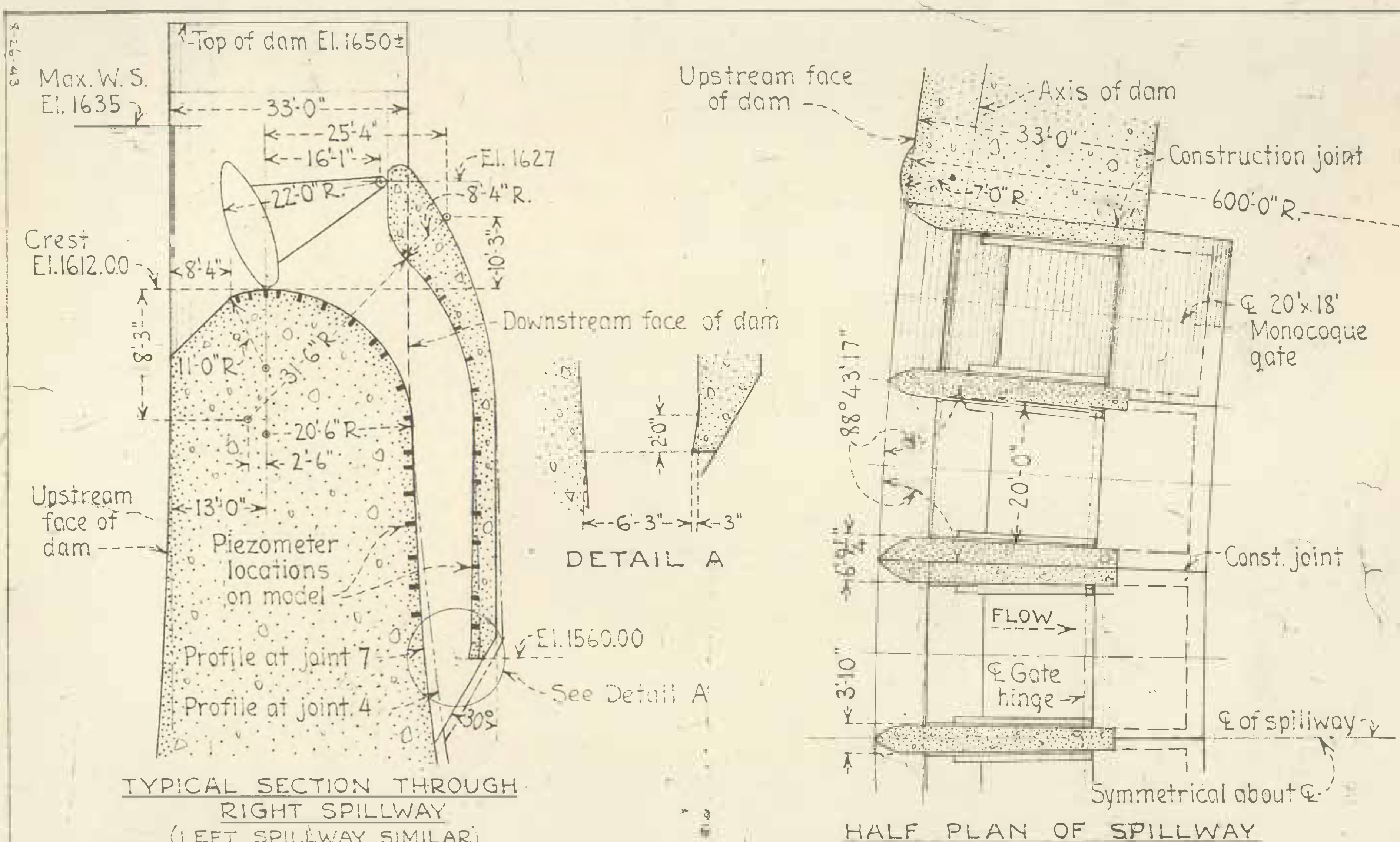
ROSS DAM
SPILLWAY PRESSURES

Overflow section 6 was overdesigned as the closed portion of the spillway did not flow full until a discharge of 125,000 second-feet was reached. It was intended that this condition would occur at 100,000 second-feet. The proportions of the closed section were arrived at by computing areas throughout based on the theoretical velocities less 15 percent for friction and impact losses. The computation was repeated, allowing a total of only 5-percent loss for a discharge of 100,000 second-feet, and the resulting shape is designated as overflow section 7. The losses were found to be extremely low, even at maximum discharge; therefore the efficiency of this spillway should compare favorably with the open, free overflow section.

11. Overflow section 7. Overflow section 7, shown on figure 14, was the same as section 6 except for the change of position and the dimensions of the deflector hood. The closed portion flowed full at a discharge of approximately 100,000 second-feet. A small lip was added to the lower end of the deflector hood to deflect the smaller discharges toward the dam face, thus reducing the dribbling previously experienced. Pressures on the overflow face and the deflector hood are plotted for overflow section 7 for free flow and also for gate operation on figure 15. The maximum negative pressure in this case is shown as 18 feet of water, which occurred at piezometer 9 for a discharge of 95,000 second-feet. The negative pressure decreased as the conduit flowed full at 100,000 second-feet. The maximum positive pressure on the deflector wall was reduced from 20 to 15 feet, but the decrease in discharge in the latter case was principally responsible for this reduction. Photographs of the 1:25 sectional spillway operating at 10,000, 40,000, 75,000, and 100,000 second-feet are shown on figure 16. The discharges as stated are based on twelve gates discharging at the same rate as the two shown. The dribbling can be seen in figure 16A with a discharge of 10,000 second-feet. As the discharge was increased, flow conditions continually improved until the ultimate desired flow condition was reached at 100,000 second-feet, at which point the conduit flowed full. The pressures on the face of the dam were positive throughout the entire discharge range.

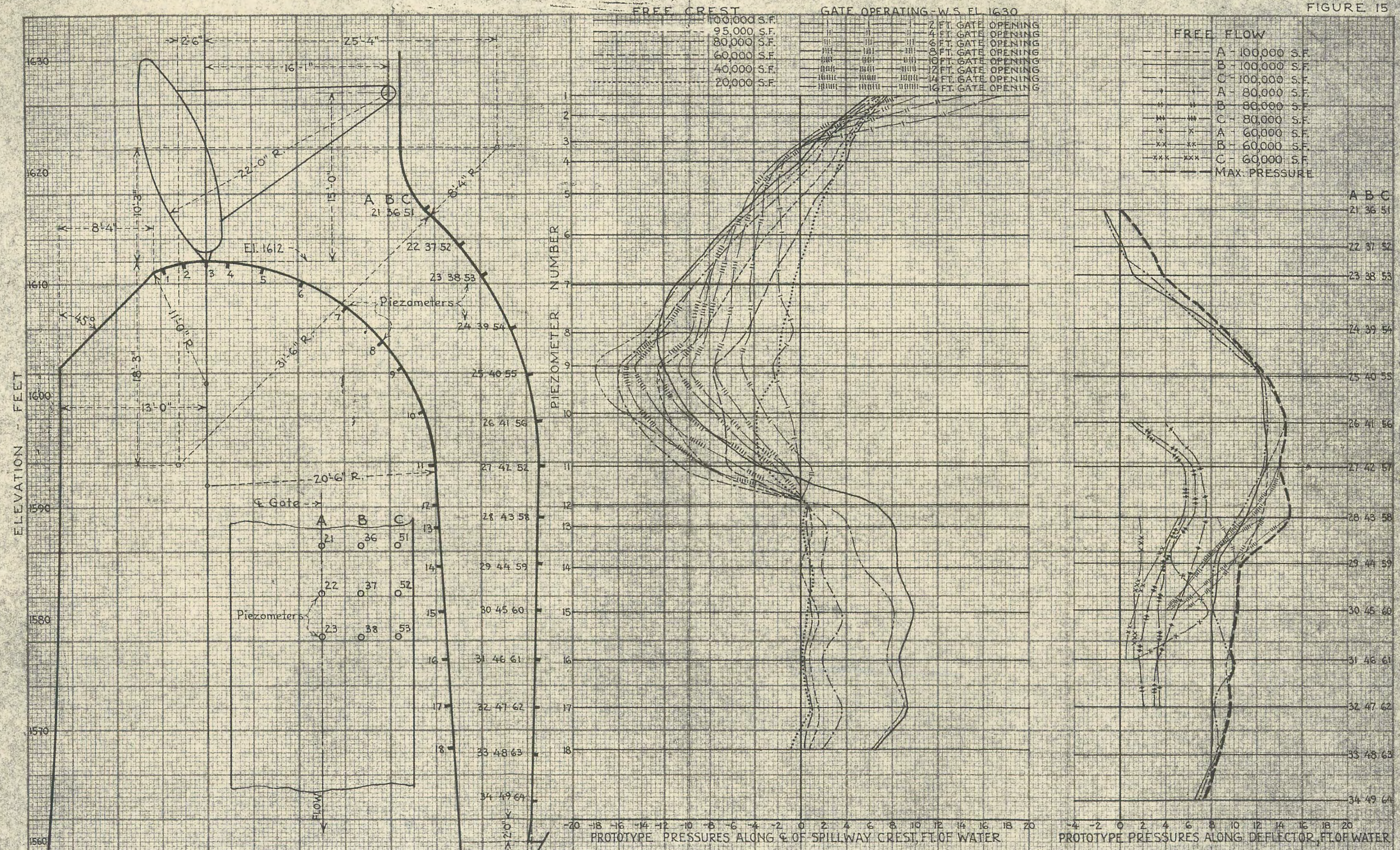
12. Overflow section 8. In an attempt to reduce the negative pressure, the constant radius in overflow section 8 was changed to a compound curve as shown on figure 17. The object was to reduce the negative pressure at piezometer 9 and increase it at other points by shortening the radius of curvature immediately downstream from the crest and lengthening it in the vicinity of piezometer 9. Assuming that

$p = \frac{V^2}{r}$ applies in this particular case (where p = pressure on face, V = velocity, and r = radius of curvature), the negative pressure at piezometer 9 should be about 15 percent less. From the curves on figure 17 the change in curvature improved the pressure at piezometer 9 by only 5.5 percent, or reduced it to a negative 17 feet of water. The



ROSS DAM SPILLWAY
OVERFLOW SECTION 7

FIGURE 14



ROSS DAM
SPILLWAY PRESSURES

FIGURE 16

A- Discharge 10,000 Second-Feet

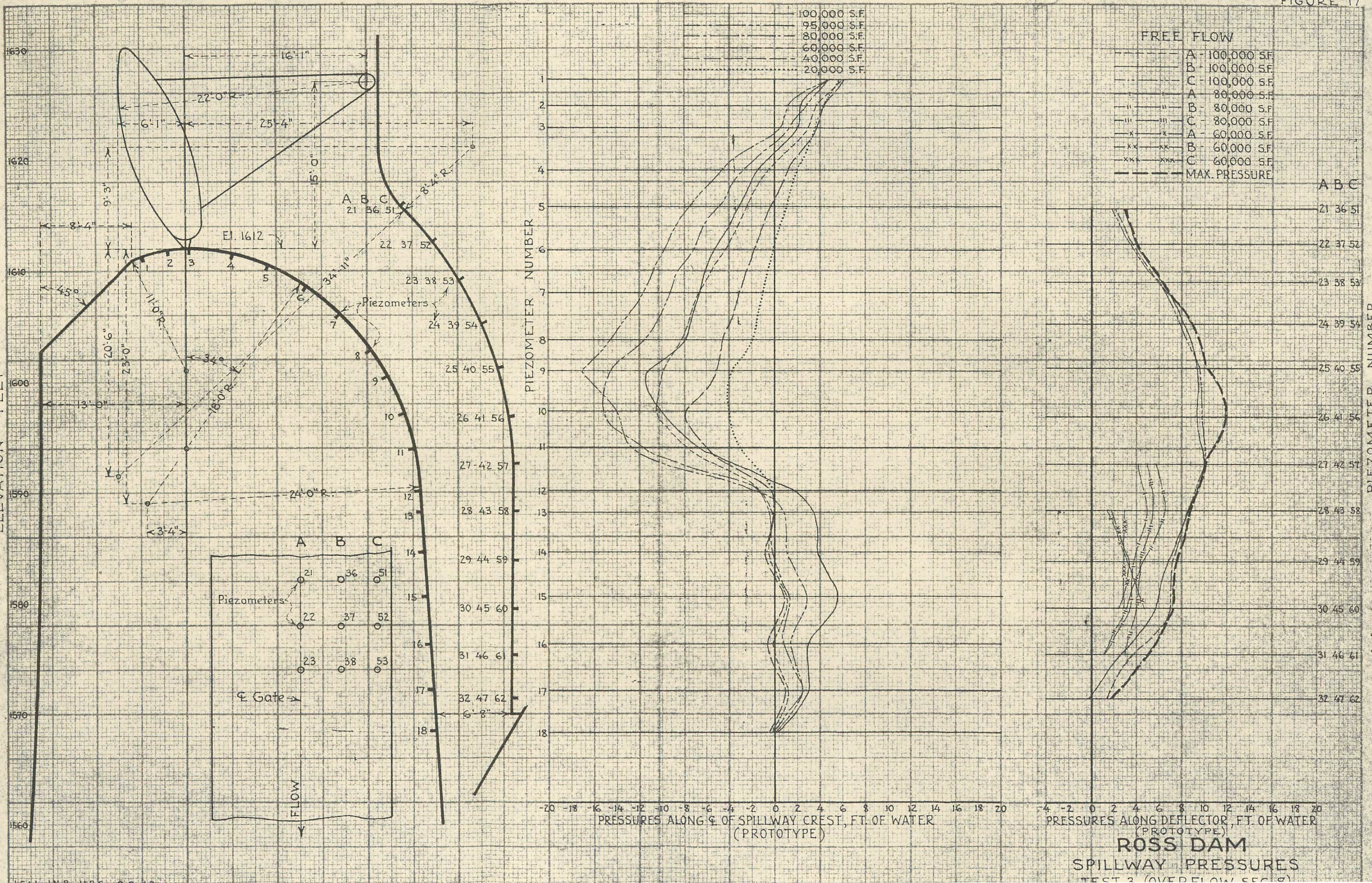
B- Discharge 40,000 Second-Feet

C- Discharge 75,000 Second-Feet

D- Discharge 100,000 Second-Feet

SECTIONAL MODEL ON 1:25 SCALE EMBODYING TWO GATES

FIGURE 17



ROSS DAM
SPILLWAY PRESSURES
TEST 3 (OVERFLOW SEC'S)

negative pressures at points above piezometer 9 did increase but not to the extent expected. The conclusion is that the theory does not completely apply as the curvature was too great for the water to follow naturally. No additional appreciable improvement in pressure conditions on the overflow face by further experimentation was apparent as long as it was necessary to comply with the established limitations. The shape as shown on figure 17 was therefore considered the final design as far as the laboratory investigations were concerned.

The maximum average positive pressure on the deflector hood (figure 17) was reduced to 12 feet of water for a discharge of 100,000 second-feet. With the alteration in shape of overflow section 8, a corresponding change was made in the dimensions of the deflector hood, which was responsible for this pressure reduction.

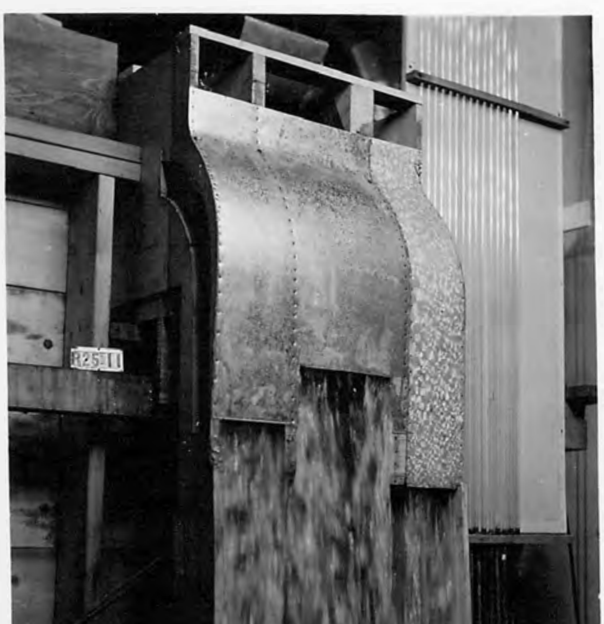
Due to the nature of this design, information was desired concerning the period of vibration and the magnitude of instantaneous pressures on the deflector hood. It was not possible to obtain comparable vibration measurements from the model which would apply to the prototype. It can be said, however, that vibration did exist in the model but was not of sufficient magnitude to cause concern. Instantaneous pressures were observed at various points on the deflector hood of the model by an oscilloscope. The fluctuation in pressure equalled the average measured pressure in most cases. In other words, for the average maximum pressure of 12 feet of water measured on the hood, the actual instantaneous pressures varied from 6 to 18 feet of water, or, the maximum force, including impact, amounted to 150 percent of the average measured pressures. It is recommended that the forces used for design be increased about 100 percent to take account of this impact.

A study was made on the overflow section to determine the most economical elevation at which to terminate the downstream extremity of the deflector hood. Figure 18 shows a portion of the hood ending at elevation 1570 with the remainder at elevation 1560. Figures 18 A and B show flows of 40,000 and 100,000 second-feet, respectively. The photographs do not depict clearly the difference in flow encountered for the two hood elevations. The jet of water leaving the longer hood was forced closer to the dam face and therefore exhibited a more desirable appearance from a hydraulic standpoint than the shorter hood. Figures 18 C and D illustrate the point more clearly. The deflector hood extremities are at elevations 1570, 1580, and 1560, viewed from left to right. The jet of water overtopped the piers with the hood ending at elevation 1580. The same condition occurred to a lesser extent for the hood ending at elevation 1570. Figure 19 shows the variation of taper with the length of hood. From a hydraulic viewpoint the most economical point to end the hood is at elevation 1560. Although the tests on overflow section 8, figure 17, were made for a hood ending at elevation 1570, it is recommended that the final design be made to conform with the section shown on figure 19. The shapes are exactly

A- Discharge 40,000 Second-Feet

B- Discharge 100,000 Second-Feet

End of Hood at Elevation 1570 on Left and 1560 on Right

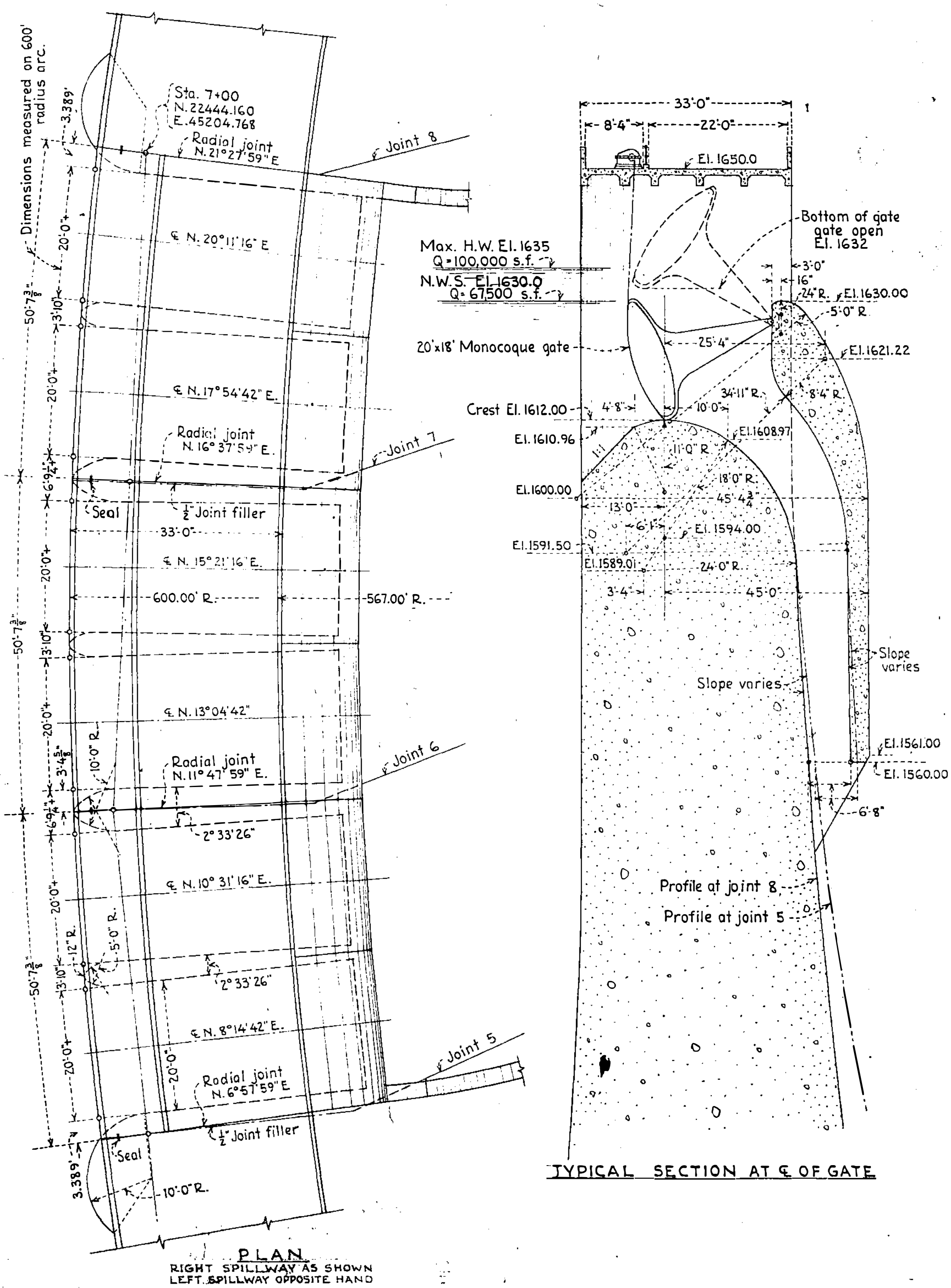


C- Discharge 40,000 Second-Feet

D- Discharge 100,000 Second-Feet

End of Hood at Elevation 1570, 1580 and 1560 - Left to Right

STUDIES TO DETERMINE LENGTH OF DEFLECTOR HOOD - 1:25 MODEL



ROSS DAM SPILLWAY
OVERFLOW SECTION 8 - AS RECOMMENDED

OCT. 16, 1943 C.J.H.-H.R.S.

the same except that on the latter figure the deflector hood has been extended to elevation 1560.

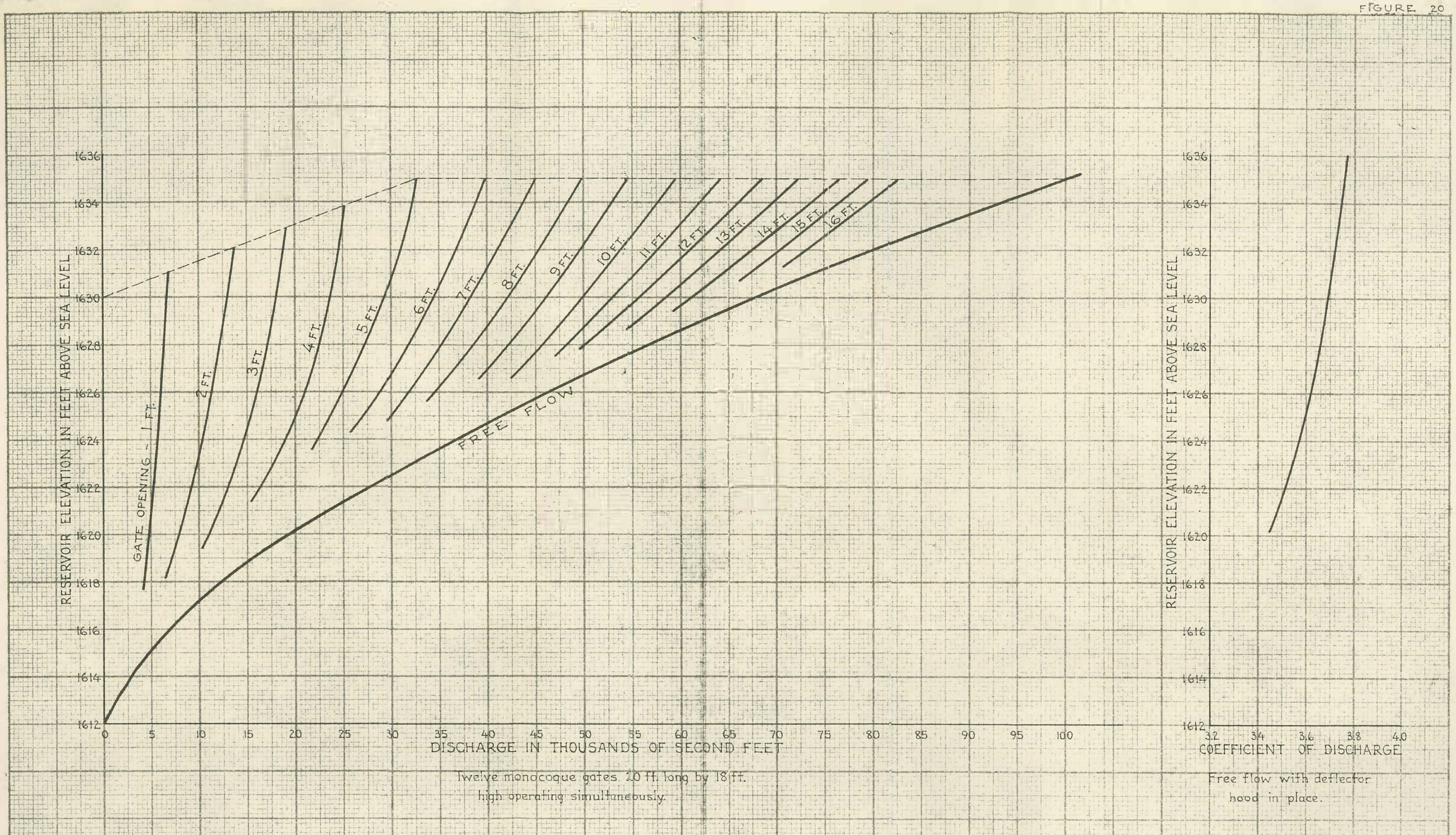
Head-discharge curves, obtained by calibrating one gate on the 1:25 model and multiplying the results by twelve, are shown on figure 20 for free flow and also for gate operation. The curves apply to both overflow sections 7 and 8. For free flow a discharge of 70,000 second-feet can be developed with reservoir elevation of 1630.5. To obtain 100,000 second-feet through the twelve gates requires a reservoir elevation of 1635.0. A head-discharge coefficient curve for free flow is also shown plotted on figure 20.

13. Final spillway design for dam to el. 1650. Upon conclusion of the above tests, overflow section 8 (figure 19) was installed at both ends of the arch on the 1:60 model and studies were commenced to determine the grades and wall positions for the channels downstream. The spillways were located as near the dam abutments as possible. The right spillway, as originally planned, was between section joints 4 and 7 and the left spillway between joints 20 and 23.

The design of the spillway channels downstream from the deflector hoods was not based on theory. Rather, the design was arbitrarily established from consideration of the arrangement of the dam and the topographic conditions downstream and then verified by model performance. A plane surface was first established for a channel, this plane intersecting the dam on a slope. A fillet was employed to blend this plane in with the downstream face of the dam. For the first trial the plane was projected parallel to the line of centers on a 1-1/4:1 slope with a 100-foot radius fillet between this slope and the dam face. The economics of the layout were not considered in this first study. The wall positions were arbitrarily located on paper, but this method was not satisfactory. The model test indicated that the wall locations were dictated primarily by the position of the channel plane. Subsequent tests also showed that considerable latitude existed in the choice of slope and direction for the channel floor, with the restriction that the surface be a plane for the entire width of the channel and that wall positions be determined separately for each layout.

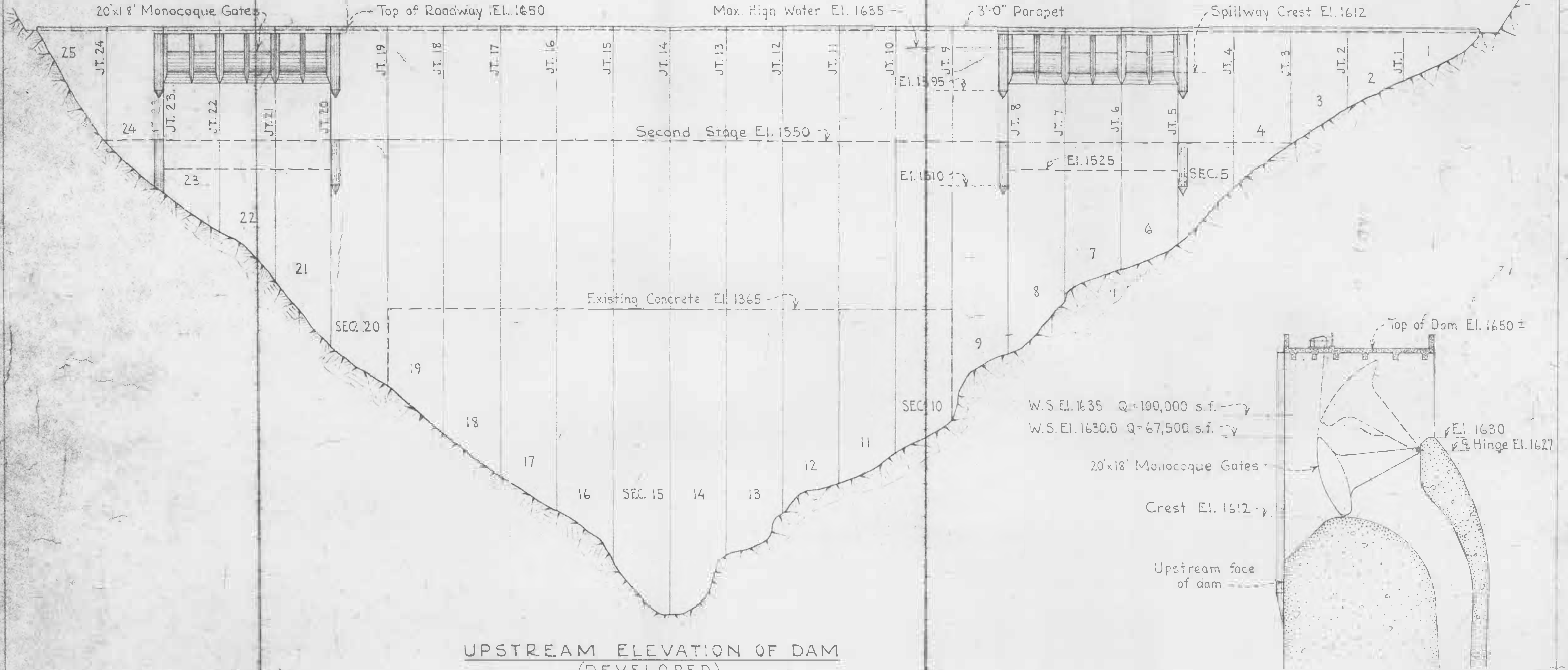
Information obtained during the course of the tests indicated that the spillway channels be lined throughout their entire lengths and because of the high cost of excavation be designed with economic balance between cut and fill as a major consideration. A cost of $8.00 a yard for excavation and $16.00 a yard for concrete fill was assumed, and estimates were made for a number of schemes in which the slope and the direction of the channel planes were varied. During the course of these studies it was found economical to shift the right spillway from its location between joints 4 and 7 to that between joints 5 and 8, as shown on figure 21. In addition to decreasing the cost, this change resulted in a symmetrical spillway layout, thus making it possible for the jets from the two channels to directly oppose one another in the river, improving the efficiency of dissipation.

FIGURE 20



ROSS DAM
SPILLWAY DISCHARGE CAPACITY
OVERFLOW SECTIONS 7 AND 8

FIGURE 21



UPSTREAM ELEVATION OF DAM
(DEVELOPED)

SECTION THRU SPILLWAY

ROSS DAM
SPILLWAY LOCATIONS - E.L. 1612

After determination of the most economical channel positions, slopes, fillets, and wall locations, each spillway was operated at discharges ranging from 200 to 50,000 second-feet while the maximum observed water surfaces were drawn on the channel walls. The dribbling action of the water from the deflector hoods, at the extremely low discharges previously discussed, dictated the heights of the upper portions of the walls, while the downstream heights were determined primarily at maximum discharge. The wall heights as finally determined consisted of the measured depths of flow at the walls plus five feet of freeboard to accommodate spray, splash, and the insufflation of air in the water. The final spillway locations, channels, wall positions, and wall heights are shown on figure 22. It is recommended that the right spillway channel be excavated on a 1.1 to 1 slope and the left spillway channel on a 1.2 to 1 slope, both measured normal to the line of centers. The alinement of the downstream walls of both spillways is shown symmetrical about the line of centers. This is not true of the upstream walls as these varied to compensate for the difference in slope of the two channels. The top surfaces of the channel walls were first shaped normal to the inside faces of the walls and later beveled so that they were parallel with the channel floor. The latter was the better from an observer's viewpoint. Photographs of the final spillways discharging at 75,000 second-feet are shown on figures 23, 24, and 25. Attention is directed to the quiet nature of the water at the base of the dam. Loose material remained unaffected by the spillway discharge, and no scour occurred at this point.

The model river channel was constructed according to the soundings submitted to this office on City of Seattle drawing No. D-13122. The extent to which the river channel will scour as a flood approaches sizable proportions is uncertain, but the model indicated that the deeper this occurs, the more desirable will be the flow in the river and the more effective the energy dissipation. If no provision were made for the dissipation basin at the foot of the spillways, the first sizable flow would excavate that portion of the river to bedrock, depositing much of the movable material in the river channel downstream. It would then be necessary to remove a large portion of this deposit from the river. It is estimated that bedrock lies 60 to 80 feet below the present river bottom and that this depth will be required for efficient operation of the dissipation basin. The pool in the model, as shown on figures 23, 24, and 25, extended down to elevation 1125. As long as it would be necessary to remove a large portion of the deposited material from the river after the first flood, it may prove more economical to excavate for the dissipation pool during construction. If the latter method is followed, the pool could be widened, which is desirable, and any irregularities of the underlying rock removed. The extent of the pool as used in the model is outlined by the dotted contour lines in the river on figure 22. For efficient dissipation the basin should be no smaller than that indicated on the above figure, but preferably deeper. Deep scouring was limited to the area shown. Deposition rather than scour will occur downstream from the pool.

HYD-125

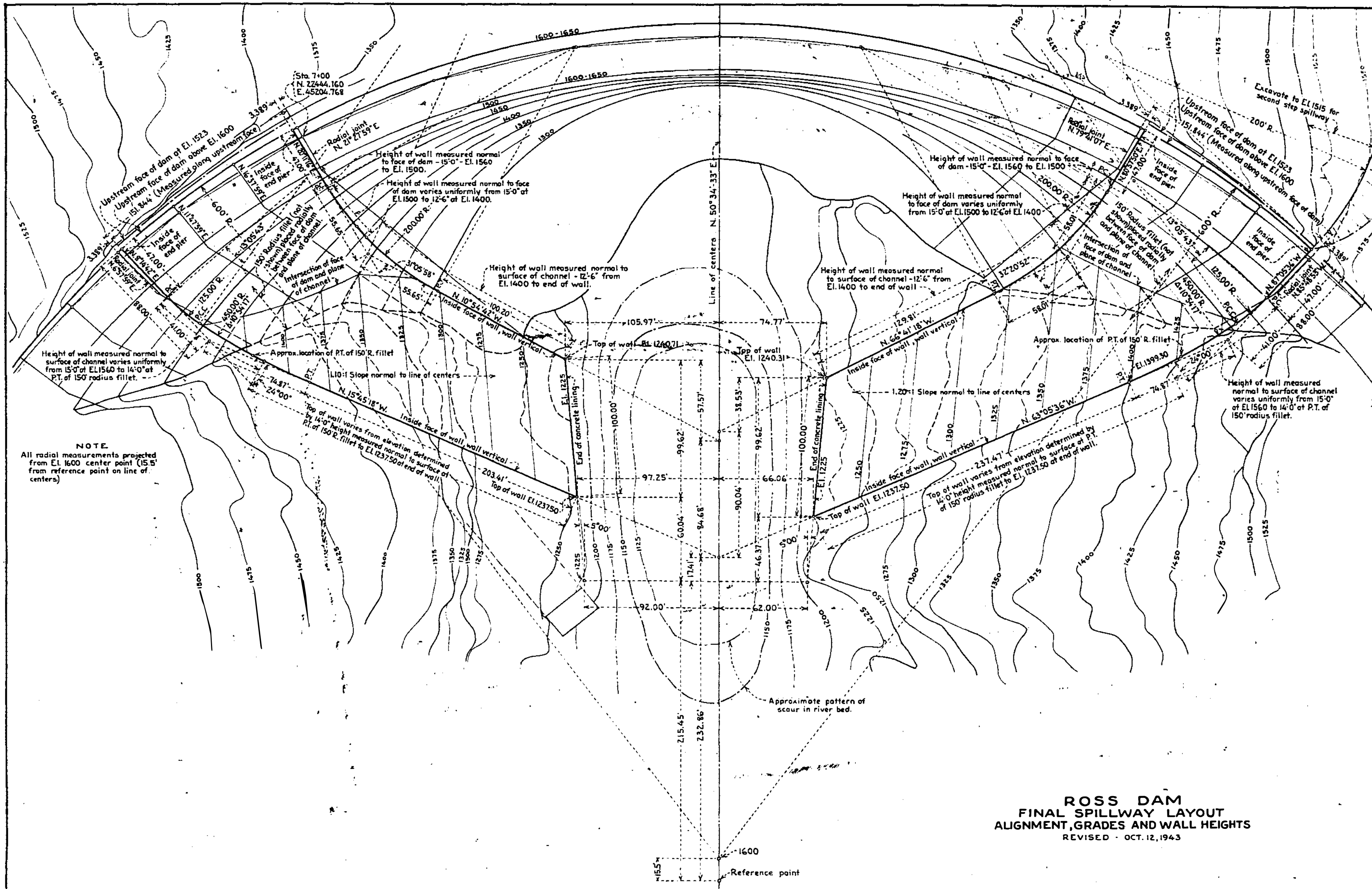


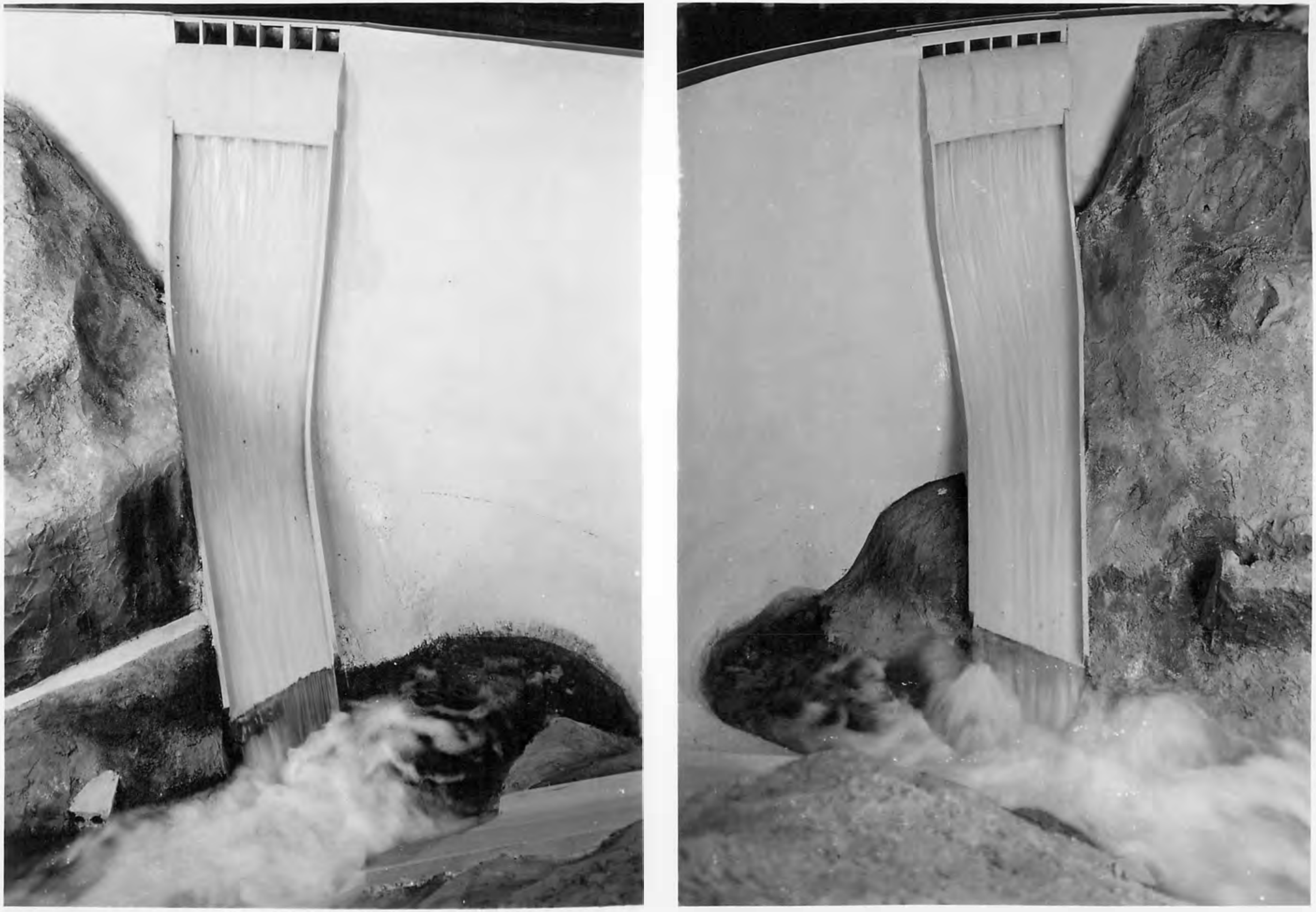
FIGURE 22

FINAL SPILLWAY DESIGN - DISCHARGE 75,000 SECOND-FEET

FIGURE 23

FIGURE 24

FINAL SPILLWAY DESIGN - TOTAL DISCHARGE 75,000 SECOND-FEET



A- Right Spillway

B- Left Spillway

FINAL SPILLWAY DESIGN - DISCHARGE 37,500 SECOND-FEET PER SPILLWAY

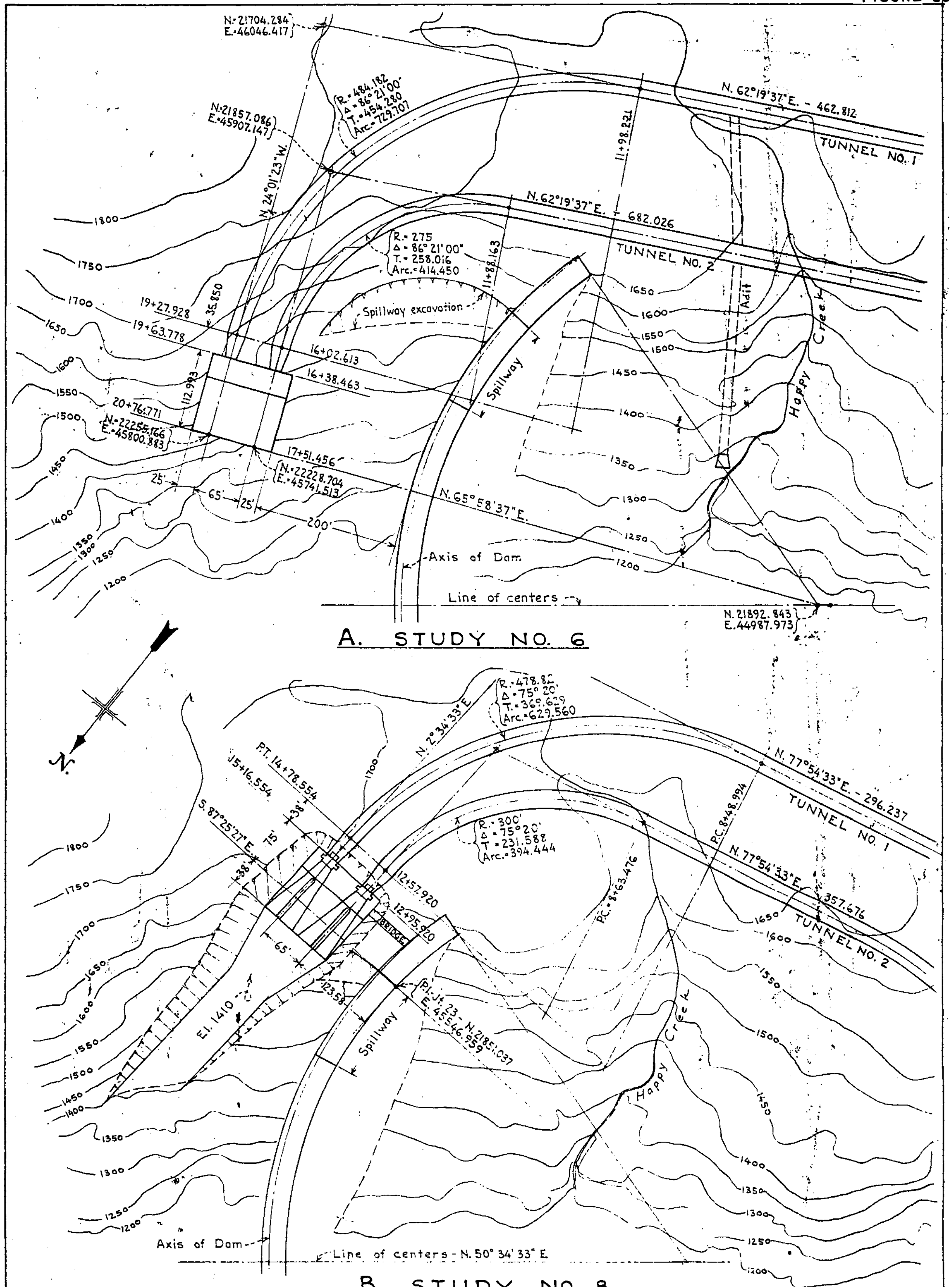
FIGURE 25

14. Location of power tunnel intakes. Of some eleven layouts made on the intake structures and tunnels leading to the powerhouse, three of the most desirable were installed in the model. The purpose was not to study flow through the intake structure but to observe the action of the water as it flowed past the structure when the left spillway was in operation. The extent of the intake structure so far as the model was concerned was merely a rack with side walls, where necessary, located as specified in the various studies. In no case did flow exist through the intake as the power tunnels were not completed. The intakes were incorporated in the model merely to make certain that adverse flow conditions would not exist in the vicinity of this structure during flood discharge over the spillway. The first intake investigated, study 6, is shown in plan on figure 26A. Very little cut was required for this design as the intake structure projected out from the canyon as shown in photograph A, figure 27. The head box was of insufficient size to accommodate the entire rack structure; so only the portion that would fit in the box is shown. Figure 27 B depicts the flow in the vicinity of the intake for a discharge of 50,000 second-feet through the left spillway. Confetti is shown on the water surface to indicate the direction of flow. Incidentally, the flow currents are not correct in this case, due to the limitation in the size of the model head box. The photographs, however, do give some idea as to the appearance of the intake in this location.

Study 8, figures 26 and 28, shows the intake structure located in a large cut in close proximity to the dam. Figure 28 B and C indicates excellent flow conditions for both discharges of 25,000 and 50,000 second-feet through the left spillway. The intake offered no obstruction to spillway flow, and it appeared that the spillway would in no way interfere with intake flow. Hydraulically speaking, this layout was excellent. From an economic standpoint the cut was excessive. Structurally, it was decided that the intake and the power tunnels were too close to the abutment of the dam.

The third layout, study 11, was somewhat of a compromise between the two above schemes. The intake was moved upstream and set in a moderate cut in the canyon wall, as shown on figures 29 and 30A. Flow conditions near the structure are shown for discharges of 25,000 and 50,000 second-feet through the left spillway on figure 30 B and C. Again, little or no interference between spillway flow and intake was discernible. Of the three intakes tested, design 11 proved the most desirable both from a structural and an hydraulic standpoint. Some doubt remained, however, as to the merits of design 6 as it was not fairly represented in the model.

15. Tailrace conditions at powerhouse. As velocities were extremely high in the river for the larger spillway discharges, it was thought advisable to construct a model of the powerhouse to study tailrace conditions. The model powerhouse was located as for study 11, shown on figure 29. Although velocities in the river were high,



A. STUDY NO. 6

B. STUDY NO. 8

ROSS DAM
POWER INTAKE STRUCTURES
DESIGNS 6 AND 8

10-12-43

A- Portion of Intake Structure **and Left** Spillway

B- Discharge 50,000 Second-Feet through Left Spillway

POWER TUNNEL INTAKE- STUDY 6

A- Intake Structure and Left Spillway

B- Discharge 25,000 Second-Feet through
Left Spillway

C- Discharge 50,000 Second-Feet through
Left Spillway

FIGURE 28

POWER TUNNEL INTAKE - STUDY 8

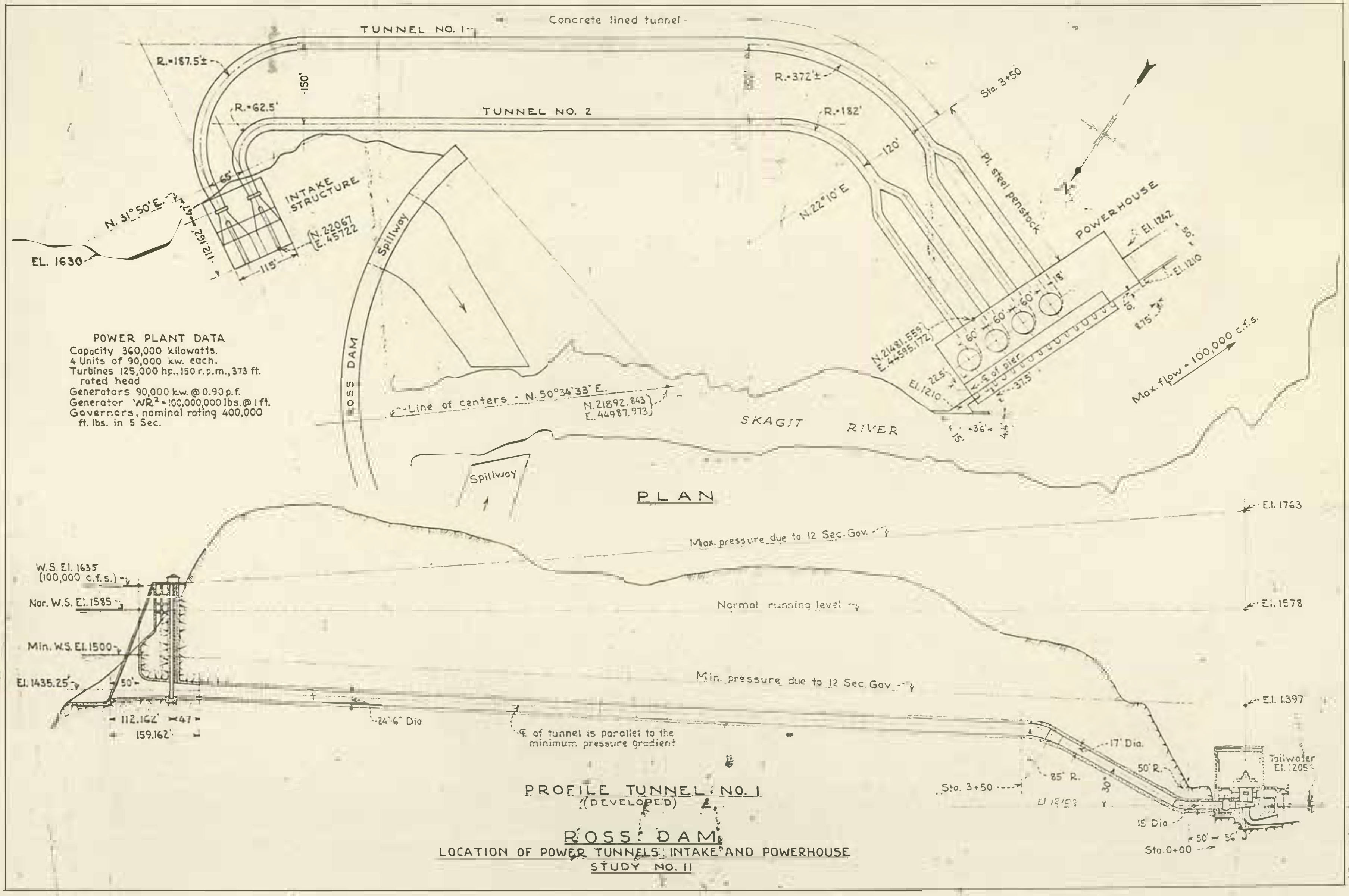
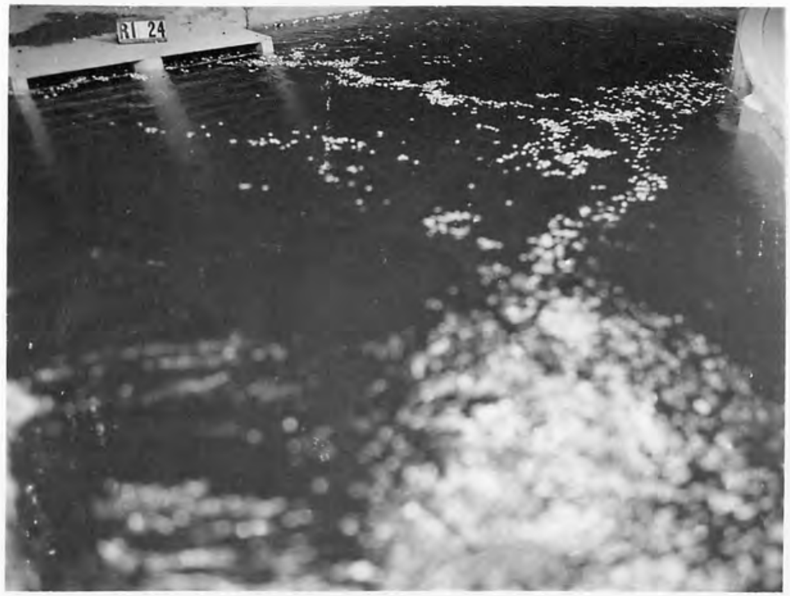

TUNNEL NO. 1
Concrete lined tunnel
R.=187.5'±
150'
R.=62.5'
TUNNEL NO. 2
R.=372'±
R.=182'
Sta. 3+50
120'
Pl. steel penstock
N. 22°10' E.
INTAKE STRUCTURE
N. 31°50' E.
65'
47'
112.162'
115'
N. 22067
E. 45722
EL. 1630
Spillway
POWERHOUSE
El. 1242
50'
El. 1210
18'
60'
60'
60'
6'
8.75'
N. 21481.559
E. 44595.172
22.5'
El. 1210
℄ of pier
37.5'
15'
36'
4.5'
Max. flow = 100,000 c.f.s.
POWER PLANT DATA
Capacity 360,000 kilowatts.
4 Units of 90,000 kw. each.
Turbines 125,000 hp., 150 r.p.m., 373 ft. rated head
Generators 90,000 k.w. @ 0.90 p.f.
Generator WR² = 100,000,000 lbs. @ 1 ft.
Governors, nominal rating 400,000 ft. lbs. in 5 Sec.
ROSS DAM
Line of centers - N. 50°34'33" E.
N. 21892.843
E. 44987.973
SKAGIT RIVER
Spillway
PLAN
El. 1763
Max. pressure due to 12 Sec. Gov.
W.S. El. 1635 (100,000 c.f.s.)
Nor. W.S. El. 1585
Normal running level
El. 1578
Min. W.S. El. 1500
El. 1435.25
50'
Min. pressure due to 12 Sec. Gov.
El. 1397
112.162'
47'
159.162'
24'-6" Dia
℄ of tunnel is parallel to the minimum pressure gradient
17' Dia.
50' R.
Tailwater El. 1205
85' R.
30'
Sta. 3+50
El. 1219.3
15' Dia
50'
56'
Sta. 0+00
PROFILE TUNNEL NO. 1
(DEVELOPED)
ROSS DAM
LOCATION OF POWER TUNNELS, INTAKE AND POWERHOUSE
STUDY NO. 11

FIGURE 29

A- Intake Structure and Left Spillway

B- Discharge 25,000 Second-Feet through Left Spillway



C- Discharge 50,000 Second-Feet through Left Spillway

POWER TUNNEL INTAKE - STUDY 11

FIGURE 30

the water surface adjacent to the powerhouse was surprisingly quiet in comparison. For 100,000 second-feet of flow in the river, the waves, measured from crest to trough, were approximately 8 feet high along the front face of the powerhouse. The addition of a wall at the upstream end and a pier at the downstream end of the powerhouse improved tailrace conditions in general. The maximum wave height in this case was reduced to approximately five feet. The dimensions of the upstream wall and the additional downstream pier are shown on figure 29. Photographs of the tailrace for discharges of 100,000, 75,000, 50,000, and 25,000 second-feet in the river are shown on figure 31. Tailwater conditions for river discharges of 50,000 second-feet and less were extremely satisfactory. Even for discharges in excess of 75,000 second-feet no major difficulties in power generation should be encountered.

A rather interesting fact was witnessed throughout these tests. Although the water surface in what represented Diablo reservoir was maintained at elevation 1205 for all river discharges, the average water surface of the tailrace adjacent to the powerhouse was consistently two feet lower, or approximately elevation 1203. This phenomenon is not uncommon. It should be stated, however, that in no case was there any water discharging into the tailrace from the powerhouse.

16. Spillways for second step dam. At the conclusion of the tests on the dam to elevation 1650, the model was revised to represent the second step dam. The overflow sections were projected vertically downward with crests at elevation 1525 so as to utilize the same channels for the second step spillways. Since the thickness of the dam at elevation 1525 was greater than at 1612, it was possible to design an overflow section with vacuumless profile for discharges up to 44,000 second-feet per spillway, which is shown dimensioned on figure 32. This shape was obtained from reliable, previously determined experimental data;[1] thus pressure measurements were omitted in these tests.

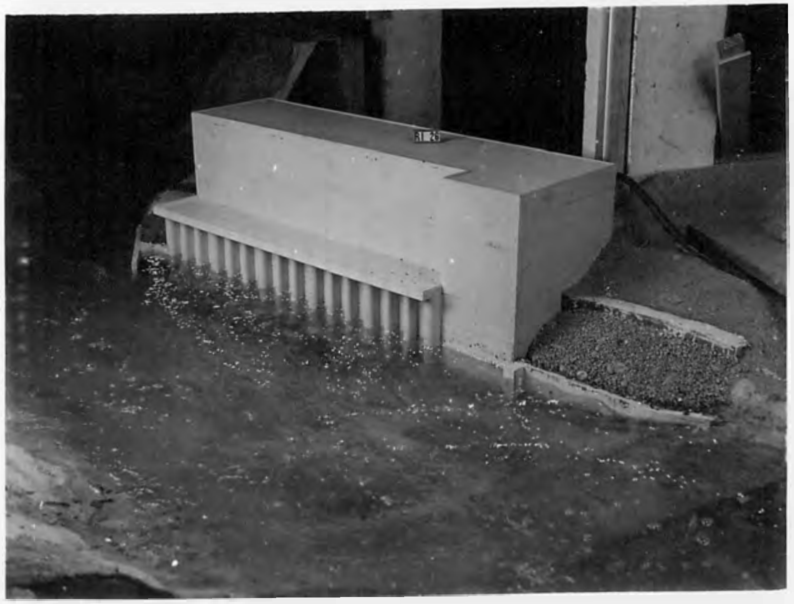
[1] Abstract of "Studied of Crests for Overfall Dams," Bulletin 3, Part VI, Hydraulic Investigations, Boulder Canyon Final Reports, HYD-118 by J. N. Bradley, December 31, 1942.

The overflow shapes were blended into the plane of each channel by a warped surface, the radial elements of which were tangents connecting the overflow profile and the channel slopes, as shown on figure 32. The warped surfaces will not be alike for the two spillways as the dam face and the channel downstream are both steeper on the right side. The overflow sections for the two spillways will be free crests, each 143 feet in length, without piers or gates. The channels downstream from the connecting warped surfaces will be the same as for the higher dam, with wall locations and wall heights identical for both stages of the dam.

A- Discharge 100,000 Second-Feet

B- Discharge 75,000 Second-Feet

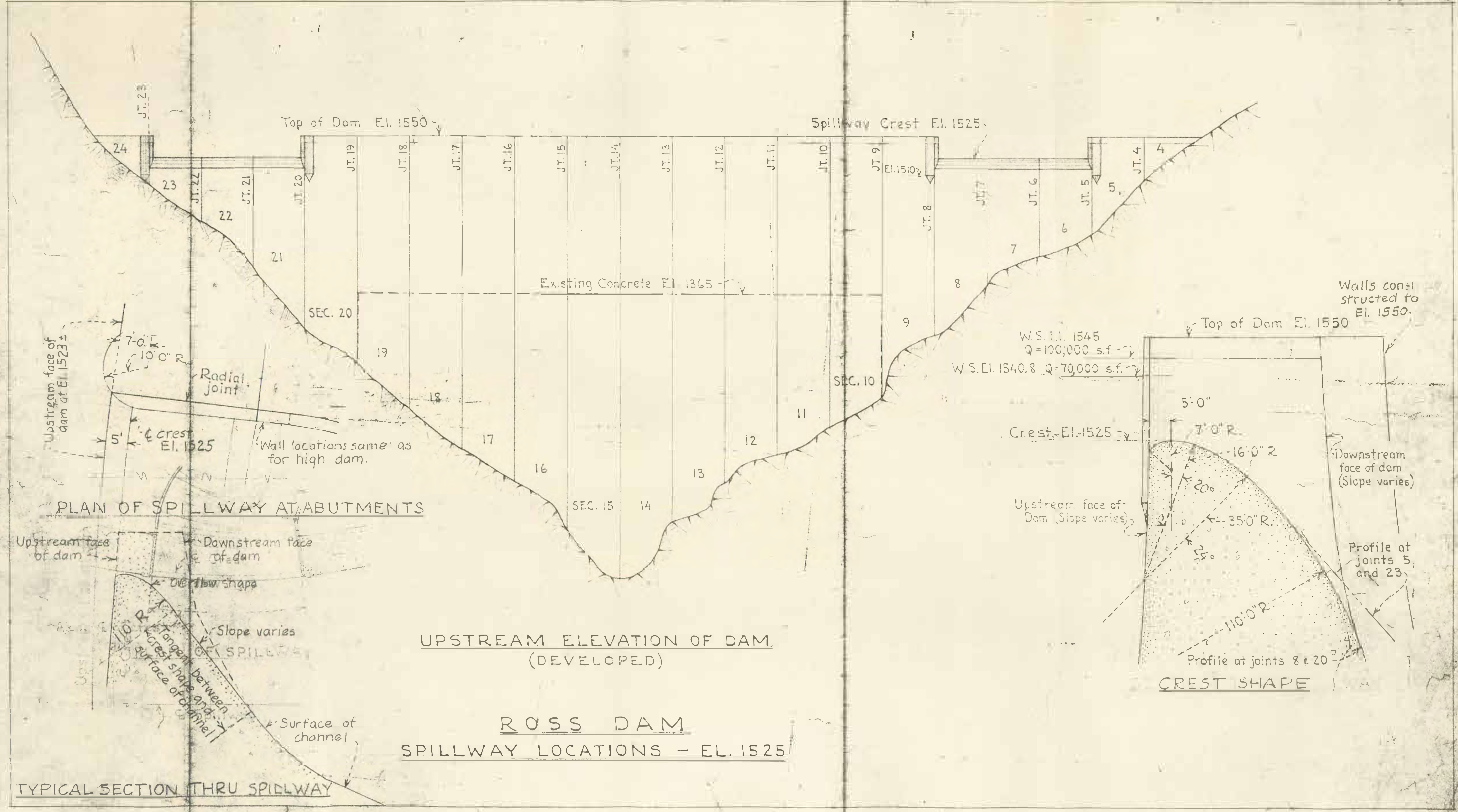
C- Discharge 50,000 Second-Feet



D- Discharge 25,000 Second-Feet

TAILRACE CONDITIONS AT POWERHOUSE

FIGURE 31



ROSS DAM
SPILLWAY LOCATIONS – EL. 1525

With the above in mind, details of the spillway channels for the second step dam can also be obtained from figure 22. The approach depth to the right spillway for the second step construction is adequate since this spillway was shifted toward the center of the dam and no excavation is necessary on this side. To provide a uniform approach to the left spillway a bench was excavated to elevation 1515 on about a 200-foot radius, as shown on figure 22. Photographs of the topography upstream from the spillways are shown for the right and the left sides on figure 33 A and C, respectively. Figure 33 B and D depicts the flow approaching the right and the left spillways for a discharge of 37,500 second-feet per spillway.

Flow conditions were generally satisfactory for both spillways for the lower flows but had a tendency to concentrate toward the downstream side of the channels as the flow increased. Flow conditions were better on the right spillway where the approach depth was greater and in all probability could be improved at the left spillway by increasing the approach excavation on that side.

Photographs of the second step spillways, as recommended, are shown in operation for a discharge of 37,500 second-feet per side on figures 34 and 35. It was fortunate indeed that the same channels and walls could be utilized for the spillways on both stages when it is considered that the velocities involved were materially different for each stage.

A photograph of the scour in the river produced by spillway operation is presented as figure 36. The depth of scour in this case was limited to elevation 1125, which was the elevation of the floor of the model. Some of the scoured material is shown in a more or less permanent deposit downstream while the loose material at the base of the dam remained undisturbed.

17. Power tunnel intake (second step dam). The flow was observed adjacent to the power tunnel intake (study 11, figure 29) with the second step spillways in operation. Although higher velocities than for the elevation 1650 dam were involved, no adverse conditions were noticeable for any discharge throughout the spillway range. Photograph C, figure 33, presents the general appearance of the intake with respect to the second step dam, and photograph D shows a flow of 37,500 second-feet approaching the left spillway.

18. Spillway discharge curves for second step dam. Head-discharge and discharge coefficient curves for the second step free-crest spillway (figure 32) are shown plotted on figure 37.

- - -

A- Approach Topography

B- Discharge 37,500 Second-Feet per Spillway

Right Spillway

C- Approach Topography

D- Discharge 37,500 Second-Feet per Spillway

Left Spillway- Power Intake Study 11

APPROACH CONDITIONS TO SECOND STEP SPILLWAYS

FIGURE 33

SPILLWAYS FOR SECOND STEP DAM - CRESTS AT ELEVATION 1525.0
DISCHARGE 37,500 SECOND-FEET PER SPILLWAY

FIGURE 34

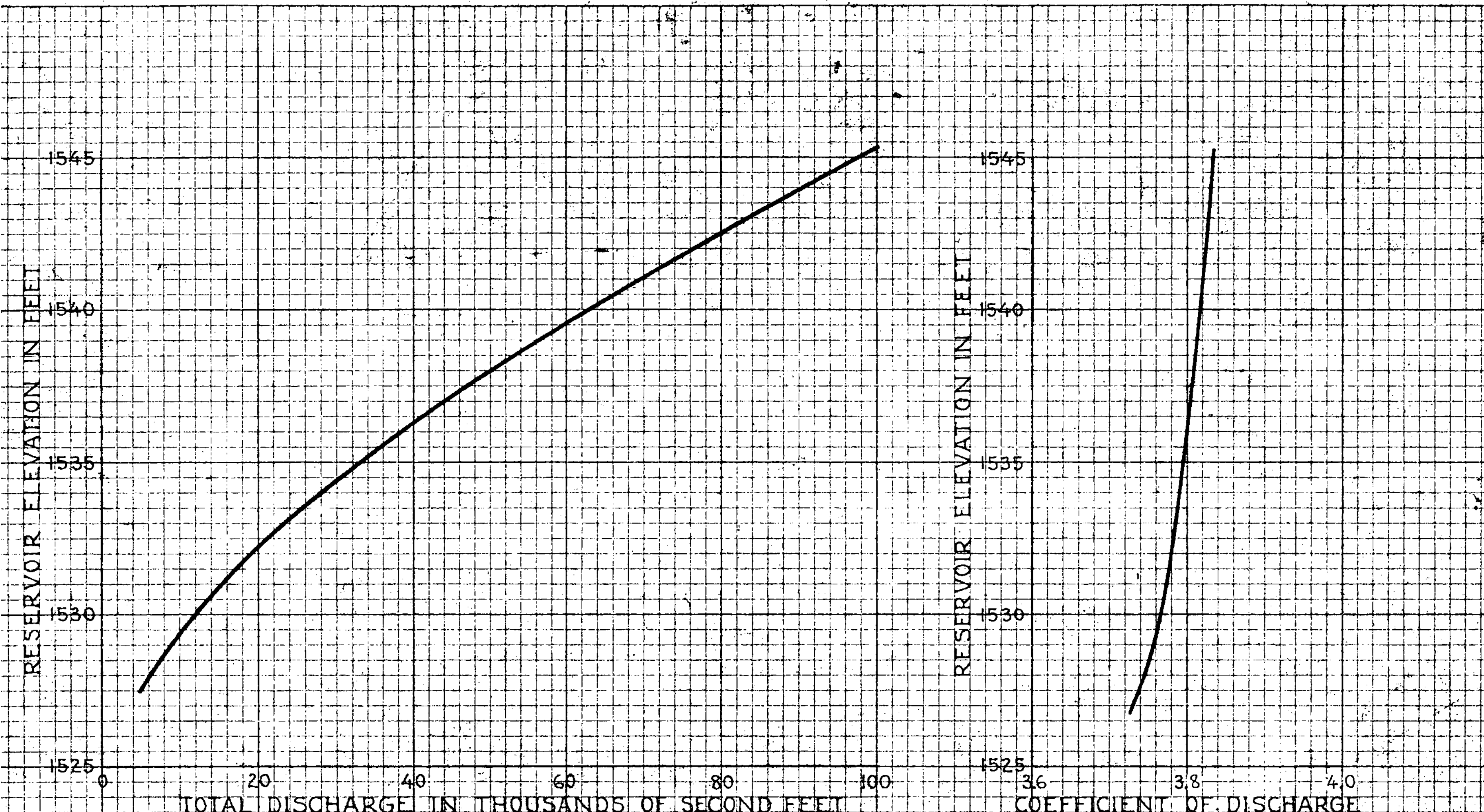
A- Right Spillway

B- Left Spillway

SPILLWAYS FOR SECOND STEP DAM - CRESTS AT ELEVATION 1525.0
DISCHARGE 37,500 SECOND-FEET PER SPILLWAY

FIGURE 35

PATTERN OF SCOUR IN RIVER AFTER A TOTAL FLOW OF 100,000 SECOND-FEET



ROSS DAM

SPILLWAY CAPACITY CURVES

SECOND STEP SPILLWAY

CRESTS AT ELEV. 1525.0

FIGURE 37