207

~~A.J. Peterka~~
Burgi

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

ROSS DAM
HYDRAULIC MODEL STUDIES ON THE SPILLWAYS AND THE HOWELL-BUNGER VALVE HOODS AND A PROPOSED ICE PREVENTION SYSTEM

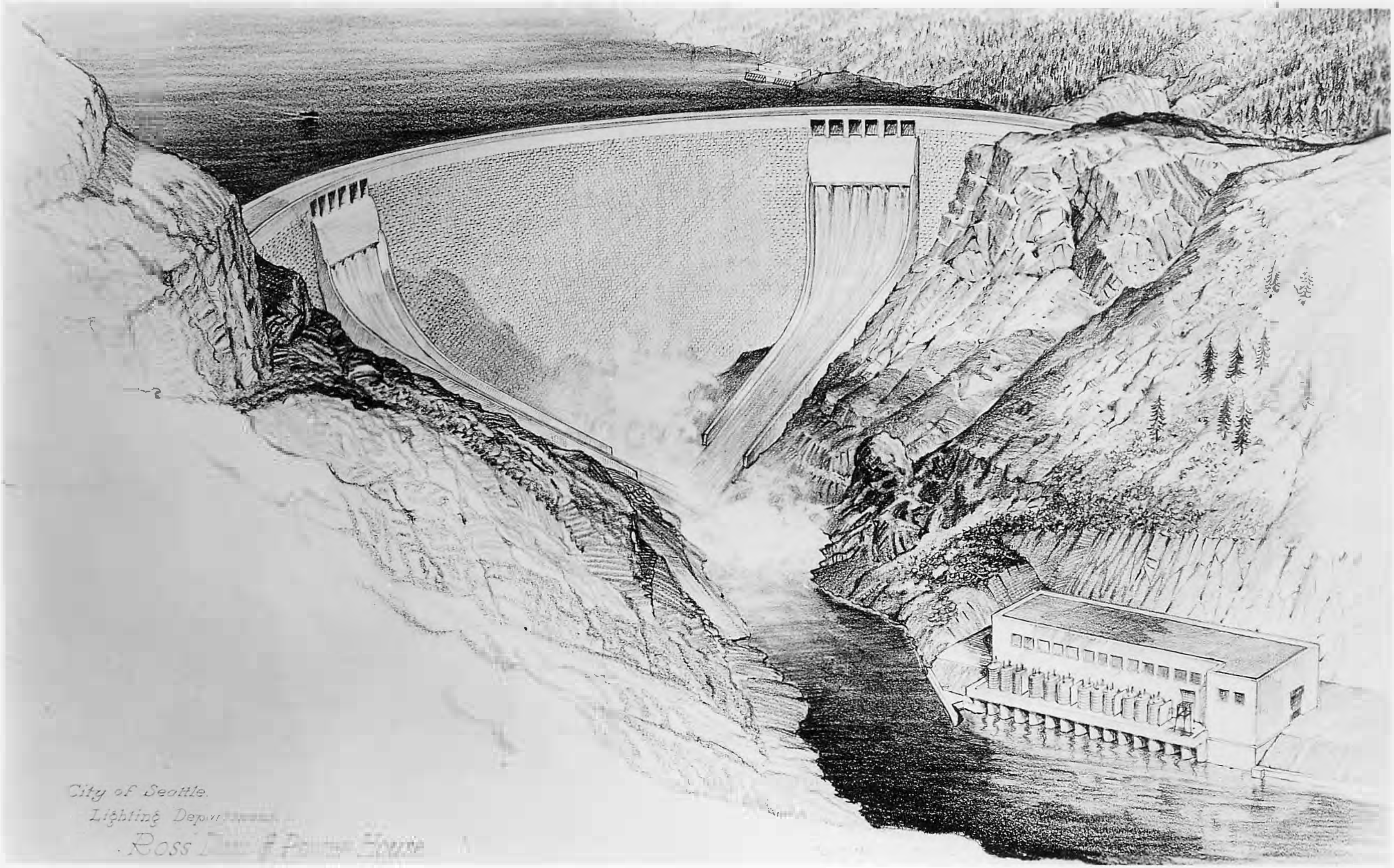
SKAGIT PROJECT, WASHINGTON

Hydraulic Laboratory Report Hyd-207

ENGINEERING AND GEOLOGICAL
CONTROL AND RESEARCH DIVISION

BRANCH OF DESIGN AND CONSTRUCTION
DENVER, COLORADO

AUGUST 21, 1946


City of Seattle.
Lighting
Ross

CONTENTS

CONTENTS (continued)

LIST OF FIGURES

LIST OF FIGURES (continued)

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

Branch of Design and Construction
Engineering and Geological Control
and Research Division
Denver, Colorado
August 21, 1946

Laboratory Report No. 207
Hydraulic Laboratory
Compiled by: A. J. Peterka
Reviewed by: J. E. Warnock
and J. N. Bradley

Subject: Hydraulic model studies on the spillways and the Howell-Bunger valve hoods and a proposed ice-prevention system for Ross Dam, City of Seattle, Washington.

SUMMARY

Spillway Studies

The spillway studies described in this report were made to determine the elements of a spillway crest and deflector for the Ross Dam with spillway crest at elevation 1582 and to revise the Stage 3 proposals where necessary to insure good spillway performance under all conditions of discharge.

Results and recommendations contained herein are based on tests of hydraulic models constructed to appropriate scales. A 1:30 scale model of part of one spillway was used to determine the crest and deflector shapes and other details. A 1:80 scale model of the entire arch dam, including the spillways, elevation 1340 outlets, and the downstream topography was used to study the overall performance of the spillways and chutes.

A spillway crest and deflector were developed, Overflow Section 9, Figures 8 and 9, which when used together, prevented excessive negative pressures on the spillway face without causing excessive positive pressures on the deflector. The spillway crest was then modified slightly, Overflow Section 10, Figure 22, so that it joined the partially complete Stage 3 spillway chutes, and no changes in the finished portions of the chutes are

necessary. The chutes were made shorter than originally proposed for Stage 3, Figure 11, because poor rock foundation was encountered in the field.

A change in the upstream pier nose radius at the ends of the spillways was found to reduce the end contractions, particularly at high discharges, and the downstream ends of the piers were streamlined where they project beyond the end of the deflector, to prevent fin formation and resulting spray on the face of the spillway. The piers beneath the deflector on the right spillway were curved in plan to improve the distribution of flow on the spillway chute. These details are shown on Figures 8 and 9. If straight piers are used in the prototype in place of the curved piers, it will be necessary to raise a portion of the outside training wall as indicated by Figure 16.

Erosion tests indicated that considerable material will be eroded at the base of the spillway chutes, but none at the base of the arch dam. Thus the safety of the dam is assured for all discharges. Extensive erosion studies on Schemes A, B, and C were made to aid in determining the best method of riverbed and spillway chute protection. Final decision as to the method to be used rests with the City of Seattle and cannot be given here. A discussion of these tests is given on pages 11 to 14.

The best order of opening the spillway gates was investigated to prevent overtopping of the training walls when only a few gates were open. The best order was found to be 4, 3, 2, 1, 5, 6. Gates on each spillway are numbered consecutively, from near the center of the arch and continuing outward. Figure 20 shows the necessity for following this schedule.

A spillway rating curve was determined from a single gate on the 1:30 scale model and multiplied by 12 to give the total discharge for both spillways, Figure 21. The free discharge curve shows that the spillways will pass the maximum discharge, 127,000 second-feet, with the reservoir at elevation 1608.

Pressure measurements were made on the final spillway shape, Overflow Section 10, Figure 22, to obtain data on the model spillway and deflector which would be strictly comparable with prototype measurements to be obtained later. Figures 23 and 24 show the location of the piezometers in the model and the pressures obtained for a range of discharges.

Howell-Bunger Valve Hood Studies

Studies to determine a satisfactory hood for jet control on the Howell-Bunger valves were made on a 1:12 scale model. A model of the 72-inch prototype valve was constructed and tested in combination with three different hood schemes. Tests were made over a considerable range of discharges and valve openings for heads up to 410 feet. Of the four schemes considered, Hood D, Figure 27, was found to be satisfactory in that it was effective in controlling spray from the valves and was not subject to cavitation damage, since all pressures measured inside the hood were positive, Figure 29. There was no unbalanced thrust on the valve since the hood was an independent unit and was in no way physically connected to the valve.

Ice-prevention System

As a result of the tests made by the Hydraulic Laboratory of the Bureau to determine the effectiveness of the air-lift system of ice prevention for Grand Coulee Dam, the laboratory was requested to determine the essential features of an ice-prevention system for installation at Ross Dam. Studies of the Grand Coulee system were made and some of its features adapted for use at Ross Dam. Modifications were made where necessary, based on experiences at other dams, and the resulting proposed system is shown on Figure 30. This schematic diagram should be considered as a preliminary proposal rather than a final design.

These investigations were conducted by the personnel of the Hydraulic Laboratory of the Bureau of Reclamation. Descriptions of tests and the results obtained are reported herein as the final report.

PART I - SPILLWAY STUDIES

Introduction

Ross Dam is located on the Skagit River in the State of Washington about 3-1/2 miles upstream from Diablo Dam and about 7 miles upstream from Gorge Dam, Figure 1, all of which constitute projects undertaken by the City of Seattle. Power projects are in operation in connection with the latter two dams, and a powerplant is also proposed at Ross Dam.

It was planned originally to build Ross Dam in four stages, with reservoir surfaces of approximately elevations 1365, 1550, 1635, and 1725, respectively. These were called Stages 1, 2, 3, and ultimate, respectively. The dam, a thin arch-type structure, was completed to Stage 1, Figure 2, before the Hydraulic Laboratory of the Bureau of Reclamation was authorized to make model studies of the spillways of Stages 2 and 3 in 1943. The results of the tests on Stages 2 and 3 are contained in Hydraulic Laboratory Report No. 136.

In Stages 2 and 3, Frontispiece, the dam was to be increased in height but not thickened. In the ultimate stage, the dam was to be increased in height and also thickened. Construction of Stage 3 had begun when trial load studies indicated that the stresses in the thin arch of Stage 3 were higher than was thought desirable. As a result, this stage was modified and a section used with the spillway crest at elevation 1582 rather than at elevation 1612 and normal reservoir surface elevation 1600. With this section, it would not be necessary to thicken the dam. At the same time, on the basis of recommendations by the Federal Power Commission, the spillways were designed for a maximum discharge of 127,000 second-feet instead of 100,000 second-feet considered for the Stage 3 model tests.

Since construction of the Stage 3 spillway chutes was well under way in the field when this investigation was started, certain obvious modifications in the spillway chutes could not be made since it was necessary that the new spillway crests and chutes join the already constructed portions of Stage 3. Instead, modifications were made at the upper end of the spillways where construction had not been started. These were found to be satisfactory.

Spillway Design

General

With the spillway crest lowered from elevation 1612 to elevation 1582, it was possible to provide a flatter crest, since the arch ring is thicker at elevation 1582 than at elevation 1612, the crest elevation for Stage 3. The thickness of the arch and the head on the crest made it possible to design the crest and deflector so that the theoretical pressures on the spillway face would not exceed minus 9 feet of water for any discharge, as compared to a minimum pressure of minus 17 feet for the earlier design. A deflector was necessary to prevent greater negative pressures, and for the larger discharges, to prevent the flow from leaving the spillway face altogether.

Practically all of the tests on both the 1:30 and 1:80 scale models were made on Overfall Section 9. This was the section recommended by the Bureau for prototype construction. However, construction in the field continued at such a rapid rate during the model tests that it was necessary for the City of Seattle to revise Overfall Section 9 slightly to fit it to the already constructed portion of the spillway. The differences in these sections are minor and for practically all purposes are the same. Rather than rely upon judgment alone, however, Overfall Section 10 was constructed to a scale of 1:30 and retested. Except for the distribution of pressure on the spillway face and deflector, the sections were identical in performance.

The Models

1:30 scale model. A 1:30 scale model of the proposed spillway and deflector was constructed with one full gate, on which spillway face and deflector pressures were measured, and a half gate on either side to insure proper flow conditions in the vicinity of the piezometers, Figure 4B. The spillway was made of concrete, the piers of wood, and the deflector and gate of sheet metal. The crest and hood were equipped with piezometers at close intervals above elevation 1515, Figure 4. The model was installed in the headbox of the 1:80 scale model for testing, Figure 5.

1:80 scale model. A second model, built to a scale of 1:80, was constructed of the entire arch dam. Figures 6 and 7 show the model as constructed, and Figure 5 shows the model limits and topography. Crest and deflector details were based on results of tests made on the 1:30 scale model, Figures 8 and 9. Other dimensions and details of construction were taken from field construction drawings supplied by the City of Seattle. The arch was constructed of wood and lined with sheet metal on the upstream side. The headbox 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 adjacent topography consisted of metal lath construction, faced with an inch of sand-cement surface coating. The metal lath was held in place by wooden supports. The box below the dam was also lined with sheet metal above the maximum tailwater level. The spillway crest was cast in rich concrete mixture and finished to a smooth surface. The deflector was cast in the same mixture and fastened to the wood piers. The spillway gates were made of sheet metal. The 72-inch outlets at elevation 1365 consisted of lengths of 1-inch pipe, reduced to the proper size at the outlet. These outlets operated only wide open, and no attempt was made to investigate partial valve openings.

Spillway and Deflector Tests, Overfall Section 9, 1:30 Scale Model

Design. From considerations of the head, discharge and thickness of the arch, it was impossible to provide a crest shape that would give satisfactory spillway face pressures for the larger discharges without the use of a deflector. In fact, without the deflector, the nappe sprung clear of the spillway for discharges above 50,000 second-feet. With the deflector in place, satisfactory performance for all discharges was obtained.

Using previously determined experimental data,1/ the spillway crest was designed to provide theoretical negative pressures along the spillway face, equal to 30 percent of head, or about minus 9 feet of water in the

---

1/ Abstract of Studies of Crests for Overfall Dams, Part VI, Hydraulic Investigations, Boulder Canyon Final Reports, Hyd.-118 by J. N. Bradley, December 31, 1942.

prototype. Negative pressures of this magnitude are not considered harmful. The spillway deflector was designed to hold the sheet of water against the spillway face for discharges above 50,000 second-feet, and at the same time prevent excessive positive pressures on the deflector. Details of the recommended crest and deflector shapes are shown on Figures 8 and 9.

Results of tests. Pressure tests indicated that for discharges between 20,000 and 127,000 second-feet over a free crest, the maximum negative pressure occurred at Piezometer 10, Figure 10. The maximum negative pressure was the equivalent of minus 10 feet of water prototype, and occurred for the maximum discharge. Pressures measured on the deflector for the same range of discharges showed a maximum of plus 12 feet of water for the maximum discharge. The complete set of pressures are shown plotted on Figure 10. In order that the deflector pressures not exceed the values shown, the horizontal distances between the spillway face and the deflector shown on Figures 8 and 9 should not be reduced. It may be seen that for 40,000 second-feet only the extreme bottom end of the deflector is in contact with the flow. For 127,000 second-feet the nappe is in contact with the deflector from about elevation 1585 to the bottom of the hood.

The fact that, in the model, the water first contacted the hood at about elevation 1585 does not mean that the prototype hood may be safely cut off at this elevation. In a small scale model, it is not possible to reproduce the insufflation, or bulking, which is known to exist to some degree in the prototype. As a result of this and other uncertainties, it is recommended that the deflector be extended above 1585 to elevation 1600. In the model, with the hood at elevation 1600, the structure was capable of discharging 140,000 second-feet without overtopping.

The spillway face pressures plotted on Figure 10 are minimum pressures obtained from a fluctuating column of water in the model manometer tube, while the deflector pressures are maximum.

Pressures for both the spillway and the deflector were also investigated for flow under a partially raised gate. These pressures were not recorded since maximum pressures on the deflector and minimum pressures on the spillway face were found to occur for discharges over a free crest.

The constriction at the end of the deflector, between the deflector and the spillway face, was incorporated into this design on the basis of previous tests which indicated that "dribbling," that otherwise occurred, was considerably reduced with the constriction in place.

Spillway Tests, Overfall Section 9, 1:80 Scale Model

General. The spillway and deflector shapes, Overflow Section 9, obtained from the 1:30 scale tests were incorporated into the 1:80 scale model, joined to the Stage 3 spillway chute, and further tests to determine the performance of the structure, as a whole, were made. Some refinements in design were found to be necessary, and these are discussed under appropriate headings. Figure 5 shows the model limits, Figures 8 and 9 the details of the spillway and chutes, and Figures 11 and 12 the layout of the spillways with respect to the arch.

Spillway pier noses. The effect of various radii on the upstream pier noses at the ends of the spillways was thoroughly investigated. The model was built with a 10-foot radius nose, as proposed for Stage 3, and considerable contraction was evident for high discharges with the gates raised free of the water surface. In an attempt to reduce the contraction, the radius of curvature was doubled, but only a small amount of improvement was evident. Other radii and arrangements were tried, and finally the curved addition on the upstream face of the dam was removed entirely and a 5-foot radius curve was made tangent to the face of the dam and the face of the pier, Figures 8 and 9. This arrangement proved entirely satisfactory and is recommended as being the best, consistent with cost. The radius shown is as large as can be used without interfering with the gate seal.

Deflector pier noses. The deflector piers were first installed with a square nose on the downstream end. Operation of the model at 50,000 second-feet or more showed that the sudden expansion of the jets, after leaving the deflector, caused a fin to form downstream from the square end, which, in the prototype, would cause needless spray on the open spillway face. Tapering the model piers, as shown on Figures 8 and 9, reduced the size of the fins until they were hardly discernible on the model, and the water surface for quite some distance below the piers appeared much smoother. This effect was particularly noticeable for 127,000 second-feet, Figure 6. Thus, the pier noses are essential in the operation of a vacuum crest.

Deflector pier curvature. An examination of the distribution of flow at the end of the right spillway chute showed that the spillway discharge was concentrated along the superelevated side of the chute. About one-third of the width of the chute was carrying approximately two-thirds of the total discharge. As a result, there was a tendency for the discharge to spill over the training wall about half way down the chute.

Since at the time of these tests the prototype chute and training walls were completed up to elevation 1490, it was not possible to change the curvature or superelevation of the chute, and improved flow conditions could be obtained only by making changes above elevation 1490. The possibilities for preventing overtopping of the downstream training wall were therefore very limited.

Improvement in the distribution of flow was evident along the entire length and at the end of the chute when the piers under the deflector hood of the right spillway were curved in plan as shown in Figure 9. Curving the piers also directed the water away from the superelevated side, and there was no tendency to spill over the training wall, Figure 13A. Distribution at the end of the chute was still not perfect, but the improvement was sufficient, from a hydraulic standpoint, to justify the installation of the curved piers.

Figure 16 shows the water surface profiles obtained with straight and with curved piers. If, for any reason, the curved piers are not constructed on the right spillway of the prototype, the training wall on the superelevated side of the right spillway chute, between elevations 1315 and 1395, should be raised to prevent flow over the wall for discharges above 100,000 second-feet. At least 2 feet of freeboard should be allowed over the water surface profile shown on Figure 16. For a discharge of 100,000 second-feet with the straight piers, the water surface profile is below the top of the wall at all points.

Further improvement might have been obtained by increasing the curvature of the piers, but it was found that this would reduce the capacity of the spillway, since the control in the bays that were made narrower would be moved from the gate to the end of the deflector. The bays that were made wider would not compensate for this reduction, since in those bays the control would remain at the gate.

Flow conditions on the left spillway chute were found to be better than those on the right chute. Some concentration of flow on the super-elevated side of the left chute was also evident, but the installation of curved piers was not considered necessary since there was no tendency for the flow to spill over the training wall, Figures 13B and 16.

Spillway chutes. During field construction of Stage 3, poor rock was found to exist at the ends of the spillway chutes. Accordingly, it was believed advisable to shorten the chutes to a point where sound rock was evident. It was planned to shorten each chute about 12 feet in plan, and this modification was incorporated into the model for testing. The ends of the model chutes before modification, however, were not built according to the plans given in the previously-mentioned Report No. 136. Instead of the uniform slope shown in the above report, the ends of the chutes were turned slightly upward with respect to the uniform grade. This feature had been incorporated into the City of Seattle drawings after Report No. 136 had been issued.

Tests on the shortened chutes with the upturned ends showed that, for all discharges, each jet was directed so as to cause excessive erosive action at the base of the opposite chute. Also, it was found that the action in the stilling-pool was slightly unstable, pulsating from side to side at regular intervals. It was believed that undermining of the structures would undoubtedly occur and that more stable operation could be obtained if the upturned lips were removed.

The lips were removed, and the uniform channel grade was maintained to the end of the chutes. Further tests showed that the pulsating condition in the stilling-pool then ceased to exist, and the jets intersected at the centerline of the channel bottom. Thus, the tendency toward undermining the structure was reduced and operation greatly improved, as indicated on Figures 6, 13, 14, and 15. The recommended layout for the spillway chutes is shown on Figure 11.

Since the ends of the chutes are some distance back from the steep rock walls of the stilling-pool, it may be necessary to excavate a channel through the rock from the end of each chute to the edge of the bluff. This excavation should be deep enough to prevent the underside of the jets from striking the exposed rock surface and sufficiently wide to provide aeration of the underside of the nappe. Any reduction from atmospheric pressure beneath the nappe would tend to depress the trajectory, and if a severe condition were encountered in the field, the performance of the stilling-pool might be affected.

Erosion tests--Scheme A. A study was made to determine the effects of the spillway discharge on an erodible, homogeneous riverbed below the dam. The model river channel was constructed according to the soundings submitted to this office on City of Seattle Drawing No. D-13122. The near-vertical sides of the channel were reproduced in mortar, and erosion of the sidewalls was not considered for these tests. The riverbed was levelled at elevation 1160 at the start of each erosion test, and the tailwater elevation was set at elevation 1205. Record tests were made with both spillways operating since this was the most serious condition. Figure 15 shows the operation of the stilling-pool with one spillway operating.

The material composing the erodible bed was a well-graded gravel which, in the prototype, would represent rocks from 6 inches to 5 feet in maximum dimension. The tests were run for 30 minutes, which would correspond to about 4-1/2 hours of prototype operation. The size of the bed material and the length of the tests are not intended to represent exactly the conditions that will obtain in the prototype. However, this information is presented to aid in interpreting the results of the erosion tests made on the model. Model tests, in general, are not conclusive as to the exact amount of material that will be moved or the depth to which erosion will progress. However, models do conclusively show where the erosion, if any, will occur and where depositions of eroded material, if any, will be made.

Erosion tests were made for discharges of 20,000, 40,000, 60,000, 80,000, and 127,000 second-feet. Photographs of the test results were made with 10-foot contour lines indicating the depth and extent of the erosion, Figure 17. It should be noted that no erosion occurred near the base of the arch dam. Maximum depths and yardages were determined for each test and are shown plotted against discharge on Figure 18.

The eroded material was deposited immediately downstream from the scoured hole and as the test progressed, moved downstream slowly. The top of the deposit was usually at elevation 1200 or thereabouts.

The effect of the elevation 1340 outlets was investigated during the erosion tests, Figure 6. The outlets were opened both with and without the spillways operating. Operation with the spillways showed that they had no measurable effect on the action in the stilling-pool. Operating alone, in spite of the concentration of flow, the action in the stilling-pool was very mild and caused less erosion than the spillways discharging 20,000 second-feet.

The bottom of the erodible bed in the model was at elevation 1110, being limited by the bottom of the outlet box. Had the gravel extended deeper, the erosion would have been deeper for discharges above 40,000 second-feet, as indicated by Figure 17. In determining the maximum erosion and the corresponding yardages for large discharges, the probable depth to which erosion would have progressed was used. This was found by projecting the existing slopes of the erosion hole until they met. For the 60,000-second-foot discharge, this method was believed to be accurate; for 80,000 second-feet, fairly accurate; and for 127,000 second-feet, purely an estimate.

The yardages given in Figure 18 do not necessarily represent the amount of erosion to be expected in the prototype, but do give a comparison of the relative amounts of erosion to be expected for various discharges.

It is known, however, that solid rock exists in the prototype at some unknown elevation beneath a layer of relatively loose material. The solid rock will erode slower than the loose material, and so some consideration should be given this condition. On the other hand, if the element of time is removed from consideration, or, in other words, if the prototype is operated for a sufficient length of time, the erosion will approach that indicated by the model, regardless of the type of material in the prototype river channel.

Erosion tests--Scheme B. From the erosion studies made on Scheme A, it was conclusively shown that no erosion was evident at the base of the arch dam for any discharge, and the safety of the dam is assured for all operating conditions. The greatest tendency toward erosion is in the center of the river channel near the downstream edge of the spillway chutes. A lesser tendency toward erosion is evident near the steep sidewalls of the channel. Thus, any progressive erosion that would tend to undermine the spillway chutes probably will not occur until the sidewalls have been exposed by the erosion of the channel bottom.

With the spillways in operation, it was evident that as erosion progressed, energy dissipation in the stilling-pool became more satisfactory. This led to the conclusion that it would be desirable to excavate and create a deep stilling-pool while construction equipment was still on the site. The City of Seattle felt that this would cost less than removing the eroded material from the channel at some later date after the construction equipment had been removed. This procedure, however, would expose the channel sidewalls to the immediate erosive effects of the first discharge that passed over the spillway, regardless of size. To reduce the possibility of undermining the chutes, it was suggested that a hole or sump be excavated downstream from the area where erosion would normally occur. Then, as erosion progressed, the eroded material would be deposited in the excavated sump, and the rock sidewalls in the vicinity of the spillway chutes would not be exposed to erosive action until some of the bottom material had actually been eroded and moved.

Tests on this scheme were made for a discharge of 60,000 second-feet. After an erosion test, similar to those made for Scheme A, the size of the eroded hole was carefully measured, then reproduced about 200 feet downstream from its original location, Figure 19A. The eroded hole was filled to elevation 1160, the discharge of 60,000 second-feet reset, and the test continued for 30 minutes. The excavated sump filled as expected, but the eroded hole below the spillway was considerably larger than for the first test. Compare Figures 17B and 19B. This was probably due to the fact that the excavated sump was too near the spillway, and the loose bed material was pushed, rather than eroded, out of place. This may or may not occur in the prototype, depending on the firmness of the material composing the riverbed. It was believed that more satisfactory results could be obtained with the sump moved farther downstream, and tests on Scheme B were abandoned.

Erosion tests--Scheme C. The test was then repeated, this time with the sump about 400 feet downstream from its original location, Figure 19C. As the test progressed, the eroded material was deposited, gradually, in the excavated sump. The final eroded hole, Figure 19D, was very similar to the one produced in the original erosion test, Figure 17B. However, more material was eroded for this test (80,000 cubic yards) than for the original (65,000 cubic yards), and the excess material was deposited above elevation 1160 in the downstream channel, thereby partially defeating the purpose of this scheme. Some advantage remained, however, since for discharges less than 60,000 second-feet or for short-duration discharges above 60,000 second-feet, the eroded material was not sufficient to fill the sump, and, consequently, no deposit was evident. At the same time, the area near the ends of the spillway chutes would not be subject to premature erosion.

If it is finally decided to excavate a stilling-pool or a sump while construction equipment is on the site, Figure 18 will be helpful in determining the amount of material to be removed after a study of costs, probable discharges, and character of bedrock are considered.

Spillway Operation

General

Operation of the model indicated that although it is preferable to operate both spillways together, either may be operated alone without causing excessive erosion. In operating either spillway, the procedure given below should be followed. The spillway rating curve, Figure 21, reservoir elevation versus discharge, may be used to aid in regulating spillway discharges.

Gate Operating Schedule

An investigation of the order of opening the spillway gates was made, and it was found that considerably better flow conditions are obtained if the proper sequence is followed in opening and closing the spillway gates. Certain gates, or groups of gates, produce flow conditions which are not desirable, and better conditions may be obtained by following the opening order 4, 3, 2, 1, 5, 6, regardless of the number of gates that are to finally remain open. The closing order should follow this sequence in reverse. The gates on each spillway are numbered consecutively from the center of the arch toward the abutments. Figure 20A shows the flow concentration caused by opening Gate 1. Figure 20B shows the improvement obtained when Gate 4 is opened first. Figure 20C shows the concentration of flow along the training wall and at the end of the chute with Gates 4, 5, and 6 open. Better distribution is evident with Gates 4, 3, and 2 open, Figure 20D. Similar studies of other combinations, not illustrated here, showed that the opening order given above was most satisfactory. The figures show the right spillway operating, but the test results are also applicable to the left spillway.

Spillway Rating Curve

A spillway rating curve, reservoir elevation versus discharge, for both spillways was obtained from the single bay of the 1:30 scale model. The single gate discharge, determined for Overfall Section 9 and checked for Overfall Section 10, was multiplied by 12 to give the discharge for the entire spillway. Two check points, for reservoir elevations 1600 and and 1608, made on the 1:80 scale model, showed reasonable agreement.

Figure 21 shows the spillway rating curve obtained. The spillway coefficient curve is also shown giving "C" in $Q = CLH^{3/2}$ in terms of reservoir elevation. In this equation, Q is the discharge in second-feet, L the length of crest measured between piers, and H the difference in elevation between the crest and the reservoir surface. At reservoir elevation 1600, top of gate, the maximum discharge was 70,500 second-feet. The maximum design discharge, 127,000 second-feet, occurs with the headwater at elevation 1608.

Exact discharges for partial gate openings could not be determined since the installation of the gates is part of a future program, and the gate details were not known. Gates will not be installed until the reservoir has filled or until shortages of gate material have been alleviated. The dashed-line curves of Figure 21 show approximate discharges to be expected for partial gate openings. These curves were determined using Overfall Section 10 and a Monocoque gate installed according to the trunnion location given for Overfall Section 8, Hydraulic Laboratory Report No. 136.

Model and Prototype Verification Tests--Overfall Section 10

General

While the model tests on Overfall Section 9 were in progress, construction of the prototype spillway chutes continued, following the plan for the Stage 3 layout. On July 22, 1946, while the 1:30 scale model of Overflow Section 9 was being tested, the prototype construction had reached elevation 1485. The drawings of Overflow Section 9, Figures 8 and 9 were made to tie in to the Stage 3 chutes at elevation 1490, but by August 23, 1946, the day the City of Seattle received these drawings, construction had already progressed to elevation 1500. Thus, it was necessary for the City of Seattle to modify Overflow Section 9 slightly to tie the two sections together. The minor changes made in the spillway shape were downstream from the crest, and they will not affect measurably the performance of the spillway. However, they were believed to be of sufficient importance to prevent a strict comparison of pressures between the model of Overflow Section 9 and the modified section being constructed in the prototype. Subsequent tests showed this to be true.

Since it is the practice of the Bureau to install necessary equipment in prototype structures to enable verification of model studies, the City of Seattle was requested, and they agreed, to install a series of piezometers in the left spillway face and deflector between Joints 22 and 23, Figure 8. The City of Seattle was requested also to submit to the Bureau the spillway section which they planned to actually construct in the vicinity of Joints 22 and 23, hereafter referred to as Overfall Section 10, so that a model of this section could be tested in the laboratory. Pressures measured on this model could then be compared with the pressures obtained at some later date on the prototype.

Pressure Measurements on Crest and Deflector

A model of Overflow Section 10, Figure 22, also referred to in the correspondence as Alternate 2, was constructed to a scale of 1:30, equipped with piezometers, as shown on Figure 24, and pressures on the spillway and deflector measured for a range of discharges with free flow over the crest. Visual investigation of pressures under a partially raised gate indicated that this condition was not as severe as for free flow over the crest. The results of the free flow tests are shown on Figures 23 and 24.

Pressures on the spillway face are shown on Figure 23 in feet of water, prototype, and are plotted against the piezometer number. For design purposes, the pressures are not significantly different than those for Overfall Section 9, differing only in degree. For discharges up to 15,000 second-feet, pressures on the spillway face were positive over most of the surface, becoming slightly below atmospheric in the region of Piezometers 9, 10, and 11. As the discharge was increased, more of the spillway was subjected to reduced pressures until for 127,000 second-feet, practically the entire face below the crest was below atmospheric pressure. The greatest negative pressure recorded was for 127,000 second-feet and was minus 6.4 feet of water at Piezometer 8. The recorded values are the average of at least three determinations, each determination being the lowest pressure indicated by the piezometer over a period of about 30 seconds.

Pressures on the deflector are shown on Figure 24. These tests also indicated results similar to those obtained for Overfall Section 9. The maximum positive pressure on the deflector was equal to 12 feet of water and occurred for the maximum discharge, 127,000 second-feet. Except for a positive "hump" in the pressure curve between elevations 1530 and 1545, the pressures increase more or less uniformly from the top to the bottom of the deflector for a given discharge. In general, the pressures also increase with the discharge. However, at a discharge of 40,000 second-feet, only the lower part of the deflector is in contact with the water.

The deflector pressures shown were obtained in the same manner as the spillway pressures except the maximum values of the pressures were used.

Considerable difficulty was encountered in obtaining consistent deflector pressure readings, especially for discharges below 127,000 second-feet. This was believed due to the variations in the flow pattern, causing "make and break" contact with the piezometer openings in the deflector. If the variations exist to the same degree in the prototype, it may be difficult to correlate model and prototype deflector pressures.

The pressures indicate that for design purposes Overfall Section 10 is the same as Overfall Section 9. For comparison of model and prototype pressures, however, differences were great enough to warrant making pressure tests on Overfall Section 10.

PART II - HOWELL-BUNGER VALVE HOOD STUDIES

Introduction

Previous to the tests described in this report, other tests on the Howell-Bunger valve installations at the Ross Dam were made by the Bureau's Hydraulic Laboratory and reported in Hydraulic Laboratory Reports Nos. 156 and 168. In these reports, the operating characteristics of the valves and the merits of two hoods were discussed. The hoods proposed by the S. Morgan Smith Company were unusable because of excessive unbalanced thrust, and the City of Seattle requested that additional tests be made to develop a workable hood for control of the jets below the valves.

The jet, emitting from the valve without a hood, spreads at an angle of about 45 degrees with the centerline of the valve and produces excessive spray, Figure 28A. In the vicinity of transformers or other electrical equipment, especially in cold weather when ice forms, this is very objectionable. The use of the hood will prevent excessive spreading of the jet and thereby reduces the spray, Figure 28B.

Hood Design

The Models

Tests to determine the efficiency of the hoods made it necessary to provide a valve, correct in every detail, to properly direct the discharge through the hood. The 6-inch model valve was made of bronze and machined to dimensions furnished by letter from S. Morgan Smith Company, dated March 2, 1945. A photostat of that letter is contained in Hydraulic Laboratory Report No. 168. The valve was constructed so that it was homologous to the prototype 72-inch valve and could be tested at any desired opening.

The valve, with a suitable length of conduit, was connected to a pressure tank which could supply water in varying quantities at heads up to 410 feet, prototype. The hoods were constructed of metal and were equipped with piezometers at points which were believed to be subject to reduced pressures.

Results of Tests

General. The criteria used to evaluate the desirability of a particular hood were (1) the effectiveness of the hood in preventing spreading of the jet after the discharge left the hood and (2) the pressures occurring inside the hood during operation. Observation of the models in operation was sufficient to determine that all hoods tested were effective in controlling the jet. Pressures obtained from piezometer readings were used to determine whether cavitation might occur in the prototype structure.

Hood A. This proposal was submitted by the City of Seattle on Drawing No. D-13132-12 and consisted of two vertical walls and a roof, enclosing both valves in a common structure. This hood was not tested since only one model valve was immediately available, and it would have been necessary to construct another model valve before the tests could be run. Before constructing another valve, it was considered advisable to test other proposals which made use of individual hoods for each valve.

Hood B. Hood B consisted of a cylinder 12 feet in diameter and 16 feet long, Figure 25, held in place by independent supports, and in no way connected to the valve. The jet issuing from the downstream end of the hood was quite satisfactory and was very effective in controlling the jet. However, the proximity of the hood periphery to the valve caused a considerable amount of water to be forced out of the upstream end of the hood, completely submerging the valve. During the winter months sufficient ice might form on the valve to make it difficult, if not impossible, to operate. Consequently, this design was abandoned.

Hood C. In this design, the hood was connected to the valve, making the hood and valve a complete unit similar to the original hood proposed by the S. Morgan Smith Company. Changes in the dimensions were made, Figure 26, to increase the pressures inside the hood. Under test conditions, this hood would be usable for prototype heads up to 200 feet. At that head, the maximum negative pressure inside the hood was minus 17 feet of water.

At this stage of the testing, the laboratory was informed that the valve was to be operated at heads up to 385 feet. This posed an entirely new problem, since at this head the pressures would be so low that cavitation would undoubtedly occur. The thrust would be increased, making it difficult, if not impossible, to close the valve after it had been opened.

Hood D--final design. As a result of the tests on Hood C, the physical connection between the valve and the hood was eliminated, and each valve was considered as an individual unit. Although only one valve and hood was constructed, in effect, a separate hood was provided for each valve. The hood was made as large as possible, taking into account the clearance between valves, and was anchored to a foundation located below the valves, Figure 27. This arrangement was a combination of the scheme used by the Tennessee Valley Authority and the proposal submitted by the City of Seattle, and proved satisfactory.

It should be noted, Figure 27, that the clearance between the model valve-sleeve flange and the inner ring of the hood is very small. The corresponding prototype clearance should also be kept to a minimum, taking into consideration bolt heads or other projecting surfaces that might affect this dimension. The diameter of the inner hood ring should not be changed, but the minimum clearance should be obtained by extending the flange of the valve sleeve. If, in spite of holding to minimum clearances, the prototype produces an excessive amount of flow between the valve sleeve flange and the inner hood ring, a seal flange could be placed on the valve sleeve, Figure 27, so that contact with the inner hood ring is maintained at all times. This should keep the area upstream from the hood dry at all times.

A complete set of pressures over the upper range of heads was obtained for this design. The piezometer locations and the operating pressures are shown on Figure 29. The pressures are expressed in terms of unit pressures $\frac{P}{H}$. To obtain the pressure for any given head, multiply the $\frac{P}{H}$ value for a particular piezometer and valve opening by the prototype head. This gives the pressure at the piezometer opening in feet of water, prototype.

The pressure curves of Figure 29 were obtained from tests in which the head varied from 173 to 410 feet prototype, and indicated that all pressures are positive over the entire operating range. Accordingly, no cavitation is expected in the prototype. Since the valve hood is independent of the valve, no unbalanced thrust can affect the normal forces necessary to open and close the valve. Since the hood is mounted on an independent foundation and is not dependent on the valve for structural support, the vibration of the entire structure should be considerably reduced.

The City of Seattle requested, through the Hydraulic Laboratory, that the Bureau check their proposal to use 1-inch thick steel plate for constructing Hood D. As a result, the Technical Engineering Analysis Section made a stress analysis of the hood, using the data from Figure 29. Their report, in the form of a memorandum from C. C. Crawford to A. J. Peterka, is given below:

"1. Introduction

The following study was made at your request to determine if one inch steel plate is adequate for fabrication of valve hoods for the Howell-Bunger Valves at Ross Dam.

"2. Conclusions

The one inch thick end plate as shown in the sketch furnished to this office is not strong enough to withstand the hydraulic loads obtained from the model tests and at the same time keep the maximum unit stress at a desirable figure.

"3. Recommendations

It is recommended that the end plate thickness be increased to 1-3/8 inches.

"4. Comments

The drawing furnished this office is only a sketch and not a complete design drawing, however, from the data at hand, it appears that radial stresses in the end plate due to hydraulic loads obtained from the model studies may reach values on the order of 20,000 pounds per square inch.

If the end plate thickness be increased to 1-3/8 inches, the radial stresses in the end plate can be reduced to values below 16,000 pounds per square inch.

An alternative design, satisfactory from the point of stress considerations, could be affected by using a 1-3/8 inch thick end plate with an outer cylindrical shell 3/8-inch thick. This design would require that a 1/2- by 6-inch stiffener ring be welded to the hood shell at its discharge end. This stiffener would have to be anchored to the concrete to be effective.

"5. Basic Data

A. Radius of hood . . . . 72 inches
B. Maximum pressure on inside of hood . . 62.1 lbs/in^2
C. Maximum average pressure on end plate. 16 lbs/in^2

"6. Technical Details

The maximum stress in the cylindrical portion of the shell due to an internal pressure of 62.1 pounds per square inch was computed from the formula $S = pr/t$ where S is the tensile unit-stress in the shell in pounds per square inch, p is the internal hydraulic pressure in pounds per square inch, r is the radius of the cylindrical shell in inches, and t is the thickness of the shell in inches. The pressure p was computed from values of p/H obtained in the model studies. The values of p/H were multiplied by 410 and 0.433 to obtain the equivalent internal pressures.

The end plate was analyzed as a circular flange fixed and supported at the outer edge with the inner edge fixed against rotation, and subjected to uniform pressure over the actual surface.

"7. References

A. Drawing titled 'Ross Dam, Howell-Bunger Valve Hood, Hood D--Final Design, Piezometer Locations and Hood Pressures.'

B. Raymond J. Roark, 'Formulas for Stress and Strain,' Second Edition."

PART III - ICE-PREVENTION SYSTEM

Introduction

Stress studies made on Ross Dam, for the stage with the crest at elevation 1582, took into account maximum flood and earthquake stresses but did not include iceload stresses. This followed the request of the Federal Power Commission that ice be prevented from forming in the vicinity of the spillway gates. In planning an ice-prevention system, it was believed advisable to not only prevent ice from forming in the vicinity of the spillways but also around the entire length of the arch.

Previous extensive investigations of an ice-prevention system for Grand Coulee Dam showed that the air-lift system was the most satisfactory. This system makes use of a series of nozzles on the upstream side of the dam, discharging air into the reservoir 10 feet or more below the surface. The resulting circulation bringing warmer water from below to the surface, prevents the formation of ice near the upstream face of the dam.

The design of the Grand Coulee system was based on tests made on a 1:1 scale model by the Bureau's Hydraulic Laboratory. From these tests, the type of nozzle, the direction of the air jet, the quantity of air necessary at each nozzle, the inside diameter and shape of the nozzle itself, the vertical spacing of the nozzles, and other details were determined. Using this and other data, the details of the Grand Coulee installation were prepared.

When an ice-prevention system for Ross Dam was suggested, the Hydraulic Laboratory was requested to make a preliminary investigation to determine the essentials of the system. During the winter season when freezing would occur, the reservoir will be between elevations 1500 and 1550. The ice-prevention system outlined in this report is similar to the Grand Coulee system and is designed to protect the entire arch from ice pressure with the reservoir above elevation 1500.

Preliminary Considerations

Much of the Grand Coulee Dam ice-prevention system may be adapted for use at Ross Dam. However, certain dissimilarities in the structures make it necessary to provide a different piping layout for the air supply. The lack of an inspection or operating gallery at a convenient elevation in Ross Dam introduces problems not present at Grand Coulee. Because of the thin arch section, it will not be feasible to construct a gallery in Ross Dam for the installation of compressors, pipes, and headers. Thus, it will be necessary to install the compressors at the roadway elevation, say, in one of the abutments, and run piping through the arch to the nozzles on the upstream face of the dam.

The thin arch section also introduces another problem in that during severe winter conditions, the temperature gradient from the front to the back of the dam may place the air supply pipes in a freezing zone. The piping should be located near the upstream face of the arch and arranged so that freezing of condensed moisture in the pipes be eliminated.

Proposed System

Design

Two horizontal rows of nozzles are considered necessary, the lower at elevation 1490 and the upper at elevation 1530. This should provide complete protection against formation of ice for reservoir elevations 1500 to 1550 and also for rare cases where the reservoir might be above elevation 1550 during the winter season. Two rows are necessary to prevent excessive pressures at the compressor.

From actual experience on Keokuk and other dams, it has been found desirable to place the nozzles between 10 and 12 feet apart, laterally, in each row. Thus, at Ross Dam, where winter conditions can be severe, the 10-foot spacing was necessary.

Assuming 1,000 feet of arch to be protected, there would be 100 nozzles in each row, or a total of 200 nozzles on the upstream face of the dam. If each nozzle discharges 2 cubic feet of free air per minute, this would make a total and maximum discharge of 200 cubic feet of free air per minute, since only one row of nozzles would be discharging at any one time. Air would be supplied by two compressors, each capable of handling 110 cubic feet of free air per minute. A third compressor, similar to the others, would be necessary as a standby unit and would be interconnected to the other two. It is suggested that these compressors be of the motor-driven air-cooled rotary type similar to those made by the Yeoman Manufacturing Company. Preliminary estimates indicate that the compressors be capable of maintaining a 40 pound per square inch gage pressure during operation of the system.

Figure 30 shows a schematic layout of a suggested installation for Ross Dam. Each of the two operating compressors discharges into a 4-inch header to which are connected four 1-1/4-inch copper distributing pipes, each serving approximately one-eighth of the arch. Each distribution pipe should serve about 12, and not more than 15, nozzles. The nozzles should be connected to the distributing pipes with 1/2-inch copper pipe.

Just below the header, each distributing line should be equipped with a regulating valve and a Bourdon-type gage. Cleanouts should be provided at each end of the distributing lines, in the header, and at each nozzle.

Effectiveness

Since no reliable data are at present available on air and water temperatures, it is not possible to predict accurately the effectiveness of the suggested system. However, based on weather conditions at Ross Dam as reported by a former resident of the area and from experiences at other dams, the suggested system should prevent the formation of ice for a distance of 10 feet upstream from the dam with intermittent operation of 4 hours on and 4 hours off, and be capable of removing 18 inches of ice with 8 hours of continuous operation.

Precautions

There are some precautions to be observed in completing the design of this system. Care should be taken to insure that no nozzle discharges appreciably more than 2 cubic feet of free air per minute. This is necessary to prevent freezing inside the nozzle and also to prevent unequal distribution to other nozzles on the same supply line. The Bourdon gage readings for various reservoir elevations should be calculated for each row of nozzles to insure the proper distribution and quantity of air. In brief, the system suggested here should be considered a preliminary proposal rather than a final design.

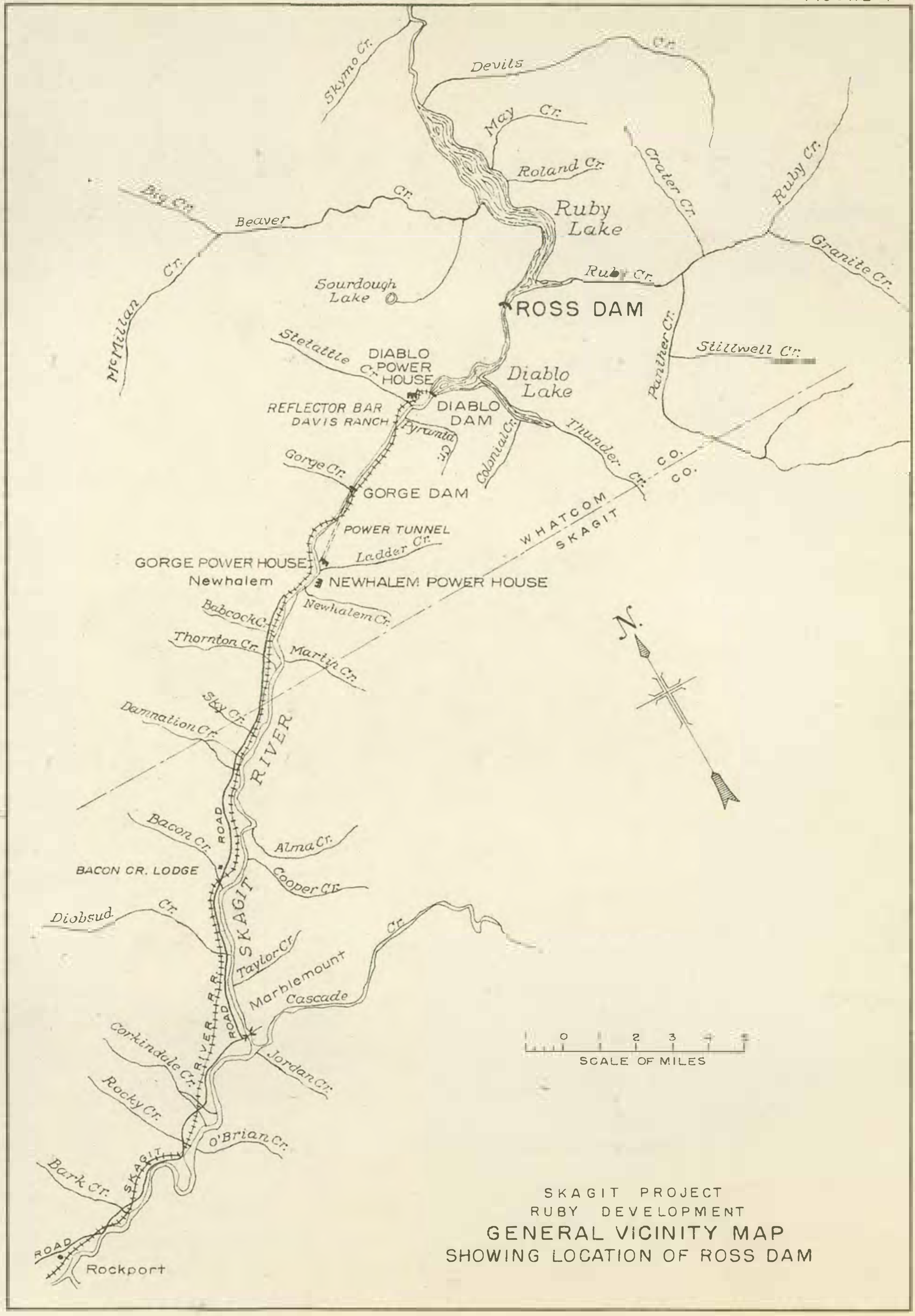
Figure 31 shows the general plan of the Grand Coulee installation, Figure 32 gives the operating diagram and instructions, Figure 33 indicates the type of air supply piping and the details of the nozzles, and Figure 34 lists the parts and sizes. These drawings should prove useful in completing the design of the Ross Dam ice-prevention system.

REFERENCES AND BIBLIOGRAPHY

The enclosed drawings, Figures 30 through 34, showing the installation at Grand Coulee Dam may be used as a guide in completing the design of the Ross Dam system.

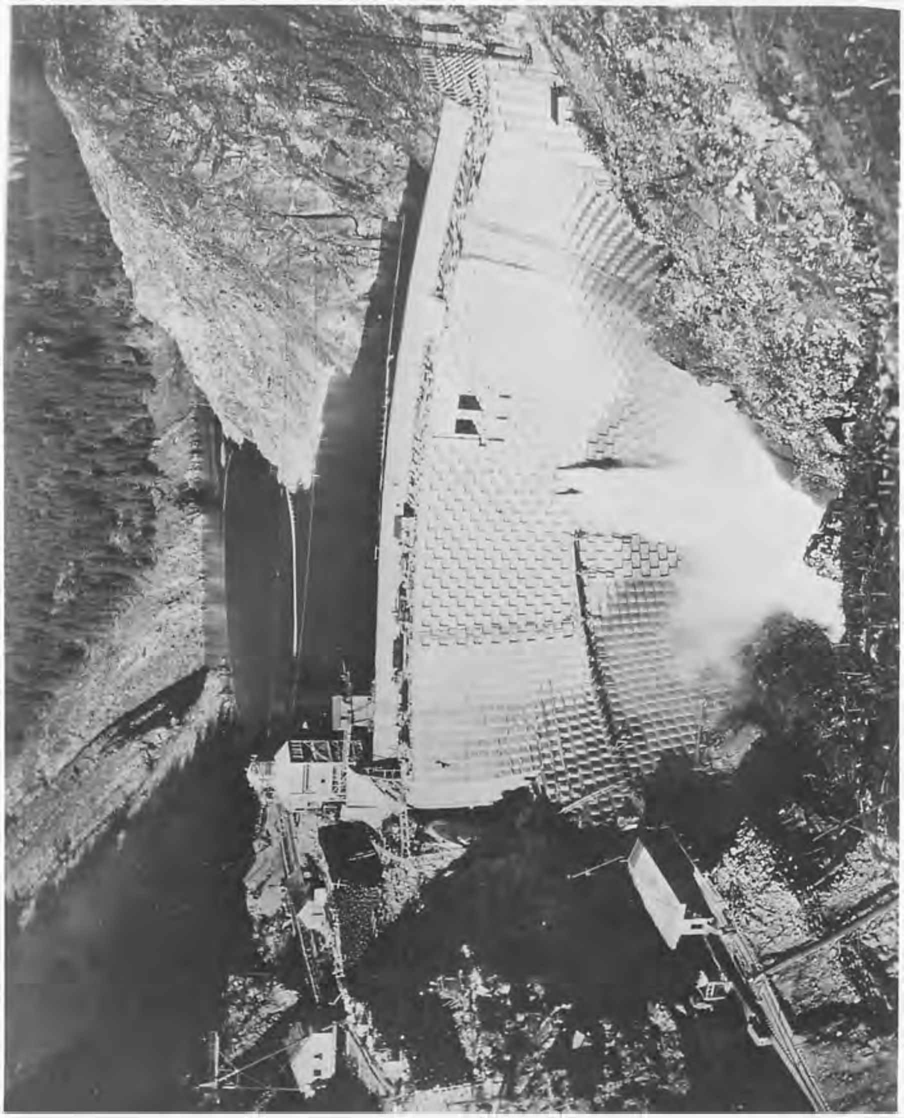
The hydraulic laboratory tests on the system installed at Grand Coulee Dam are discussed in Hydraulic Laboratory Report No. 68. This report appears in another form as a paper by T. C. Owen in the Transactions of the A.S.M.E., April 1942, Volume 64, No. 3. Included with this paper is a discussion of the Grand Coulee system by P. J. Bier, who directed the mechanical design work.

Another reference to the Grand Coulee system is contained in the Design Manual, tentative edition, "Penstock and Pipe Design Section of the Mechanical Division of the Branch of Design and Construction, Bureau of Reclamation."



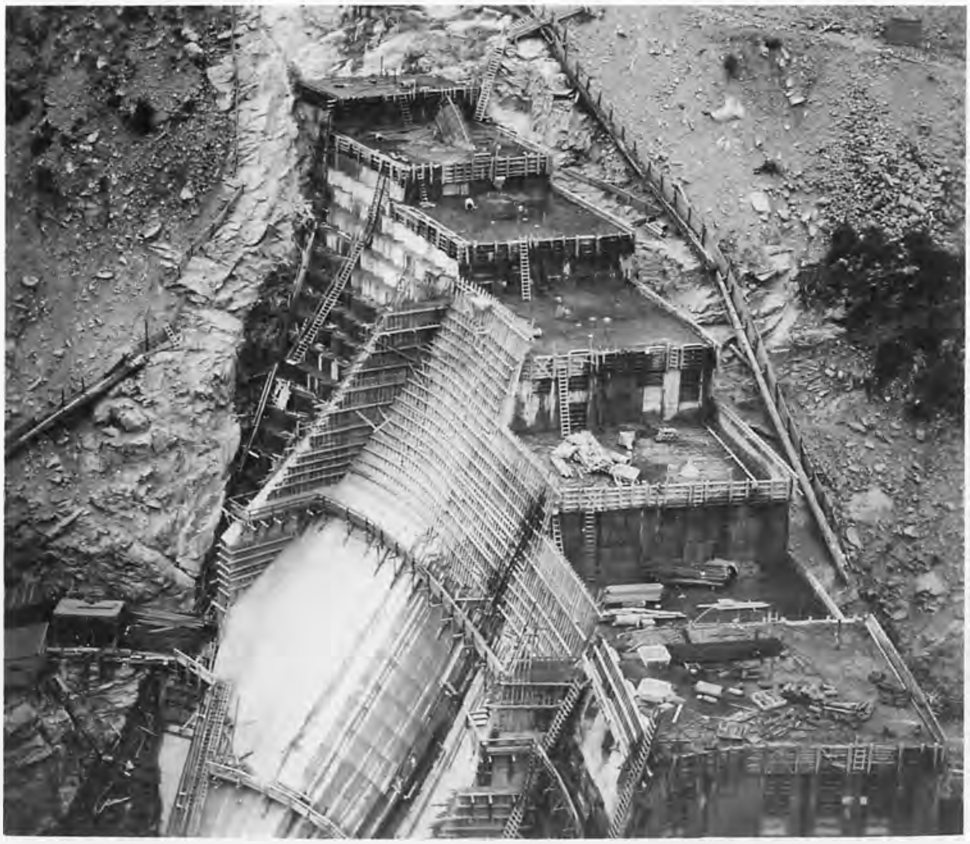
SKAGIT PROJECT
RUBY DEVELOPMENT
GENERAL VICINITY MAP
SHOWING LOCATION OF ROSS DAM

FIGURE 2



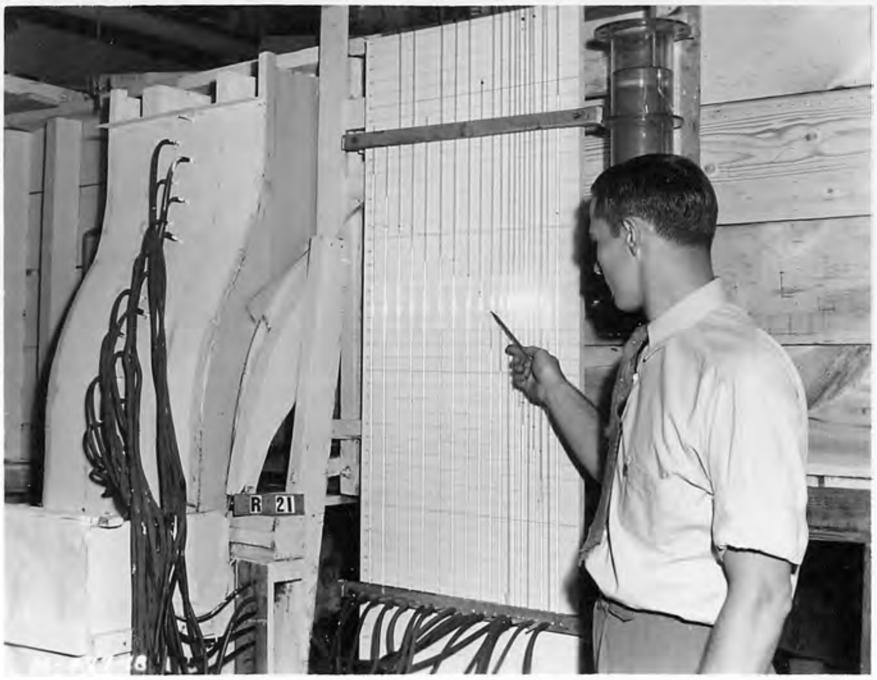
ROSS DAM--INITIAL DEVELOPMENT
TO ELEVATION 1365

View looking upstream.



Right spillway and training-wall.

ROSS DAM
CONSTRUCTION PROGRESS--AUGUST 13, 1946

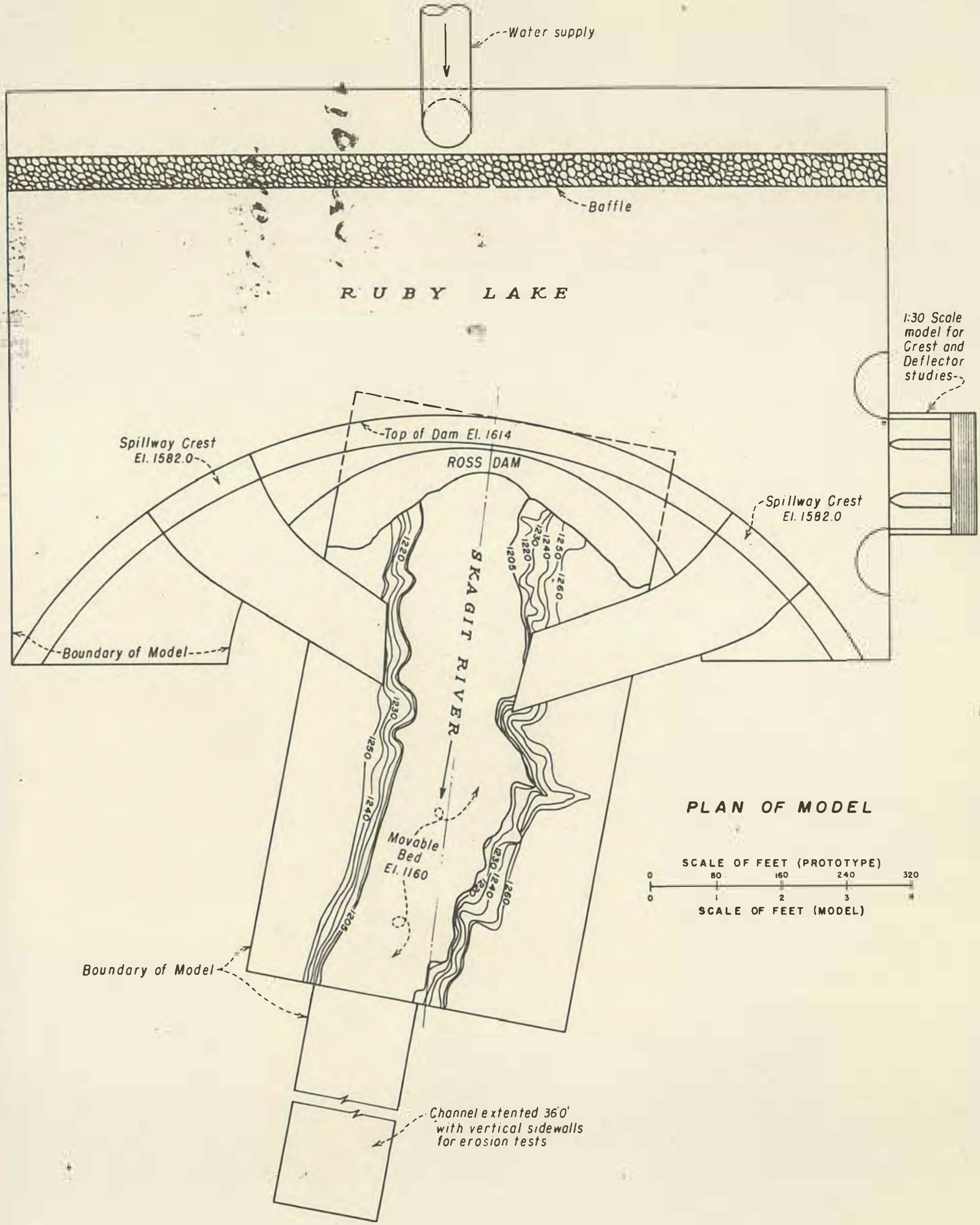


A. Pressure measurements on overflow
Section 9, deflector in place.

B. Crest shape and piezometers
with deflector removed.

ROSS DAM
1:30 SCALE SPILLWAY AND DEFLECTOR MODEL

HYD-207



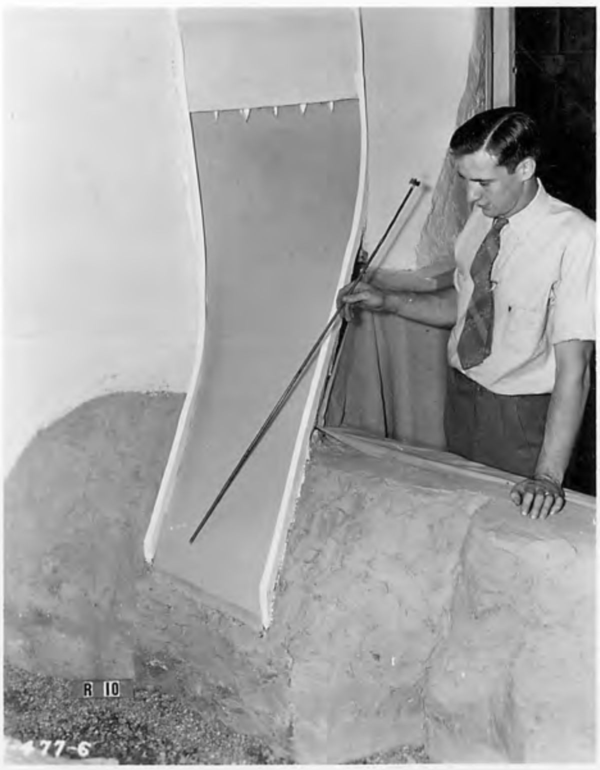
ROSS DAM

EXTENT AND LAYOUT OF 1:80 AND 1:30 SCALE MODELS—OVERFLOW SECTION 9

ROSS DAM
1:80 SCALE MODEL--SPILLWAYS DISCHARGING 127,000 SECOND-FEET

FIGURE 6

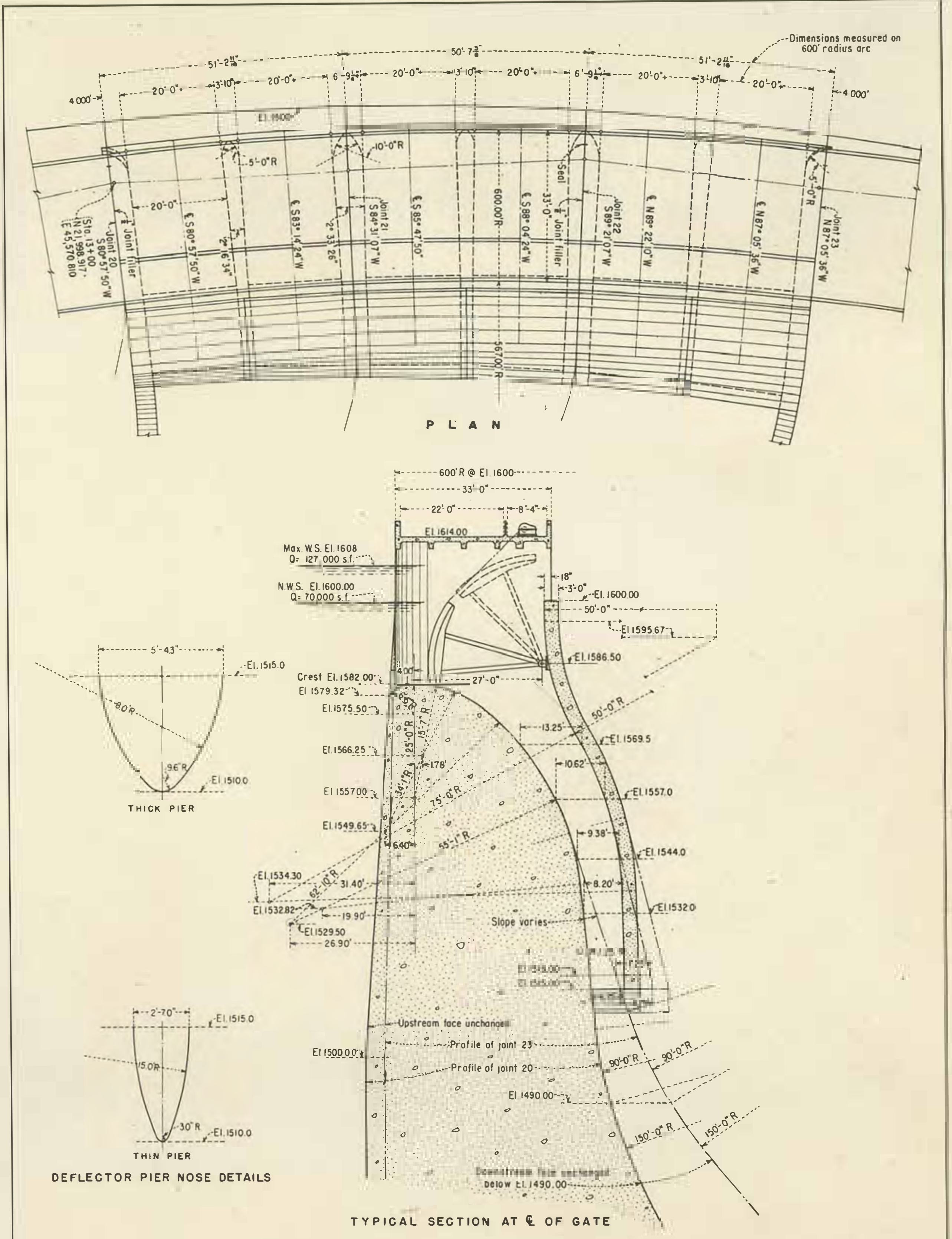
A. Right spillway and Howell-Bunger valve location.



B. Left spillway.

ROSS DAM
1:80 SCALE MODEL

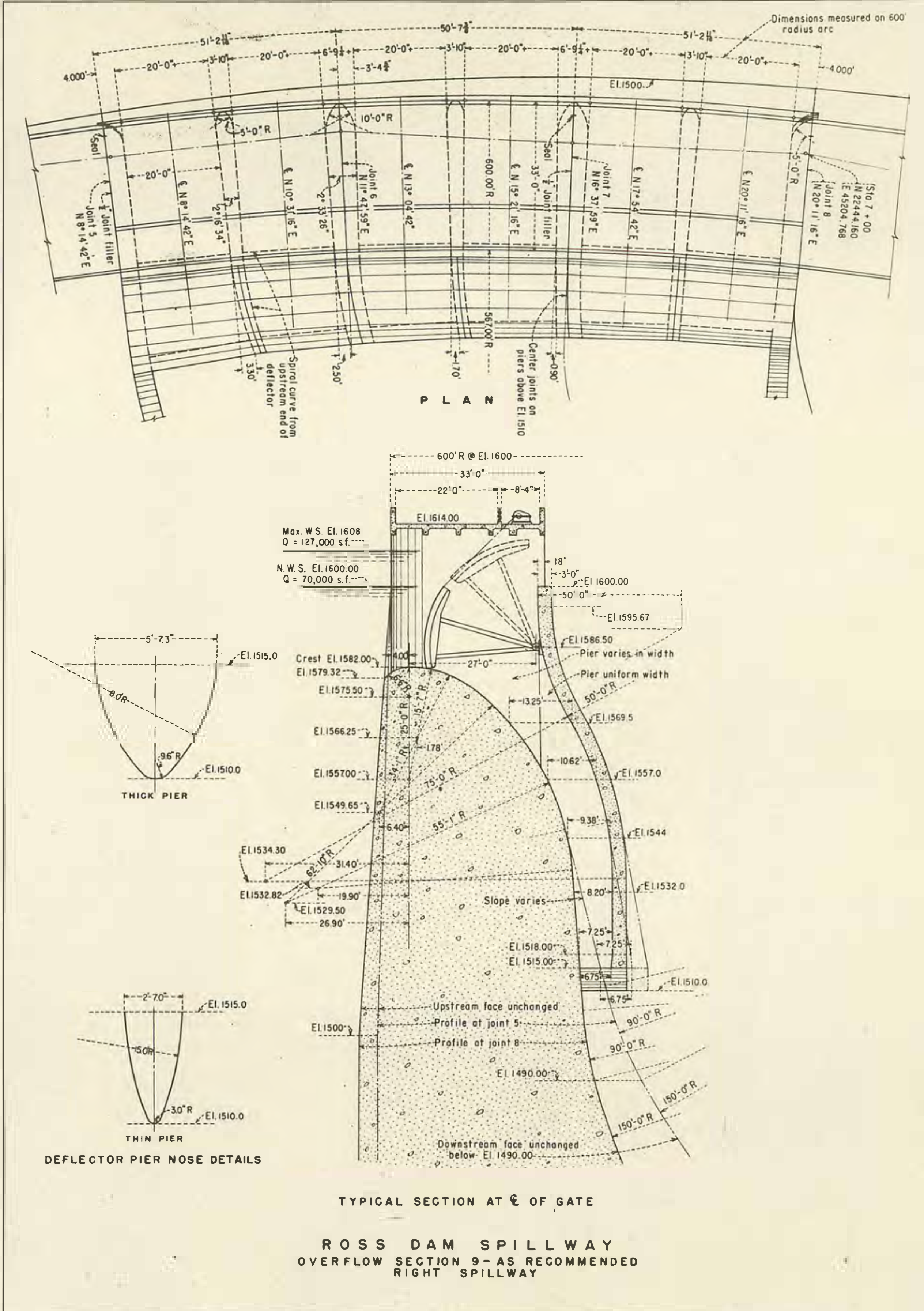
FIGURE 8



ROSS DAM SPILLWAY
OVERFLOW SECTION 9 - AS RECOMMENDED
LEFT SPILLWAY

HYD-297

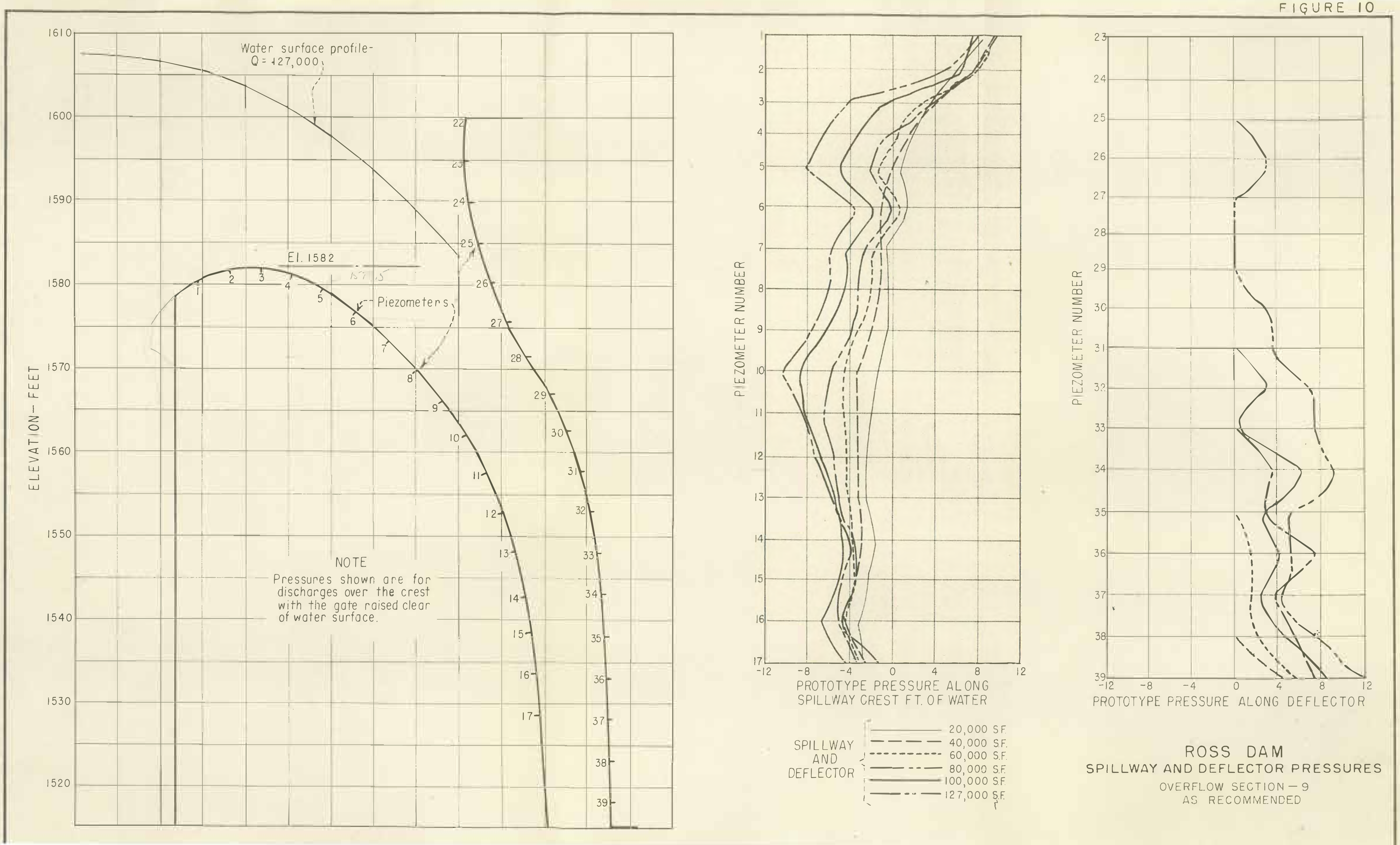
FIGURE 9



ROSS DAM SPILLWAY

OVERFLOW SECTION 9 - AS RECOMMENDED

RIGHT SPILLWAY



ROSS DAM
SPILLWAY AND DEFLECTOR PRESSURES
OVERFLOW SECTION - 9
AS RECOMMENDED

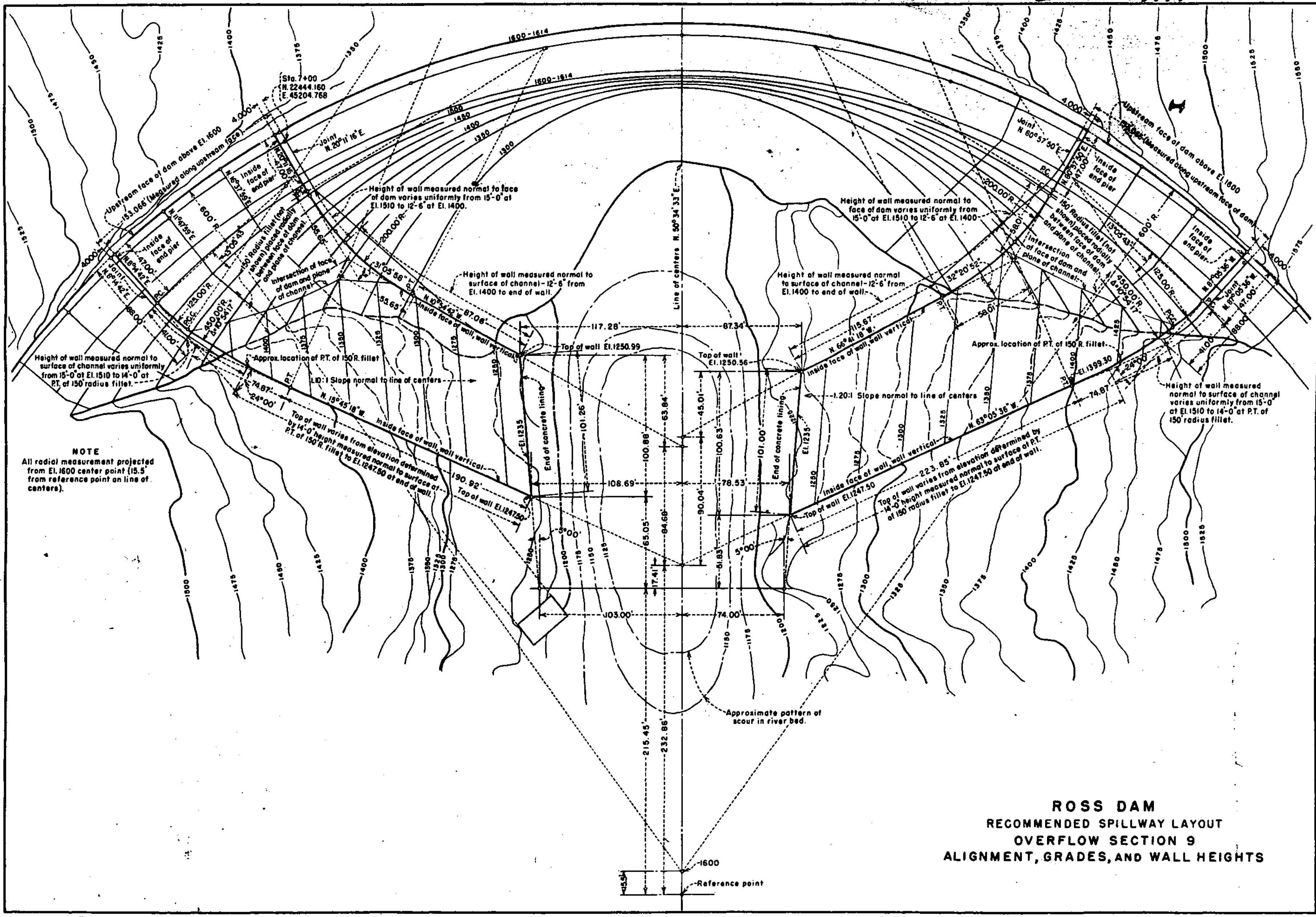
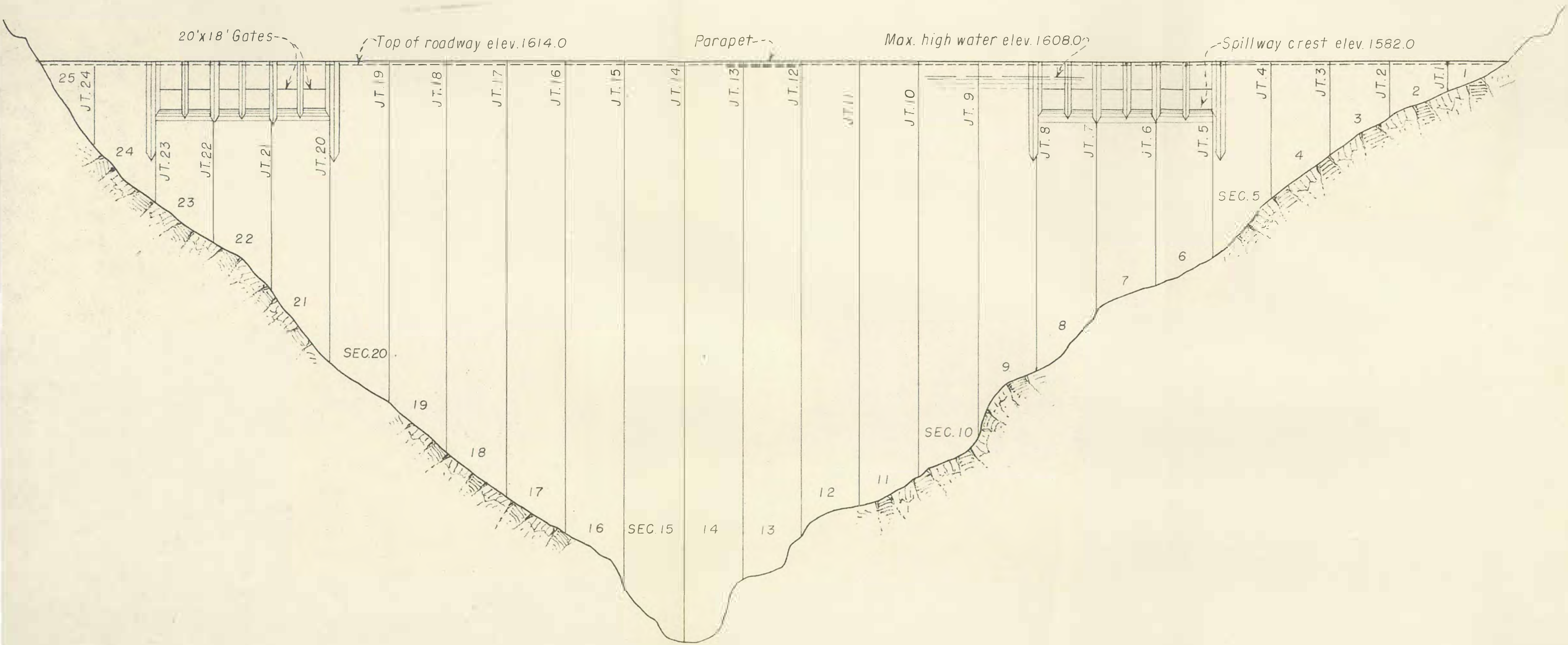

ROSS DAM
RECOMMENDED SPILLWAY LAYOUT
OVERFLOW SECTION 9
ALIGNMENT, GRADES, AND WALL HEIGHTS
NOTE
All radial measurement projected from El. 1600 center point (15.5' from reference point on line of centers).
Line of centers N. 30° 34' 33" E.
Sta. 7+00
N. 22444.160
E. 45204.768
Upstream face of dam above El. 1600
153.066' (Measured along upstream face)
Height of wall measured normal to face of dam varies uniformly from 15'-0" at El. 1510 to 12'-6" at El. 1400.
Height of wall measured normal to surface of channel - 12'-6" from El. 1400 to end of wall.
Height of wall measured normal to surface of channel varies uniformly from 15'-0" at El. 1510 to 14'-0" at P.T. of 150' radius fillet.
Approx. location of P.T. of 150' R. fillet
1.10:1 Slope normal to line of centers
1.20:1 Slope normal to line of centers
Top of wall El. 1250.99
Top of wall El. 1250.56
Top of wall El. 1247.50
Top of wall varies from elevation determined by 14'-0" height measured normal to surface at P.T. of 150' R. fillet to El. 1247.50 at end of wall.
Inside face of wall, wall vertical
End of concrete lining
Approximate pattern of scour in river bed.
Reference point

FIGURE 11



UPSTREAM ELEVATION OF DAM
(DEVELOPED)

ROSS DAM

SPILLWAY LOCATIONS - SPILLWAY CREST EL. 1582
OVERFLOW SECTION 9

A. Flow on right spillway.

B. Flow on left spillway.

ROSS DAM
SPILLWAYS DISCHARGING 127,000 SECOND-FEET

FIGURE 14

A. Discharge 40,000 second-feet.

B. Discharge 60,000 second-feet.

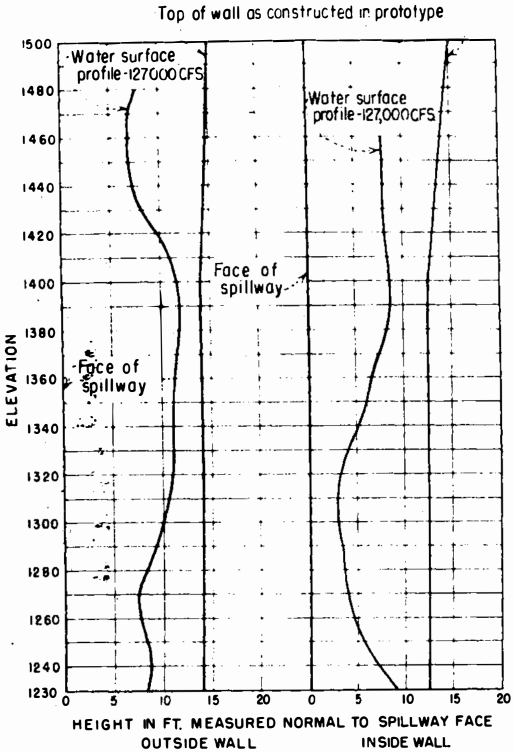
ROSS DAM
STILLING POOL OPERATION

A. Right spillway only operating.

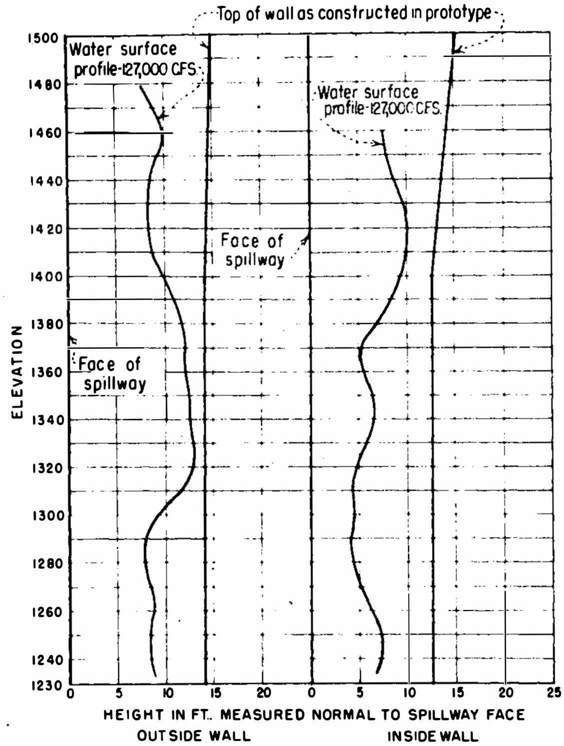
B. Left spillway only operating.

ROSS DAM
SINGLE SPILLWAY DISCHARGING 63,500 SECOND-FEET

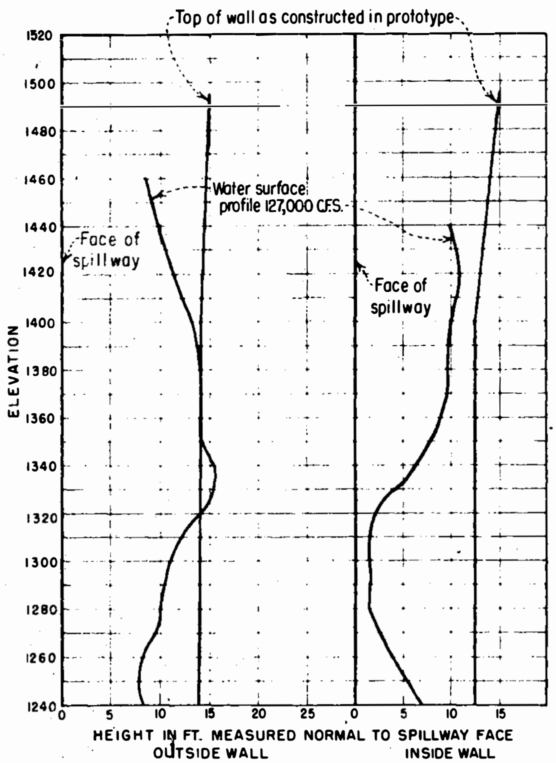
HYD-207



RIGHT SPILLWAY
CURVED PIERS



LEFT SPILLWAY
STRAIGHT PIERS



RIGHT SPILLWAY
STRAIGHT PIERS

NOTE

Training walls constructed in prototype according to data given on Figure 22 of Hydraulic Laboratory Report No.136.

ROSS DAM
WATER SURFACE PROFILES
OVERFLOW SECTION-10

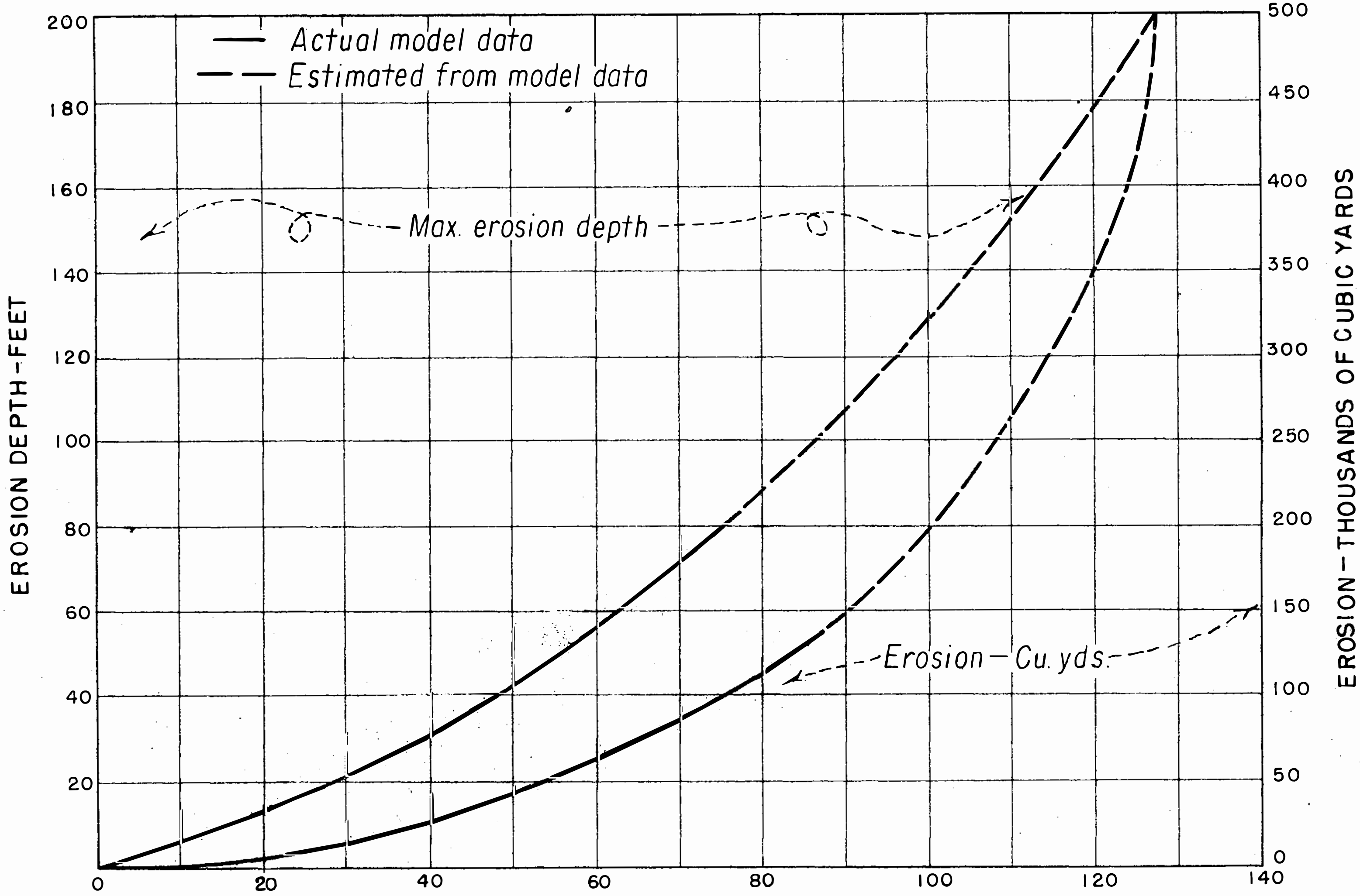
A. After 40,000 second-feet.

B. After 60,000 second-feet.

C. After 80,000 second-feet.

D. After 127,000 second-feet.

ROSS DAM
EROSION TESTS



ROSS DAM—EROSION TEST RESULTS

1:80 SCALE MODEL

FIGURE 18

A. Excavated hole 200 feet from spillways.

B. Resulting erosion pattern after 60,000 second-feet.

C. Excavated hole 400 feet from spillways.

D. Resulting erosion pattern after 60,000 second-feet.

ROSS DAM
EROSION EXPERIMENT

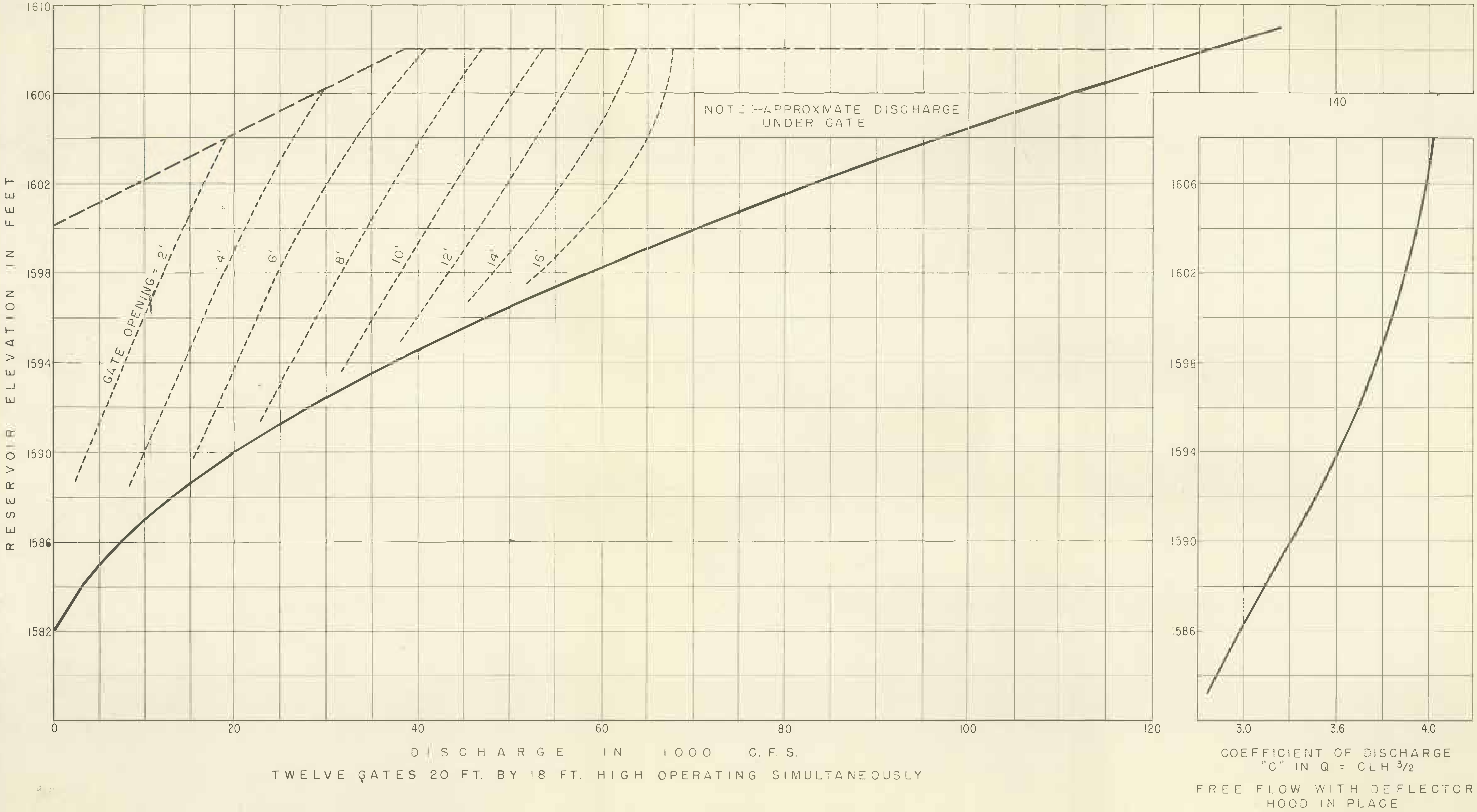
A. Flow concentrated.
Gate 1 open.

B. Flow well distributed.
Gate 4 open.

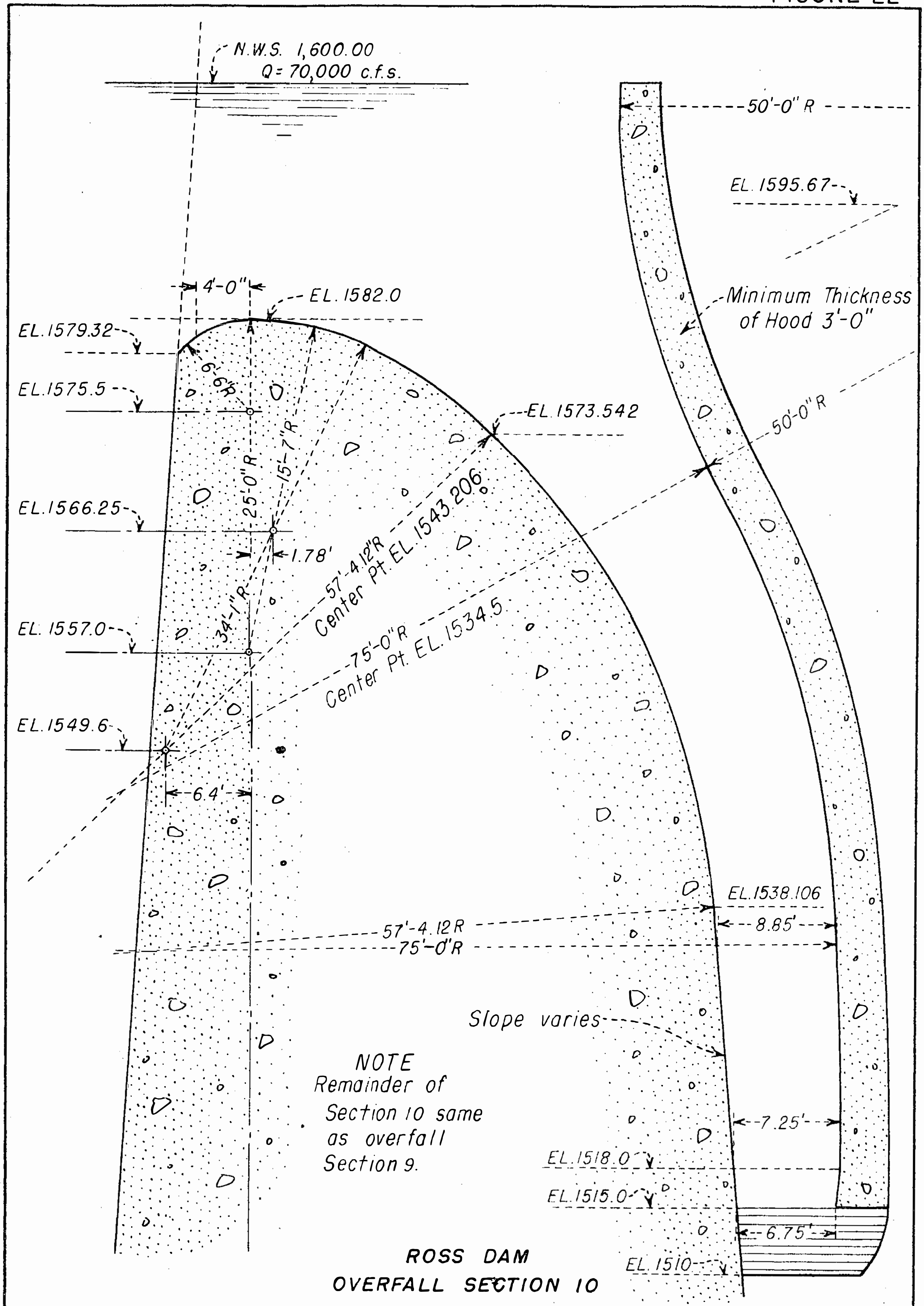
C. Flow climbs training-
wall. Gates 4, 5, 6
open.

D. Flow well distributed.
Gates 4, 3, 2 open.

ROSS DAM
TESTS TO DETERMINE ORDER OF OPENING GATES

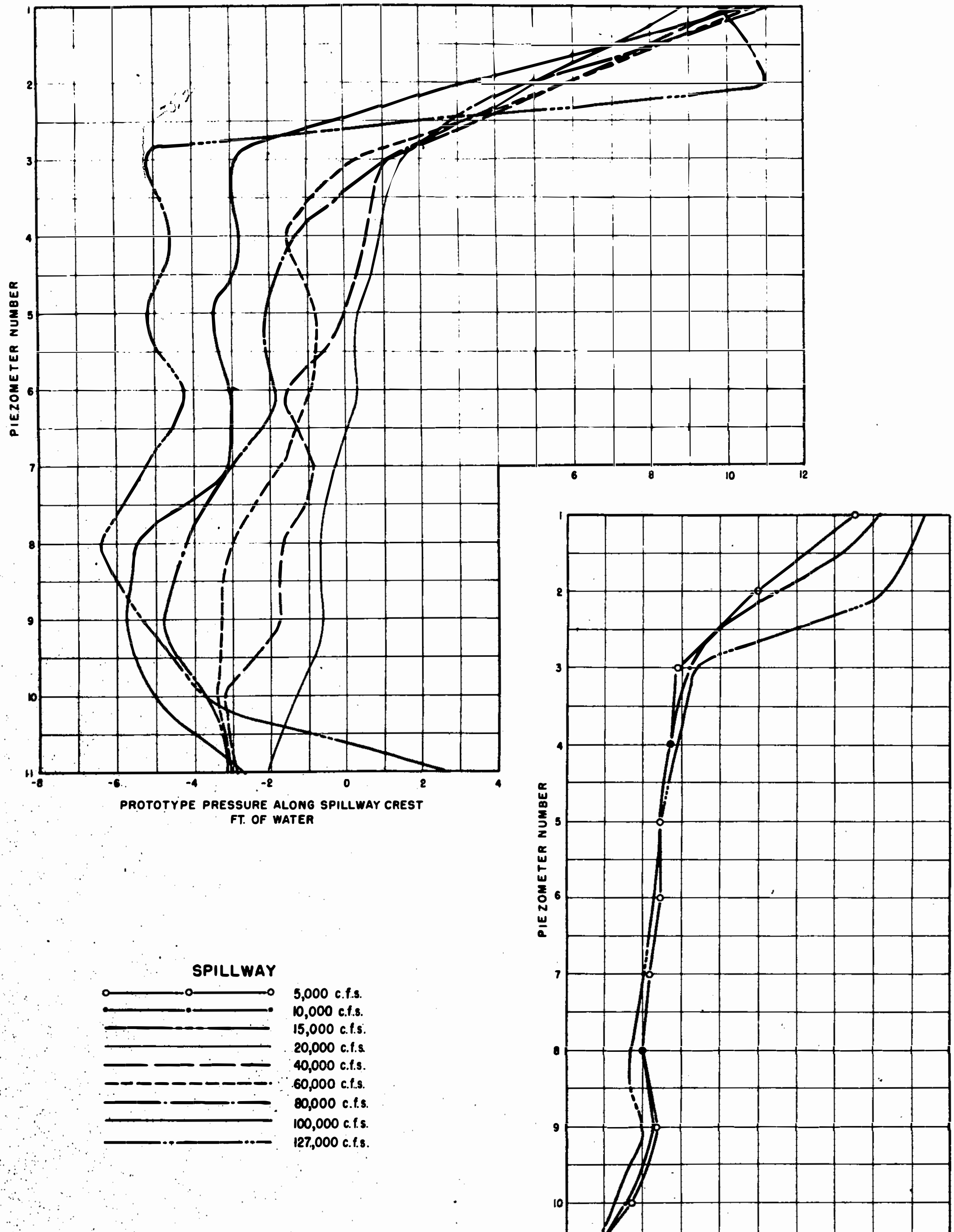


ROSS DAM SPILLWAY DISCHARGE CURVES

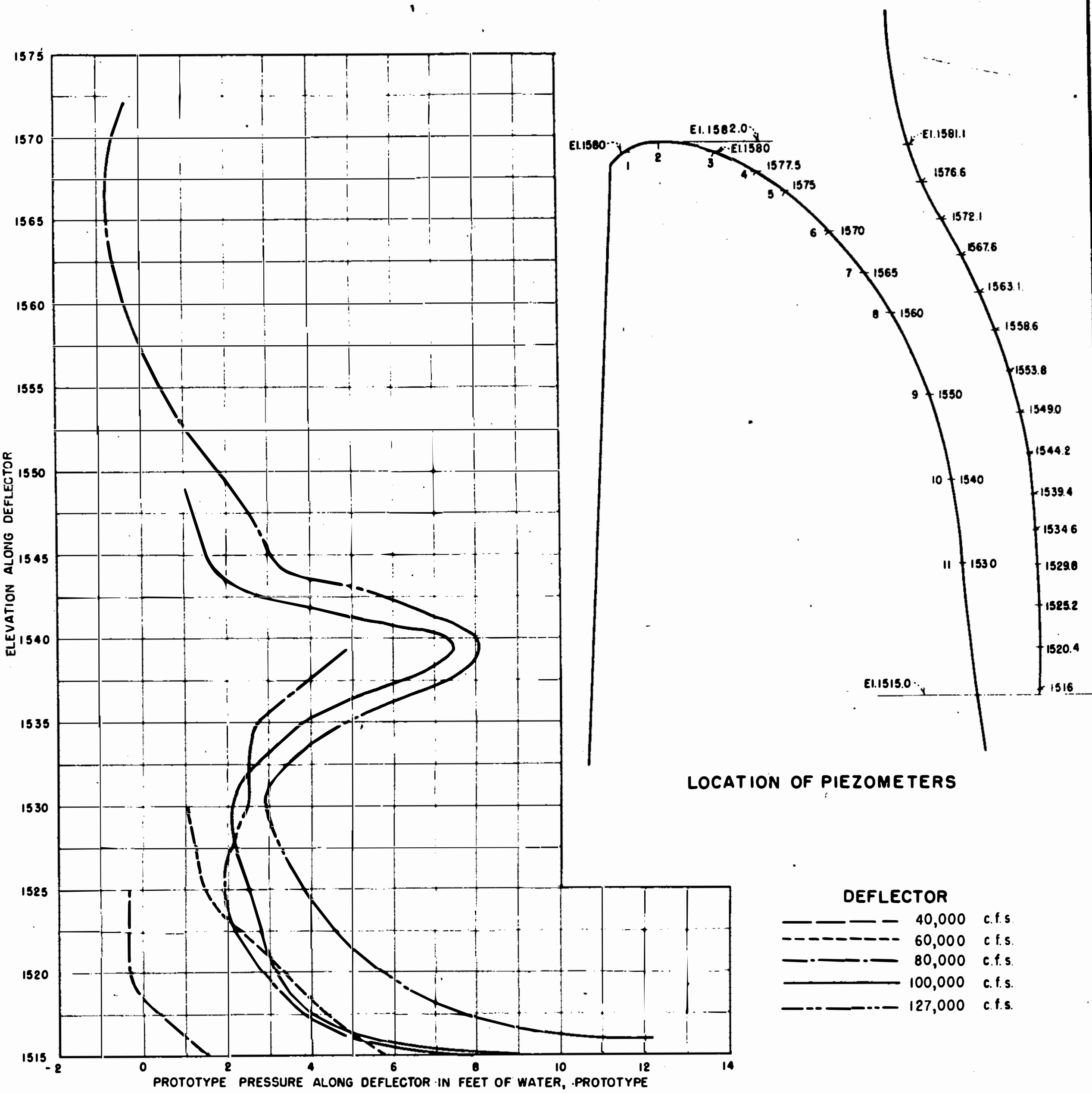


ROSS DAM
OVERFALL SECTION 10

HYD-207

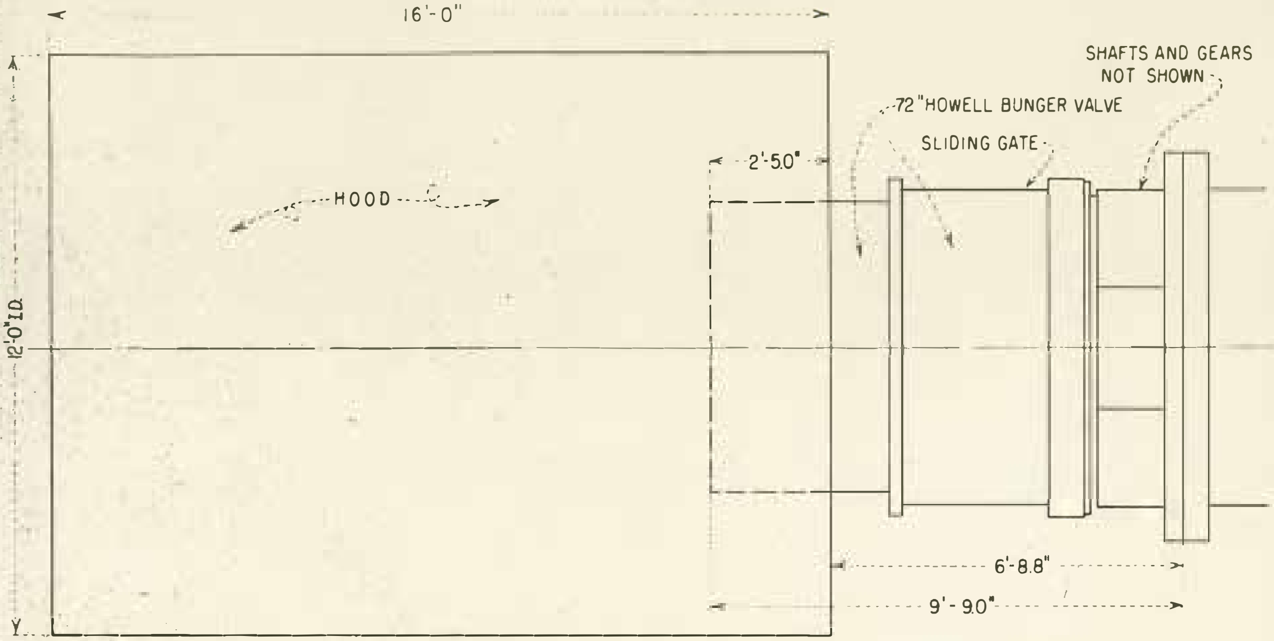


ROSS DAM
SPILLWAY PRESSURES
OVER FALL SECTION-10

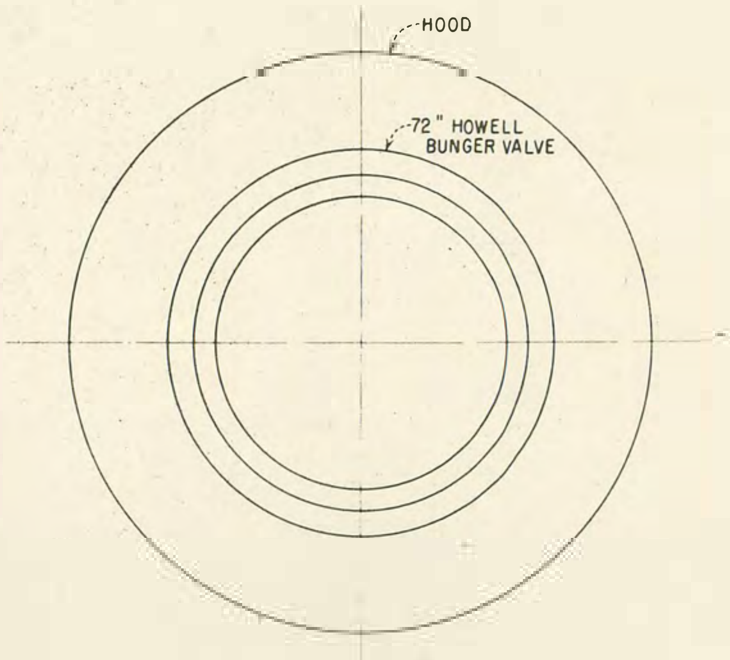


ROSS DAM
DEFLECTOR PRESSURES
OVER FALL SECTION-10

HYD-207



ELEVATION

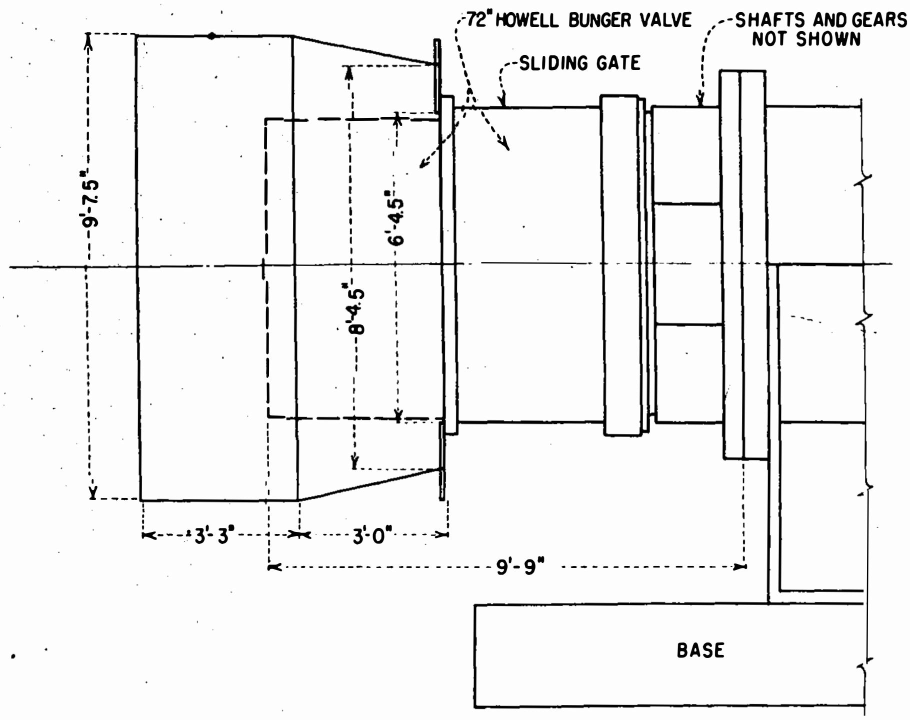


END VIEW

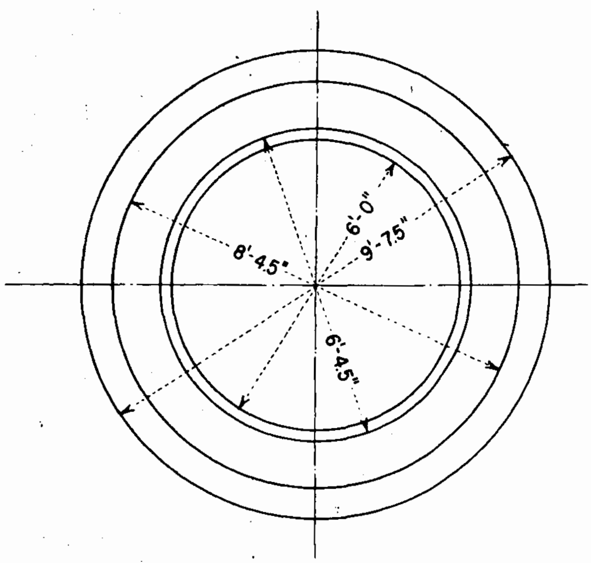
ROSS DAM

HOWELL - BUNGER VALVE HOOD

HOOD B - PRELIMINARY DESIGN



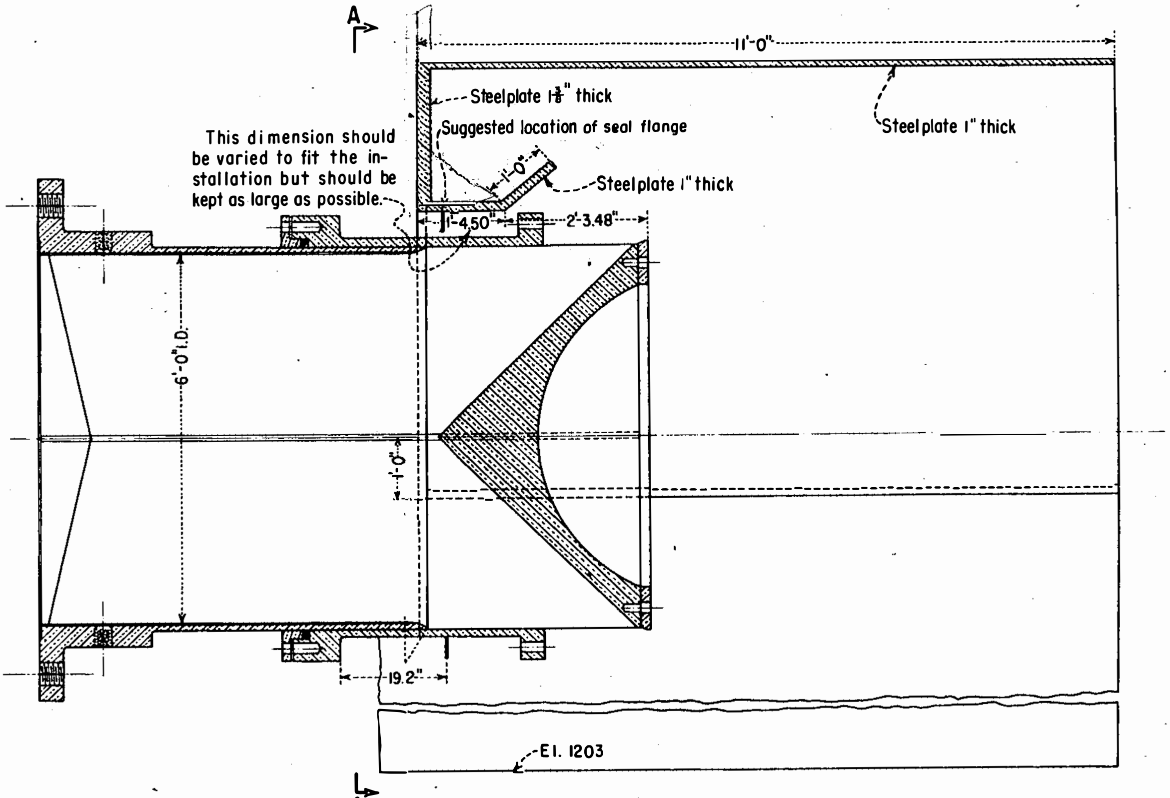
ELEVATION



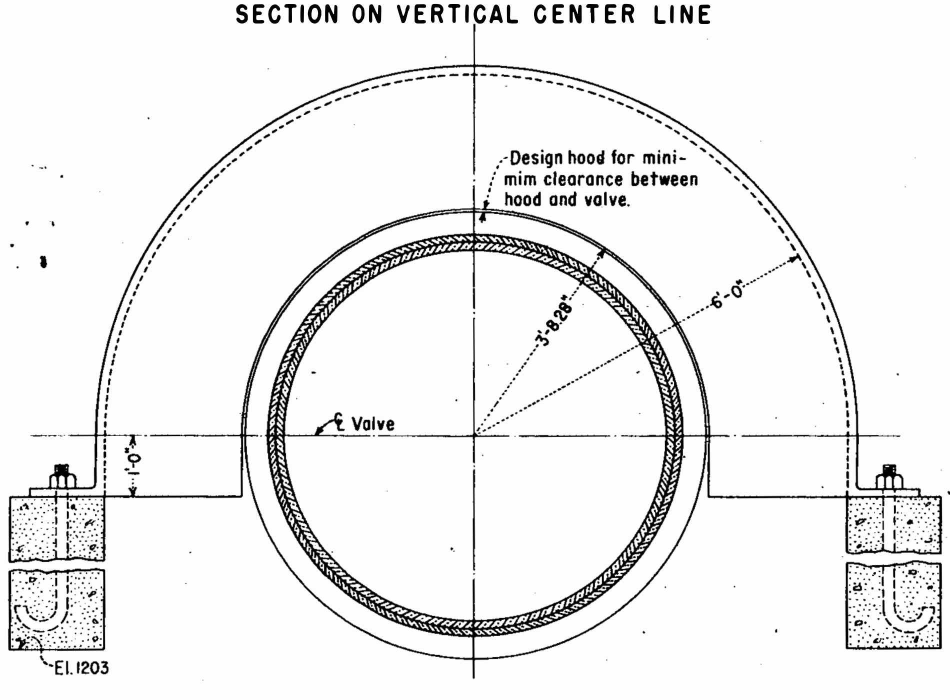
END VIEW

ROSS DAM
HOWELL - BUNGER VALVE HOOD
HOOD C - PRELIMINARY DESIGN

FIGURE 27



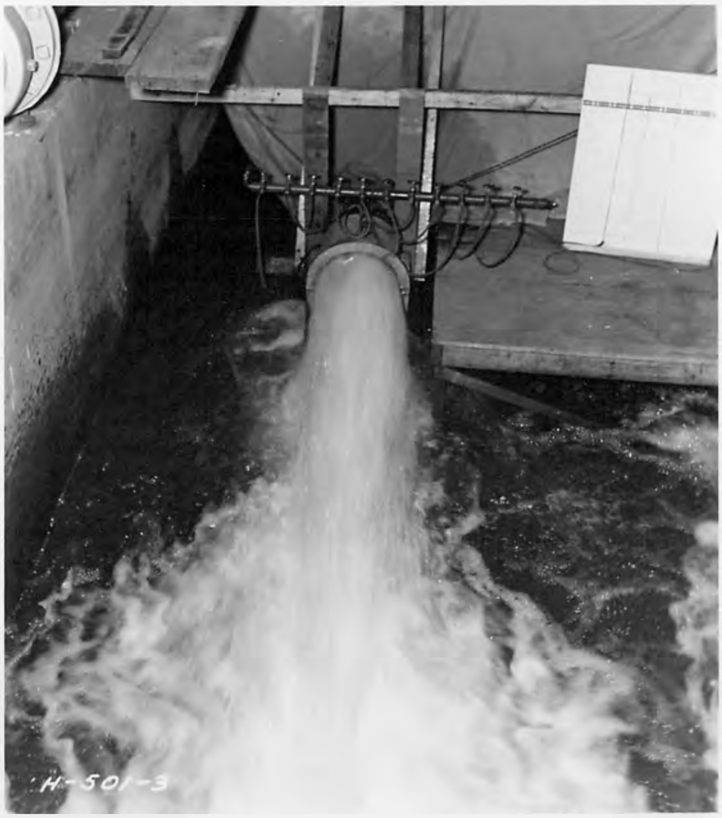
SECTION ON VERTICAL CENTER LINE



SECTION A-A

ROSS DAM
HOWELL-BUNGER VALVE HOOD
HOOD D - FINAL DESIGN

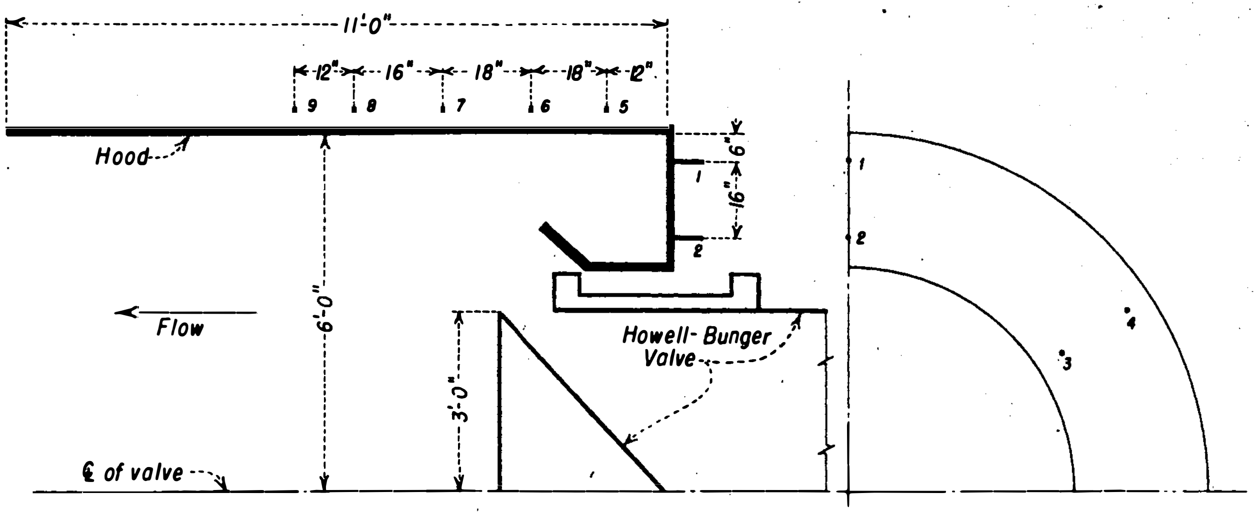
A. Valve discharging with hood removed.



B. Valve discharging with Hood D in place.

ROSS DAM
HOWELL-BUNGER VALVE OPERATION

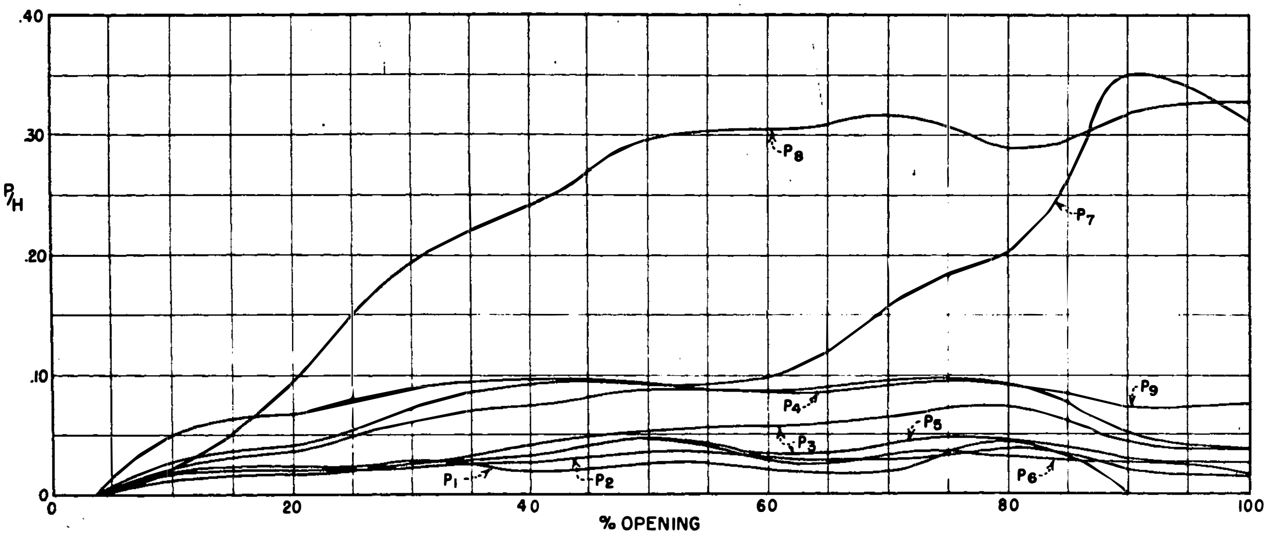
HYD-207



ELEVATION

SECTION

LOCATION OF PIEZOMETERS

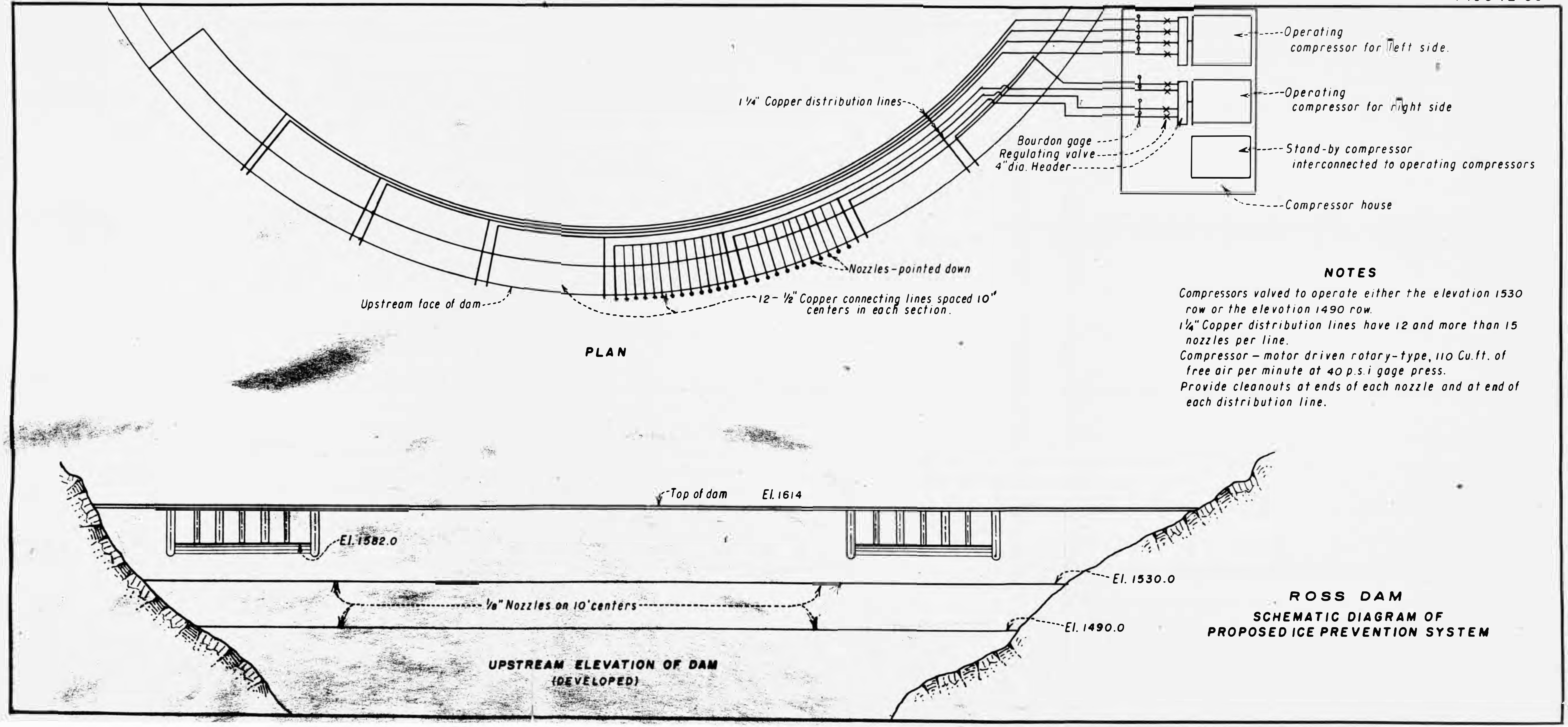


NOTES

$\frac{P}{H}$ Values obtained from 1:12 model tests. H range was from 173 to 410 feet of prototype head. To obtain prototype pressures multiply $\frac{P}{H}$ by prototype head.

ROSS DAM
HOWELL - BUNGER VALVE HOOD
HOOD D - FINAL DESIGN
PIEZOMETER LOCATIONS AND HOOD PRESSURES

FIGURE 30



NOTES

Compressors valved to operate either the elevation 1530 row or the elevation 1490 row.

1 1/4" Copper distribution lines have 12 and more than 15 nozzles per line.

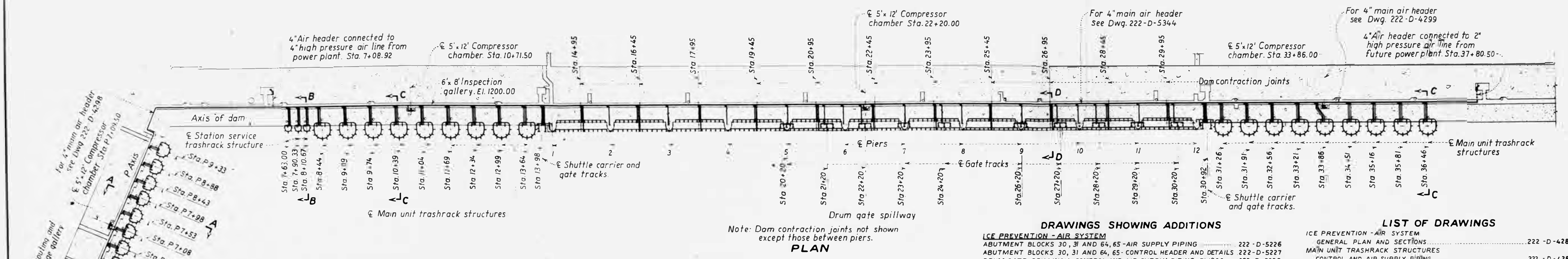
Compressor – motor driven rotary-type, 110 Cu.ft. of free air per minute at 40 p.s.i gage press.

Provide cleanouts at ends of each nozzle and at end of each distribution line.

ROSS DAM

SCHEMATIC DIAGRAM OF PROPOSED ICE PREVENTION SYSTEM

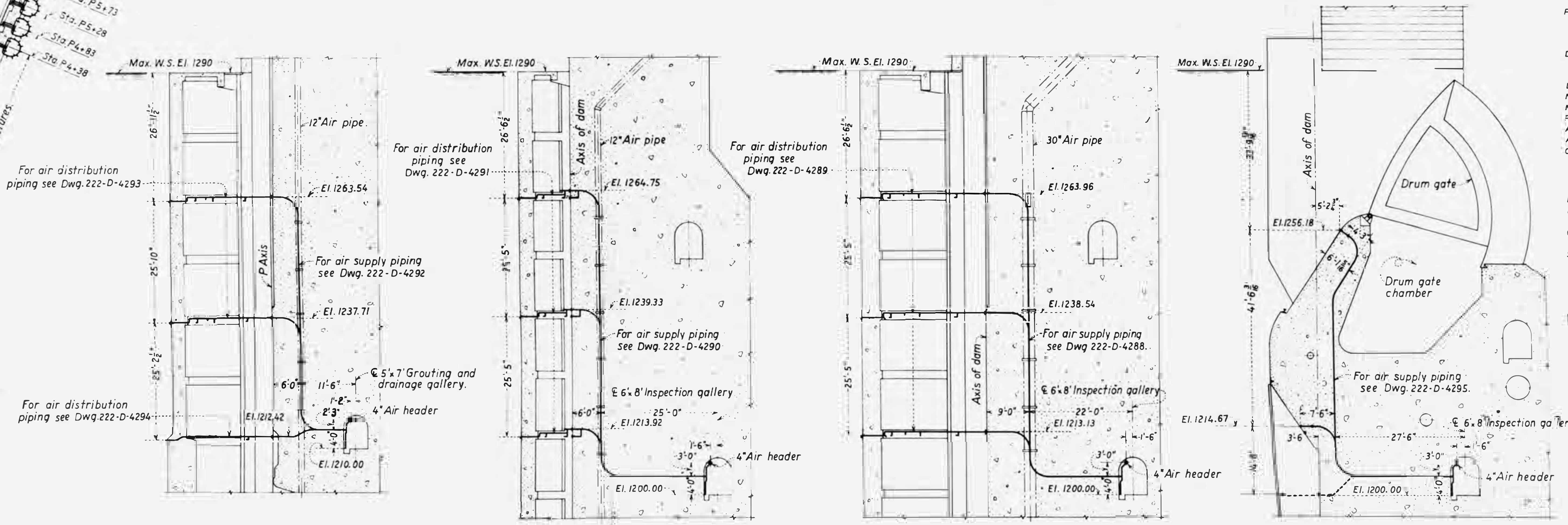
FIGURE 31



DRAWINGS SHOWING ADDITIONS

ICE PREVENTION - AIR SYSTEM

ABUTMENT BLOCKS 30, 31 AND 64, 65 - AIR SUPPLY PIPING	222-D-5226
ABUTMENT BLOCKS 30, 31 AND 64, 65 - CONTROL HEADER AND DETAILS	222-D-5227
DRUM GATE SPILLWAY - CONTROL AND AIR SUPPLY PIPING - El. 1200	222-D-5228
ADDITIONAL LIST OF PARTS AND GENERAL NOTE	222-D-5229
MAIN AIR HEADER STA. 13+95 TO STA. 30+95	222-D-5344



SECTION A-A
TYPICAL SECTIONAL ELEVATION
PUMPING PLANT TRASHRACK STRUCTURES

SECTION B-B
TYPICAL SECTIONAL ELEVATION
STATION SERVICE TRASHRACK STRUCTURES

SECTION C-C
TYPICAL SECTIONAL ELEVATION
MAIN UNIT TRASHRACK STRUCTURES

SECTION D-D
TYPICAL SECTIONAL ELEVATION
DRUM GATE SPILLWAY

LIST OF DRAWINGS

ICE PREVENTION - AIR SYSTEM	
GENERAL PLAN AND SECTIONS	222-D-4287
MAIN UNIT TRASHRACK STRUCTURES	
CONTROL AND AIR SUPPLY PIPING	222-D-4288
AIR DISTRIBUTION PIPING	222-D-4289
STATION SERVICE TRASHRACK STRUCTURES	
CONTROL AND AIR SUPPLY PIPING	222-D-4290
AIR DISTRIBUTION PIPING	222-D-4291
PUMPING PLANT TRASHRACK STRUCTURES	
CONTROL AND AIR SUPPLY PIPING	222-D-4292
AIR DISTRIBUTION PIPING	222-D-4293
AIR DISTRIBUTION PIPING	222-D-4294
DRUM GATE SPILLWAY	
AIR SUPPLY PIPING	222-D-4295
CONTROL HEADER AND DETAILS	222-D-4296
LIST OF PARTS AND GENERAL NOTE	222-D-4297
MAIN AIR HEADER - STA. P4+34.83 TO STA. 13+95	222-D-4298
MAIN AIR HEADER - STA. 30+95 TO STA. 37+80.50	222-D-4299
PIPE SUPPORTS AND CLAMPS	222-D-4300
AIR COMPRESSORS, PIPING ASSEMBLY, AND LIST OF PARTS	222-D-6301
AIR COMPRESSOR, PIPING ASSEMBLY, AND PIPE SUPPORTS	222-D-6302
OPERATING DIAGRAM AND INSTRUCTIONS	222-D-4303

REFERENCE DRAWINGS

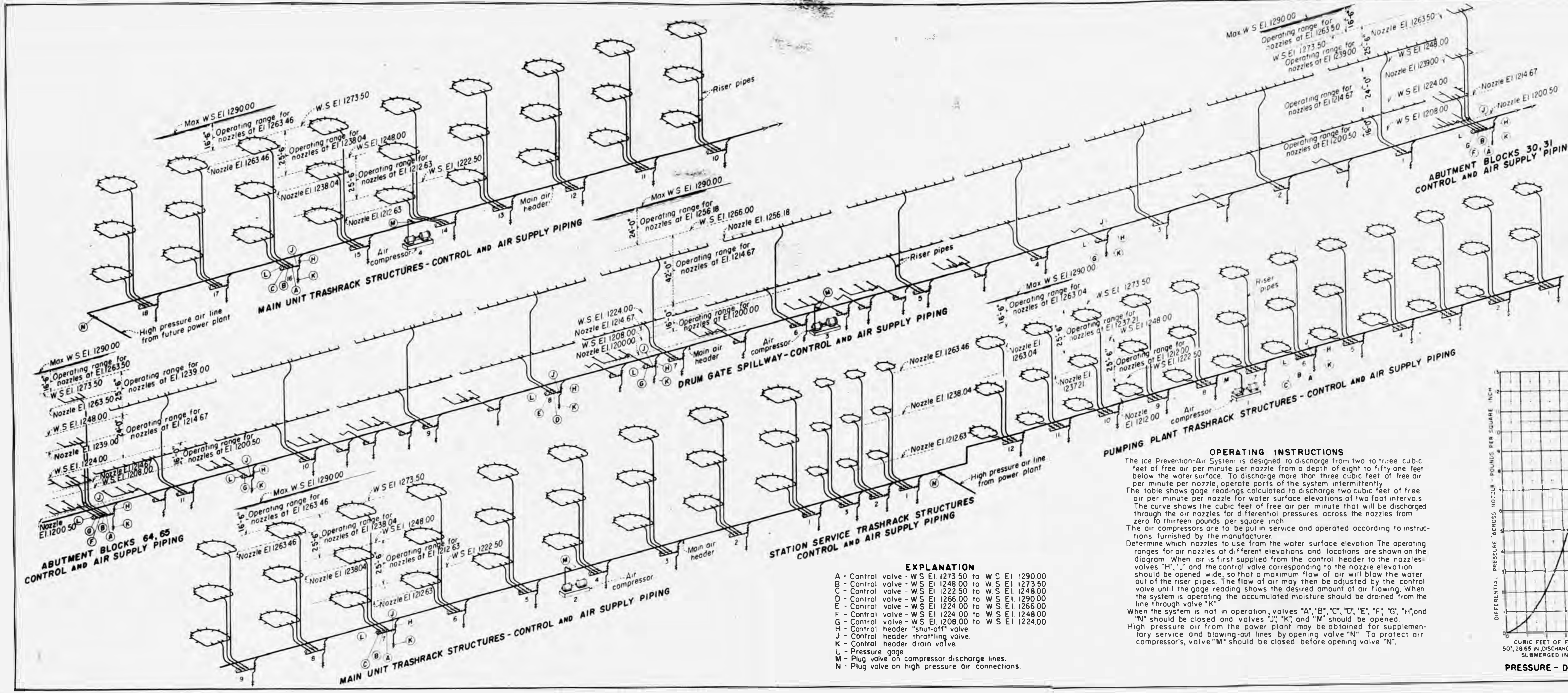
DRUM GATE SPILLWAY - PLAN AND SECTIONS	222-D-1934
POWER AND STATION SERVICE PENSTOCKS - BY-PASS AND AIR PIPING	222-D-2125
GROUTING AND DRAINAGE GALLERY - PLAN AND ELEVATION. STA. P4+17 TO STA. 5+74	222-D-2172
STATION SERVICE	
TRASHRACK STRUCTURE	222-D-2215
TRASHRACK STRUCTURE REINFORCEMENT ABOVE EL. 1023.13	222-D-2217
MAIN UNITS	
TRASHRACK STRUCTURE ABOVE EL. 1061.25	222-D-2222
TRASHRACK STRUCTURE - REINFORCEMENT ABOVE EL 1061.25	222-D-2313
135' x 28' DRUM GATE - EMBEDDED GATE ANCHORAGE	
ANCHOR BOLT LOCATION DIAGRAMS	222-D-2653
GATE CHAMBER CONTOUR - ANCHOR BOLT SETTING	222-D-2657
GALLERY AND SHAFT LAYOUT - PLAN - EL. 1200	222-D-2804
PUMPING PLANT	
TRASHRACK STRUCTURE BELOW EL. 1212.33	222-D-3100
TRASHRACK STRUCTURE ABOVE EL. 1212.33	222-D-3101
TRASHRACK STRUCTURE - REINFORCEMENT ABOVE EL. 1177.77	222-D-3105
BY-PASS AND AIR PIPING	222-D-3640
SHUTTLE CARRAGE SUPPORTS - BLOCKS 30, 31 AND 64, 65	222-D-4947

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT - WASHINGTON

GRAND COULEE DAM
ICE PREVENTION - AIR SYSTEM
GENERAL PLAN AND SECTIONS

DRAWN: J.C.W. SUBMITTED: L. J. McClellan
TRACED: R.E.B. RECOMMENDED: [illegible]
CHECKED: W.H.A. APPROVED: [illegible]
DENVER, COLORADO, SEPT. 25, 1939
SHEET 1 OF 1
222-D-4287

FIGURE 32



TABLE

WATER SURFACE ELEVATION	CALCULATED GAGE READINGS: MAIN UNIT TRASHRACK STRUCTURES	STATION SERVICE TRASHRACK STRUCTURES	PUMPING PLANT TRASHRACK STRUCTURES	DRUM GATE SPILLWAY	ABUTMENT BLOCKS 30, 31, AND 64, 65
1290	13.9	13.3	13.8	16.8	13.1
1288	13.1	12.5	13.0	16.0	12.3
1286	12.4	11.7	12.2	15.2	11.5
1284	11.6	10.9	11.4	14.4	10.7
1282	10.8	10.1	10.6	13.6	9.9
1280	10.0	9.3	9.8	12.8	9.1
1278	9.2	8.5	9.0	12.0	8.3
1276	8.4	7.7	8.2	11.3	7.5
1274	7.7	6.9	7.5	10.5	6.6
1272	16.7	16.3	16.8	9.7	15.8
1270	15.9	15.5	16.0	8.9	15.0
1268	15.2	14.7	15.2	8.1	14.1
1266	14.4	13.9	14.4	7.3	13.3
1264	13.6	13.0	13.6	23.0	12.5
1262	12.8	12.2	12.8	22.2	11.7
1260	12.0	11.4	12.0	21.4	10.9
1258	11.2	10.6	11.2	20.6	10.1
1256	10.4	9.8	10.4	19.8	9.3
1254	9.6	9.0	9.6	19.0	8.5
1252	8.8	8.2	8.8	18.2	7.6
1250	8.0	7.4	8.0	17.4	6.8
1248	7.2	6.6	7.2	16.5	6.0
1246	16.3	16.0	16.4	15.7	15.3
1244	15.5	15.2	15.6	14.9	14.5
1242	14.7	14.4	14.8	14.1	13.6
1240	13.9	13.5	14.0	13.3	12.8
1238	13.1	12.7	13.2	12.5	12.0
1236	12.3	11.9	12.4	11.7	11.2
1234	11.5	11.1	11.6	10.9	10.4
1232	10.7	10.3	10.7	10.1	9.6
1230	9.9	9.5	9.9	9.3	8.8
1228	9.1	8.7	9.1	8.5	8.0
1226	8.3	7.9	8.3	7.7	7.2
1224	7.6	7.1	7.5	6.9	6.4
1222	6.8	6.2	6.7	11.5	11.2
1220				10.7	10.4
1218				9.9	9.6
1216				9.1	8.7
1214				8.3	7.9
1212				7.5	7.1
1210				6.6	6.3
1208				5.8	5.5

OPERATING INSTRUCTIONS

The Ice Prevention-Air System is designed to discharge from two to three cubic feet of free air per minute per nozzle from a depth of eight to fifty-one feet below the water surface. To discharge more than three cubic feet of free air per minute per nozzle, operate parts of the system intermittently.

The table shows gage readings calculated to discharge two cubic feet of free air per minute per nozzle for water surface elevations of two foot intervals. The curve shows the cubic feet of free air per minute that will be discharged through the air nozzles for differential pressures across the nozzles from zero to thirteen pounds per square inch.

The air compressors are to be put in service and operated according to instructions furnished by the manufacturer.

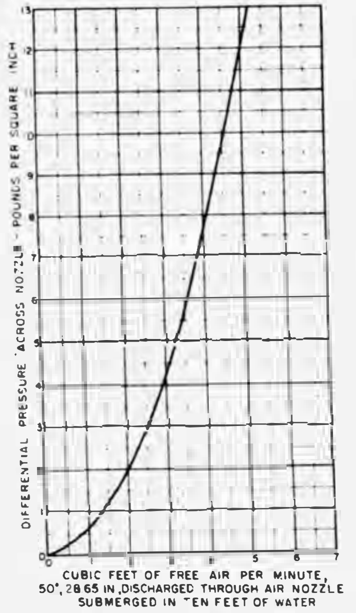
Determine which nozzles to use from the water surface elevation. The operating ranges for air nozzles at different elevations and locations are shown on the diagram. When air is first supplied from the control header to the nozzles valves "H", "J" and the control valve corresponding to the nozzle elevation should be opened wide, so that a maximum flow of air will blow the water out of the riser pipes. The flow of air may then be adjusted by the control valve until the gage reading shows the desired amount of air flowing. When the system is operating the accumulated moisture should be drained from the line through valve "K".

When the system is not in operation, valves "A", "B", "C", "D", "E", "F", "G", "H", and "N" should be closed and valves "J", "K", and "M" should be opened.

High pressure air from the power plant may be obtained for supplementary service and blowing-out lines by opening valve "N". To protect air compressor's, valve "M" should be closed before opening valve "N".

EXPLANATION

A - Control valve - W.S. El. 1273.50 to W.S. El. 1290.00
B - Control valve - W.S. El. 1248.00 to W.S. El. 1273.50
C - Control valve - W.S. El. 1222.50 to W.S. El. 1248.00
D - Control valve - W.S. El. 1266.00 to W.S. El. 1290.00
E - Control valve - W.S. El. 1224.00 to W.S. El. 1266.00
F - Control valve - W.S. El. 1224.00 to W.S. El. 1248.00
G - Control valve - W.S. El. 1208.00 to W.S. El. 1224.00
H - Control header "shut-off" valve.
J - Control header throttling valve.
K - Control header drain valve.
L - Pressure gage
M - Plug valve on compressor discharge lines.
N - Plug valve on high pressure air connections.



PRESSURE - DISCHARGE CURVE

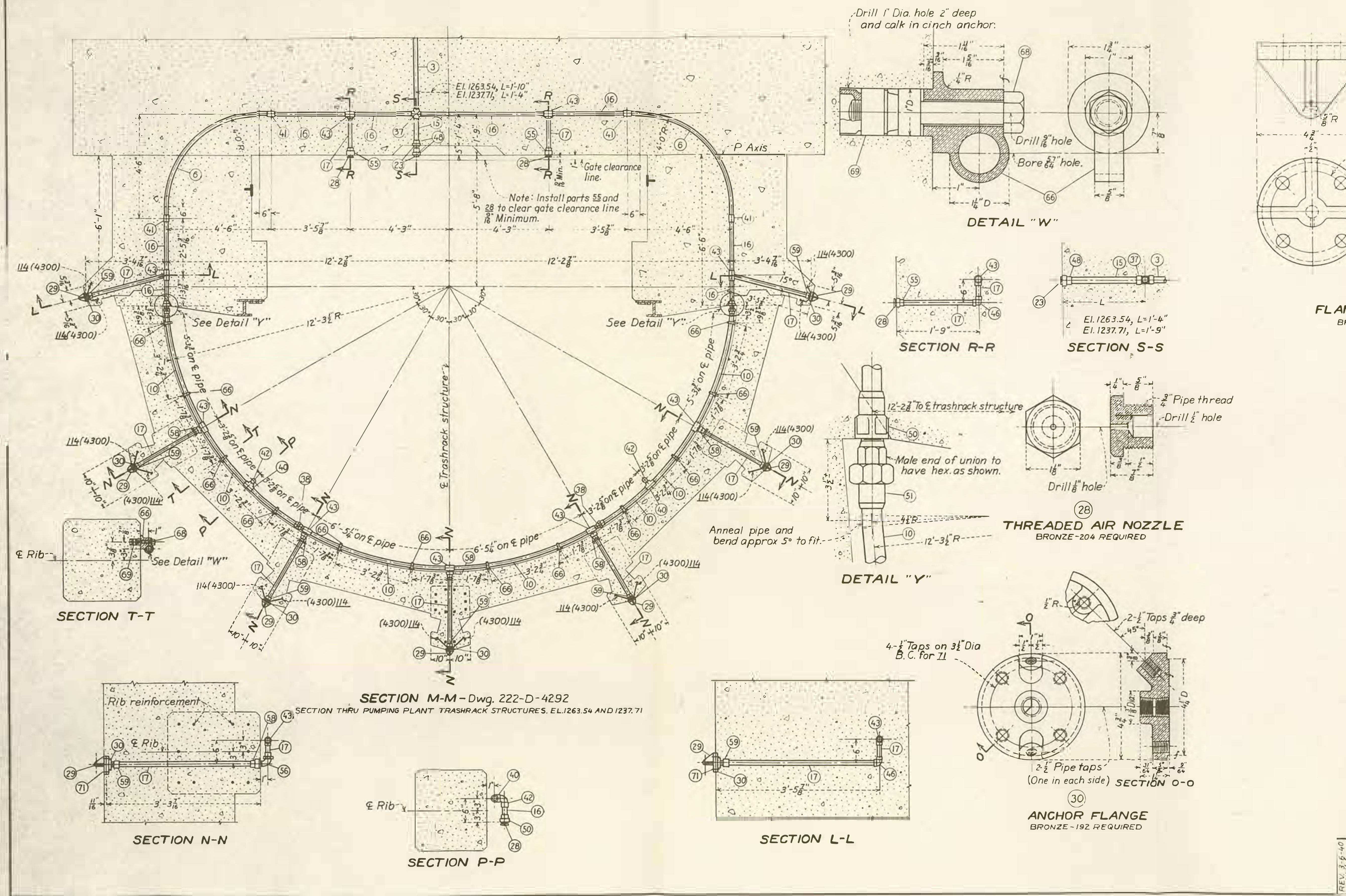
REFERENCE DRAWING
GENERAL PLAN AND SECTIONS 222-D-4287

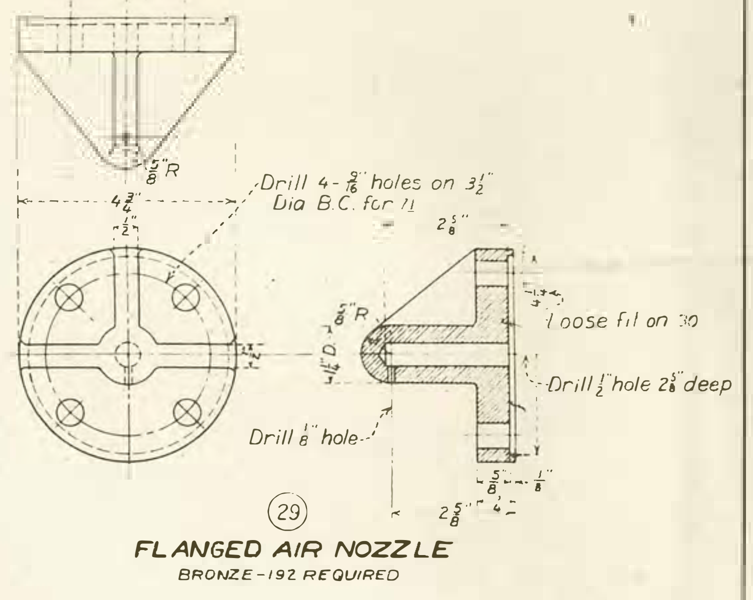
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT - WASHINGTON

GRAND COULEE DAM
ICE PREVENTION-AIR SYSTEM
OPERATING DIAGRAM AND INSTRUCTIONS

DRAWN | TRACED | CHECKED | SUBMITTED | RECOMMENDED | APPROVED
DENVER, COLORADO, NOV. 4, 1938. 222-D-4303

HYD-207





INSTALLATION NOTES

All pipe threads and flange faces shall be coated with a thin coat of red lead.

Before placing concrete close nozzle outlets with the countersunk plugs 25 or 33 and protect face of flange 30 so flange air nozzle 29 may be properly installed.

During concreting and pressure grouting maintain hydrostatic pressure in all embedded pipes as directed by the contracting officer.

After installation each unit of piping shall be air-tested at 50#/□" internal pressure.

All exposed solder and threaded joints to be soap tested while the pipe is under air pressure.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT-WASHINGTON

GRAND COULEE DAM
ICE PREVENTION-AIR SYSTEM
PUMPING PLANT TRASHRACK STRUCTURE
AIR DISTRIBUTION PIPING

DRAWN W.G.S. SUBMITTED L. N. McClellan
TRACED E.M.W. RECOMMENDED
CHECKED W.H.S. APPROVED

REV. 3-8-40

DENVER, COLORADO, SEPT. 25, 1939 SHEET 2 OF 3 222-D-4293

HYD-207

FIGURE 34

LIST OF PARTS

PART No.	DESCRIPTION	MATERIAL	LOCATION OF PARTS: MAIN UNIT TRASHRACK STRUCTURES		STATION SERVICE TRASHRACK STRUCTURES		PUMPING PLANT TRASHRACK STRUCTURES		DRUM GATE SPILLWAY		TOTAL QUANTITY REQ'D ON SUPPLY CONTRACT	PART No.
			QUANTITY	DWG. No. 222-D-	QUANTITY	DWG. No. 222-D-	QUANTITY	DWG. No. 222-D-	QUANTITY	DWG. No. 222-D-		
1	1" Pipe bend (detailed)	Copper	108	4288							108	1
2	1" Pipe bend (detailed)	Copper	3	4288							3	2
3	1" Pipe bend (detailed)	Copper			18	4290	48	4292	44	4296	110	3
4	1" Pipe bend (detailed)	Copper					12	4292			12	4
5	1" Pipe bend (detailed)	Copper							11	4296	11	5
6	3/4" Pipe bend	Copper	108	4289			72	4293			180	6
7	3/4" Pipe, 20'-0" lengths, bent to 16'-9" radius	Copper	135	4289							* 138	7
8	3/4" Pipe bend	Copper			18	4291					18	8
9	3/4" Pipe 20'-0" lengths, bent to 5'-11" radius	Copper			9	4291					9	9
10	3/4" Pipe 20'-0" lengths, bent to 12'-3 1/2" radius	Copper					44	4293			* 45	10
11	3/4" Pipe 20'-0" lengths, bent to 12'-8 7/8" radius	Copper					23	4294			* 24	11
12	1/2" Pipe bend	Copper			18	4291					18	12
13	1/2" Pipe bend (detailed)	Copper					60	4294			60	13
14	1" Pipe. Std. I.P.S. random lengths, threaded and coupled	Brass	115 ft.	4288	20 ft	4290	70 ft	4292	70 ft.	4296	* 300 ft.	14
15	1" Pipe 20'-0" lengths	Copper	120	4288	21	4290	48	4292	52	4295	* 247	15
16	3/4" Pipe 20'-0" lengths	Copper	100	4289	7	4291	45	4293-94	142	4295	* 300	16
17	1/2" Pipe 20'-0" lengths	Copper	101	4289	6	4291	47	4293-94	26	4295	185	17
18	1"-45°-125# Elbow, screwed I.P.S.	Bronze	54	4288	9	4290	36	4292	33	4296	* 138	18
19	1"-150# Union, screwed. I.P.S.	Bronze	36	4288	6	4290	24	4292	22	4296	* 92	19
20	1" Close nipple. Std. I.P.S.	Bronze	72	4288	12	4290	48	4292	44	4296	* 184	20
21	1"-125# Street tee, screwed. I.P.S.	Bronze	18	4288	3	4290	12	4292	11	4296	* 45	21
22	1"-125# Y type strainer, screwed. I.P.S.	Bronze	18	4288	3	4290	12	4292	11	4296	44	22
23	1" Countersunk plug. I.P.S.	Bronze	54	4289	9	4291	36	4293-94	22	4295	* 126	23
24	3/4" Countersunk plug I.P.S.	Bronze			18	4291					* 20	24
25	3/4" Countersunk plug. I.P.S.	Bronze	270		9		204		264		765	25
26	3/4" x 1/2"-90°-125# Street elbow, screwed. I.P.S.	Bronze	54	4289							* 56	26
27	1/2" x 1/4" Bushing, screwed. I.P.S.	Bronze	18	4288	3	4290	12	4292	11	4296	* 45	27
28	Threaded air nozzle (detailed)	Bronze	270	4289	9	4291	204	4293	264	4296	* 765	28
29	Flanged air nozzle (detailed)	Bronze	378	4289	45	4291	192	4293			* 625	29
30	Anchor flange (detailed)	Bronze	378	4289	45	4291	192	4293			* 625	30
31	1" Pipe flange (detailed)	Bronze	54	4288	9	4290	36	4292	22	4296	* 125	31
32	1" Copper x 4 1/4" O.D. companion flange	Bronze	54	4288	9	4290	36	4292	22	4296	* 125	32
33	1/2" Countersunk plug. I.P.S.	Bronze	378		45		192				630	33
34	Tee, 1" copper x 1" copper x 1" copper	Bronze	54	4288	9	4290	36	4292	22	4296	* 125	34
35	Expansion joint, 1" copper x 1" copper	Bronze	27	4288	3	4290					* 31	35
36	Coupling, 1" copper x 1" copper	Bronze	264	4288	45	4290	132	4292	151	4295	* 610	36
37	Cross, 1" copper x 1" copper x 3/4" copper x 3/4" copper	Bronze	54	4289	9	4291	36	4293-94	22	4295	* 125	37
38	Union, 3/4" copper x 3/4" copper	Bronze	108	4289	9	4291	72	4293-94			* 195	38
39	Expansion joint, 3/4" copper x 3/4" copper	Bronze							66	4295	* 67	39
40	Tee, 3/4" copper x 3/4" copper x 3/4" copper	Bronze	108	4289	36	4291	48	4293			* 198	40
41	Coupling, 3/4" copper x 3/4" copper	Bronze	451	4289	52	4291	255	4293-94	142	4295	* 925	41
42	90° Elbow, 3/4" fitting x 3/4" copper	Bronze	108	4289			48	4293			* 162	42
43	Tee, 3/4" copper x 3/4" copper x 1/2" copper	Bronze	486	4289	36	4291	264	4293-94	220	4295	* 1025	43
44	90° Elbow, 3/4" copper x 1/2" copper	Bronze			18	4291			44	4295	* 64	44
45	Coupling, 1/2" copper x 1/2" copper	Bronze	101	4289	6	4291	47	4293-94	26	4295	* 194	45
46	90° Elbow, 1/2" copper x 1/2" copper	Bronze	216	4289	9	4291	144	4293-94			* 378	46
47	Coupling, 1" copper x 1" outside I.P.S.	Bronze	18	4288	3	4290	12	4292	11	4296	* 46	47
48	Coupling, 1" copper x 1" inside I.P.S.	Bronze	54	4289	9	4291	36	4293-94	22	4295	* 125	48
49	90° Elbow, 1" copper x 1/2" inside I.P.S.	Bronze	18	4288	3	4290	12	4292	11	4296	* 46	49
50	Coupling, 3/4" copper x 3/4" inside I.P.S.	Bronze	216	4289	36	4291	120	4293-94			* 382	50
51	Union, 3/4" copper x 3/4" outside I.P.S.	Bronze	108	4289			72	4293-94			* 185	51
52	90° Union elbow, 3/4" copper union end x 3/4" outside I.P.S.	Bronze			18	4291					* 19	52
53	Tee, 3/4" copper x 3/4" copper x 3/4" inside I.P.S.	Bronze					24	4294			* 25	53
54	Union tee (detailed)	Bronze					60	4294			* 62	54
55	Coupling, 1/2" copper x 3/4" inside I.P.S.	Bronze	108	4289	9	4291	132	4293-94	264	4295	* 525	55
56	90° Union elbow, 1/2" copper union end x 1/2" outside I.P.S.	Bronze	216	4289	27	4291	120	4293			* 372	56
57	Union tee, 1/2" outside I.P.S. x 1/2" inside I.P.S. x 1/2" copper union outlet	Bronze	54	4289							* 56	57
58	Coupling, 1/2" copper x 1/2" inside I.P.S.	Bronze	270	4289	27	4291	180	4293-94			* 488	58
59	Coupling, 1/2" copper x 1/2" outside I.P.S.	Bronze	378	4289	45	4291	192	4293-94			* 630	59
60	1"-150# Wedge disc gate valve, screwed. I.P.S.-R.S.	Brass	18	4288	3	4290	12	4292	11	4296	44	60
61	1"-150# Plug type disc globe valve, screwed. I.P.S.	Brass	36	4288	6	4290	24	4292	22	4296	88	61
62	150# Wedge disc gate valve, 1" copper x 1" copper. R.S.	Brass	54	4288	9	4290	36	4292	22	4296	121	62
63	Gage cock, tee head. 1/4" inside I.P.S. x 1/4" outside I.P.S.	Brass	18	4288	3	4290	12	4292	11	4296	44	63
64	Pressure gage	Brass	18	4288	3	4290	12	4292	11	4296	44	64
65	1" Pipe clamp (detailed)	Copper	36	4288	6	4290	24	4292	22	4296	* 90	65
66	3/4" Pipe support (detailed)	Bronze	648	4289	36	4291	432	4293			* 1140	66
67	3/8" Dia. x 2 1/2" long drive in stud type concrete anchor with hex. nut	Steel, galv.	72	4288	12	4290	48	4292	44	4296	* 180	67
68	1/2" Dia. x 3 1/4" long hex. head cap screw	Naval Brass	648	4289	36	4291	432	4293			* 1150	68
69	1/2"-2 Unit threaded cinch anchor Phospher bronze male parts	Bronze, Lead	648	4289	36	4291	432	4293			* 1150	69
70	1/2" Dia. x 1" long hex. head cap screw	Naval Brass	216	4288	36	4290	144	4292	88	4296	* 510	70
71	1/2" Dia. x 1 3/8" long hex. head cap screw	Naval Brass	1512	4289	180	4291	768	4293-94			* 2510	71
72	Hair felt. 1" thick x 3'-0" wide		12 ft.	4288	2 ft	4291			24 ft.	4295	* 40 ft.	72
73	Solder, 1/8" round wire. 5# spools (95% Tin and 5% Antimony)		15		2		8		5		* 30	73
74	Solder flux. 1# tins		9		1		5		3		* 20	74

GENERAL NOTE

All solder joint type of fittings shall be made of cast bronze alloy containing 85% copper, 5% tin, 5% zinc, and 5% lead. All joints indicated as copper connections are to be properly sized and finished for capillary type of solder joints. All joints indicated as iron pipe size connections shall have full clean-cut American Standard pipe threads. Companion flange <u>32</u> shall be plain faced with flange diameter, thickness, and drilling in accordance with M.S.S. 150 pound SP Bronze Flange Standard All solder joint type of fittings to be air tested under water at 90#/□" gage pressure.

All iron pipe size fittings shall be 125# American Standard banded fittings made from cast bronze containing 85% copper, 5% tin, 5% zinc, and 5% lead. Threads shall be clean-cut American Standard type of pipe threads. All fittings shall be air tested under water at 125#/□" gage pressure.

All unions shall be of the ground joint type.

Threaded air nozzle, flanged air nozzle, anchor flange, 1" pipe flange, and 3/4" pipe support; part numbers 28, 29, 30, 31 and 66 respectively, shall be made of cast bronze alloy containing 85% copper, 5% tin, 5% zinc, and 5% lead.

All copper pipe shall be type "K", hard copper, and conform to Federal Specification WW-T-799 and be furnished in length specified. All bends to be made up complete by manufacturer. A light anneal of the pipe for making the bends will be permitted, so that the bending will return the pipe to it's approximate initial hardness.

All Standard iron pipe size brass pipe shall conform to the requirements for grade "A" water pipe as prescribed in Federal Specification WW-P-351 and be furnished semi-annealed. Pipe to be furnished in random lengths, threaded and coupled.

All cap screws shall be hex. head semi-finished naval brass cap screws with U.S. Standard threads.

Parts shall be marked or tagged with drawing numbers and part numbers, thus: 4289-27, 4291-27, 4293 & 94-27, etc.

* Total quantities listed include excess for loss or damage of parts by the field.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT - WASHINGTON

GRAND COULEE DAM

ICE PREVENTION - AIR SYSTEM

LIST OF PARTS AND GENERAL NOTE

DRAWN J.C.W. SUBMITTED L. N. McClellan
TRACED F.C.-R.L.A. RECOMMENDED J. L. Savage
CHECKED W.H.S. P.B. APPROVED [illegible]

DENVER, COLORADO, SEPTEMBER 25, 1939