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Boulder Dam

HYD 7.1

J. E. WARNOCK HYD-7.1

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

Technical Memorandum No. 525

HYDRAULIC MODEL EXPERIMENTS FOR THE DESIGN OF THE BOULDER DAM

BOOK 5

MODEL STUDIES OF APPURTENANT STRUCTURES

By

J. N. BRADLEY, ASSISTANT ENGINEER
and
J. E. WARNOCK, RESEARCH ENGINEER

Denver, Colorado,
June 4, 1936.

UNITED STATES
DEPARTMENT OF THE INTERIOR
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MEMORANDUM TO CHIEF DESIGNING ENGINEER

SUBJECT: HYDRAULIC MODEL EXPERIMENTS

FOR THE DESIGN OF THE BOULDER DAM

BOOK 5

MODEL STUDIES OF APPURTENANT STRUCTURES

- - -

By J. N. BRADLEY, ASSISTANT ENGINEER

AND

JACOB E. WARNOCK, RESEARCH ENGINEER

- - -

Under direction of

E. W. LANE, RESEARCH ENGINEER

- - -

TECHNICAL MEMORANDUM NO. 525

Denver, Colorado,

June 4, 1936.

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PREFACE

The hydraulic model experiments for the design of the Boulder Dam, as described in this report, were made in the laboratory of the Colorado Agricultural Experiment Station, Fort Collins, Colorado, under the direction of Jacob E. Warnock, Research Engineer, with the exception of a portion of the experiments described in chapter VI. These were made at the laboratory of the U. S. Bureau of Reclamation, Montrose, Colorado, with W. M. Borland, Associate Engineer, in charge.

The tests described under chapters IV and V were conducted and the report prepared by J. N. Bradley, Assistant Engineer, with the exception of section 4, chapter IV which was written by S. P. Wing, Engineer. Chapters VI and VII were prepared by Jacob E. Warnock, Research Engineer. The work was instigated under the general supervision of E. W. Lane, former Research Engineer, and was completed under the direction of Jacob E. Warnock, now acting in that capacity. These studies were made under the general supervision of J. L. Savage, Chief Designing Engineer. All engineering work of the Bureau of Reclamation was under the direction of R. F. Walter, Chief Engineer, and all activities of the Bureau were under the direction of Dr. Elwood Mead, Commissioner.

It is desired to acknowledge the cooperation of the Colorado State Board of Agriculture (governing board of Colorado State College of Agriculture and Mechanical Arts) and the staff of the U. S. Bureau of Agricultural Engineering in extending the Bureau of Reclamation the use of the hydraulic laboratory of the Colorado Agricultural Experiment Station for conducting the experiments on the models of the Boulder Dam and its appurtenant structures.

As this report was made possible by the cooperative efforts of a number of individuals, it is desired to acknowledge the activities of W. H. Price, W. M. Borland, C. W. Thomas, J. W. Ball, A. N. Smith, W. J. Colson, and W. O. Parker who assisted in the construction work and laboratory testing, and H. W. Brewer and J. D. McCrum who assisted in the preparation of this report. Appreciative acknowledgment is also made to S. P. Wing, Engineer, who contributed to this report and cooperated in the analysis of the results.

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CONTENTS (continued)

CONTENTS (continued)

FIGURES

FIGURES (continued)

FIGURES (continued)

TABLES

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PHOTOGRAPHS (continued)

I. INTRODUCTION

The Boulder Dam structures as outlined in the preliminary designs far surpassed any similar structures constructed in the past. In so far as it was possible before proceeding to the final designs, careful investigation was made as to the validity of the existing formulas. Hydraulic formulas and behavior were particularly in question, since, as is well known, the formulas are largely empirical. Early hydraulic model tests of the side-channel spillways described in Books 1 to 3 of "Hydraulic Model Experiments for the Design of the Boulder Dam" having given results of great value for design purposes, it was decided to test by model other features. This report describes the hydraulic tests for the following:

(1) The junction between the 13-foot penstocks and the 30-foot penstock header.

(2) The hydraulic losses and pressure conditions in the intake towers.

(3) The flow characteristics of the needle valves in the tunnel plug outlets.

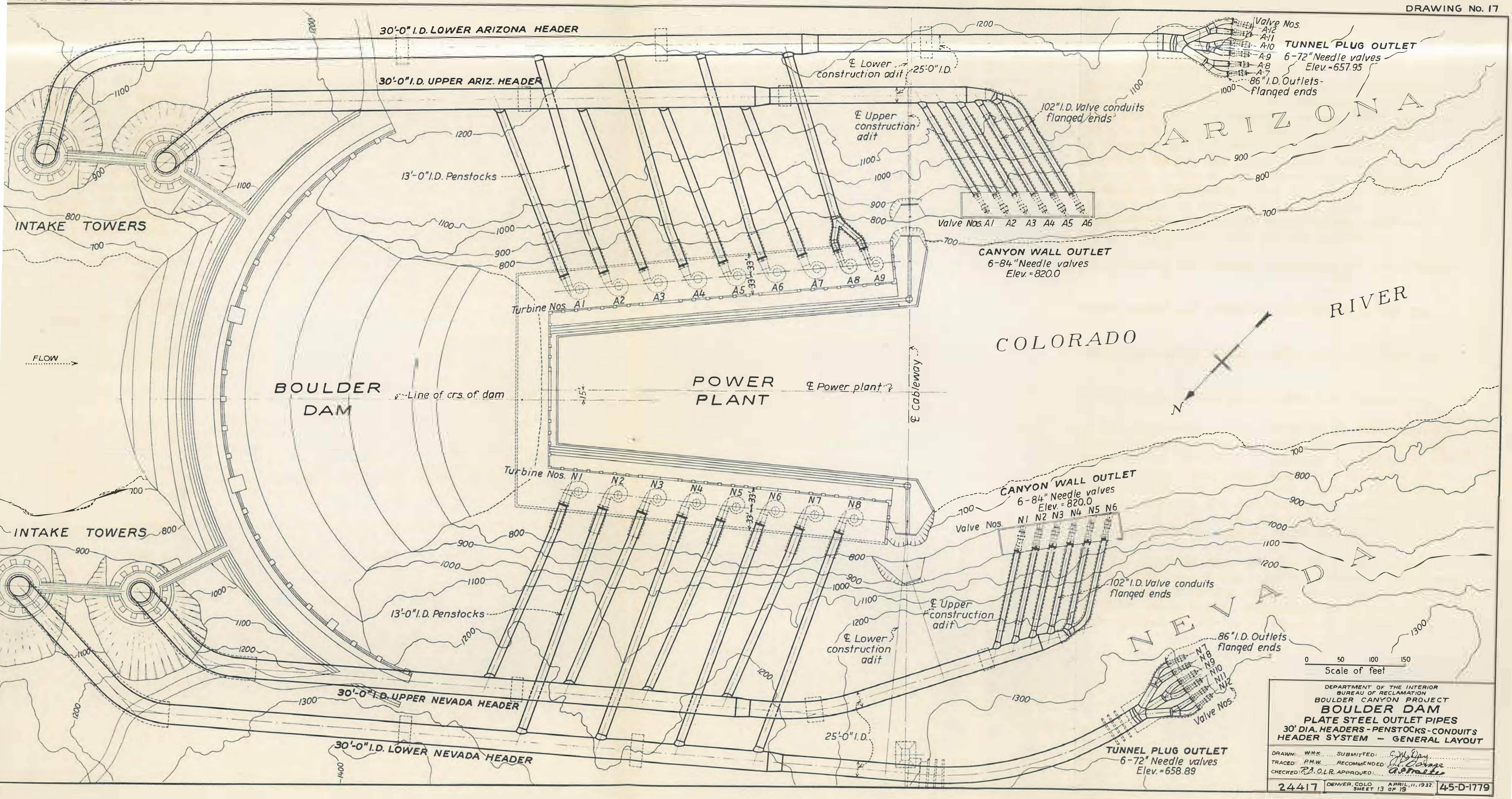
(4) Hydraulic conditions in the river below the powerhouse.

Aside from the validation of the features of the hydraulic design, these tests gave information which could have been obtained in no other way concerning the formation of eddies, distribution of pressures and hydraulic losses. They made possible a material improvement of the flow conditions in the 50-foot concrete-lined tunnel below the tunnel-plug outlets, the elimination of necessary piping and detail, and the formulation of an operating program for different combinations of flow from the outlet works and the spillways.

This report was compiled as a description of the laboratory procedure and as an analysis of the results procured. The location and relationship of the structures studied are shown on figure 1. The design data from which these models were constructed were obtained from the design offices of the U. S. Bureau of Reclamation, Denver, Colorado.

FIG. 1

DRAWING No. 17



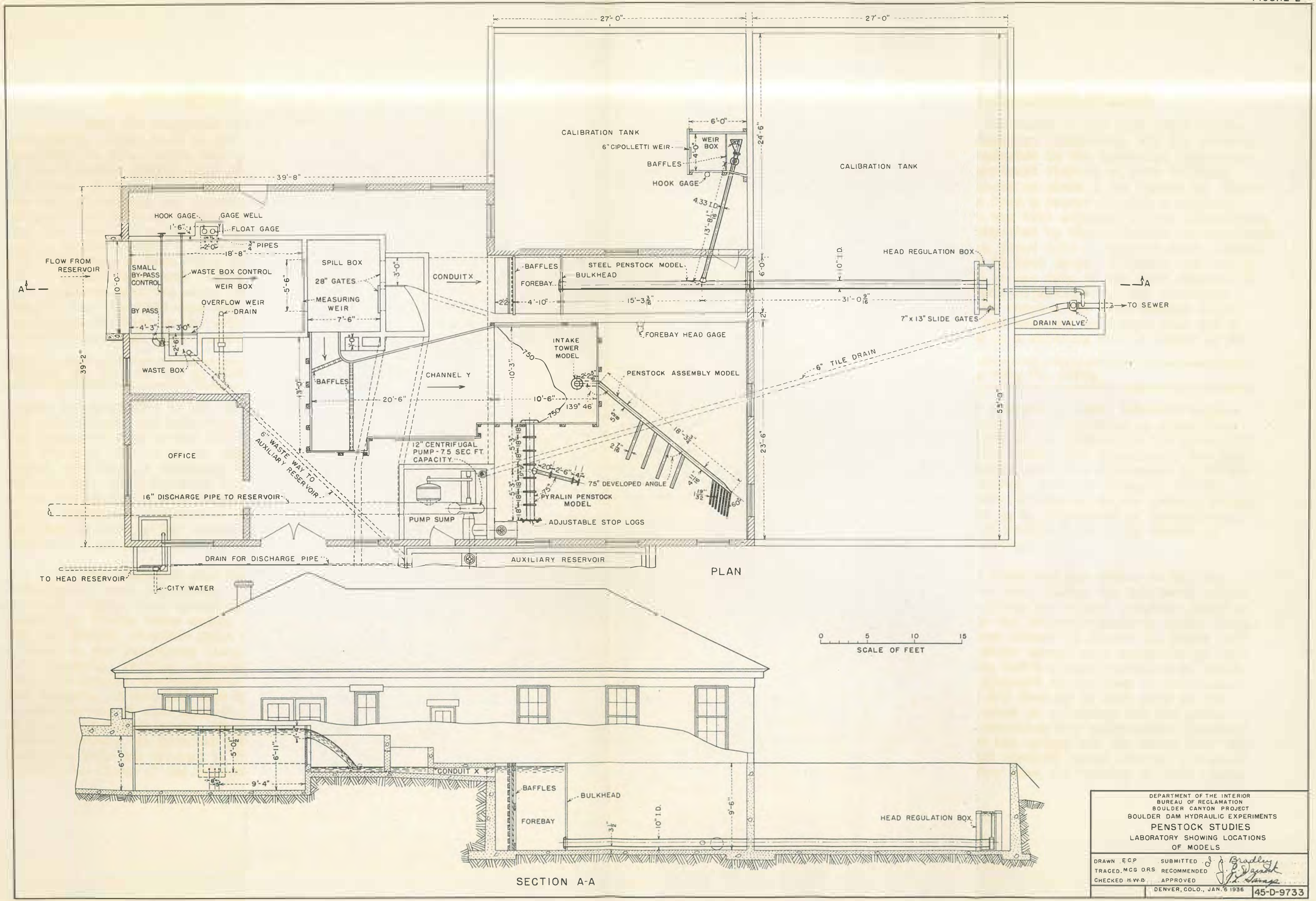
II. THE LABORATORY AND GENERAL EQUIPMENT

With the exception of the tests on the 1:20 model of the tunnel-plug outlet made in the Montrose laboratory, all the studies described in this report were conducted in the hydraulic laboratory of the Colorado Agricultural Experiment Station at Fort Collins, Colorado. A general layout is shown on plate I and figure 2. Water for the experiments was supplied from a reservoir with a capacity of 30,000 cubic feet, located on the hill adjacent to the laboratory. The flow from the reservoir, controlled by three 12-inch hand-operated gates, passed through a diverging flume into a concrete weir channel, $19\frac{1}{2}$ feet long, 10 feet wide, and $7\frac{1}{4}$ feet deep. A bypass gate and a 4-inch bypass valve were located in one side of the weir channel, 13 feet upstream from the weir. Small adjustments of the discharge passing over the weir were made by varying the flow through these bypasses. The head on the weir was measured by a hook gage and a Cornell-type float gage[1] located in a stilling well 9 inches by 24 inches and connected to the weir channel by three 3/4-inch pipes. The weir channel and measuring weir are illustrated on plate I-C. The bypass weir is on the left side of the channel in the photograph and the gage well is on the right. During the experiments two weir plates were used, a 90-degree V-notch weir for discharges up to two second-feet, and a two-foot Cipolletti weir for flows of two to eight second-feet. Both weirs were calibrated in the setting shown, by the Irrigation Division of the U. S. Department of Agriculture, and were checked by the laboratory staff of the U. S. Bureau of Reclamation prior to the model testing.

[1] Trans. Am. Soc. C. E., p. 1154, vol. 83, 1920.

The equipment thus far described was common to all the experimental work performed. From this point, the equipment varied for each model. When operating either the visual penstock junction model, the intake tower model, or the penstock assembly model (fig. 2), the 28-inch diverter gates were closed to divert the water into a flume on the laboratory floor which served as a reservoir as well as a channel. In this, adjustable baffles were installed to eliminate undesirable cross-currents produced by the bend at the head of the channel. A tank $10\frac{1}{2}$ feet by $10\frac{1}{4}$ feet by 12 feet deep at the downstream end of this channel served as a forebay for the group of models just mentioned. When operating the quantitative penstock junction model, one diverter gate was opened and the water from the measuring weir was allowed to flow straight ahead through a channel beneath the floor of the laboratory into the forebay of this model.

FIGURE 2



CALIBRATION TANK
6" CIPOLLETTI WEIR
WEIR BOX
BAFFLES
HOOK GAGE
CALIBRATION TANK
HOOK GAGE
GAGE WELL
FLOAT GAGE
3" PIPES
FLOW FROM RESERVOIR
SMALL BY-PASS CONTROL
WASTE BOX CONTROL
WEIR BOX
SPILL BOX
28" GATES
CONDUIT X
BAFFLES
STEEL PENSTOCK MODEL
FOREBAY
BULKHEAD
HEAD REGULATION BOX
TO SEWER
7" x 13" SLIDE GATES
DRAIN VALVE
OVERFLOW WEIR DRAIN
BY PASS
MEASURING WEIR
FOREBAY HEAD GAGE
INTAKE TOWER MODEL
6" TILE DRAIN
PENSTOCK ASSEMBLY MODEL
WASTE BOX
BAFFLES
CHANNEL Y
OFFICE
6" WASTE WAY TO AUXILIARY RESERVOIR
12" CENTRIFUGAL PUMP - 7.5 SEC FT. CAPACITY
75° DEVELOPED ANGLE
PYRALIN PENSTOCK MODEL
16" DISCHARGE PIPE TO RESERVOIR
PUMP SUMP
ADJUSTABLE STOP LOGS
DRAIN FOR DISCHARGE PIPE
AUXILIARY RESERVOIR
TO HEAD RESERVOIR
CITY WATER
PLAN
0 5 10 15
SCALE OF FEET
CONDUIT X
BAFFLES
BULKHEAD
FOREBAY
10" I.D.
HEAD REGULATION BOX
SECTION A-A
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS
PENSTOCK STUDIES
LABORATORY SHOWING LOCATIONS OF MODELS
DRAWN E.C.P.
TRACED M.C.G. O.R.S.
CHECKED H.W.B.
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLO., JAN. 6 1936
45-D-9733

A one-inch round rod extending from the ceiling of the laboratory to the floor of the laboratory tanks was installed in a central location as a mounting for a forebay head gage. By means of a pin and clamp, a hook gage with a range of two feet could be mounted on the rod at any hole. A stilling well in which the hook operated was held in position on the rod by a friction clamp and could be connected hydraulically by a 3/4-inch rubber hose to the forebay of any model. The forebay heads, which varied over a wide range, were obtained in this manner.

III. DEFINITION OF SYMBOLS

Penstock Studies

Q_a = Total discharge, second-feet.
Q_s = Discharge through branch, second-feet.
Q_b = Discharge in main pipe below junction, second-feet.
A_a = Area of main pipe above junction, square feet.
A_s = Area of branch, square feet.
A_b = Area of main pipe below junction, square feet.
$V_a = \frac{Q_a}{A_a}$ = Mean velocity in main pipe upstream from junction, feet per second.
V_s = Mean velocity in branch, feet per second.
V_b = Mean velocity in main pipe below junction, feet per second.
D_a = Average diameter of main pipe above junction, feet.
D_s = Average diameter of branch, feet.
D_b = Average diameter of main pipe below junction, feet.
$\frac{Q_s}{Q_a}$ = Ratio of branch discharge to total discharge.
h_v = Velocity head, feet.
P_1 = Pressure head at piezometer 1, feet.
P_2 = Pressure head at piezometer 2, feet, etc.
P_x = Average of pressure heads at piezometers 2, 3, 4, and 5, feet.
S = Friction loss per foot of straight pipe, feet of water.
$h_{f(x-7)}$ = Pipe friction loss from P_x to piezometer 7, feet of water.
$h_{f(6-36)}$ = Pipe friction loss from piezometer 6 to 36, etc.
J_b = Junction loss in main pipe, feet.
J_s = Junction loss in branch, feet.
$\frac{J}{V^2/2g}$ = Junction loss coefficient.

$R = \frac{V.D}{\nu}$ = Reynolds' number.

$F = \frac{V^2}{g \cdot D}$ = Froude's number.

ν = Kinematic viscosity $= \frac{u}{\rho}$, square feet per second.

u = Absolute viscosity $= \frac{0.00003716}{0.4712 + 0.01435T + 0.0000682T^2}$, $\frac{\text{lb. sec.}}{\text{ft.}^2}$

where T is the temperature in degrees Fahrenheit.

ρ = Density of water at T degrees Fahrenheit, $\frac{\text{lb. sec.}^2}{\text{ft.}^4}$

β $\bar{v}$ = Average velocity where

$\beta = \frac{\int v^2 dA}{V^2 A}$ = Coefficient of velocity where $V = \frac{Q}{A}$ = mean velocity.

αH_v = True velocity head where

$$\alpha = \frac{\int v^3 dA}{V^3 A} = \text{Energy head coefficient.}$$

Intake Tower Studies

d_1 = Difference in elevation between the water surface outside and inside the tower, feet (model).

d_2 = Difference in elevation between the reservoir surface and the reading of piezometer 45 (model).

d_3 = Difference in elevation between the reservoir surface and the reading of any one of the five piezometer rings A, B, C, D, and E located below the tower (model).

D_1 = Difference in elevation between the water surface outside and inside the tower (prototype).

h_t = Total loss through tower, feet of water.

h_{eu} = Entrance loss through upper gate, feet of water.

h_{eb} = Entrance loss through lower gate, feet of water.

h_{pu} = Average head required inside tower to change direction of flow at upper gate (model).

h_{pb} = Average head required inside tower to change direction of flow at lower gate (model).

H_p = Average head required inside tower to change direction of flow at either gate (prototype).

$h_{f(45-u)}$ = Pipe friction from piezometer 45 to bottom of the upper gate (model).

$h_{f(C-b)}$ = Pipe friction from piezometer ring C to the bottom of the lower gate (model).

h_b = Compound bend loss in main header directly below intake tower.

IV. PENSTOCK JUNCTION LOSS TESTS

1. PURPOSE

While analyzing the design of the 30-foot headers, turbine penstocks, and outlet works, it was realized that the junctions between the 30-foot headers and the 13-foot turbine penstocks were the potential sources of excessive losses of head which might be reduced and thus increase the effective head on the turbines. Losses of head below the turbine penstocks would reduce the discharge capacity of the outlet needle valves and any unnecessary loss of head between the reservoir and the turbine penstocks will be reflected in the power output at such time in the future as the total output of the power plant has been absorbed by the market. With the long radius bends incorporated in the design of the 30-foot headers, practically the only possibility of decreasing the head loss would be to improve the efficiency of the junctions between the headers and turbine penstocks. It was for this purpose that a series of extensive tests both qualitative and quantitative were made on models of one of these junctions.

As a preliminary and qualitative study, a model representing the junction between the 30-foot upper Arizona header and the 13-foot penstock leading to turbine A-1 (fig. 1) was built of transparent pyralin for observation. This particular junction was selected as it was possible to have at this point a wide variation of flow ranging from 100 percent of the total through the main penstock and no flow through the branch to 100 percent through the branch and no flow in the main penstock below the junction. The actual ratio of the discharge in the penstock to the flow in the branch depends entirely upon the manner of operation downstream. With this junction as a major point of interest, it was desired to determine visually the path of the water after it had entered the branch for various combinations of flow in the system.

In the qualitative or visual tests, it was desired to determine the location of eddies and disturbances in the branch for the varying ratios of the discharge in the branch to the discharge in the main header. Filler blocks of different dimensions were used to alter the physical shape of the junction in an effort to reduce the disturbances and hence the losses. Tests made for this purpose are grouped as follows:

(1) Tests on the junction as designed with different increments of discharge and discharge ratios by means of which the form of the entrance eddy was determined.

(2) Tests outlining the eddy for the case in which a deflecting

block was introduced into the main pipe in an endeavor to reduce the eddy loss in the branch even though it increased the loss in the main header.

(3) Tests in which the space occupied by the eddy in the branch was filled by a solid block with the anticipation of reducing the hydraulic losses and decreasing the construction cost.

The quantitative tests were made on a model of the same junction. This model was carefully built of galvanized iron so that the actual losses in the junction could be evaluated numerically. Incidental tests were made to determine the actual effect attained by the installation of the filler blocks designed as a result of the observations on the visual model.

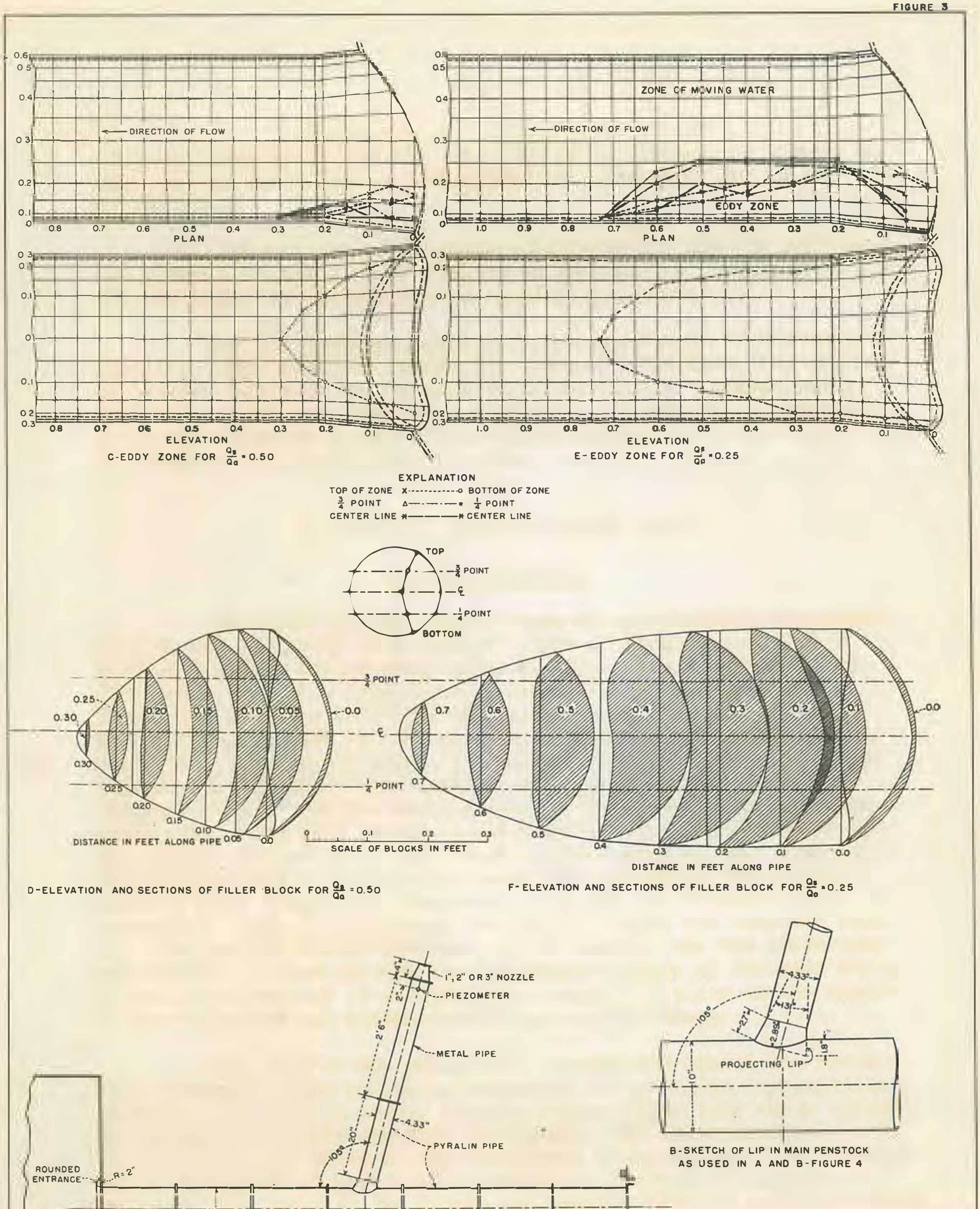
2. PENSTOCK JUNCTION VISUAL TESTS

A. The Apparatus

The model for the visual tests was constructed on a scale of 1:36 and consisted of a straight piece of 10-inch pipe with a 4.33-inch branch intersecting it at an angle of 105 degrees in a downstream direction (fig. 3A and plate II). A portion of the branch and also the main pipe was graduated at intervals of five-hundredths of a foot in both the longitudinal and circumferential direction, as shown on plate II. Three metering nozzles (1, 2, and 3 inches in diameter) were used interchangeably at the end of the branch to obtain various combinations of discharge at the junction, while small stop logs served to regulate the flow at the end of the main pipe. A piezometer was installed two inches up the branch from the upstream end of the conical nozzle for measuring the pressure at this point (fig. 3A). The total discharge was measured over the laboratory weir, the discharge through the branch was computed from the observed piezometer pressures at the nozzle, and the difference was assumed to flow through the downstream portion of the main pipe. It was not necessary to calibrate the nozzles in place as the nature of the tests did not warrant that degree of accuracy.

The path of the water as it entered the branch was traced by inserting a color solution, consisting of potassium permanganate crystals dissolved in water, through a long copper tube which extended through the nozzle and into the branch. The tube was moved about in the branch according to the directions of an observer who recorded the results with respect to the coordinate lines on the pipes. The action of the water in the branch was readily detected by this method.

FIGURE 3



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BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS

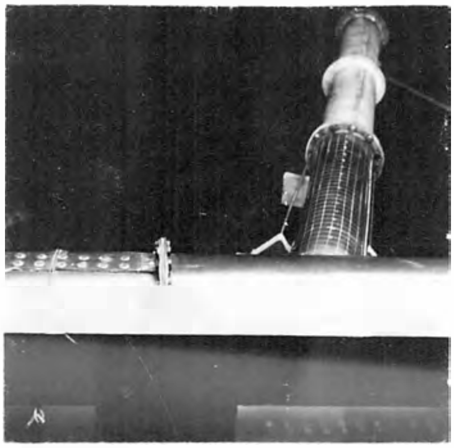
PENSTOCK STUDIES

VISUAL TESTS ON JUNCTION LOSSES

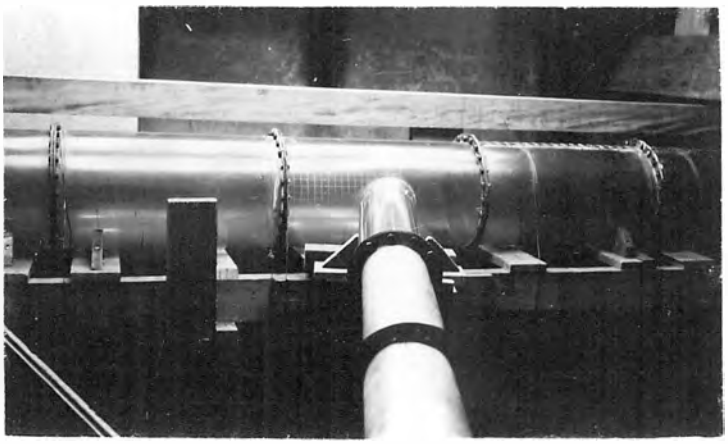
DRAWN J.N.B. SUBMITTED J. N. Bradley
TRACED S.P.C.B. O.R.S. RECOMMENDED
CHECKED J.D.M. APPROVED
DENVER, COLO., JAN. 7, 1935
45-D-9708

PLATE II

A. LOOKING DOWN THE MAIN PIPE



B. LOOKING DOWN THE BRANCH PIPE



C. LOOKING UP THE BRANCH PIPE

MODEL A

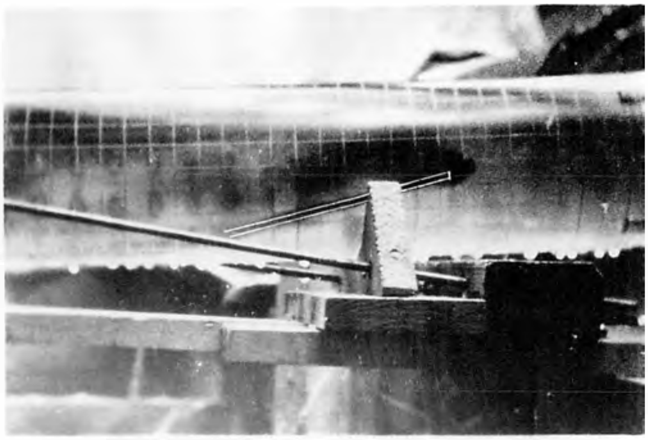
B. Results and Conclusions

As the water flowed from the main pipe into the branch, there was a zone of high velocity on the right-hand side (looking downstream on the branch, fig. 3A) while on the left-hand side a zone of relatively quiet water formed which, for lack of a better term, shall be called an "eddy zone", since it was filled with eddying water. When the color was inserted in the stream of high velocity water it was dissipated quickly by being carried down the tube and out of the nozzle, while any color inserted in the eddying zone was much more slowly dispersed. By moving the color tube back and forth the line of demarcation between the two zones could be defined. An examination of plate III will make the method clear.

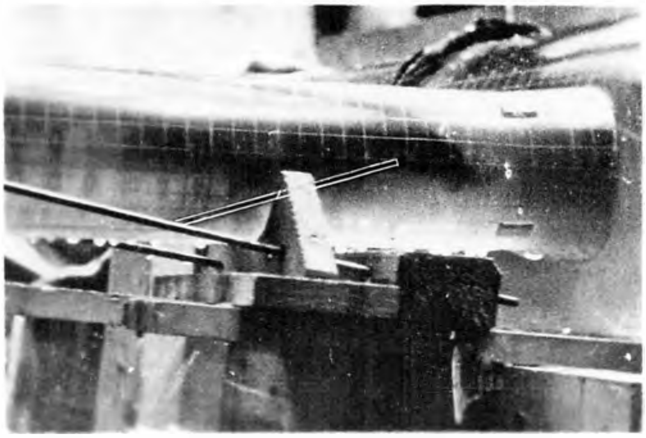
It was found that an eddy zone of relatively quiet water existed in the branch for all combinations of discharge, although the dimensions of the zone decreased as the ratio of the quantity in the branch, Q_s, to the quantity in the main pipe above the branch, Q_a, increased. A study of the various runs plotted on figures 4 and 5 will show the limits of the eddying zones. The ratio $\frac{Q_s}{Q_a}$ will in the future be referred to as the "discharge ratio."

It was thought that a lip protruding into the main pipe on the downstream side of the branch entrance might reduce the loss of head through the branch pipe, although it was expected to cause an increase in the loss in the main pipe. This lip, as shown on figure 3B, was installed during tests 1 to 3, inclusive. Eight runs were made on the three tests, and these are plotted on figures 4A and 4B. The remaining tests on this model were performed with the lip removed, and are plotted on figures 4C and 5. A summary of the conditions under which each run was made and the results obtained is shown on table I.

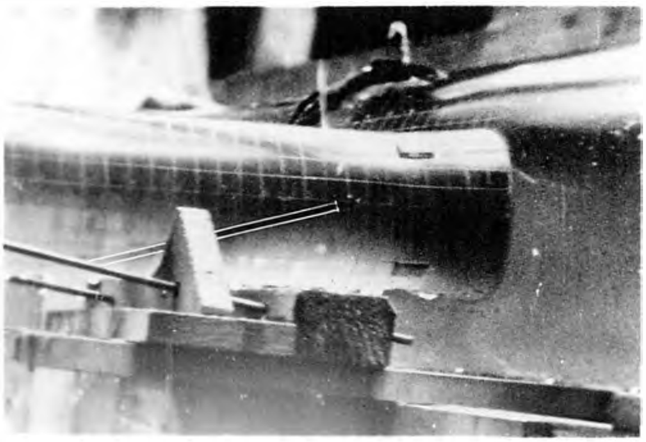
On this table, comparisons have been made, where possible, between similar runs with and without the lip to determine its effect. A comparison of tests 2-3, 2-2, 3-3, 3-2, and 3-1, respectively (made with the lip in the main pipe and denoted by letter A), with tests 4-3, 4-2, 5-3, 5-2, and 5-1, respectively (made with the lip removed and identified by the same symbol), indicates that although there is some difference in the location of the apex of the eddy zone for runs of similar discharge and similar discharge ratios, the variation is slight and occurs in both directions. That fact would indicate that the lip in the main pipe made no material difference in the shape and volume of the eddy zone, or in other words, the head loss through the branch was not materially different for either condition. As it was difficult to accurately determine the limits of the eddy zone, some variation was expected.



A. DOWNSTREAM FROM THE DEAD WATER ZONE
DYE MOVES DOWNSTREAM



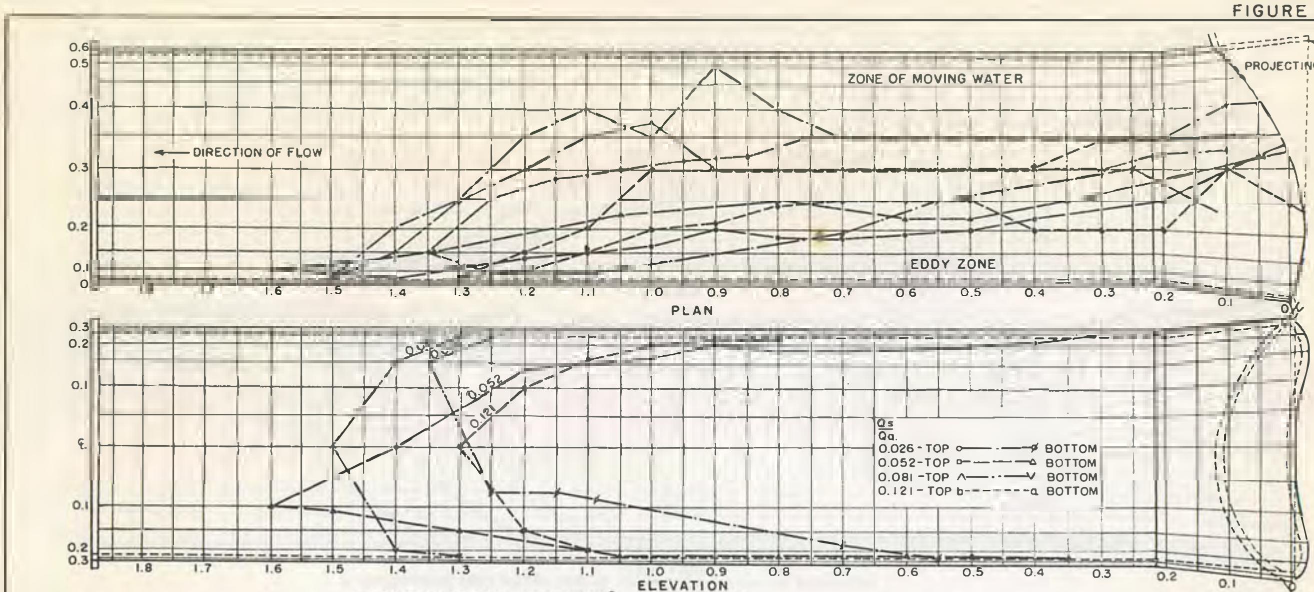
B. AT DOWNSTREAM EXTREMITY OF DEAD WATER ZONE
DYE IS DIFFUSED IN BOTH DIRECTIONS



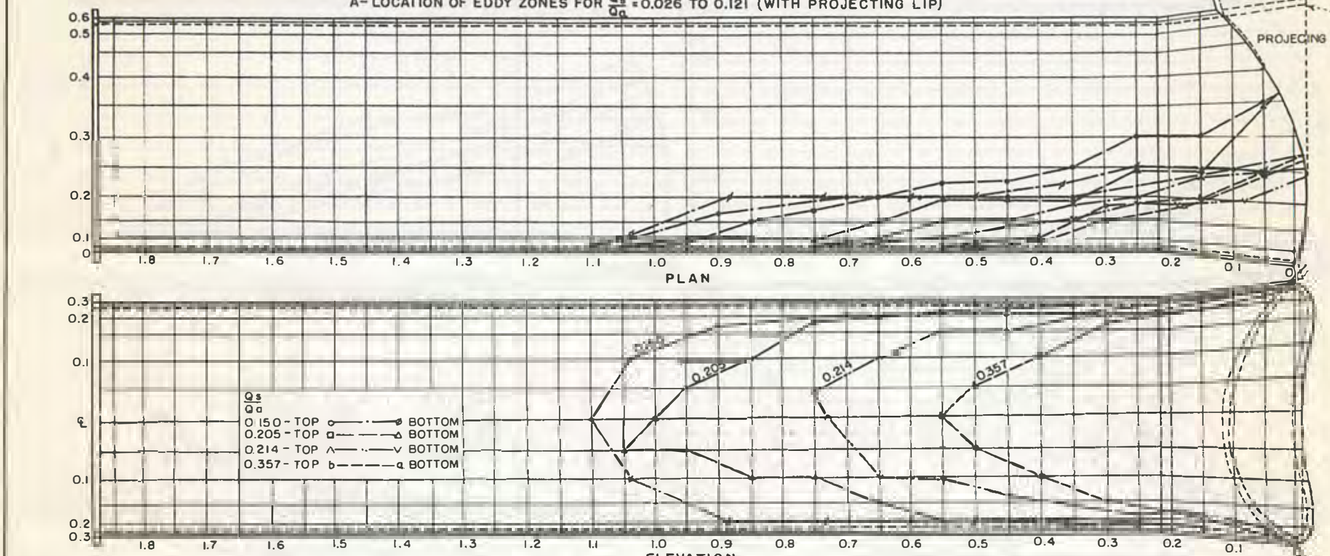
C. DYE REMAINS IN DEAD WATER ZONE

COLOR METHOD OF LOCATING ZONE OF DEAD WATER

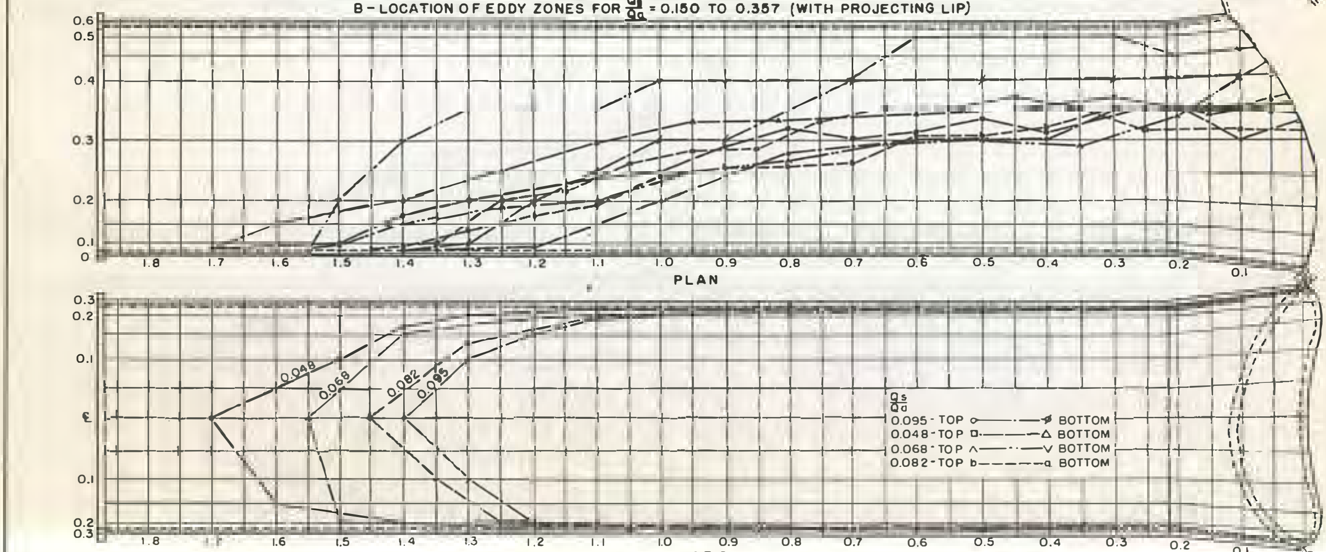
FIGURE 4



A - LOCATION OF EDDY ZONES FOR $\frac{Q_s}{Q_a}$ = 0.026 TO 0.121 (WITH PROJECTING LIP)



B - LOCATION OF EDDY ZONES FOR $\frac{Q_s}{Q_a}$ = 0.150 TO 0.357 (WITH PROJECTING LIP)



C - LOCATION OF EDDY ZONES FOR $\frac{Q_s}{Q_a}$ = 0.048 TO 0.095 (LIP REMOVED)

NOTE

Line and symbols indicate boundaries of eddy water zones for various ratios of discharge through branch penstock. Outside of pipe graduated in five hundredths of a foot.

DEPARTMENT OF THE INTERIOR
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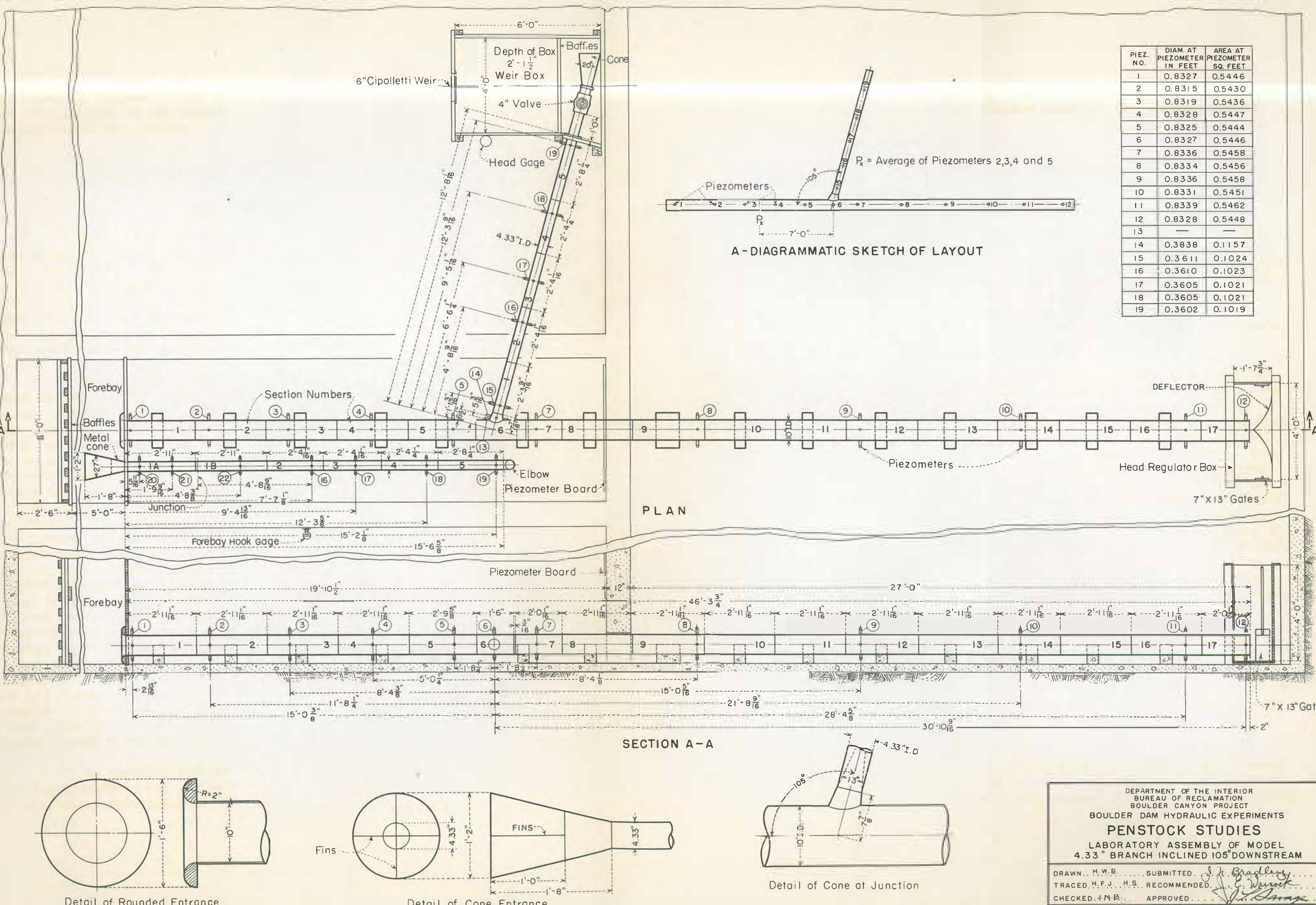
BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS

PENSTOCK STUDIES

VISUAL TESTS ON JUNCTION LOSSES

DRAWN J.N.B. SUBMITTED J. N. Bradley
TRACED S.P.C.B. H.S. RECOMMENDED E. Darrick
CHECKED J.P.M. APPROVED J. L. Savage

DENVER, COLO., JAN. 7, 1936 45-D-9709



PIEZ. NO.	DIAM. AT PIEZOMETER IN FEET	AREA AT PIEZOMETER SQ. FEET
1	0.8327	0.5446
2	0.8315	0.5430
3	0.8319	0.5436
4	0.8328	0.5447
5	0.8325	0.5444
6	0.8327	0.5446
7	0.8336	0.5458
8	0.8334	0.5456
9	0.8336	0.5458
10	0.8331	0.5451
11	0.8339	0.5462
12	0.8328	0.5448
13	—	—
14	0.3838	0.1157
15	0.3611	0.1024
16	0.3610	0.1023
17	0.3605	0.1021
18	0.3605	0.1021
19	0.3602	0.1019

DEPARTMENT OF THE INTERIOR
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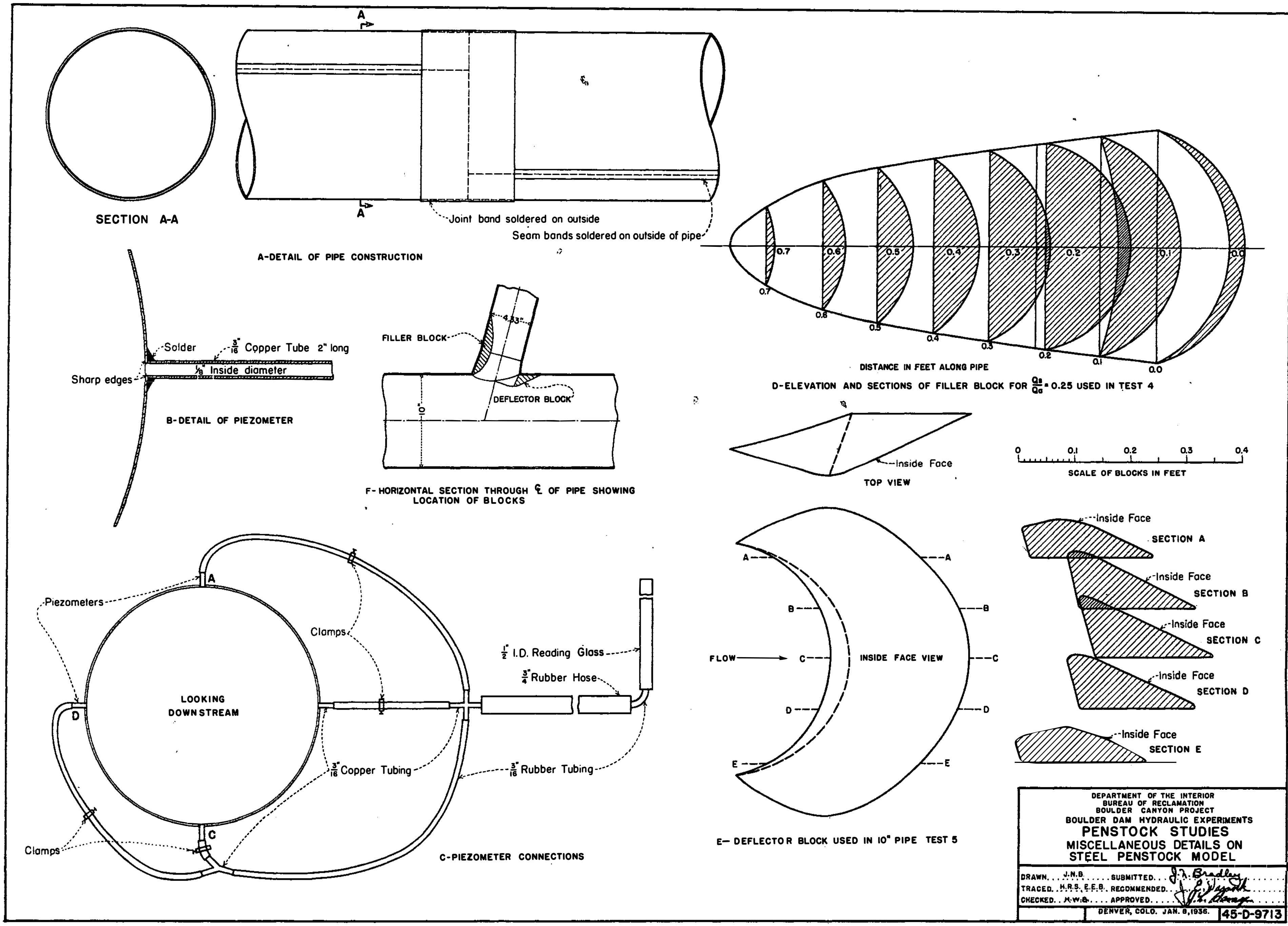
BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS

PENSTOCK STUDIES

LABORATORY ASSEMBLY OF MODEL
4.33" BRANCH INCLINED 105° DOWNSTREAM

DRAWN H.W.B. SUBMITTED J. N. Bradley
TRACED H.F.J. H.S. RECOMMENDED E. Wenrock
CHECKED J.N.B. APPROVED J. L. Savage

DENVER, COLO., JAN. 8, 1936 45-D-9712



SECTION A-A
A-DETAIL OF PIPE CONSTRUCTION
Joint band soldered on outside
Seam bands soldered on outside of pipe
Solder
3/16" Copper Tube 2" long
1/8" Inside diameter
Sharp edges
B-DETAIL OF PIEZOMETER
FILLER BLOCK
4.33"
DEFLECTOR BLOCK
10"
F-HORIZONTAL SECTION THROUGH ℄ OF PIPE SHOWING LOCATION OF BLOCKS
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
DISTANCE IN FEET ALONG PIPE
D-ELEVATION AND SECTIONS OF FILLER BLOCK FOR Qs/Qa = 0.25 USED IN TEST 4
Inside Face
TOP VIEW
SCALE OF BLOCKS IN FEET
Piezometers
Clamps
LOOKING DOWNSTREAM
1/2" I.D. Reading Glass
3/4" Rubber Hose
3/16" Copper Tubing
3/16" Rubber Tubing
C-PIEZOMETER CONNECTIONS
FLOW
INSIDE FACE VIEW
E- DEFLECTOR BLOCK USED IN 10" PIPE TEST 5
SECTION A
SECTION B
SECTION C
SECTION D
SECTION E
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS
PENSTOCK STUDIES
MISCELLANEOUS DETAILS ON
STEEL PENSTOCK MODEL
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TRACED. H.R.S. E.E.B. RECOMMENDED.
CHECKED. H.W.B. APPROVED.
DENVER, COLO. JAN. 8, 1936.
45-D-9713

FIGURE 8

A. MAIN PIPE WITH 75° BRANCH

B. THE BRANCH PIPE

C. MAIN PIPE WITHOUT BRANCH

MODEL B

pipe to regulate the discharge through the branch. A weir box, 4 feet wide, 6 feet long, and 2 feet deep, equipped with stilling baffles and a 6-inch Cipolletti weir was located at the end of the branch to measure the discharge at that point. A gage well attached to the outside of the weir box was connected by a 3/4-inch rubber hose to a hole in the box 20 inches upstream from the weir, and a hook gage was used in the well to measure the head on the weir. As this weir box differed from the standard form, it was necessary to calibrate the 6-inch Cipolletti weir in place. This was done by completely closing the downstream end of the main pipe with a blank flange and diverting all water through the branch and over the 6-inch Cipolletti weir. The calibration was made by comparison with the 90-degree calibrated V-notch measuring weir. The purpose of the metal cone at the end of the branch pipe was to recover a portion of the energy which would have been lost at the end of the pipe, and thus increase the discharge through the branch.

Rings of piezometers were placed at intervals along the pipes, as shown on figure 7. Four piezometers constituted a ring, and connections were so that individual piezometers could be read separately. Each piezometer consisted of a 3/16-inch outside diameter copper tube 2 inches long with a 1/8-inch bore. In construction, a 1/8-inch hole was carefully drilled in the pipe at the proper location. A copper tube was then accurately placed over the hole, normal to the pipe, and heavily soldered in place. The inside of the tube and hole was then reamed to remove any irregularities which might have developed during the process. Piezometer openings were of the sharp-corner type and care was taken to remove all burrs and to keep the corners sharp and flush with the inside of the pipe (fig. 8B). Where piezometers were installed on cones, they were in all cases set normal to the surface. As it was desired to read each piezometer separately, the connections were made as shown on figure 8C. Three-sixteenths inch rubber tubing was used for the shorter connections, while each ring was connected to the piezometer board by a 3/4-inch garden hose. This arrangement made it possible to use one glass reading tube to each ring of piezometers. Each ring was equipped with three screw clamps which could be alternated between the four rubber tubes to segregate the piezometer to be read.

In reading piezometers, all A piezometers were read simultaneously with all others closed. In rotation, all B, C, and D piezometers were read in the same manner with the others closed. This procedure was repeated four times during a run, making a total of 16 readings to a ring. Observations were made as rapidly as the board could be read. About forty minutes were required to make a complete set.

The 3/4-inch hose which extended from the rings to the piezometer reading board sloped upward toward the board to facilitate the removal of entrapped air. Before each run, all piezometer clamps were loosened, and all tubes leading to the A piezometers were disconnected to allow water to flow freely through them. The 3/4-inch hoses were vibrated to drive any entrapped air toward the reading board. Finally, the hoses were disconnected from the glass reading tubes to allow water to flow freely through them. This routine for eliminating air required from fifteen to twenty minutes before each run, but was a necessary procedure as all the piezometers were located on a central board making some of the hoses connections 30 feet long.

To identify the experimental data, each alteration or setup of the model was assigned a test number and each run of a test, a run number. In like manner, each picture was given a picture number. Thus, 10-B-6 is interpreted as test 10, picture B of run 6.

B. Pipe Friction Calibration (4.33-inch Branch)

The friction losses in the 4.33-inch and 10-inch pipes were determined by connecting the pipes to the bulkhead separately and making calibration runs (tests 1 and 2). In test 1, the branch pipe was connected as shown on figure 7, with two extra joints of pipe on the upstream end to give additional length for reducing irregularities of flow caused by entrance conditions. A conical entrance (figure 7) was used on the upstream end of the 4.33-inch pipe with a vertical and a horizontal fin extending from the large end two-thirds of the distance up the length of the cone to prevent vortex action. An elbow was connected to the downstream end of the pipe in a vertical plane to keep the pipe flowing full at all times and to increase the pressure in the pipe so that it could be read on the gage board. It was necessary, in the case of smaller discharges, to further retard the flow by loosely bolting a blank flange onto the end of the elbow to procure a sufficient number of runs to plot a calibration curve. In other cases, additional vertical sections of pipe were bolted to the elbow.

Eleven runs were made on test 1 with discharges ranging from 0.55 to 1.70 second-feet. During a run, the two head gages on the 90-degree V-notch measuring weir were read simultaneously with the forebay head gage at two-minute intervals. The temperature of the water was recorded once during a run. A run was terminated when each piezometer on the model had been read four times, as previously described. The 16 readings from each piezometer ring were averaged and the respective computed mean velocity heads added. These values for each ring were then plotted with respect to the length of the pipe. A sample sheet showing the method for a few runs is shown on figure 9. Due to the fact that the entrance loss effect extends some 40 diameters downstream, the lines drawn through the points have a slight curvature but would eventually terminate in straight lines were the pipe sufficiently long. The slope of these straight lines represents the friction loss per foot of straight pipe. As the pipe was not long enough to allow these lines to completely straighten out, it was necessary to draw a tangent to each of the curves near its downstream extremity. These tangents are shown drawn on figure 9. The slope of each represents the straight pipe friction per foot for a particular discharge and water temperature. A constant has been consistently subtracted in some of the runs to consolidate the data for comparison as the slopes of the lines are the major item of interest. These constants in no way effect the slope of the lines.

With the friction loss, discharge and temperature known for each run, the friction factor, f, in the formula $h_f = \frac{fLV^2}{2gD}$ was computed and plotted with respect to Reynolds' number on figure 10. The friction factor was computed, using the equation $f = \frac{2gSD}{V^2}$, where S is the friction slope (fig. 9) and L is unity. It has been customary in recent years to design conduits from curves of this type, obtained from experimental results. In using these curves it is only necessary to know the range of Reynolds' numbers that will be encountered in the field and the type of pipe to be used. This being known, the factor, f, can be taken from a curve similar to those plotted on figure 10 for a pipe having comparable roughness[2].

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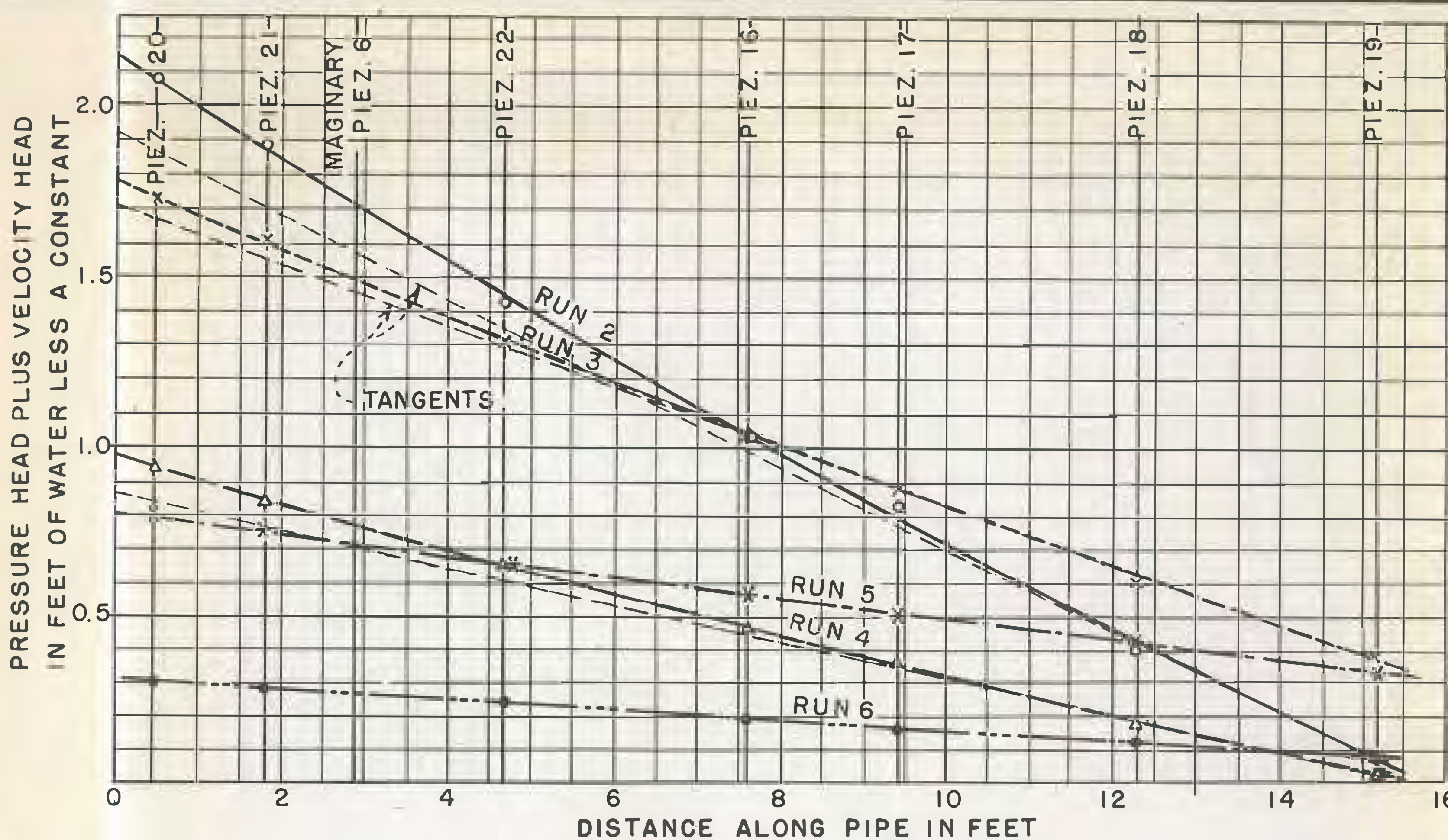
[2] "The Flow of Fluids in Closed Conduits", by R.J.S. Pigott, Mechanical Engineering, August 1933.

As the temperature varied considerably throughout the series of tests, each run was corrected to the base temperature of 60 degrees Fahrenheit by means of figure 11A. The temperature correction graph was constructed by choosing a number of discharges at various temperatures for the 4.33-inch pipe and computing the Reynolds' numbers for each. From figure 10, the friction factors were obtained for the various conditions and the friction loss per foot of pipe was computed. From this information, figure 11A, which shows the variation in the friction loss due to temperature, was constructed. For example, with a discharge of 1.6 second-feet, water temperature of 70 degrees, and a pipe friction of 0.2000 foot per foot of pipe, figure 11A shows that this friction loss at a temperature of 60 degrees would be 0.2000 - 0.0037 = 0.2037 foot per foot of pipe.



SYMBOL	RUN NO.	DISCHARGE IN SEC. FT.	SLOPE OF LINE
o———o	2	1.454	0.1297
x- - - - - - -x	3	1.203	0.0919
△—— —— -—△	4	0.953	0.0617
——-—	5	0.657	0.0306
●——-··——●	6	0.442	0.0149

NOTE

Slope of Tangent indicates average friction loss per foot of pipe.

TEST I

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT

BOULDER DAM HYDRAULIC EXPERIMENTS
PENSTOCK STUDIES
METHOD USED IN DETERMINING
STRAIGHT PIPE FRICTION IN 4.33" BRANCH

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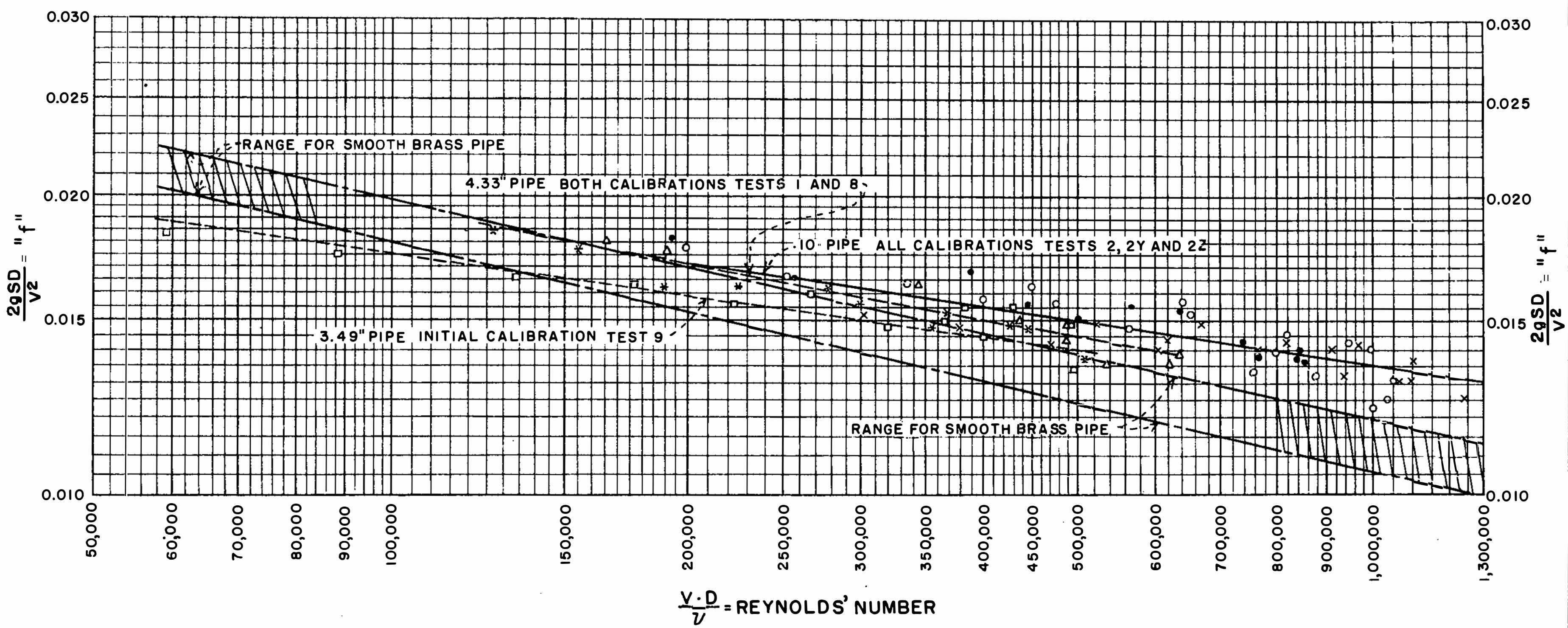
DENVER, COLO. 12-31-35 | 45-D-9742

FIGURE 9



EXPLANATION

4.33" PIPE	TEST 1	✳
	TEST 8	△
10" PIPE	TEST 2	○
	TEST 2Y	×
	TEST 2Z	•
3.49" PIPE	TEST 9	□

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BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS

PENSTOCK STUDIES

LOSSES DUE TO PURE PIPE FRICTION

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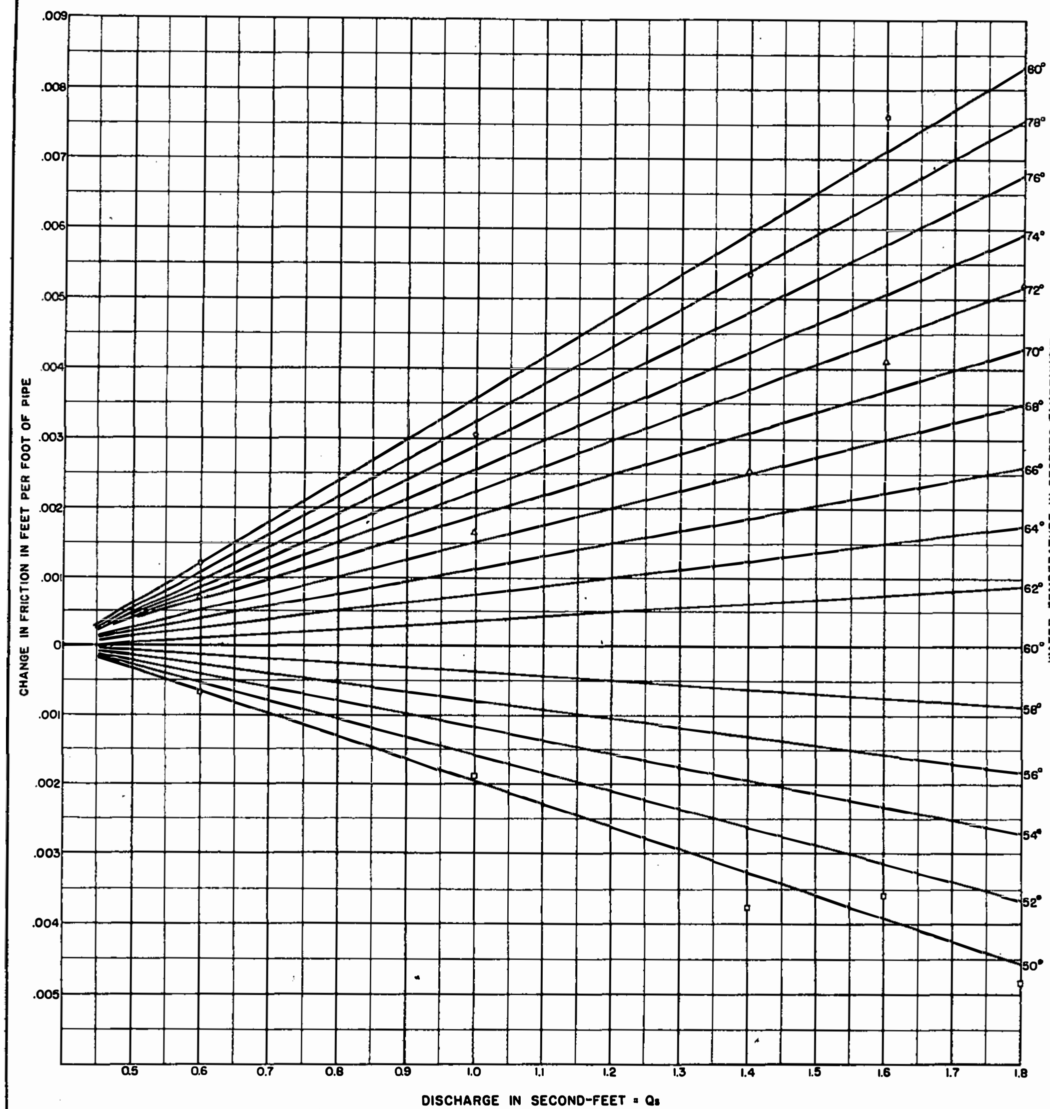
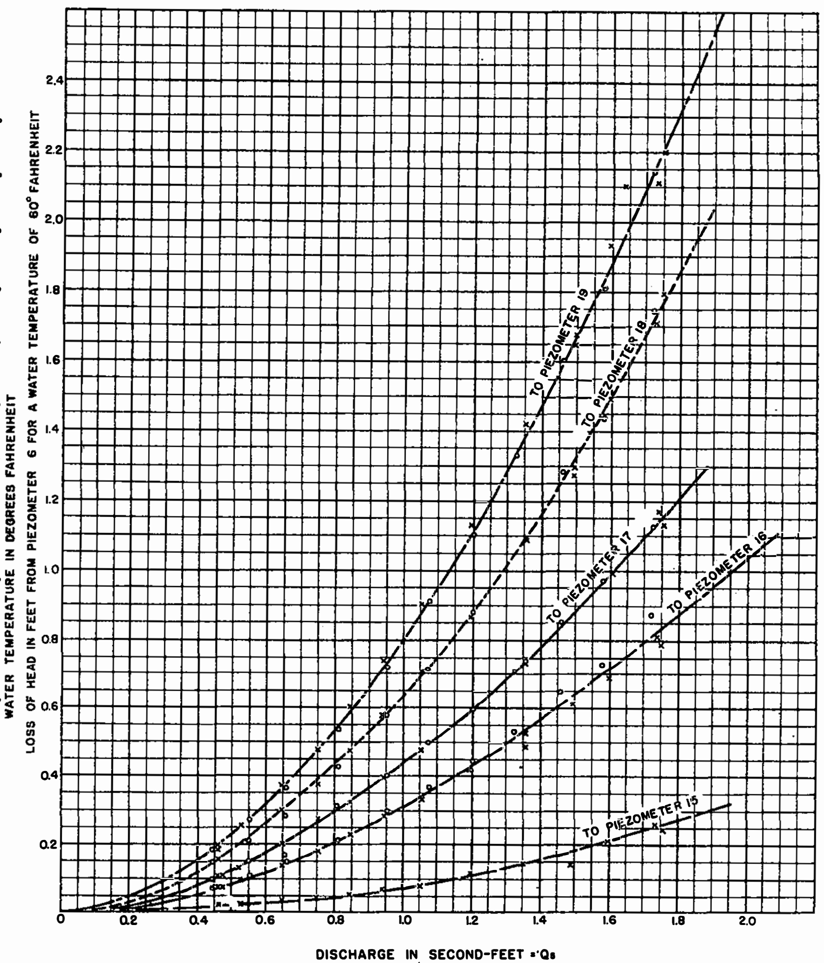
DENVER, COLO., JAN. 6, 1936 45-D-9743

FIGURE 10



A-TEMPERATURE CORRECTION CURVES FOR 4.33" BRANCH



B- CALIBRATION CURVES FOR FRICTION IN 4.33" BRANCH

EXPLANATION

Test-1 ∘ First calibration
Test-8 × Second calibration

NOTE

Pipe friction is measured from piezometer 6. This is the same as assuming that the branch pipe extended to the ℄ of the main pipe.

DEPARTMENT OF THE INTERIOR
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BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS

PENSTOCK STUDIES

FRICTION CALIBRATION CURVES AND TEMPERATURE CORRECTION CURVES FOR 4.33" BRANCH PIPE

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DENVER, COLO. DEC. 31, 1938. 45-D-9714

FIGURE 11

BOULDER DAM PENSTOCK STUDIES
CALIBRATION OF EXPERIMENTAL BRANCH PIPE
4.33" INSIDE DIAMETER
COMPUTATION OF FRICTION DROP FROM JUNCTION

Run No.	Discharge Sec. ft.	Temp. °F	Temp. corr. Per ft. of pipe	Plotted energy at Junction	Piezometer No. 16		17		18		19	
					Computed energy	Drop to Junction	Computed energy	Drop to Junction	Computed energy	Drop to Junction	Computed energy	Drop to Junction
1	1.7232	60.5	-.00020	2.372	-.001 +1.498	.875	-.001 +1.245	1.128	-.002 +.627	1.747	-.002 +.228	2.146
2	1.4542	60.2	-.00007	1.678	-.003 +1.033	.648	-.005 +.830	.853	-.007 +.402	1.283	-.009 +.083	1.604
3	1.2030	60.3	-.00007	1.479	-.003 +1.038	.444	-.005 +.892	.592	-.007 +.605	.881	-.009 +.387	1.101
4	.9529	59.8	+.00005	.761	+.002 +.469	.290	+.003 +.363	.395	+.005 +.179	.577	+.006 +.036	.719
5	.6570	59.5	+.00005	.710	.002 .565	.143	.003 .517	.190	.005 .423	.282	.006 .341	.363
6	.4425	59.4	0	.266	---- .193	.073	.173	.093	.125	.141	.082	.184
7	1.5729	57.5	+.00094	2.280	.004 1.548	.728	.006 1.302	.972	.009 .831	1.440	.012 .455	1.813
8	1.3254	57.0	+.00090	1.432	.004 .898	.530	.006 .718	.708	.008 .376	1.048	.011 .092	1.329
9	1.0721	57.1	+.00067	.991	.003 .625	.363	.004 .491	.496	.006 .271	.714	.008 .082	.901
10	.8067	57.2	+.00040	.615	.002 .402	.211	.003 .302	.310	.004 .187	.424	.005 .073	.537
11	.5504	57.5	+.00015	.477	+.001 +.369	.107	+.001 +.330	.146	+.001 +.266	.210	+.002 +.206	.269
12	1.8355	--	---	---	---	--	---	--	---	--	---	--

Note: The word "junction" is defined as the point at which the center line of the branch intersects the center line of the main pipe.

TABLE II

45-D-9760

the runs of higher discharge. The entrance to the main pipe consisted of a circular collar.

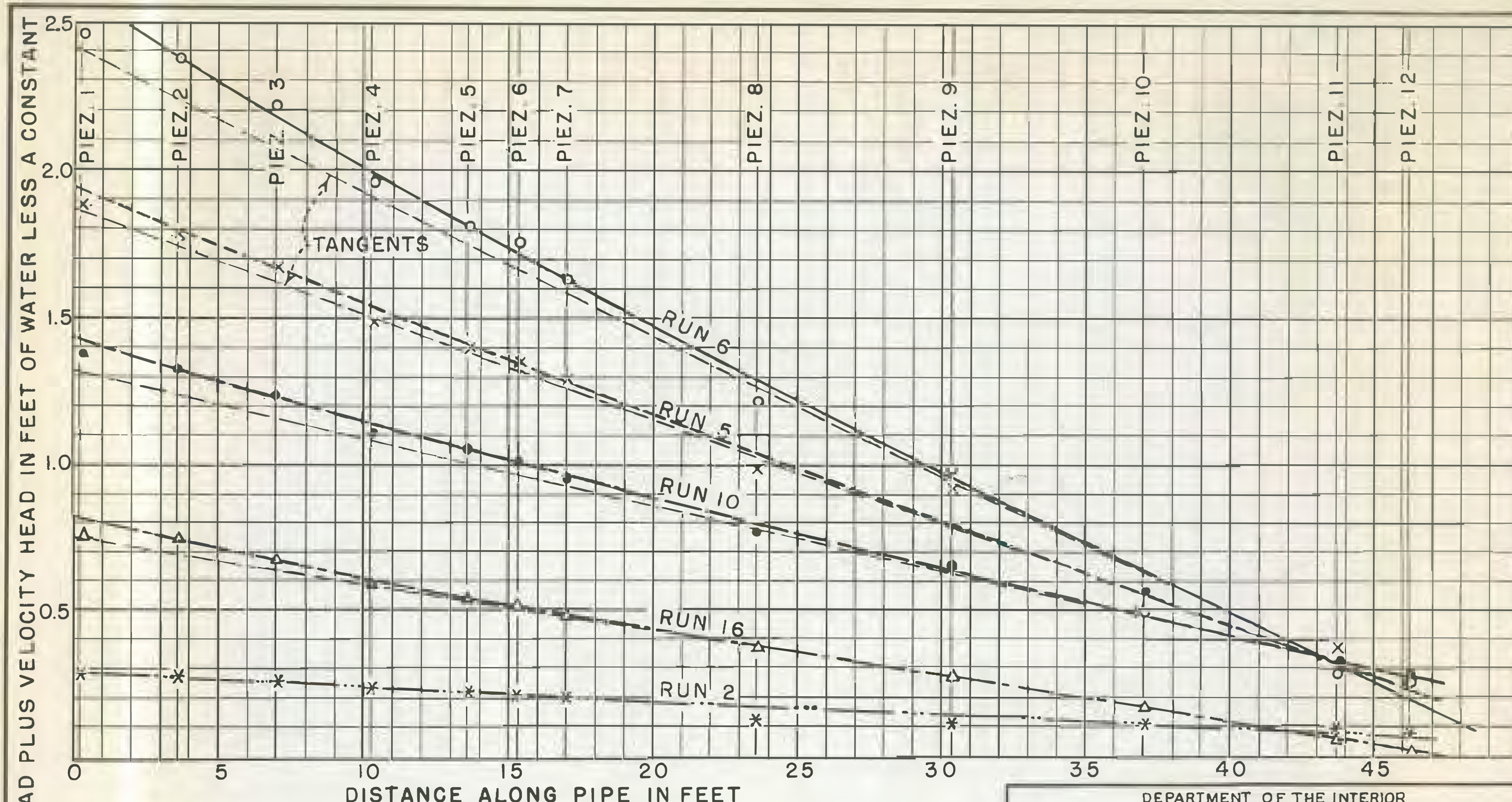
The friction slopes shown for a few discharges on figure 12 were drawn to obtain the average friction loss per foot in the main pipe. Again tangents were drawn to the downstream end of the curves and the slopes of these considered the average friction loss per foot of pipe. The temperature correction curves used in evaluating the results obtained on the 10-inch pipe are shown on figure 13, and the friction losses as measured upstream and downstream from piezometer 6 for various discharges at a temperature of 60 degrees Fahrenheit are shown on figure 14. As in the case of the 4.33-inch pipe, piezometer 6 at the theoretical junction was used as a reference and the friction loss was measured upstream and downstream from this point. The friction factor, f, for the 10-inch pipe is plotted on figure 10 with respect to Reynolds' number.

A set of check runs under test 2Y was made on the 10-inch pipe four weeks after test 2 was completed, and the results checked the original set. Both sets of points are plotted on figures 10 and 14.

It is apparent from a study of figure 10 that the 10-inch pipe shows a greater surface roughness than the 4.33-inch branch for the same value of Reynolds' number even though both were constructed in the same manner and of the same material. Theoretically, one would expect the friction factor, f, to be smaller in the larger pipe. The explanation may lie in the fact that the 10-inch pipe had a joint to every 3.5 diameters, while the 4.33-inch pipe had a joint to about every 7.0 diameters.

D. Evaluation of Junction Losses

With the straight pipes calibrated, they were connected together as shown on figure 7 and studies were made to evaluate the loss of head in the junction. Runs were made using total discharges from 1.5 to 8.0 second-feet. By adjusting the 4-inch valve at the end of the branch in relation to the slide gates in the head regulator box, it was possible to divert part of the total discharge through the branch. For large discharge ratios, $\frac{Qs}{Qa}$, the total discharges were necessarily low, while for small discharge ratios practically any total discharge could be used up to 8 second-feet. During a run, readings were taken simultaneously at two-minute intervals on the two head gages in the large weir box, the forebay head gage, and the head gage for the 6-inch Cipolletti weir. During the same period of time, each piezometer on the pipe was read four times.



SYMBOL	RUN NO.	DISCHARGE IN SEC.-FT.	SLOPE OF LINE
o———o	6	7.498	0.0511
×------×	5	6.136	0.0354
•— — —•	10	5.040	0.0245
△—·—·—△	16	4.166	0.0161
✳—···—✳	2	2.034	0.0045

NOTE

Slope of Tangent indicates average friction loss per foot of pipe.

TEST 2

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT

BOULDER DAM HYDRAULIC EXPERIMENTS
PENSTOCK STUDIES
METHOD USED IN DETERMINING
STRAIGHT PIPE FRICTION IN 10" PIPE

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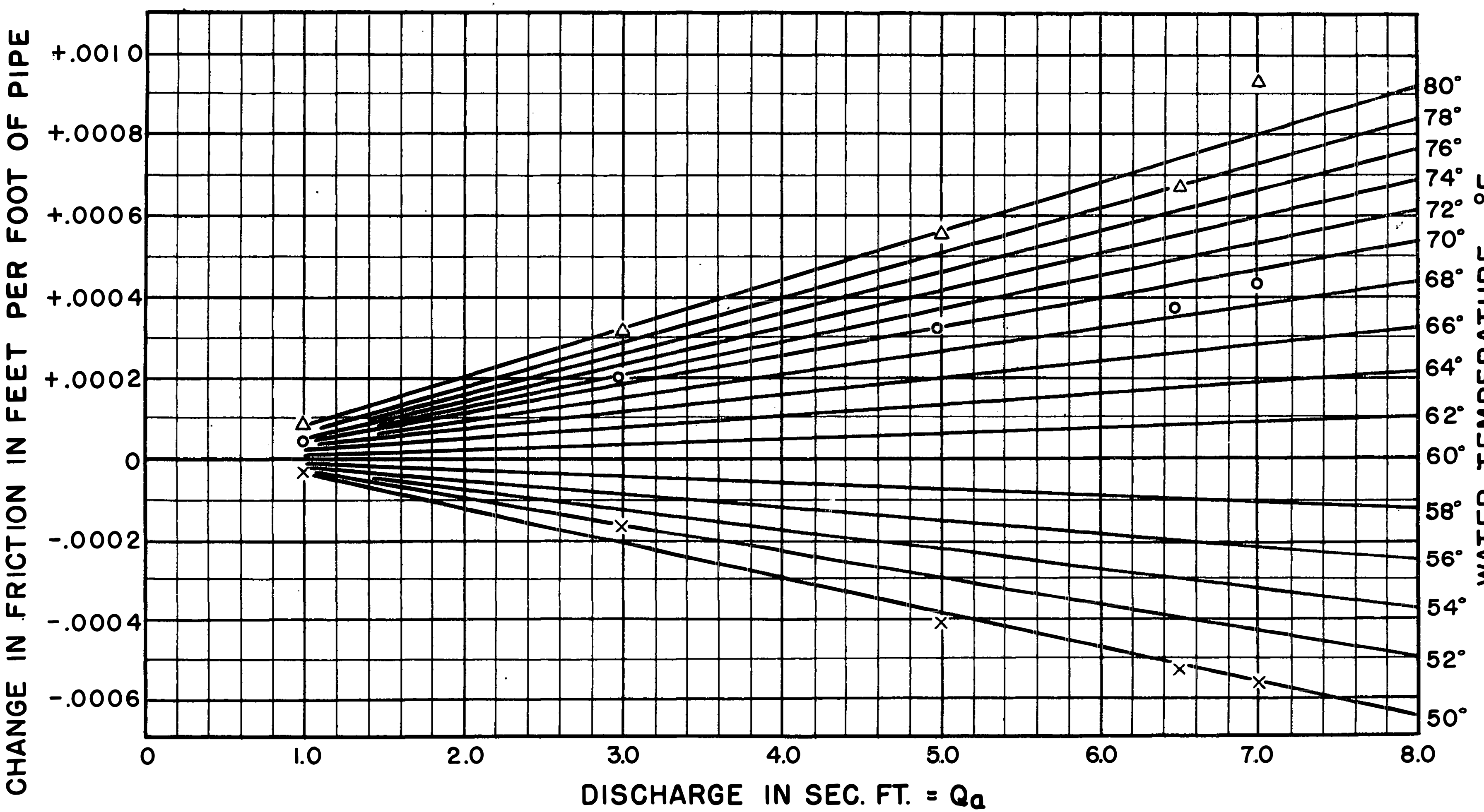
DENVER, COLO., 12-30-35 | 45-D-9738

FIGURE 12



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BOULDER DAM HYDRAULIC EXPERIMENTS

PENSTOCK STUDIES

TEMPERATURE CORRECTION CURVES FOR 10" MAIN PIPE

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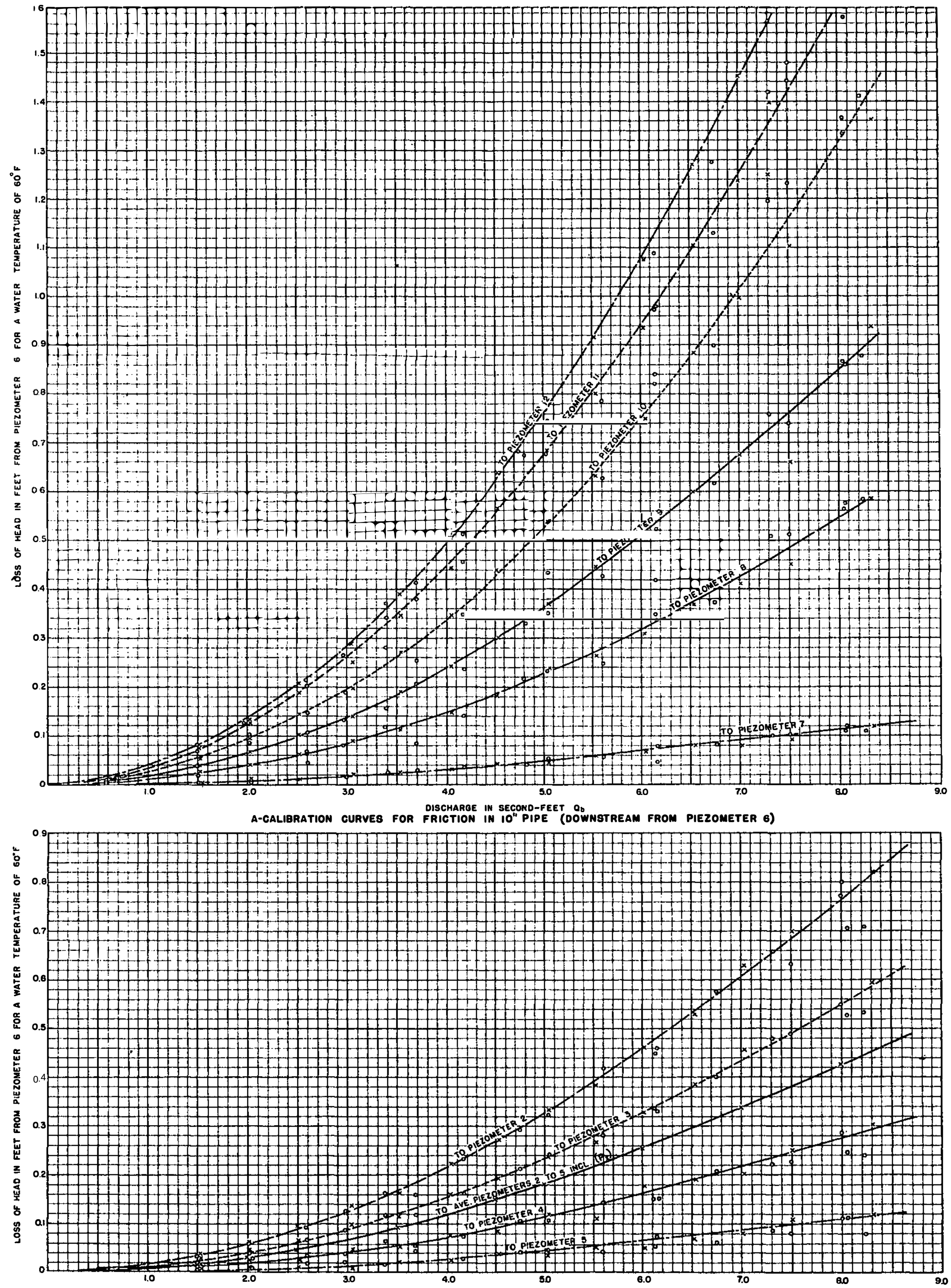
DENVER, COLO. 1-6-36 45-D-9739

FIGURE 13



A-CALIBRATION CURVES FOR FRICTION IN 10" PIPE (DOWNSTREAM FROM PIEZOMETER 6)

B-CALIBRATION CURVES FOR FRICTION IN 10" PIPE (UPSTREAM FROM PIEZOMETER 6)

NOTE
All pipe friction is measured from piezometer 6

EXPLANATION
Test-2 o First calibration.
Test-2Y x Second calibration.

DEPARTMENT OF THE INTERIOR
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BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS
PENSTOCK STUDIES
FRICTION CALIBRATION CURVES FOR 10" PIPE

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DENVER, COLO. JAN. 20, 1936. 45-D-9715

For the purpose of these experiments, junction losses are defined as follows: The junction loss shall be the sum of the pressure drop plus the drop in nominal velocity head, less the friction loss between points upstream and downstream from the junction sufficiently removed to be free from effects caused by the junction. In determining the friction loss, pipe distances were measured to the center line of the intersection. The following equations express this definition (fig. 7A).

Junction loss chargeable to the main pipe:

$$J_b = \left\{ P_x + \frac{V_a^2}{2g} \right\} - \left\{ P_{10} + \frac{V_b^2}{2g} + h_f\ (x-10) \right\}$$

Junction loss chargeable to branch:

$$J_s = \left\{ P_x + \frac{V_a^2}{2g} \right\} - \left\{ P_{18} + \frac{V_s^2}{2g} + h_f\ (x-18) \right\}$$

Rather than choose a single piezometer from which to measure the pressure head above the junction, the average of piezometers 2, 3, 4, and 5 was used and is denoted as P_x. The location of P_x is approximately 7.0 feet or 8.4 diameters upstream from piezometer 6.

Although energy losses determined in accordance with the above definition are subject to an error in the order of $0.05 \frac{V^2}{2g}$ due to the use of the nominal velocity head instead of that integrated over the section, and due to the subtraction of friction losses to the center of the intersection, the definition seems a practical one. The error involved, however, should be kept in mind in interpreting the results.

The tests on the 105-degree junction were divided into two groups. Test 3 was a normal pipe junction, where, as in tests 4 to 7, inclusive, the physical shape of the junction was modified by filler blocks in accordance with the earlier visual tests in an attempt to improve the hydraulic efficiency of this junction.

Twenty-six runs were made on test 3 with discharges through the branch ranging from 0 to 100 percent of the total. The test procedure was the same as in the calibration runs except for reading the additional piezometers and the small weir gage. A computation sheet for one of the runs on test 3 is shown on table III. From the computed energy at P_x, the energy at each piezometer downstream from the junction was subtracted independently. This method tends to eliminate

BOULDER DAM PENSTOCK STUDIES
COMPUTATIONS OF JUNCTION LOSS

Test No. 3
Run No. 11
Date 6/11/32

Q_a = 1.9964 | Q_s = 1.2033 | Q_b = 0.7931 | Q_s/Q_a = 0.6027 | Temp. 67.5°F

Px Average of Piezometers 2-3-4 & 5 = 3.955
Average $Va^2/2g$ at Piezometers 2-3-4 & 5 = 0.209
Energy at Px = 4.164

	PIEZOMETER NO.									
	7	8	9	10	11	12	16	17	18	19
Ave. Piezometer Reading	4.064	4.070	4.066	4.060	4.049	4.041	1.241	1.051	.780	.519
Velocity Head	.033	.033	.033	.033	.033	.033	2.151	2.161	2.161	2.169
Pipe Friction Above Piez.6	.029									
Temperature Correction	.001									
Corrected	.028	.028	.028	.028	.028	.028	.028	.028	.028	.028
Pipe Friction Below Piez.6	.001	.008	.012	.019	.021	.028	.434	.592	.880	1.112
Temperature Correction	0	0	0	0	.001	.001	.009	.012	.018	.023
Corrected	.001	.008	.012	.019	.020	.027	.425	.580	.862	1.089
Junction Loss	+.038	+.025	+.025	+.034	+.034	+.035	.319	.344	.334	.359
$J_b/V_a^2/2g$	+.182	+.120	+.120	+.163	+.163	+.167	1.526	1.646	1.598	1.718
$J_s/V_s^2/2g$							0.148	0.159	0.155	0.166

Average $V_s^2/2g$ = 2.161

TABLE III

45-D-9758

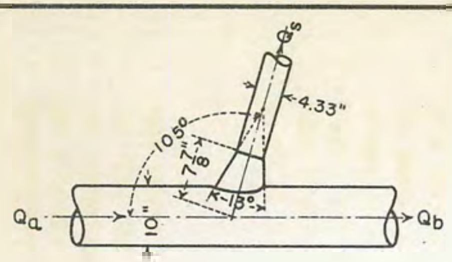
piezometer discrepancies. The differences in each case represent pipe friction plus junction losses. With the discharge known, the pipe friction for the main pipe for a temperature of 60 degrees was obtained from the curves on figure 14 and the pipe friction for the branch was secured from figure 11B. The temperature correction curves on figures 11A and 13 were used to correct the pipe friction values for temperature. With the corrected pipe friction subtracted, the remaining loss is that which was charged to the junction. This is the procedure followed on table III. Junction losses in the main pipe were computed in terms of the velocity head upstream from the junction. Junction losses in the branch are expressed in terms of the velocity head upstream from the junction and also in terms of the velocity head in the branch.

The results of the junction losses for test 3 chargeable to the main pipe plotted with respect to the discharge ratios are shown on figure 15. On figure 15B, the discharge ratio is related to the junction losses obtained from each of the individual downstream piezometers. The separate piezometer curves (fig. 15B) are in close agreement, with the exception of piezometers 7 and 12, which in most cases were disregarded when drawing average curves due to their undesirable locations. The curve for test 3 (fig. 15A) was obtained from the average of the individual piezometer curves on figure 15B.

The average results of the junction losses chargeable to the branch plotted with respect to the discharge ratios are shown on figure 16. On figure 16A, the junction losses are in terms of the velocity head in the branch and on figure 16B they are in terms of the velocity head in the main pipe upstream from the junction. The separate piezometer curves for the branch were in good agreement. In computing these junction losses, the straight pipe friction charged to the branch was computed on the assumption that the branch pipe extended in to the center line of the main pipe. The reason for this assumption becomes apparent when an attempt is made to design a junction in a large pipe line from experimental data.

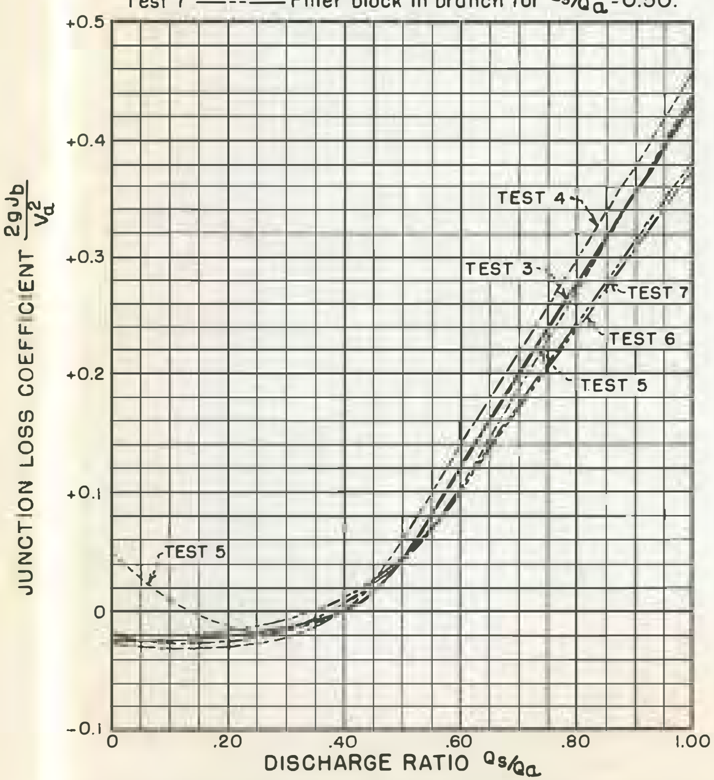
For $\frac{Q_s}{Q_a} = 0$ (fig. 15), all the flow goes down the main pipe and within the limits of experimental error no loss is found caused by the intersection of the branch in the line although the diameter of opening is more than half that of the main pipe. The results on figure 15 verify experiments made by Professor D. Thoma[3] on small

[3] "Hydraulic Losses in Pipe Fittings" by Prof. D. Thoma. Transactions of the Tokyo Sectional Meeting, World Power Conference, Tokyo, October 29 to November 7, 1929. This article has been translated and incorporated in the U.S.B.R. Tech. Memo. No. 325. For a more detailed description of these experiments see "Losses in Oblique Angled Pipe Branches" by Franz Peterman, Mitteilungen des Hydraulischen Instituts der Technischen Hochschule Munchen Bulletin 1, p. 75 ff; and Bulletin 2, p. 61 FF. These articles have been translated in the "Transactions of the Munich Hydraulic Institute" and published by the American Society of Mechanical Engineers as Bulletin No. 3.



EXPLANATION

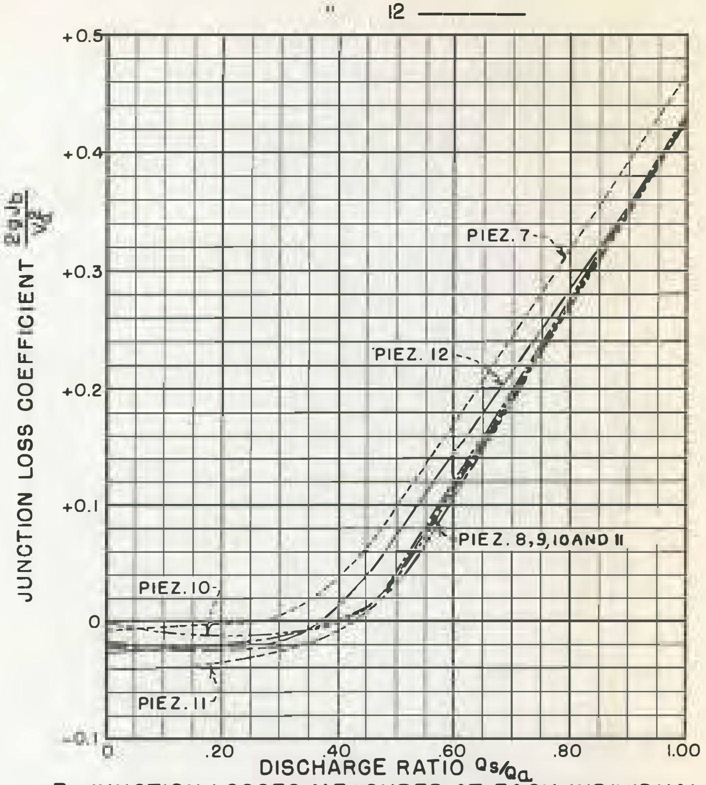
Test 3 ———— Original setup - no filler blocks.
Test 4 — — — Filler block in branch for $Q_s/Q_a = 0.25$.
Test 5 --------- Horseshoe block in main pipe.
Test 6 -·—·—·— Improved filler block in branch for $Q_s/Q_a = 0.25$.
Test 7 ——·-·—— Filler block in branch for $Q_s/Q_a = 0.50$.



A-JUNCTION LOSSES IN MAIN PIPE OBTAINED FROM AVERAGE OF INDIVIDUAL PIEZOMETER CURVES

TEST NO. 3
EXPLANATION

Piezometer 7 ----------
" 8 — — — — —
" 9 -—·—·—-
" 10 ---—---—--
" 11 ---------—----
" 12 ————————



B-JUNCTION LOSSES MEASURED AT EACH INDIVIDUAL DOWNSTREAM PIEZOMETER

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS

PENSTOCK STUDIES
JUNCTION LOSSES IN MAIN PENSTOCK

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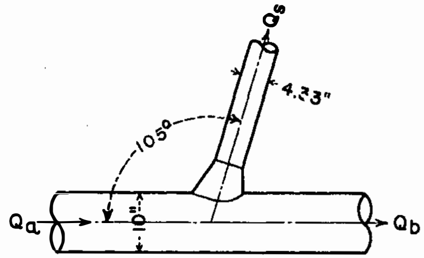
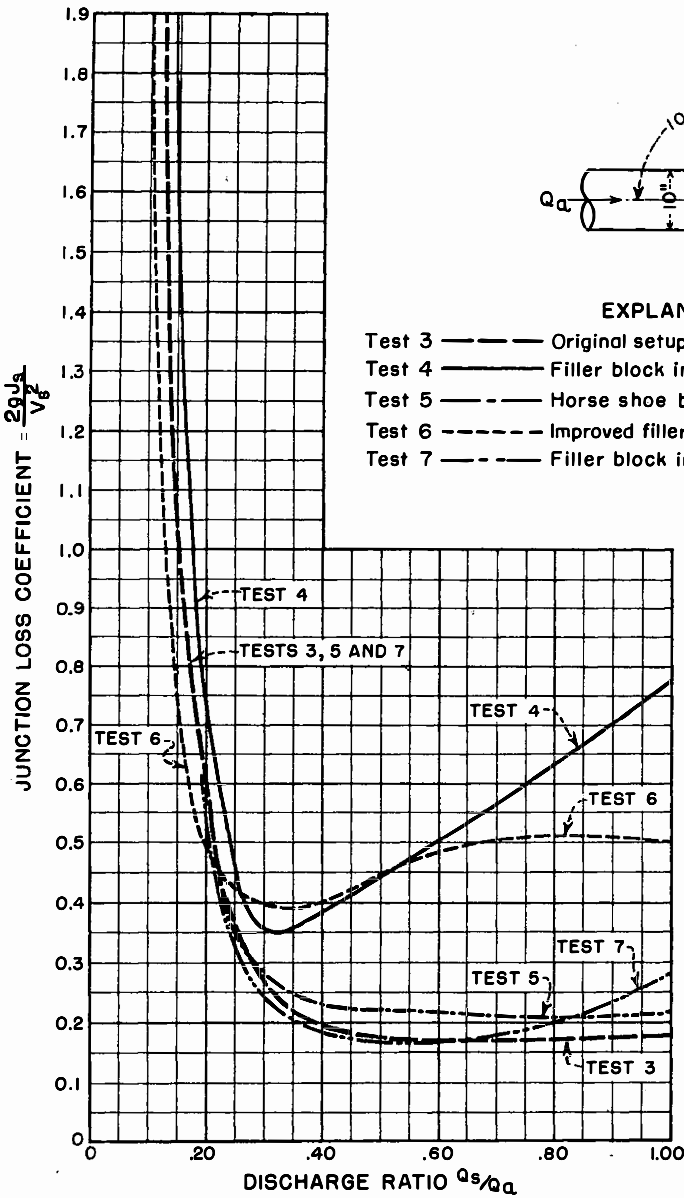
TRACED D.W.S.	RECOMMENDED J. E. Warnock
CHECKED H.W.B.	APPROVED J. L. Savage
DENVER, COLO., DEC. 30, 1935	45-D-9741

FIGURE 15

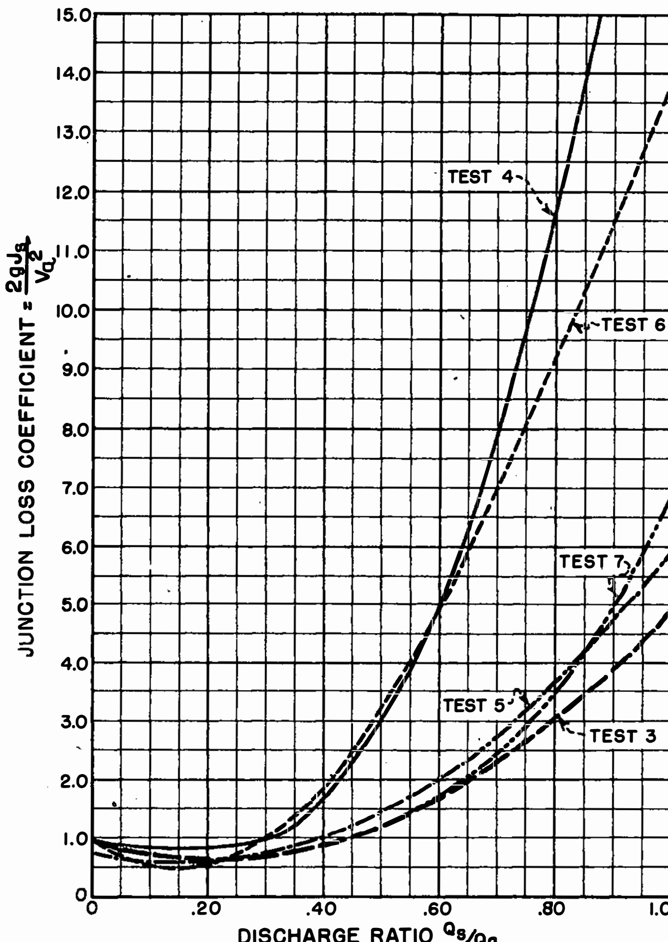


EXPLANATION

Test 3	— — —	Original setup – no filler blocks.
Test 4	———	Filler block in branch for Q_s/Q_a = 0.25.
Test 5	—·—	Horse shoe block in main pipe.
Test 6	- - - - -	Improved filler block in branch for Q_s/Q_a = 0.25.
Test 7	—··—	Filler block in branch for Q_s/Q_a = 0.50.



A - JUNCTION LOSSES IN BRANCH IN TERMS OF VELOCITY HEAD IN BRANCH



B - JUNCTION LOSSES IN BRANCH IN TERMS OF VELOCITY HEAD IN MAIN PIPE

JUNCTION LOSSES IN BRANCH PIPE OBTAINED FROM AVERAGE OF INDIVIDUAL PIEZOMETER CURVES

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS

PENSTOCK STUDIES

JUNCTION LOSSES IN BRANCH PENSTOCK

DRAWN H.W.B. SUBMITTED J. N. Bradley
TRACED H.R.S.-D.W.S. RECOMMENDED J. E. Warnock
CHECKED J.N.B. APPROVED J. L. Savage

DENVER, COLO., JAN. 7, 1936 | 45-D-9734

FIGURE 16

brass pipe in the respect that for discharge ratios of less than 0.40, an apparent gain of head is recorded in the main pipe downstream from the junction. This gain in head is partly due to the manner in which the loss coefficients were determined and partly due to the pressure reaction from the water diverted into the branch. It is, of course, not a gain in energy.

It is possible to compute the junction loss chargeable to the branch (fig. 16A) within reasonable limits by assuming that the entrance to the branch is an ordinary sharp-edged conical pipe entrance. The average textbook indicates that the entrance loss for a cone of this angle and type when connected to a still reservoir should be in the vicinity of $0.15 \frac{V_s^2}{2g}$. As this entrance does not connect to a still reservoir but to a pipe in which varying conditions of flow exist, there is an additional loss to be added to that above which, for the lower ratios of $\frac{Q_s}{Q_a}$ is approximately $0.4 \frac{V_a^2}{2g}$. This value is not a constant for all conditions of flow, but its presence becomes negligible as the ratio of $\frac{Q_s}{Q_a}$ increases.

The total junction loss then computed in this manner would be

$$J_s = 0.15 \frac{V_s^2}{2g} + 0.4 \frac{V_a^2}{2g} \qquad (1)$$

Expressing the entire loss in terms of the velocity head in the branch, the procedure is as follows:

$$V_s = \frac{Q_s}{A_s} \quad \text{and} \quad V_a = \frac{Q_a}{A_a}$$

$$\frac{V_s^2}{V_a^2} = \frac{\left(\frac{Q_s}{A_s}\right)^2}{\left(\frac{Q_a}{A_a}\right)^2} = \left(\frac{Q_s \; A_a}{Q_a \; A_s}\right)^2$$

$$\frac{V_a^2}{2g} = \frac{V_s^2}{2g} \left(\frac{Q_a}{Q_s}\right)^2 \left(\frac{A_s}{A_a}\right)^2$$

Substituting this value in equation (1) the junction loss coefficient

$$\frac{J_s}{V_s^2/2g} = 0.15 + 0.4 \left(\frac{Q_a}{Q_s}\right)^2 \left(\frac{A_s}{A_a}\right)^2 \qquad (2)$$

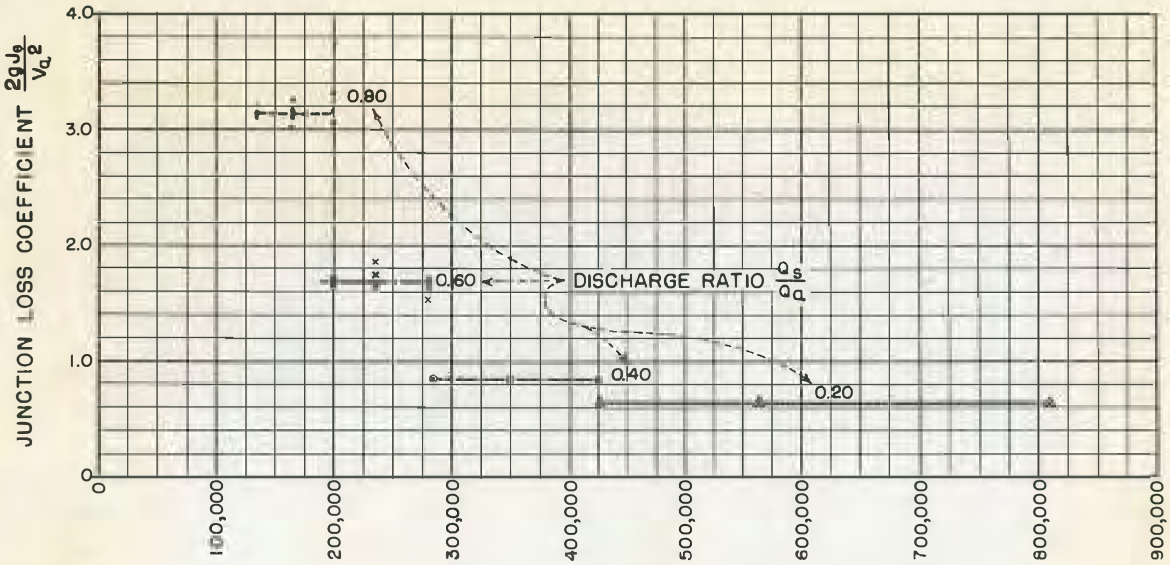
By substituting various values of $\frac{Q_a}{Q_s}$ in equation (2), a curve is obtained which agrees closely with the one for test 3 on figure 16A. The value of $\frac{A_s}{A_a}$ is equal to 0.1875 in this case.

Although data are meagre for the purpose in mind the following point of interest is cited. In the visual tests on the pyralin model, it was stated that the volume of the eddy zone in the branch seemed to be independent of the discharge, depending only on the discharge ratio $\frac{Q_s}{Q_a}$. Thoma makes a statement to this effect, that the junction loss coefficient is independent of the total discharge and depends entirely upon the ratio $\frac{Q_s}{Q_a}$. On figure 17, there has been plotted the variations in the loss coefficients for different values of Reynolds' number. The curves on figure 17A show the junction losses in the branch to vary only with the discharge ratio $\frac{Q_s}{Q_a}$ and not with the total discharge or Reynolds' number. The curves on figure 17B show a similar indication for the junction losses in the main pipe but the points are more scattered due to the exaggerated scale of the graph. These results agree with Thoma's work and with the visual tests in which the eddy at the junction appeared to remain the same size for a given discharge ratio, regardless of the total discharge.

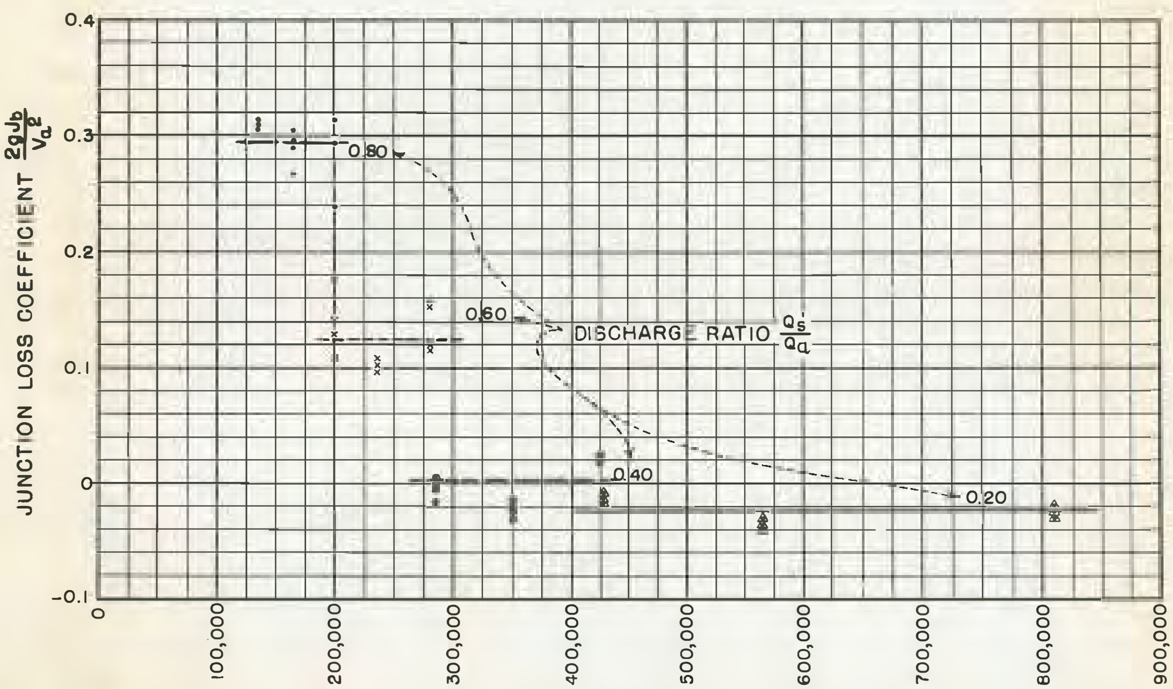
E. Reduction of Junction Losses

An effort was made in tests 4 to 7, inclusive, to reduce junction losses at discharge ratios of 0.25 and 0.50. In test 5, a deflecting block was installed in the main pipe. In tests 4, 6, and 7, wooden filler blocks made to the same size and shape as the eddy zones encountered in the pyralin model were installed in the branch pipe immediately below the junction.

For test 5, the deflecting block shown on figure 8E was installed in the main pipe, as shown on figure 8F. The block was made to represent as nearly as possible the lip that extended out into the main pipe in the preliminary tests on the pyralin model. The results from the six runs made on this test are plotted on figures 15A, 16A, and 16B. This block produced no improvement in



A - JUNCTION LOSSES IN BRANCH IN TERMS OF VELOCITY HEAD IN MAIN PIPE



B - JUNCTION LOSSES IN MAIN PIPE IN TERMS OF VELOCITY HEAD IN MAIN PIPE

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BOULDER DAM HYDRAULIC EXPERIMENTS
PENSTOCK STUDIES
JUNCTION LOSSES IN TERMS OF REYNOLD S' NUMBER FOR TEST 3

DRAWN J.N.B.	SUBMITTED J. N. Bradley
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DENVER, COLO., DEC. 31, 1935	45-D-9735

the junction loss coefficients in either the branch or the main pipe nor did it seem to produce any detrimental effects for the lower discharge ratios.

The filler block shown on figure 8D was designed from the curves on figure 6 for a discharge ratio of 0.25, and was installed in the branch pipe in the location shown on figure 8F for test 4. The results plotted on figures 15A, 16A, and 16B reveal that this block failed to reduce the junction loss coefficient, but instead increased it as compared with test 3 for all discharge ratios.

Another attempt was made in test 6 to improve conditions at the junction by using a filler block for a discharge ratio of 0.25. Special care was exercised in obtaining the shape of this block in the pyralin model, which is shown on figure 3F. Better results were produced by this block, which was somewhat smaller and more accurate than the one used in test 4. The curves on figure 16 actually show a small improvement over test 3 for discharge ratios below 0.22. Due to the restriction of area that these blocks offered in the branch, the junction losses chargeable to the branch would be expected to increase for discharge ratios above that for which they were designed.

In test 7, the filler block shown on figure 3D for a discharge ratio of 0.50 was used. When compared with test 3 on figure 16, practically no change in the junction loss coefficient is noticeable up to a discharge ratio of 0.60. In fact, this block shows a slight improvement in the coefficient when compared with test 3 for discharge ratios of less than 0.60. Above this ratio, the junction losses increase in the branch while in the main pipe (fig. 15A), the losses show a decrease for discharge ratios above 0.50.

Summing the results of the above tests, it can be said that in the majority of cases the filler blocks actually did decrease the junction loss coefficients for discharge ratios under that for which they were designed. It was of course expected that the junction loss would increase for values greater than the designed ratio. The improvement, contrary to expectations, was so small that the benefit derived from filler blocks of this type would hardly be worth the expense of installation. It is suggested, however, that for penstock installations, where the discharge ratios for the branches will remain practically constant and power head at the turbines is at a premium, filler blocks may prove to be of some value. These, however, would not be a practical installation at the Boulder Dam. Had the filler blocks shown promising results they may have been incorporated in the penstock design.

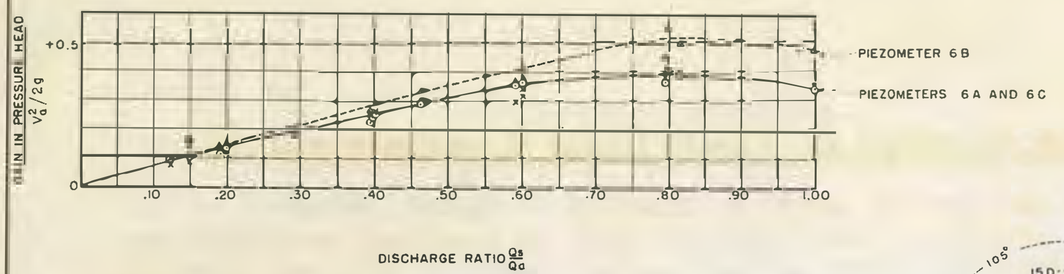
F. Investigation of Static Pressure at 105-Degree Junction

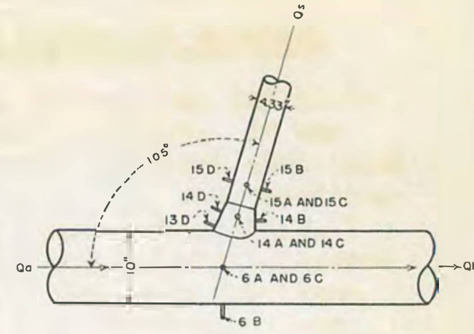
In the design of thin-walled pipes, the maximum and minimum pressures developed at junctions are quite important. It was to determine more in detail these pressure conditions that additional piezometric connections were installed in the model of the junction. Piezometer 13D and piezometer rings 14 and 15 were placed (figs. 7 and 18) in the section of the branch immediately below its intersection with the main pipe. During test 3, these were read simultaneously with the other piezometers. Three of the pressure outlets in piezometer ring 6 were also read, the fourth outlet being eliminated by the installation of the junction of the branch pipe.

The pressure at each of these points was related to the average pressure condition at piezometer ring 6, measured during previous runs without the branch installed. The drop in pressure between each piezometer and piezometer ring 6 was expressed in terms of velocity head in the main pipe upstream from the junction in order to produce dimensionless quantities. This ratio of drop in pressure head to velocity head is plotted against the discharge ratio on figure 18. As it is possible in design to determine quite accurately the pressure at any point in a straight pipe line, which is not true after a junction is installed, the pressure obtained at ring 6 (without the branch connected) was used as the reference pressure in these experiments.

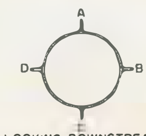
A slight increase in pressure was found at piezometers 6A, 6B, and 6C (fig. 18) due to the introduction of the junction. For discharge ratios up to 0.20, a decided increase is shown at piezometer 14B, but for higher ratios, the pressure diminishes rapidly. For the lower ratios, the gain in pressure indicated by this piezometer was probably caused by impact, while for the larger ratios, the loss in pressure may be attributed to turbulence and complicated eddy formations.

As an aid in visualizing the pressure distributions in a junction of this type, diagrams of the actual observed pressure intensities of piezometer rings 14 and 15 for a few runs on test 3 are shown on figure 19. The pressures were measured in feet of water above the center of the pipe and are plotted both vertically and horizontally, using the intersection of the two axes as the reference point. The circumferential pressure at piezometer ring 15 is quite uniform in each case. The pressures recorded at ring 14, however, varied greatly with the discharge conditions. Considerable variation can be noted in the pressure distribution for the extreme cases where $\frac{Q_s}{Q_a}$ = 0.151 and 1.00.

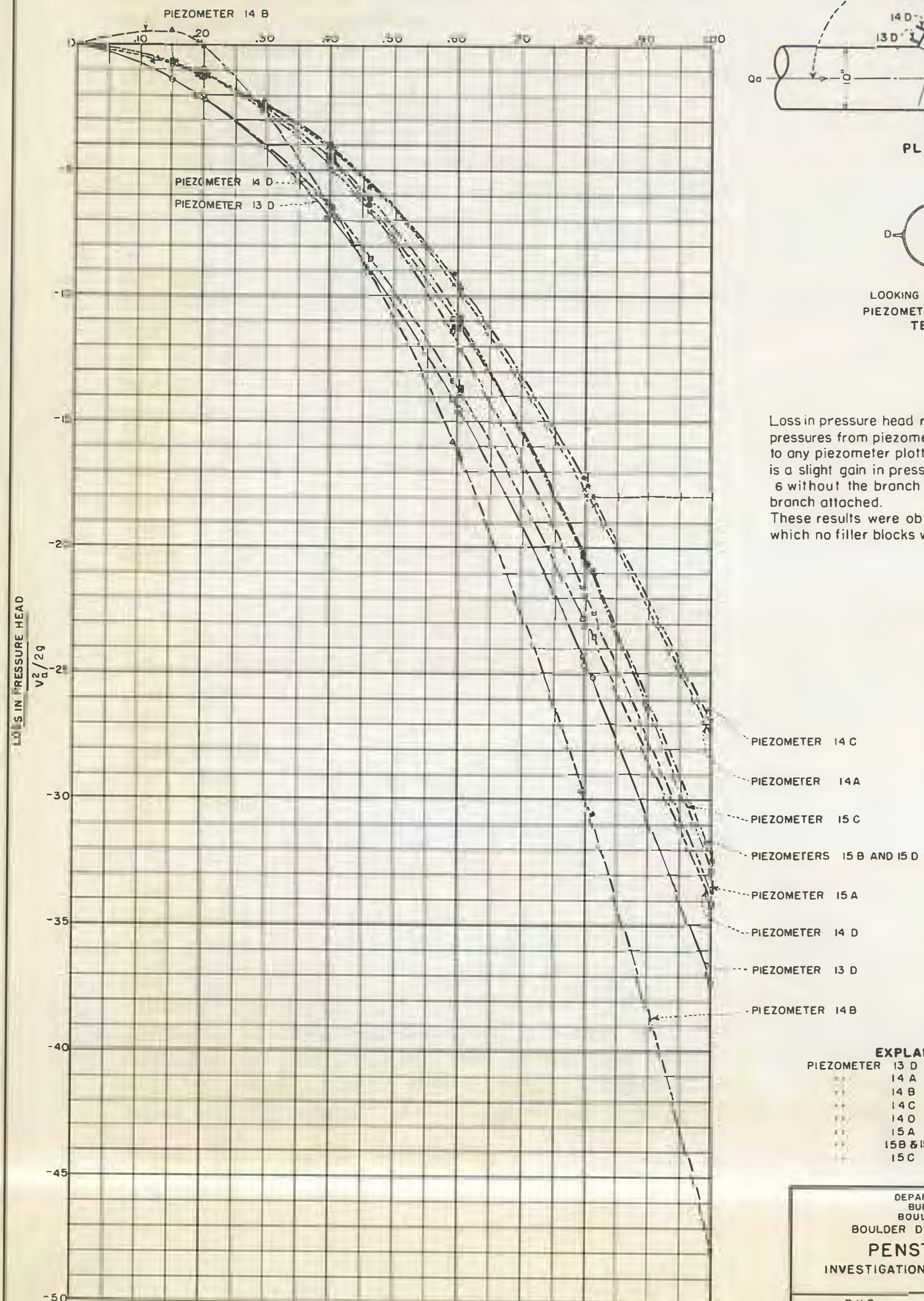




PLAN



LOOKING DOWNSTREAM
PIEZOMETER LOCATIONS
TEST 3



NOTE

Loss in pressure head represents the difference in pressures from piezometer 6 (without branch attached) to any piezometer plotted. Notice that the difference is a slight gain in pressure head between piezometer 6 without the branch and piezometer 6 with the branch attached.
These results were obtained for Test 3 during which no filler blocks were used.

EXPLANATION

PIEZOMETER	13 D	⊙———⊙
"	14 A	X- - - - - - X
"	14 B	△— — — —△
"	14 C	✳— - —— - —✳
"	14 D	⊡——- - - -⊡
"	15 A	⊔————⊔
"	15 B & 15 D	⊗— —- — —⊗
"	15 C	⊠——- · -——⊠

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PENSTOCK STUDIES

INVESTIGATION OF PRESSURES AT JUNCTION
FOR TEST 3

DRAWN R.H.O.	SUBMITTED J. N. Bradley
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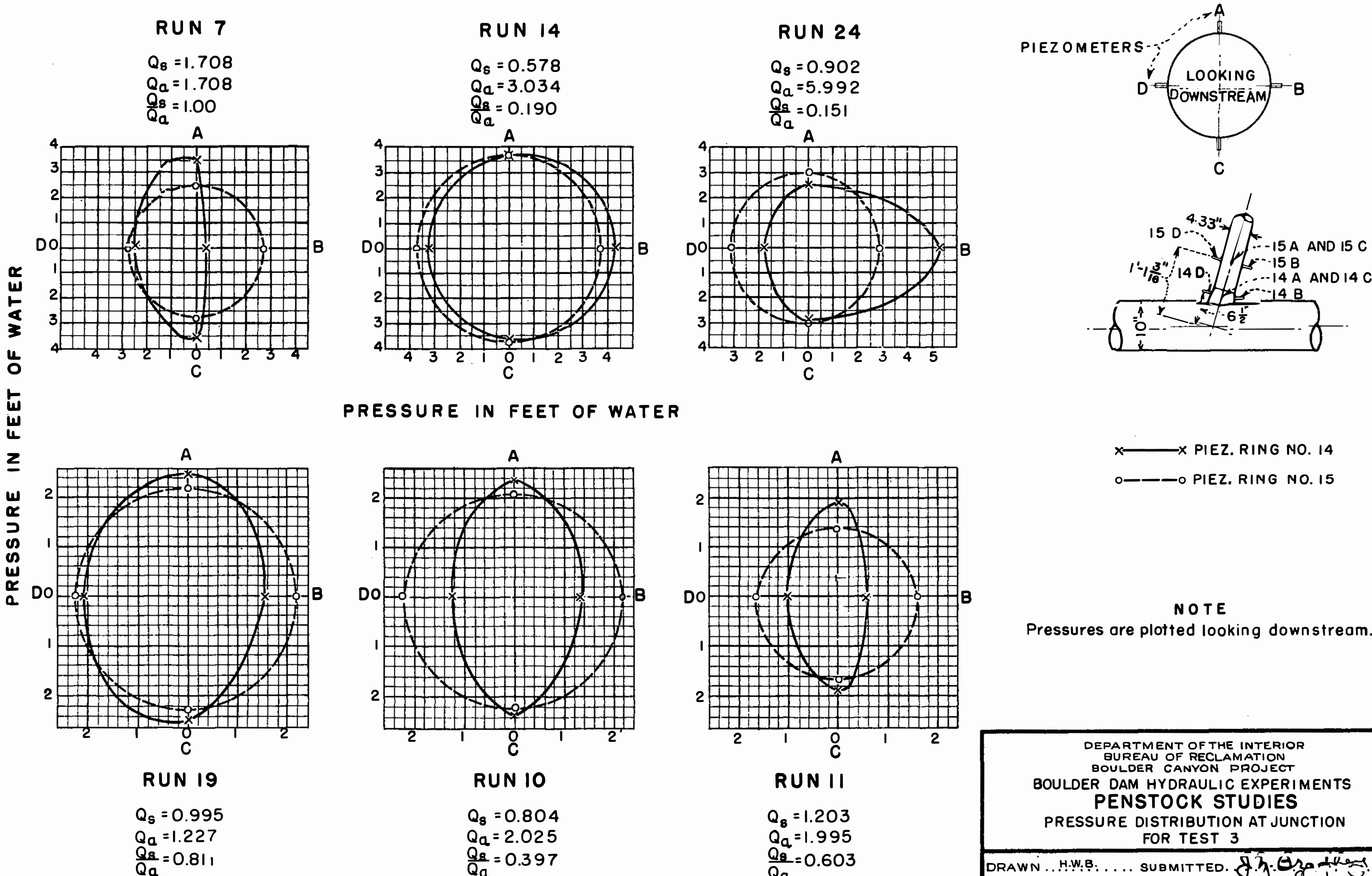
CHECKED J.N.B.	APPROVED J. L. Savage
DENVER, COLO., JAN. 2, 1936	45-D-9716



NOTE

Pressures are plotted looking downstream.

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PENSTOCK STUDIES

PRESSURE DISTRIBUTION AT JUNCTION
FOR TEST 3

DRAWN H.W.B. SUBMITTED J. H. Bradley
TRACED H.S. RECOMMENDED [illegible]
CHECKED J.N.B. APPROVED J. L. Savage

DENVER, COLO., JAN. 8, 1936 | 45-D-9740

FIGURE 19

G. Tests on a Right-Angle Junction

As previously mentioned, experiments of a similar nature have been performed by Professor Dr. - Ing. Thoma and his associates at Munich, and translations of their work have been used for reference in design of pipe junctions. This was especially the case in the early stages of the designs of the Boulder Dam penstocks. There was some question as to the validity of applying Thoma's data to such extremely large penstocks, particularly in view of the fact that his experiments were performed on relatively small brass pipes[4].

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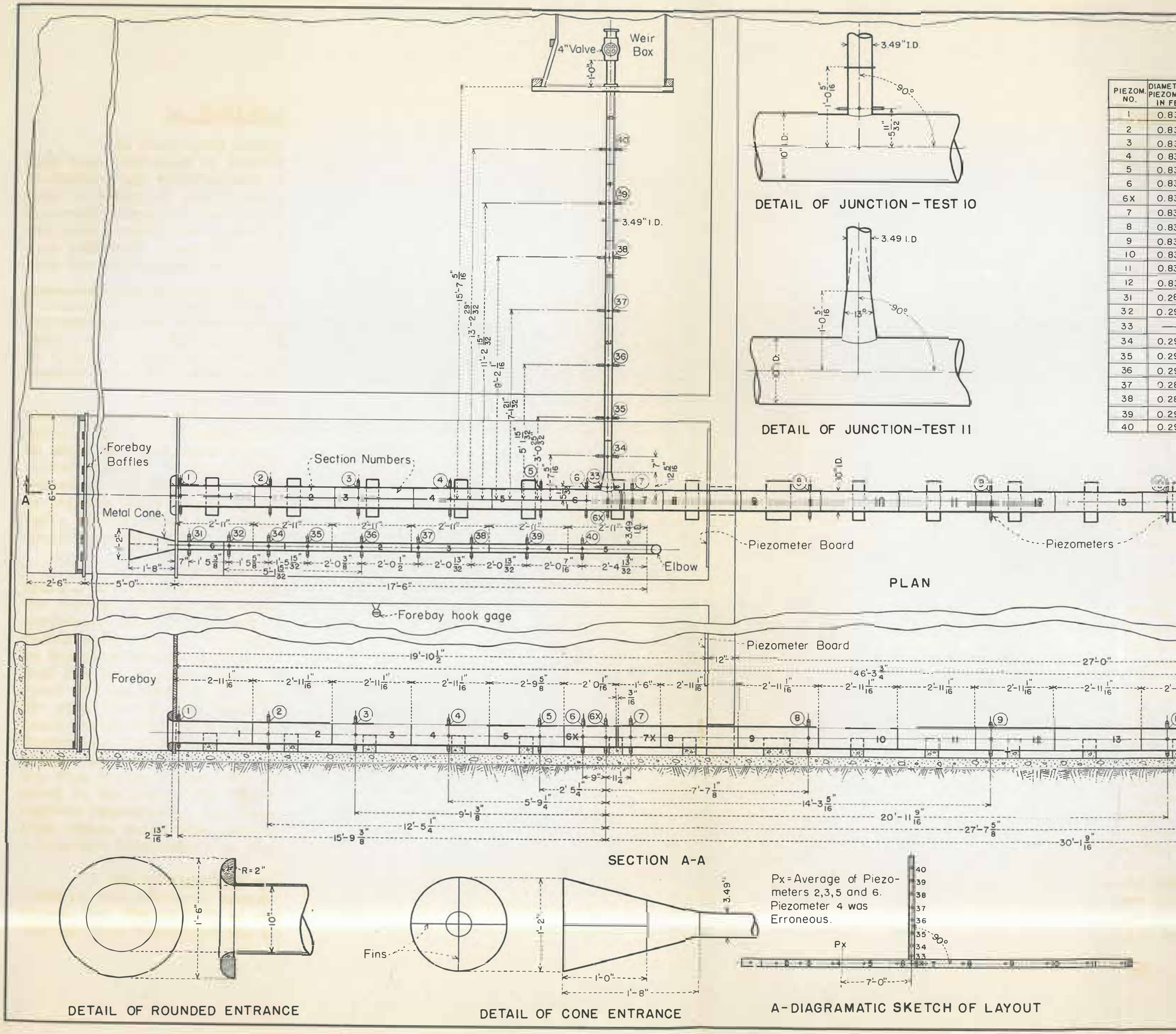
[4] "Losses in Right-Angle Pipe Tees" by Gustav Vogel, Dip. Engr., Mitteilungen des Hydraulischen Instituts der Technischen Hochschule. Part 1 - 1926. This article has been translated and incorporated in U.S.B.R. Tech. Memo. No. 299.

In an effort to make a definite check on Thoma's experiments, two tests were made for the determination of junction losses, using a 3.49-inch branch pipe connected to the 10-inch main pipe at an angle of 90 degrees (fig. 20). In test 10, the branch was connected directly to the main pipe, while in test 11, the branch was joined to the main pipe by a cone having a total central angle of 13 degrees and a length equal to 2.1 D_s. The ratios of the diameters D_s/D_a of 0.35 ± and the proportions of the cone used were directly comparable with two of Thoma's experiments.

Before investigating the junction losses, the 3.49-inch branch, with an extra section of pipe upstream, was connected to the forebay bulkhead and calibrated. The procedure was the same as previously used to calibrate the 4.33-inch branch. The friction factor curve for, f, from this calibration is plotted with respect to Reynolds' number as test 9 on figure 10. This curve falls below that for the 10-inch pipe. Two months later a check calibration (test 9Z) was made on the same pipe under the same conditions and the pipe friction was found to have increased considerably. It was later proven that this second calibration was in error and it was, therefore, disregarded. Friction loss curves plotted against discharge for test 9 are shown on figure 21A. These curves represent the pipe friction from piezometer ring 6X (fig. 20A) to any piezometer in the branch for various discharges at a temperature of 60 degrees Fahrenheit. The temperature correction curves for the 3.49-inch branch which were used to convert the pipe friction losses to a constant temperature on figure 21A are shown on figure 21B.

The calibrations of the 10-inch pipe before and after the tests on the 90-degree branch are indicated as tests 2Y and 2Z on figure 10. Practically no change in friction was evidenced in the 10-inch pipe throughout the entire set of experiments. Calibration



PIEZOM. NO.	DIAMETER OF PIEZOMETER IN FEET	AREA AT PIEZOMETER SQUARE FT.
1	0.8327	0.5446
2	0.8315	0.5430
3	0.8320	0.5436
4	0.8328	0.5447
5	0.8325	0.5444
6	0.8328	0.5447
6X	0.8331	0.5451
7	0.8330	0.5450
8	0.8334	0.5456
9	0.8336	0.5458
10	0.8331	0.5451
11	0.8339	0.5462
12	0.8328	0.5448
31	0.2898	0.0660
32	0.2902	0.0661
33	——	——
34	0.2900	0.0660
35	0.2901	0.0661
36	0.2903	0.0662
37	0.2896	0.0659
38	0.2895	0.0658
39	0.2903	0.0662
40	0.2903	0.0662

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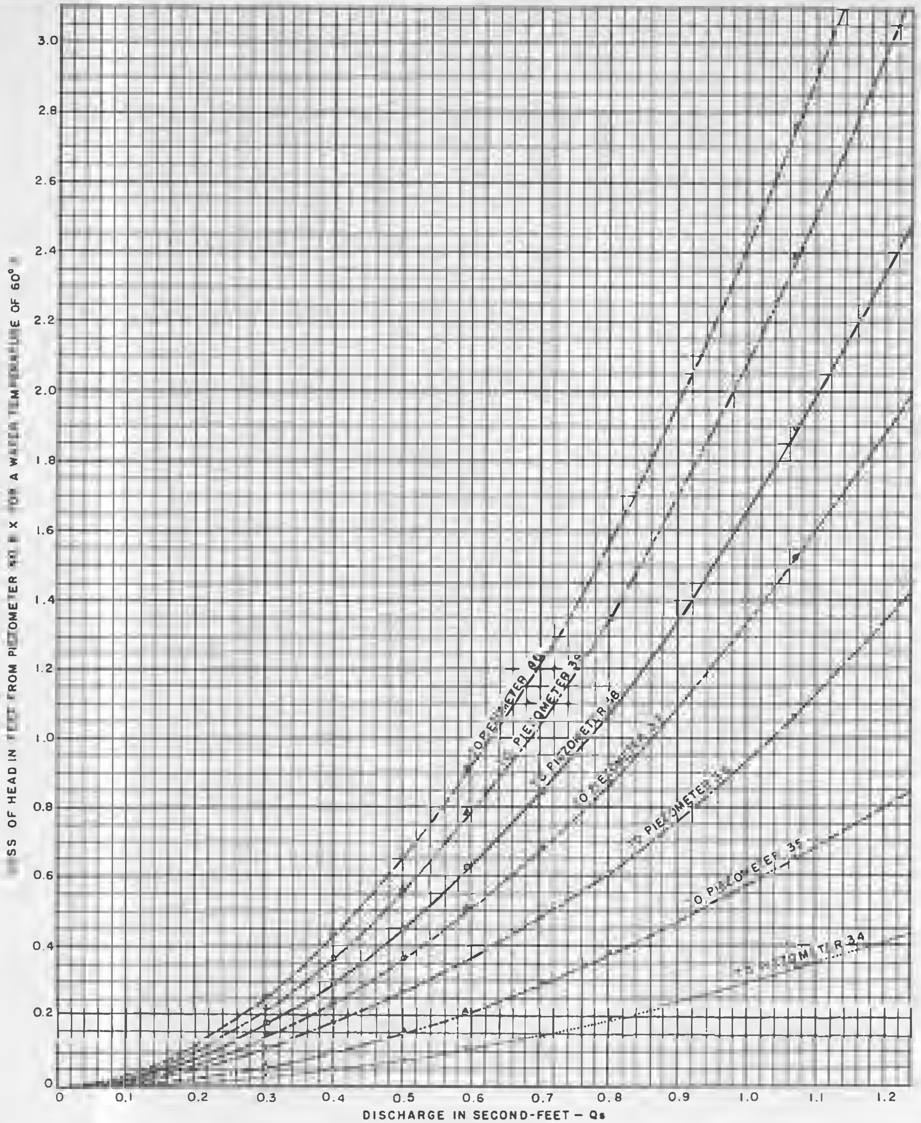
PENSTOCK STUDIES

LABORATORY ASSEMBLY OF MODEL
3.49" BRANCH AT 90° WITH MAIN PENSTOCK

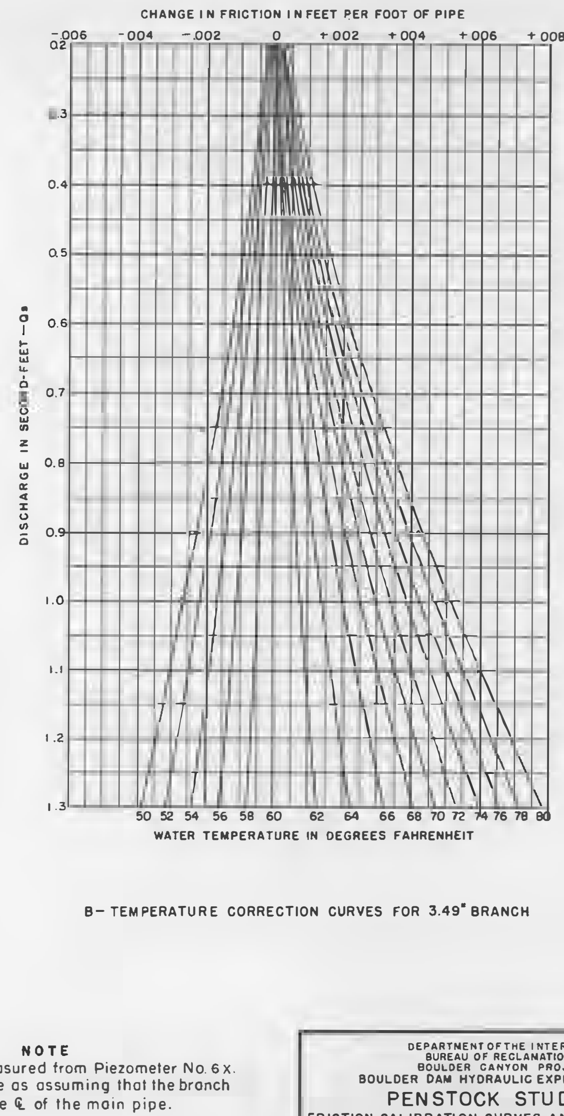
DRAWN. H.W.B. SUBMITTED J.N. Bradley
TRACED. M.F.W. E.E.B. RECOMMENDED E. Warnock
CHECKED J.N.B. APPROVED J.L. Savage

DENVER, COLO. JAN. 6, 1936. 45-D-9717



TEST 9

A– CALIBRATION CURVES FOR FRICTION IN 3.49" BRANCH
TEST 9 – FIRST CALIBRATION



B– TEMPERATURE CORRECTION CURVES FOR 3.49" BRANCH

NOTE

Pipe friction is measured from Piezometer No. 6 X. This is the same as assuming that the branch extended to the ℄ of the main pipe.

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PENSTOCK STUDIES

FRICTION CALIBRATION CURVES AND TEMPERATURE CORRECTION CURVES FOR 3.49" BRANCH

DRAWN. H.W.B. SUBMITTED.
TRACED. E.F.J. H.S. RECOMMENDED.
CHECKED. J.D.M. APPROVE.
DENVER, COLO., JAN. 3, 1932

45D-9718

FIGURE 21

curves representing the pipe friction from piezometer 6X to any piezometer in the main pipe for different discharges at a temperature of 60 degrees are shown on figure 22.

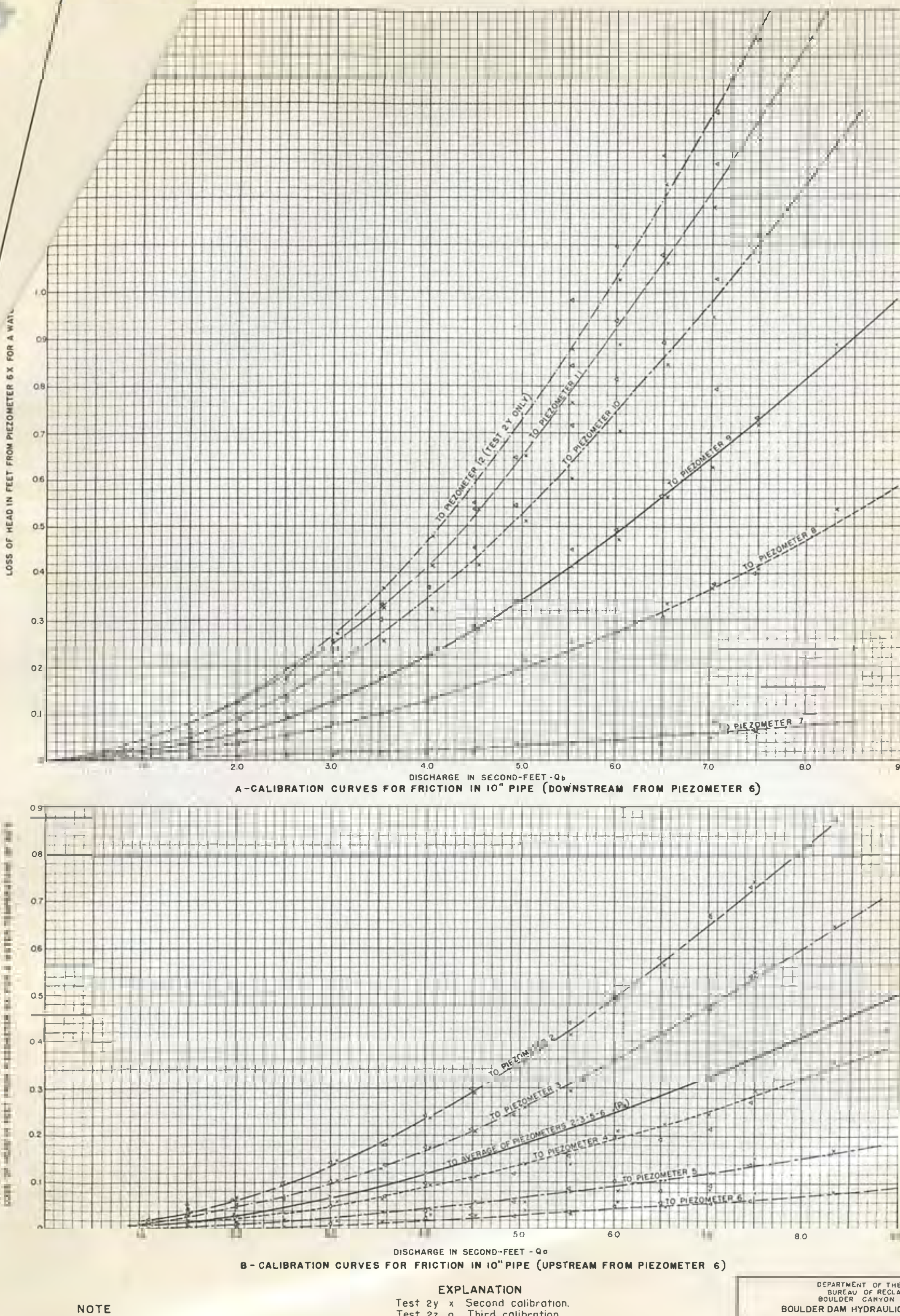
The test procedure for determining junction losses in tests 10 and 11 was the same as in test 3 except that the additional piezometer ring 6X was used as the reference point and P_x in tests 10 and 11 now represented the average readings of piezometers 2, 3, 5, and 6 rather than piezometers 2, 3, 4, and 5. Piezometers on ring 4 had been damaged and readings from it were no longer included in the average.

The runs on tests 9, 10, 11, and 9Z unfortunately were spread out over a period of two of the warmest summer months, as work of greater importance made it necessary to complete these tests as circumstances permitted. The following schedule shows the calendar dates on which the various runs were made.

Test No.	Run No.	Date Performed	Nature of Experiment
9	All runs	7/22 and 7/23	Initial friction calibration
10	1 to 8, incl.	7/27 and 7/28	90° branch, without cone
10	9 to 14, incl.	8/8 and 8/9	90° branch, without cone
10	15 to 18,incl.	8/26	90° branch, without cone
11	All runs	9/2 to 9/7, incl.	90° branch, with cone
9Z	All runs	9/28 and 9/29	Final friction calibration

The pipes were located from 4 to 6 inches above the floor of the laboratory calibration tanks. Consequently they were intermittently submerged while other experiments were in progress in the laboratory. This condition and the warm weather were conducive to the propagation of algae growth and the formation of rust spots in the pipes. Tests 10, 11, and 9Z showed that as time advanced the friction in the branch increased which was entirely possible due to the above factors. Strangely, the 10-inch pipe, which was subjected to the same conditions for a longer period of time, showed practically no change in pipe friction.

It was assumed that the junction losses obtained for runs 1 to 8, inclusive, of test 10 were probably correct as these runs were commenced within four days after the test 9 friction calibration. It was also assumed that the results from test 11 were presumably correct as the pipes were cleaned previous to these runs. Junction losses for these runs are plotted on figures 23 and 24.



A-CALIBRATION CURVES FOR FRICTION IN 10" PIPE (DOWNSTREAM FROM PIEZOMETER 6)

B-CALIBRATION CURVES FOR FRICTION IN 10" PIPE (UPSTREAM FROM PIEZOMETER 6)

NOTE
Pipe friction is measured in both directions from piezometer 6X

EXPLANATION
Test 2y x Second calibration.
Test 2z o Third calibration.

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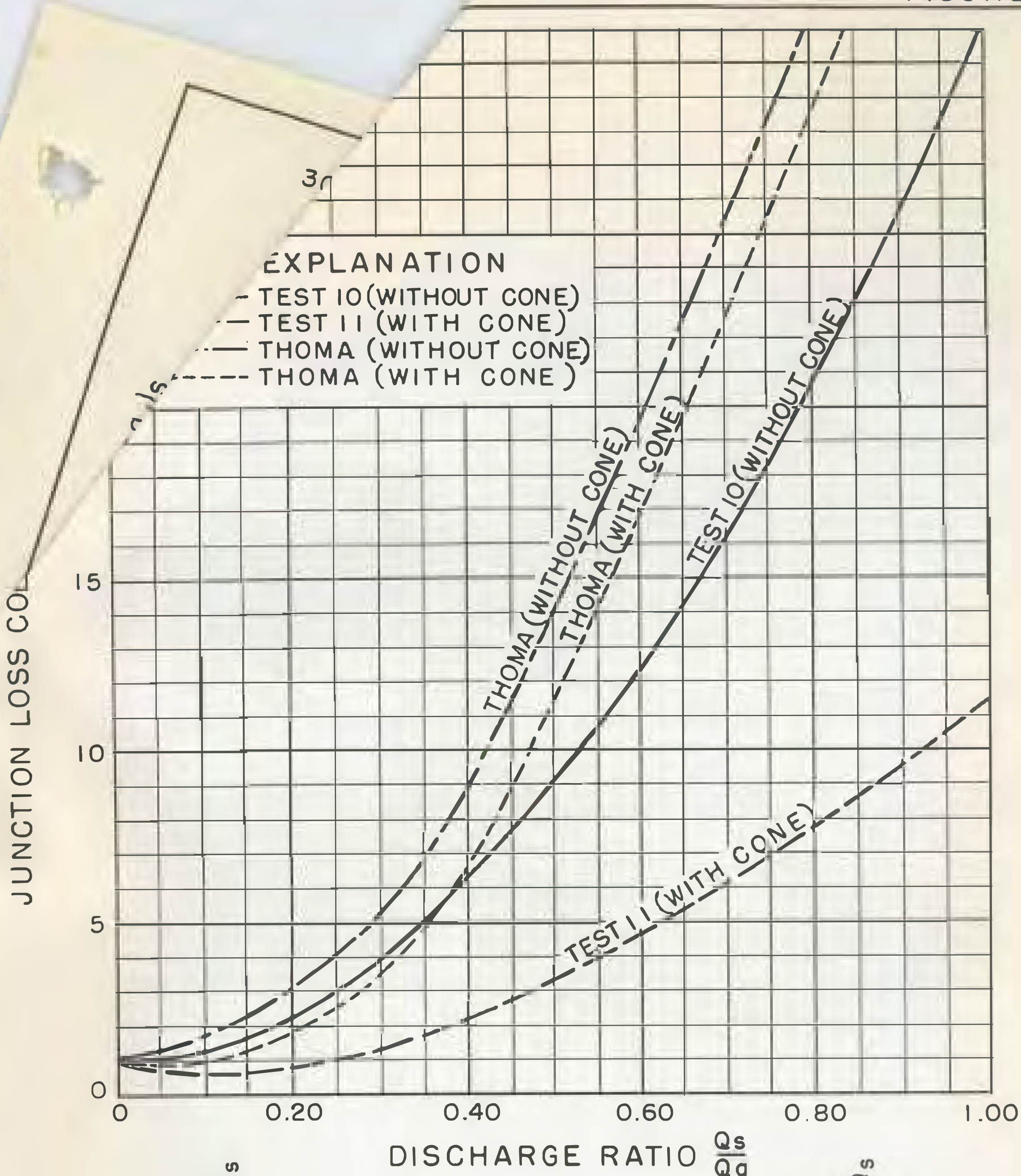
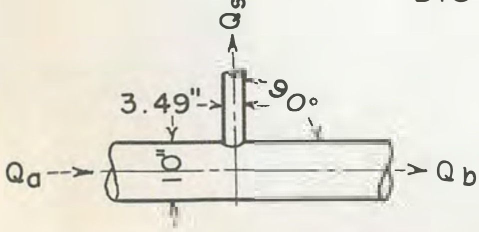
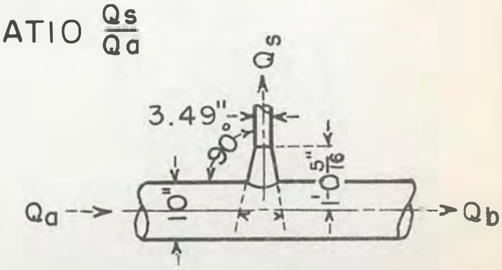
PENSTOCK STUDIES
FRICTION CALIBRATION CURVES FOR 10" PIPE

DRAWN H.W.B. SUBMITTED J.N. Bradley
TRACED H.S. RECOMMENDED E. Wanick
CHECKED J.N.B. APPROVED J.L. Savage
DENVER, COLO., DEC 30, 1935 45-D-9719





JUNCTION FOR TEST 10



JUNCTION FOR TEST 11

Junction loss in branch in terms of velocity head in main pipe upstream of junction.

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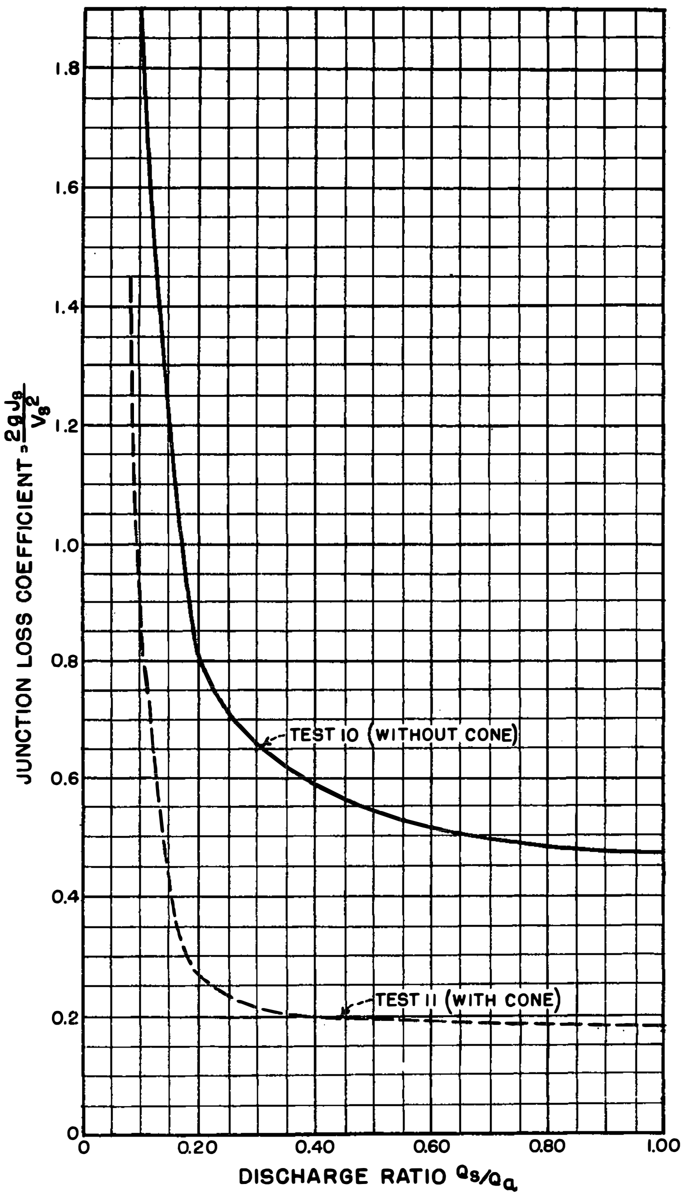
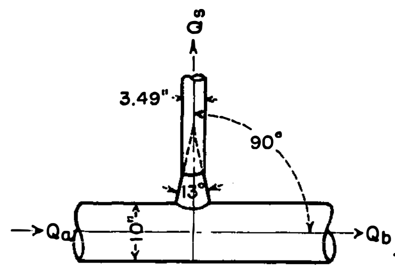
PENSTOCK STUDIES
JUNCTION LOSSES IN BRANCH PIPE

DRAWN. J.N.B. .... SUBMITTED. J. N. Bradley
TRACED. E.F.J. H.S. RECOMMENDED J. E. Warnock
CHECKED. J.D.M. APPROVED. J. L. Savage

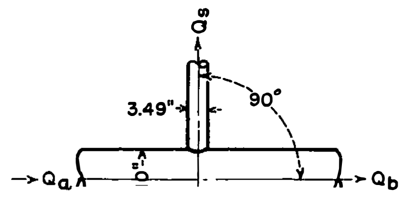
DENVER, COLO., APR. 18, 1936 | 45-D-9736



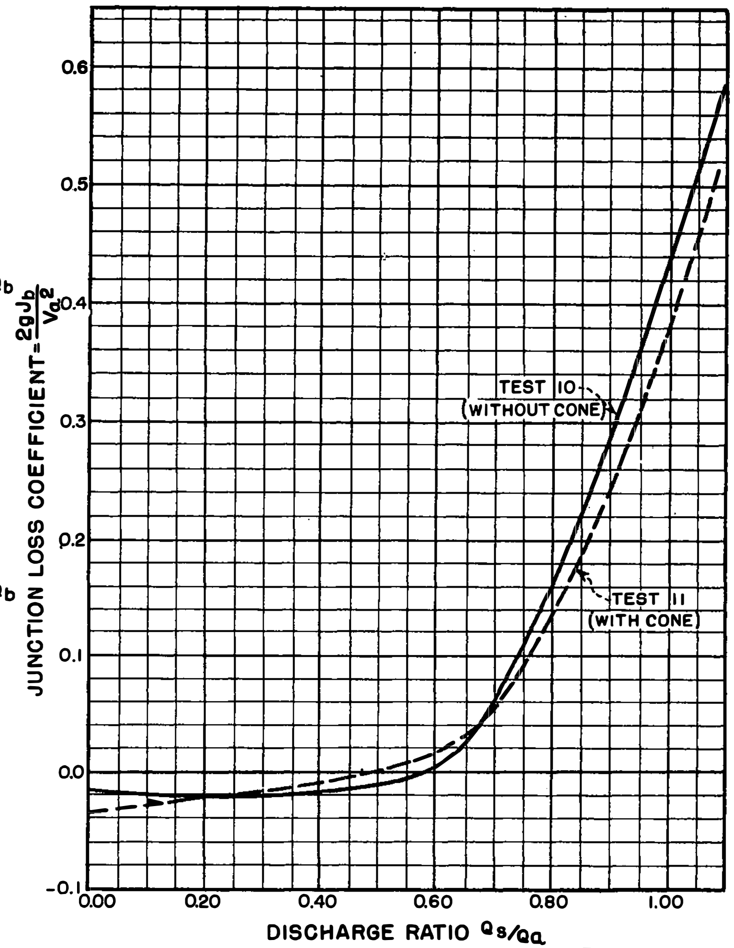
A– JUNCTION LOSS IN BRANCH IN TERMS OF VELOCITY HEAD IN BRANCH



TEST 11 — WITH CONE



TEST 10 — WITHOUT CONE



B – JUNCTION LOSS IN MAIN PIPE

NOTE

Junction losses in branch and main pipe obtained from average of individual piezometer curves.

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BOULDER DAM HYDRAULIC EXPERIMENTS

PENSTOCK STUDIES

JUNCTION LOSSES IN BRANCH AND MAIN PIPES

DRAWN H.W.B.	SUBMITTED J. N. Bradley
TRACED H.R.S.-D.W.S.	RECOMMENDED E. Wenzak
CHECKED J.N.B.	APPROVED J. L. Savage
	DENVER, COLO., JAN. 6, 1936 45-D-9745

FIGURE 24

A check on the results of tests 10 and 11 was made by another method. The pipe friction for the 3.49-inch branch and the 10-inch main pipe was obtained for each run by plotting the piezometer readings plus the corresponding velocity heads upstream and downstream from the junction and drawing average lines representing the friction slopes through the points to the junction. The difference in elevation of these lines at the junction represented the losses at that point. By this method, the junction losses and straight pipe friction were obtained simultaneously from the same runs, thus eliminating errors produced by the changing pipe friction. The check method showed that runs 1 to 8, inclusive, (test 10), and all runs of test 11 were reasonably correct as computed from the test 9 friction calibration. By the check method, all runs on test 10 fell approximately on the same curve. Cleaning the pipes previous to test 11 evidently reduced the pipe friction to agree with the test 9 friction calibration. Three weeks elapsed between the completion of test 11 and the second friction calibration (test 9Z) which afforded ample opportunity for the pipe friction again to increase in this length of time, which it did. The check method definitely indicated that the friction observed in calibration 9Z was too large to be applicable in the junction loss computations and was therefore discarded.

As a result of this investigation, runs 1 to 8, test 10 (without the cone) and all runs on test 11 (with the cone) are plotted in three forms on figures 23 and 24. Figure 23 shows the junction loss measured in the branch in terms of the velocity head in the main pipe upstream from the junction and figure 24A shows these same losses in terms of velocity head in the branch. The junction losses measured in the main pipe expressed in terms of the velocity head in the main pipe upstream from the junction are exhibited on figure 24B.

Thoma's junction loss coefficients for an exactly similar layout in which he used a 43-millimeter smooth brass pipe with a 90-degree 15-millimeter branch (with and without cone) are plotted on figure 23. These results are considerably higher than those obtained by the Bureau on larger pipes which would indicate that viscous effects in small junctions as well as in small pipes are appreciable. Although this incident does not constitute definite proof, it indicates that experimental data on small junctions for small values of Reynolds' number are not especially applicable for computing losses in large penstocks where large values of Reynolds' number are involved. Thoma's work dealt with Reynolds' numbers up to 100,000, while the Boulder Dam penstocks involve Reynolds' numbers as large as 90,000,000. Considerable recent data are available which indicate that the tendency is for losses to decrease with the use of larger models in spite of the fact that in a model of given size, little change in the loss coefficient is observed for varying velocities. If junction losses in large smooth penstocks such as those at Boulder Dam were computed on the basis of Thoma's experiments, the existing losses would undoubtedly be somewhat less than the computed losses.

Thoma's curve for the junction loss coefficient for the cone installed is unquestionably in error as a few rough computations will show. The addition of the cone should produce a very noticeable improvement in the entrance conditions to the branch. The Bureau's experiments show that the addition of a cone reduced the junction loss coefficient to approximately one-third of its original value.

As a matter of interest, it is possible to compute the junction losses for the layouts in tests 10 and 11 with a limited degree of accuracy by a method similar to that used previously for test 3.

The layout in test 10 (without cone) will be considered first. For an ordinary sharp-edged pipe entrance leading from a quiet reservoir, handbooks show the entrance loss to be approximately $0.5 \frac{V_s^2}{2g}$. In the case of a junction, various conditions of flow exist at the entrance to the branch, and an additional loss which is in the vicinity of $0.8 \frac{V_a^2}{2g}$ should be combined with the entrance loss. The total junction loss then as computed for the branch is

$$J_s = 0.5 \frac{V_s^2}{2g} + 0.8 \frac{V_a^2}{2g}$$

The second term is a variable but is approximately $0.8 \frac{V_a^2}{2g}$ for the lower values of $\frac{Q_s}{Q_a}$ where it is effective (fig. 23). For larger values of $\frac{Q_s}{Q_a}$ this term is negligible compared to the first. Expressing this equation in a more practical form the junction loss coefficient for the branch is

$$\frac{J_s}{V_s^2/2g} = 0.50 + 0.8 \left(\frac{Q_a}{Q_s}\right)^2 \left(\frac{A_s}{A_a}\right)^2$$

where $\frac{A_s}{A_a} = 0.1216$.

Upon substituting different values of $\frac{Q_a}{Q_s}$ in the above equation a curve can be obtained which will approximate the one for test 10 on figure 24A.

The junction loss for test 11 (with the cone) may be estimated in the same manner by substituting the proper coefficients in the above equation. Upon comparing the entrance to the branch in test 11 with test 3, one would expect the entrance for test 3 to be slightly superior to that in test 11. Therefore, it seems logical to substitute the following coefficients in the formula

$$J_s = 0.2 \frac{V_s^2}{2g} + 0.5 \frac{V_a^2}{2g}$$

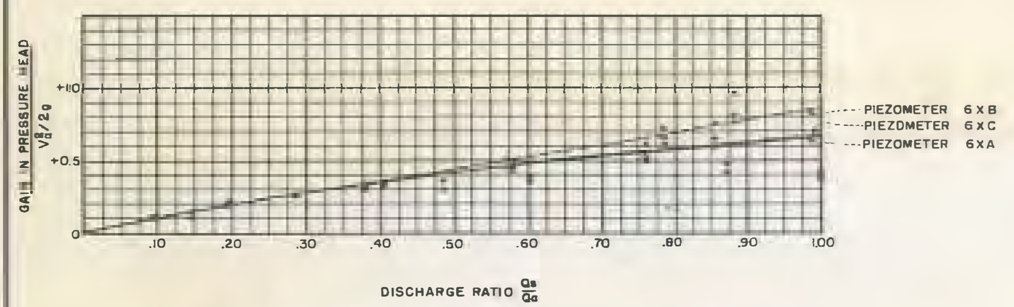
Expressing this in the other form, the junction loss coefficient becomes

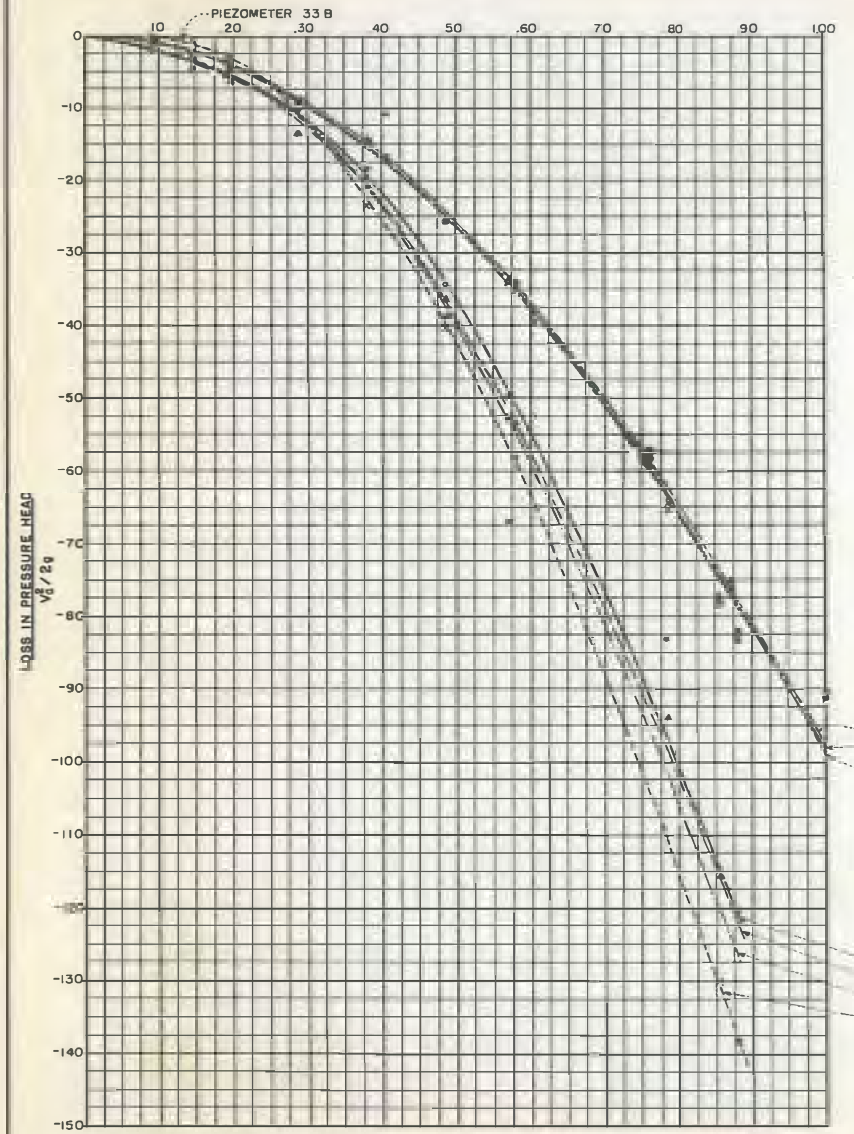
$$\frac{J_s}{V_s^2/2g} = 0.20 + 0.5 \left(\frac{Q_a}{Q_s}\right)^2 \left(\frac{A_s}{A_a}\right)^2$$

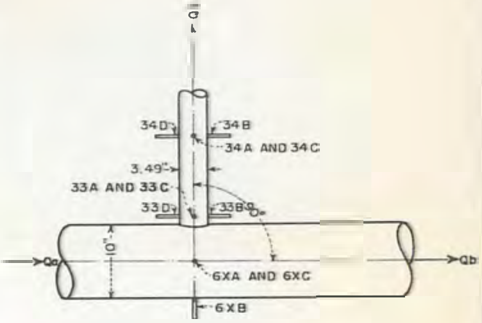
The curve obtained by substituting values of $\frac{Q_a}{Q_s}$ in this equation will be found to practically coincide with the curve for test 11 on figure 24A.

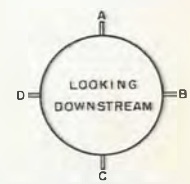
H. Investigation of Static Pressures at Right-Angle Junction

Piezometer rings 33 and 34 were installed in the branch near the junction (fig. 20) for the purpose of studying the pressures in that zone and were read with the other piezometers during tests 10 and 11. The drop in pressure between each of these piezometers and the piezometers at the theoretical junction are plotted on figures 25 and 26, respectively. The difference in pressure, expressed in terms of velocity head in the main pipe above the junction, is plotted with respect to the discharge ratio. In both tests 10 and 11, piezometers 6XA, 6XB, and 6XC show a pressure above the average obtained at ring 6X without the branch attached. A slightly higher pressure was developed in the main pipe for test 10 than for test 11. The curve for piezometer 33B (fig. 25) is quite similar to the curve for piezometer 14B on figure 18, although there was no increase in pressure developed in the first. The curve for piezometer 33B on figure 26 shows the effect on the pressure at this point produced by the addition of the cone. An increase in pressure was









PIEZOMETER LOCATIONS
TEST 10
(WITHOUT CONE)

EXPLANATION

PIEZOMETER 33 A
33 B
33 C
33 D
34 A
34 B
34 C
34 D

NOTE

Loss in pressure represents the difference in pressures from piezometer 6 x (without branch attached) to any piezometer plotted. Note that the difference is a slight gain in pressure between piezometer 6 x (without the branch) and piezometer 6 x (with the branch attached.)

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PENSTOCK STUDIES

INVESTIGATION OF PRESSURES AT JUNCTION
FOR TEST 10

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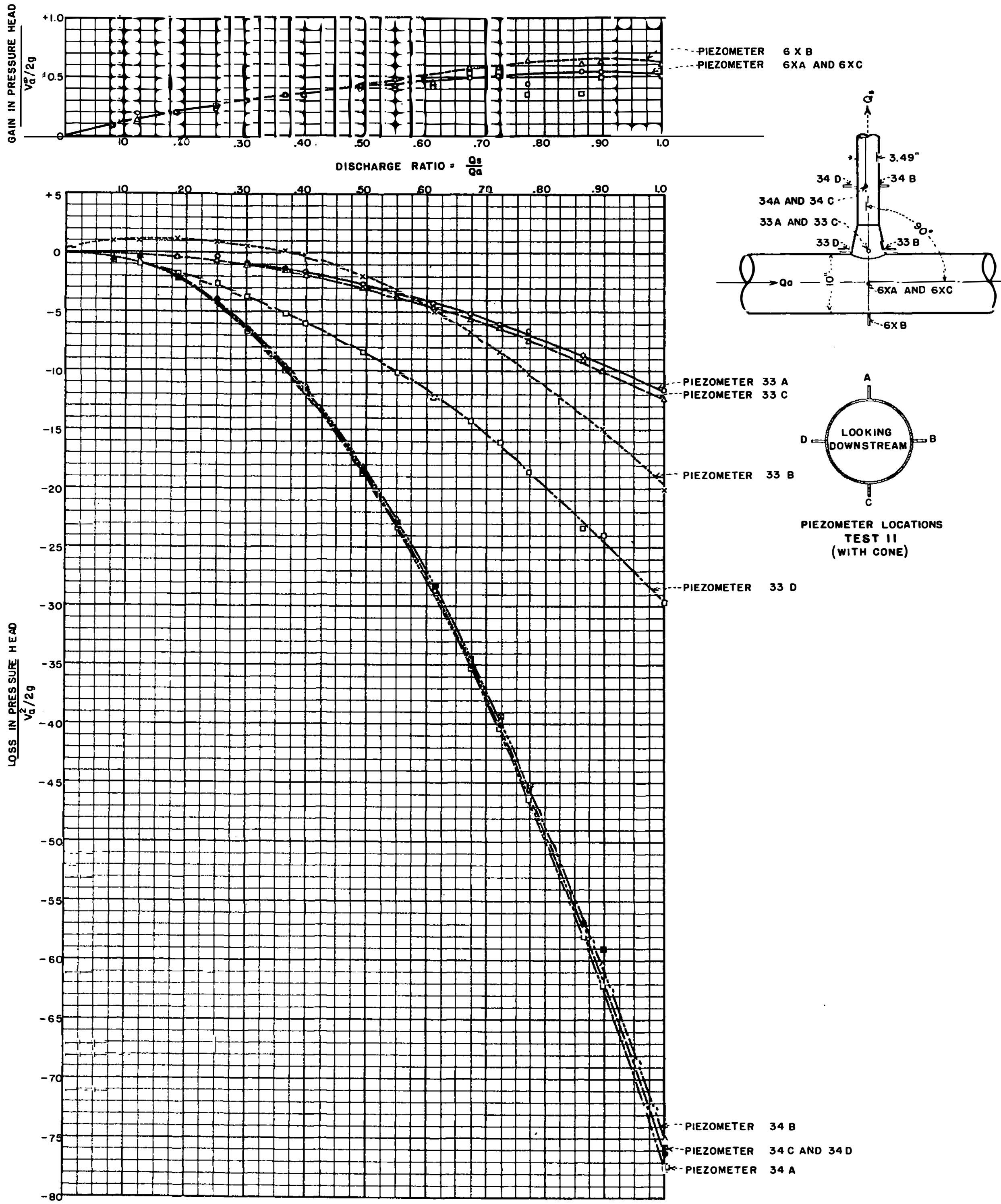
DENVER, COLO., JAN. 2, 1936 45-D-4720

FIGURE 26



PIEZOMETER LOCATIONS
TEST 11
(WITH CONE)

EXPLANATION

PIEZOMETER	33 A	o———o
"	33 B	x- - - - -x
"	33 C	△— — —△
"	33 D	□—-—□
"	34 A	⊔—-—⊔
"	34 B	^-----^
"	34 C	⊠—-—⊠
"	34 D	⊠—-—⊠

NOTE

Loss in pressure represents the difference in pressure from piezometer 6X (without branch attached) to any piezometer plotted. Note that the difference is a slight gain in pressure between piezometer 6X (without the branch) and piezometer 6X (with the branch.)

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BOULDER DAM HYDRAULIC EXPERIMENTS

PENSTOCK STUDIES

INVESTIGATION OF PRESSURES AT JUNCTION FOR TEST 11

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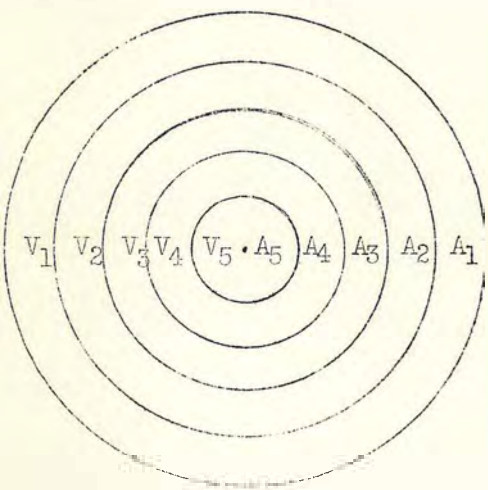
DENVER, COLO. JAN 2, 1936 45-D-9721

registered at piezometer 33B for discharge ratios below 0.40 and only a small drop was observed for ratios above this value.

I. Velocity Distribution in the Main Pipe

At the conclusion of the experiments on the quantitative model of the pipe junction, a study was made of the velocity distribution in the 10-inch pipe with the branch removed. The pitot tube (plate VA) was mounted near piezometer 6 or approximately 18 diameters downstream from the pipe entrance and traverses made in both a horizontal and a vertical plane passing through the center line of the pipe. Five pitot-tube readings were made at each point shown on figure 27A.

The actual velocities measured at the several points are plotted on figure 27 for six discharges. Isovels representing points of equal velocity were drawn and the areas between these lines obtained by the use of a planimeter. The nonuniformity in the velocity distribution is probably due to the unsymmetrical entrance conditions to the pipe.



A = Total area of pipe

$V = \frac{Q}{A}$ = Mean velocity in pipe

βV = Average velocity in pipe

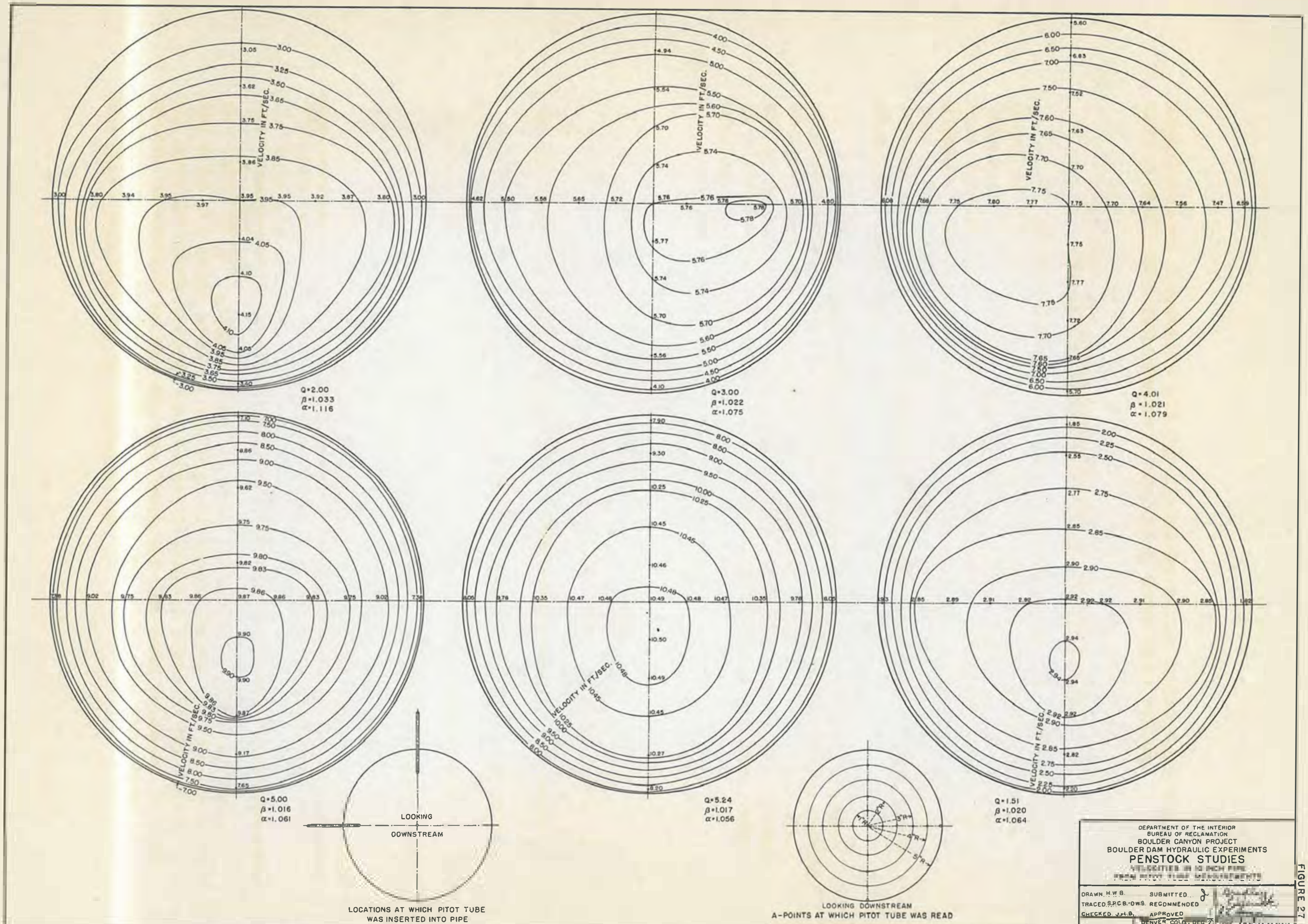
H_V = Total mean velocity head in pipe

Q = Total discharge in second-feet

The coefficient of velocity in a circular pipe is expressed[5]

[5] "Branch Losses in Pipes by the Momentum Method" by Ralph W. Powell, U.S.B.R. Library.

as $\beta = \frac{\int v^2 \, dA}{V^2 A}$, where V is the mean velocity in feet per second and A is the total area of the pipe in square feet. If the areas of the annular rings between the velocity contour lines in the above sketch be denoted as a_1, a_2, a_3, etc., and the mean velocity in each respective area be indicated as v_1, v_2, v_3, etc., the numerator of the above equation can be expressed as


VELOCITY IN FT./SEC.
Q=2.00
β=1.033
α=1.116
Q=3.00
β=1.022
α=1.075
Q=4.01
β=1.021
α=1.079
Q=5.00
β=1.016
α=1.061
Q=5.24
β=1.017
α=1.056
Q=1.51
β=1.020
α=1.064
LOOKING
DOWNSTREAM
LOCATIONS AT WHICH PITOT TUBE
WAS INSERTED INTO PIPE
LOOKING DOWNSTREAM
A-POINTS AT WHICH PITOT TUBE WAS READ
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PENSTOCK STUDIES
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APPROVED
DENVER, COLO., DEC. 2
45-D-9722
FIGURE 27

$$\sum(v_1{}^2a_1 + v_2{}^2a_2 + v_3{}^2a_3 + --) \text{ or } 2g \sum(h_{v_1}a_1 + h_{v_2}a_2 + h_{v_3}a_3 + ---)$$

$$= 2g \sum(h_v a).$$

The mean velocity, V, in the pipe is equal to

$$\frac{\sum(v_1a_1 + v_2a_2 + v_3a_3 + ---)}{A}$$

$$\text{or} \quad \frac{\sum(q_1 + q_2 + q_3 + ---)}{A} = \frac{Q}{A}$$

The denominator of the equation is then simply $\frac{Q^2}{A}$ which can also be expressed as $2g\ H_v A$, where H_v is the total mean velocity head in the pipe.

Substituting the numerator and denominator into the original equation, the result is

$$\beta = \frac{\sum(h_v a)}{H_v A}$$

The experimental values were substituted in the above equation for the six discharges investigated and it was found that for these discharges the value of β ranged from 1.016 to 1.033, as indicated on figure 27.

The energy head correction factor[6] expressed as $\alpha = \frac{\int v^3 dA}{V^3 A}$, where v = velocity in each individual area bounded by isovels, V = mean velocity in pipe in feet per second, and A = total area of pipe in square feet, was computed for the six runs shown on figure 27. The integration of the expression $\int v^3 dA$ was accomplished by plotting a mass diagram, the coordinates of which were the area of the pipe and the cube of the individual velocities at isovels. The numerator of the above equation was then obtained by planimetering the area under the curve. The denominator was procured by multiplying the total area of the pipe by the sum of the cubes of the individual velocities between isovels. The velocity head coefficient α ranged from 1.020 to 1.116 and these values are shown listed under the separate runs on figure 27. The true velocity head is then αH_v, where H_v = total mean velocity head in the pipe.

The junction loss results would have appeared slightly

[6]"Velocity-Head Correction for Hydraulic Flow", by M. P. O'Brien and J. W. Johnson, Engineering News-Record, August 16, 1934.

different had the true velocity head been used in the computations rather than the mean velocity head, as in a short pipe such as this, the coefficient varies as the distance down the pipe. It is evident, therefore, that numerous measurements would have been required to obtain a true interpretation of the velocity conditions throughout the pipes. The results of these experiments together with others indicate that the velocity head coefficient for a given pipe decreases with an increase in Reynolds' number, and that for a given Reynolds' number the coefficient decreases as the size of pipe increases.

It was realized that additional studies of the velocity distribution would have been interesting, and perhaps valuable, but the requirements of this particular problem were not such as to justify, and the time limitations prohibited these additional studies. Furthermore, such additional work would have required a material refinement of the apparatus.

J. Summary and Conclusions on Junction Tests

The foregoing tests may be summarized as follows:

1. The installation of a filler block at a junction to supplant the existing eddy formation at this location produced no material improvement in the efficiency of the junction.

2. Comparison of results obtained by Thoma and the Bureau on a similar junction of different size indicated that the junction loss decreased as the size of the junction increased. The larger loss indicated by the smaller junction is primarily due to viscous effects and to the fact that the straight pipe friction is seldom to scale, although the junctions may be geometrically similar. It is, therefore, logical to assume that the greatest deviations in the junction losses occur for the small sizes.

3. The addition of a cone connecting a right angle branch with the main pipe reduces the junction loss coefficient to approximately one-third of its original value.

In conclusion, it may be well to emphasize the fact that the foregoing experiments were performed for a specific purpose and were not intended to constitute a general study of the subject. The results of these tests may prove of value for a similar case in the future which is within the limitations of these experiments. The majority of the graphs have been expressed such that the values are dimensionless, thus it would appear that the same values indicated by the graphs apply for both model and prototype. Strictly speaking this is not entirely the case, especially on very small models, due to the factors mentioned above. It is expected that the junction losses in the Boulder Dam penstocks will be slightly less than those obtained on the model, in the same way that the losses in the model were less than those obtained by Thoma on a smaller but similar junction. The magnitude of the variation is expected to be less pronounced in the former case as the model was of sufficient size to largely eliminate these viscous effects.

4. ACCURACY OF EXPERIMENTAL RESULTS

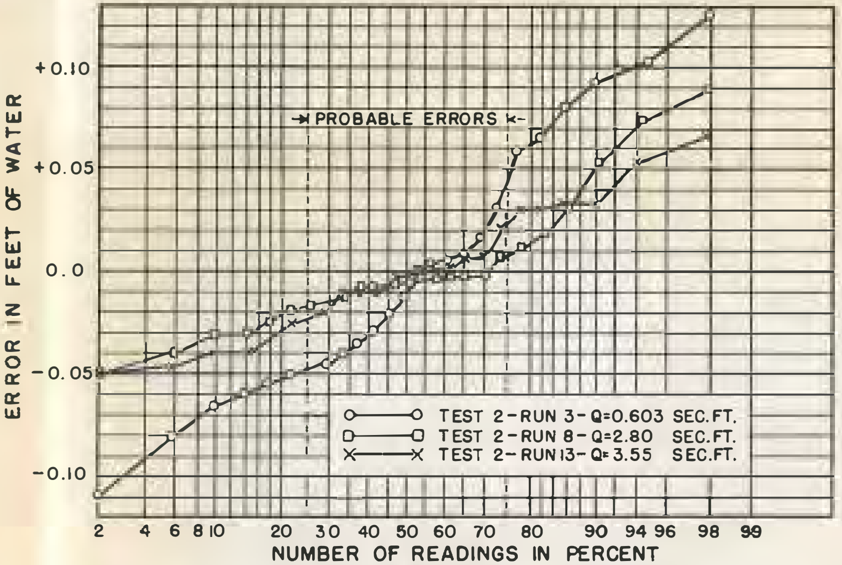
A. General

The model tests on the junction losses of the Boulder Dam penstocks comprised approximately 25,000 separate observations. These observations, made with equipment of the greatest practical accuracy, with multiple readings of piezometers located every four to six diameters over the entire length of the pipe tested, afford an exceptional opportunity to evaluate experimental errors, to analyze the distribution of hydraulic losses, and to develop a procedure of maximum efficiency for future tests.

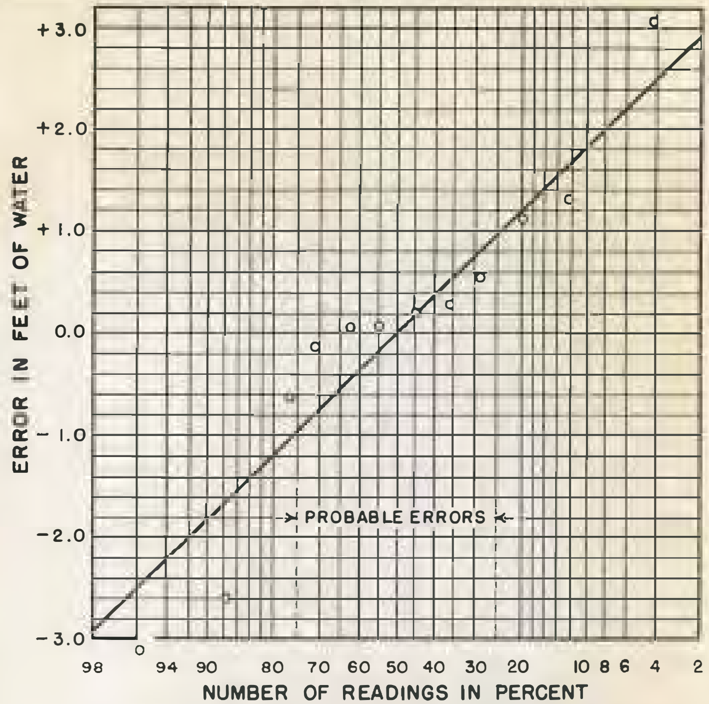
In this report, the theoretical energy loss due to a fitting has been defined as the sum of the piezometric drop and the difference in mean velocity heads, less the normal friction loss between the two piezometer stations. Theoretically, the two piezometer stations should be located sufficiently distant upstream and downstream from the fitting that normal velocity distribution exists in the stream cross sections. To satisfy this definition, a complete loss determination requires the simultaneous measurement of two mean velocity heads, of two piezometeric elevations, and of the loss which would exist in the same total length of pipe if it were straight.

B. Errors in the Mean Velocity Head

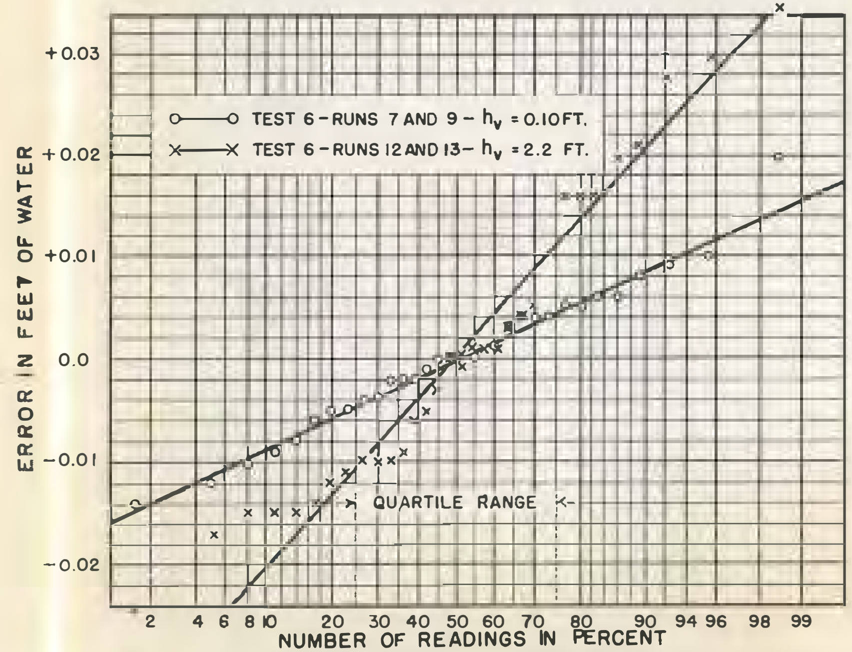
The rate of discharge in these tests was measured by weirs which had previously been calibrated volumetrically. It is thought that absolute errors in the discharge as determined from the measurements did not exceed one-half of one percent. From the weir, the water passed to a stilling basin and thence to equipment under test. Although nearly an hour was allowed before each test for the flow to stabilize, fluctuations persisted in the stilling basin. The water rose and fell slowly with a probable variation of 0.025 foot from the mean elevation (fig. 28A). At the end of each run, therefore, the average discharge computed from the mean of the weir readings was corrected to allow for any storage which had taken place in the forebay. The correction for even the lowest discharge was less than 0.2 percent. However, an observational error was introduced by the momentary changes in the total head on the apparatus. Assuming the total head as $3h_v$, with velocity heads of one foot and over, the error for a single observation becomes $\frac{1}{3} \times \frac{0.025}{1.0} = 0.8$ percent of h_v. On the theory that the probable error in the velocity head due to forebay fluctuations is re-



A-PROBABILITY DISTRIBUTION OF FLUCTUATIONS IN RESERVOIR ELEVATIONS



C-DISTRIBUTION OF MEAN ERRORS DUE TO PHYSICAL MAKEUP OF PIEZOMETERS



B-PROBABILITY DISTRIBUTION OF FLUCTUATIONS IN PIEZOMETER READINGS

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PENSTOCK STUDIES
PROBABLE ERRORS IN JUNCTION LOSS TESTS

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CHECKED J.N.B. APPROVED J. L. Savage

DENVER, COLO. JUNE 3, 1938 | 45-D-10285

FIGURE 28

duced in inverse proportion to the square root of the number of observations,[7] the sixteen readings made on each piezometer ring re-

[7] For a detailed discussion of the application of the theory of least squares to the interpretation of errors, see Root: "Mathematics of Engineering", Williams and Wilkins Co., Baltimore.

duce this error to $\frac{0.8}{\sqrt{16}}\ h_v = 0.2$ percent of h_v.

Repeated microcaliper measurements showed that the diameters of the pipes did not vary more than 0.003 inch or 0.1 percent for the 3.49-inch branch. This error would cause a deviation of approximately 0.2 percent in the mean velocity head. The previously mentioned error of 0.5 in the weir discharge is equivalent to an error of $(1.005)^2 - 1.00$ or one percent in the mean velocity head. Combining these with the error caused by the forebay fluctuations, there is obtained a probable total error in the mean velocity head of $\sqrt{(.01)^2+(.002)^2+(.002)^2}$ or only slightly in excess of one percent.

Although in accord with the usual practice, the mean velocity head was used in this report for the computation of kinetic energy head, it is known that because of the nonuniform distribution of the velocity, the kinetic energy would be from two to nine percent in excess of this for Reynolds' numbers between 50,000 and 1,000,000. Furthermore, the corrections may be much larger downstream from fittings below which contractions of the stream have been produced. Information concerning the magnitude of this correction at the location just upstream from the junction in the 10-inch pipe tests (approximately 18 diameters below the entrance) is given in Chapter IV, section 3-I of this report. It need only be stated here that the numerical values of the losses given in this report have been computed on a basis of mean velocity heads. The error involved in this procedure will be further discussed subsequently.

C. Errors in the Determination of the Hydraulic Grade

The determination of the hydraulic grade at any point was subject to the following errors:

(1) Fluctuations in the headwater elevations.
(2) Fluctuations due to turbulence.
(3) Errors in the mechanical construction of the piezometers.
(4) Errors due to lack of parallelism in flow.
(5) Various errors due to capillarity, temperature, entrained air, etc.

With regard to errors (1) and (2), since approximately half of the total pressure head on the apparatus was transformed into velocity head at the entrance, the effect of reservoir fluctuations on the fluctuations in the pipe piezometers might be expected to be reduced one-half. Figure 28A indicates that the probable fluctuation in the reservoir was about 0.025 foot; so the expected fluctuation in a piezometer is of the order of 0.012 foot. Results showing the piezometer fluctuations for two different discharges and at 16 different piezometers are plotted on figure 28B. These indicate a probable error for a single observation of 0.008 foot. Since these data take into account changes in piezometric levels caused both by headwater and turbulence fluctuation, and the error is less than that expected by headwater fluctuations alone, a reasonable conclusion appears to be that turbulence surges were of relatively little importance.

Four readings of each piezometer were taken at about ten-minute intervals during each run. The fluctuation error for the set of readings for a single piezometer would therefore be $\frac{0.008}{\sqrt{4}} = 0.004$ foot. Since four piezometers comprised a ring, the error of the piezometric grade as determined by the ring and caused by fluctuations only would be 0.002 foot.

Due to slight roughness at the edges of the openings or nonparallelism of flow (error 3), most piezometers exhibited positive or negative deviations varying with the velocity head. Assuming the correct piezometric elevation at a given ring to be that of the mean of the four piezometers, each read four times, a study has been made of the variation of the individual piezometers from the mean. For this purpose the experiments made to determine the friction losses in the 10-inch straight pipe were used, the analysis being based on the piezometer errors of rings 6, 10, and 11, located 18, 44, and 52 diameters downstream from the rounded intake.

Eight runs with velocity heads varying from 1.5 to 3.5 feet were utilized and the differences from the mean piezometric level of each run for each ring were expressed in terms of the velocity head. The distribution of the mean error for the twelve piezometers is shown on figure 28C. The indicated probably error for a single piezometer due to mechanical defects is $0.01\ h_v \pm 15$ percent. With a mean velocity head equal to 2.5 feet, the corresponding error is 0.025 foot. With a ring containing four piezometers, the probable error is 0.012 foot which is comparable to the 0.002 foot caused by headwater fluctuations. Although the error due to mechanical defects of the piezometers may appear large, previous work on the subject[8] indicates that

[8] "Piezometer Investigations" - Allen and Hooper - Trans. A.S.M.E. Hyd. 54-1 - 1932.

it does not exceed the limit of practical accuracy.

In connection with lack of parallelism in the flow (error 4), there is reason to believe that water entering a pipe system through an imperfectly shaped mouthpiece passes through the same phenomena of contraction and expansion as those experienced by a free jet issuing from an orifice. In the straight pipe experiments on 3.49-, 4.33-, and 10-inch pipes, with piezometers located 3.5 to 7 diameters along the pipes, the experimental data indicate that points of low and high pressure analogous to waves occurred downstream from the entrance at intervals of from 6 to 8 diameters. The amplitude of the pressure deviation was about one percent of the velocity head, and it was not entirely damped in a length of even 50 diameters. Although the experimental data is not conclusive, it is estimated that there was a probable error from this cause in the order of 0.7 percent of the velocity head at the upper end and 0.2 percent of the velocity head at the low er end.

No attempt has been made to evaluate errors due to temperature, entrained air, and capillary effects (error5). In comparison with other errors it is believed that these may be neglected.

D. Errors in Obtaining Energy Grade

For any given run, the energy grade at any piezometer ring was determined by taking the mean of the 16 separate readings for the four piezometers over a period of perhaps 40 minutes and adding to it the mean velocity head for the same period. The probable error in the result would then be as follows:

$$\gamma^2 = \text{velocity head error}^2 + \text{error due to standing pressure waves}^2 + \text{mechanical error of piezometer}^2 + \text{fluctuation error}^2.$$

Assuming the fluctuation error to be equal to or less than 0.002 h_v for heads of one foot and over, the experimental error in the energy grade at any piezometer ring for a given run becomes

$$\gamma = h_v \sqrt{(.002)^2+(.002)^2+(.005)^2+(.002)^2} = 0.6 \text{ percent of } h_v.$$

For velocity heads of three feet or more this error amounts to 0.02 foot. The variations of this magnitude revealed by the plot of energy grades (fig. 12) are common. These errors are exclusive of those previously mentioned as resulting from inaccuracies in the weir measurements.

E. Errors in Determination of Losses in Straight Pipe

At the start of the experimental program it was contemplated that the losses in straight pipe required by the experimental procedure would be obtained "once for all" by tests for this sole purpose on a straight pipe as long as the laboratory facilities would permit. The following apparatus was available:

(1) 3.49-inch pipe: Entrance 29-degree cone with 16:1 entrance-to-throat area ratio; calming length, 38 diameters containing 6 piezometer rings; end length, 14 diameters with 3 piezometer rings.

(2) 4.33-inch pipe: Entrance, 27 degree cone with 10:1 entrance-to-throat area ratio; calming length, 26 diameters containing 4 piezometer rings; end length, 16 diameters with 3 piezometer rings.

(3) 10-inch pipe: Entrance, semicircular beading on face of wall with radius equal to 0.4 that of pipe; entrance-to-throat area ratio 2:1; calming length, 36 diameters containing 8 piezometer rings; end length, 16 diameters with three piezometer rings.

Since the pipe used was of the same quality throughout and all piezometer rings were of equal accuracy, the distinction between calming length and measuring section is a nominal one, but is useful for analyzing errors.

According to Nikuradse, the velocity distribution is nearly uniform at the throat of a well-rounded pipe entrance whose shape is such that no contraction occurs; but for smooth pipes, the distribution changes into the typical velocity profile within a distance of from 25 to 40 diameters downstream. The energy losses during this transition must be very small and may be even less than in the portions of the pipe subjected to normal flow. The change in velocity distribution, however, results in a conversion of potential to kinetic energy. The hydraulic grade line for this case, therefore, tends to take the form of a falling curve, concave upward, steeper at the upstream end, and flattening into a straight slope descending toward the outlet. If, now, the entrance conditions are not ideal so that a tendency toward contraction exists, the kinetic energy at the contracted section is much higher than would be indicated by the calculation from the mean velocity corrected for normal distribution. The static pressures would show a drop from the absolute entrance to the vena contracta, followed by a sharp rise and subsequent gradual drop, the latter possibly subject to fading oscillations. The experimental measurements of static pressures as made, begin approximately at the probable location of this contraction, and show the sharp rise and succeeding drop with oscillations. When a constant kinetic energy term based on downstream conditions is added to this pressure distribution, the resulting nominal energy grade-line shows the same variations as the measured pressures, and is thus too low at the first point, may be too high at its peak, and may oscillate about the true grade-line with diminishing amplitude thereafter.

It is evident, therefore, that to obtain the normal loss in a straight pipe, either the pressure drop must be obtained at a location sufficiently distant from the entrance that the velocity distribution is normal and the hydraulic grade is a straight line, or the true value of the kinetic energy at the measured cross sections must be known. Unfortunately, in the Bureau's tests, the length of pipe which could be accommodated was limited so that only 14 to 15 diameters at the downstream end could be considered as having approximately normal velocity distribution. The measured energy drop in this length approximated $0.23\ h_v$. Since it has been shown that the accidental error in the mean results of a set of readings on a single piezometer ring is of the order of $0.006\ h_v$, for a single determination of the friction coefficient from the drop between two rings, the accidental error is $\frac{\sqrt{2} \times 0.006\ h_v}{0.23\ h_v} = 3.7$ percent of the energy drop. If the error in the kinetic energy head as calculated on a basis of the mean velocity corrected by the arbitrary factor, α is as much as 3 percent, which is the range of the empirical values of α, then the possible error may be as much as $\frac{(0.23 + .0085)(1 + .03)\ h_v - 0.23\ h_v}{0.23 h_v}$, or 6.8 percent of the energy drop.

F. Entrance Losses

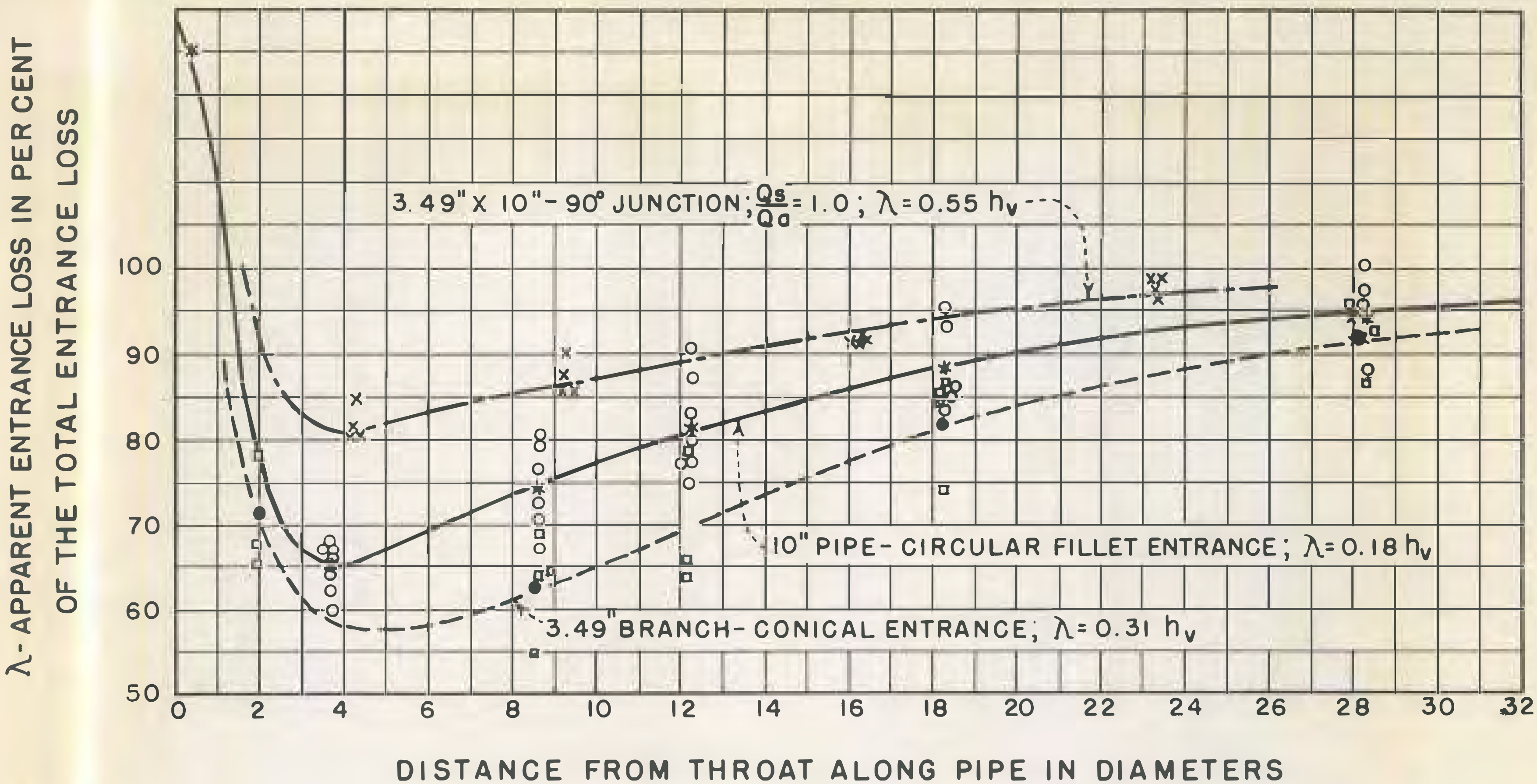
During the tests for friction losses in straight pipes, advantage was taken of the opportunity to measure the nominal entrance losses and their rate of development. The results obtained from this series of observations and their maximum deviation from a mean are as follows:

(1) 3.49-inch pipe entrance, loss $0.14\ h_v \pm 2.5$ percent
(2) 4.33-inch pipe entrance, loss $0.11\ h_v \pm 3.3$ percent
(3) 10-inch pipe entrance, loss $0.18\ h_v \pm 2.5$ percent

The accumulative percentage of the total apparent entrance loss at various diameters below the entrance is shown on figure 29. It is noted that, even at 30 diameters, only 95 percent of the total entrance loss has occurred.

Since this data was obtained from apparatus whose total length was limited, there is reason to believe that the determinations of both the entrance losses and the straight pipe losses include large persistent errors. Thus, if only five percent of a given entrance loss, assumed to be $0.20\ h_v$, occurs in the end 22 diameters downstream from the 30-diameter point, the straight pipe loss determined from this section will have a persistent positive error of $\frac{0.05 \times 0.20}{0.23 \times \frac{22}{16}} = 3.2$ percent. This effect, combined with the fact that the entrance losses were much higher for the 10-inch pipe than for the 3.49-inch pipe, may account for the higher straight-pipe losses indicated by the 10-inch pipe.

In these tests the difference between the straight pipe and entrance loss is distinguished by constructing a tangent to the hydraulic grade line at a point about 40 diameters below the entrance. The straight pipe loss in the upstream 40 diameters is thus $40 \times 0.015\ h_v = 0.60\ h_v$. Since the accidental errors would be practically the same fraction of h_v regardless of the pipe size,



DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS
PENSTOCK STUDIES
DISTRIBUTION OF ENTRANCE LOSSES

DRAWN S.P.W. SUBMITTED J. C. Wagner
TRACED E.F.J. H.S. RECOMMENDED R. F. Blanks
CHECKED J.N.B. APPROVED J. L. Savage

DENVER, COLO., JUNE 3, 1936 45-D-10271

FIGURE 29

and since, as has been shown, the accidental error may amount to as much as 3.7 percent of h_v, the accidental error in the determination of the entrance loss for the 4.33-inch pipe, for which the total entrance loss is 0.11 h_v, might amount to $\frac{0.037 \times 0.60\ h_v}{0.11}$ or about 20 percent. The repetition of readings would reduce this to $\frac{.20}{\sqrt{25}}$ or about 4 percent which is comparable to the 3.3 percent given in a preceding paragraph.

G. Branch Loss Errors

Although absolute values of losses were not required to satisfy the major purpose of these tests which was to obtain qualitative results for different types of branch connections, and although the reduction of accidental errors by repetition of readings was considered sufficient, there remain many circumstances under which a determination of the true values of these losses is important. In a closer analysis, therefore, the following persistent errors must be considered:

(1) Errors in mean velocity head.
(2) Errors due to neglect of energy head correction factor.
(3) Errors in the determination of the straight pipe loss, including the effect of too short a calming length.

Where the absolute value of the loss which is to be determined is small, lack of control of these factors introduces large errors in the result and is probably responsible for much of the apparent discrepancies in reported experiments. Consider the following two examples:

(1) Figure 15A shows the coefficients of junction loss in the main pipe caused by diversion into the branch in terms of the velocity head in the main pipe. The equation by means of which the coefficients were experimentally determined may be written

$$\lambda = (1 - K_2) + (P_x - P_o) - K_1 K_2 m = 0.12$$

where the symbols and experimental values found for a Reynolds' number of 200,000 and for a discharge ratio, $\frac{Q_s}{Q_a}$, of 0.6 are as follows:

λ = coefficient of junction loss in terms of $\frac{V_a^2}{2g}$

$(P_x - P_o)$ = drop in hydraulic grade as a fraction of $\frac{V_a^2}{2g}$ (= -0.64)

K_1 = coefficient of straight pipe loss in terms of $\frac{V_a^2}{2g}$ (= 0.019)

m = length of pipe in diameters between the branch and the downstream piezometers (= 25)

K_2 = ratio of mean velocity head below the branch to that above (= 0.16)

$0.12 = (1-0.16) - 0.64 - 0.019 \times 0.16 \times 25$

If the difference in true kinetic energy heads be used for the first term instead of the difference of mean velocity heads, the first term becomes, $(1.12 - 1.05 \times 0.16) = 0.95$ and $\lambda = 0.23$ which is $0.11\ h_v$ in excess of that reported on figure 15A. This is a 90 percent increase.

In this example, the energy correction factor upstream was obtained from Pitot tube measurements while that downstream was estimated from data of Nikuradse. It may be seen, however, that in cases where the downstream velocity head is small as compared with that upstream, large errors may be made in both the friction loss and the downstream energy correction factor without changing the result.

(2) Figure 16B shows the coefficients of the junction loss in the branch in terms of the velocity head in the main pipe. Using the same equation as in (1) for $\frac{Q_s}{Q_a} = 0.6$ and a Reynolds' number of 200,000 the terms are:

$$1.7 = (1 - 10.2) + 14.7 - 0.0145 \times 10.2 \times 26$$
$$= -9.2 + 14.7 - 3.8$$

Again if the energy correction factors for velocity distribution are 1.09 and 1.05 respectively, the first term becomes $(1.09 - 1.05 \times 10.2) = -9.6$ or a correction of $0.4\ h_v$. Thus, the true loss is 23 percent less than the nominal loss as tabulated. In contrast to the previous example, this case indicates that the energy correction factor upstream is of small importance in comparison with that downstream. On the other hand, if a persistent error of +10 percent occurs in the straight pipe loss of $3.8\ h_v$, the junction loss would be increased by 23 percent.

H. Errors Due to Changing Straight Pipe Friction

Finally, mention should be made of errors due to change in the straight pipe friction during the tests. Definite evidence exists that beginning in the middle of July, as the water warmed, the friction loss in the 3.49-inch pipe, which was then under test, increased about 30 percent in two months. After cleaning, the loss approached its original value. Along with the increase in pipe roughness, the entrance loss was doubled, a phenomenon similar to that found by Thoma in his tests on bend losses. The increase in roughness apparently was caused by a growth of algae in the pipe walls.

It is evident that if the roughness of a pipe changes under test that measurements of straight pipe losses must be made simultaneously with the measurements of fitting losses if large errors are to be avoided. It is possible that fitting losses are only applicable to pipes of similar roughness.

I. Conclusions on Errors

From a study of the errors in these tests it is thought that the following conclusions are warranted:

(1) Where only qualitative conclusions are to be drawn from hydraulic tests, extremely large errors may be introduced if the roughness of the pipe changes under test.

(2) The size of the accidental error for the energy grade at a given ring for a single run was largely controlled by the mechanical error in the given piezometer. If in future tests, the piezometers are placed in a spiral around the pipe over a length of about 3 diameters, errors due to pressure waves will be eliminated. Where fluctuations exist similar to those found in these tests, if piezometers are read to the nearest 0.01 of a foot and the number of readings reduced by one-half, there will be no appreciable change in the accuracy.

(3) By moving piezometers from a location near a fitting to the location where the straight pipe loss is determined, a considerable increase in the accuracy of an experiment is possible.

(4) Where quantitative losses are sought, errors in the energy correction factor and in the straight pipe losses will probably control the accuracy. The energy correction factor is of particular importance where anticipated losses are small and this is probably responsible for many of the discrepancies in existing data.

V. INTAKE TOWER AND PENSTOCK ASSEMBLY

1. INTAKE TOWER HYDRAULIC MODEL

A. Introduction

The water for irrigation and power purposes will be controlled by the four intake towers adjacent to the upstream face of the Boulder Dam[9]. Each of the intake towers is provided with two

[9] "Hydraulic Valves and Gates for Boulder Dam" by P. A. Kinzie, Mechanical Engineering, vol. 56, July 1934, pp. 387-414.

cylinder gates, 32 feet in diameter, the lower in the base of the tower at elevation 895 and the upper at elevation 1045. These gates will serve to control the flow through the 30-foot steel penstock headers that extend from the base of each tower to the turbines in the power plant and to either the canyon-wall outlet works or the tunnel-plug outlet works.

The closure of the upper and lower gates in any one intake tower will permit the unwatering of the steel penstock and all its appurtenances for the purpose of inspection and maintenance.

To determine the hydraulic action of these towers and control gates under the various conditions of discharge to which they may be subjected, a model of one of the towers on a scale of 1 to 64 was constructed (plate VI) and tested in the hydraulic laboratory of the Colorado Agricultural Experiment Station, Fort Collins, Colorado.

By means of this model the distribution of the flow through the two sets of gate openings was determined, methods of reducing the entrance losses studied, and the necessity of air vents below the lower cylinder gate investigated.

B. The Apparatus

The model consisted of a complete assembly of the upper Arizona 30-foot penstock header, intake tower, and branch penstocks leading to the turbines and needle valves (figs. 30 to 33, inclusive).

The intake tower and the surrounding topography was located in the 10.5- by 10.25-foot tank, as shown on figure 2. The inner portion of the tower consisted of galvanized sheet-metal cylindrical shells, accurately built and carefully soldered together with butt

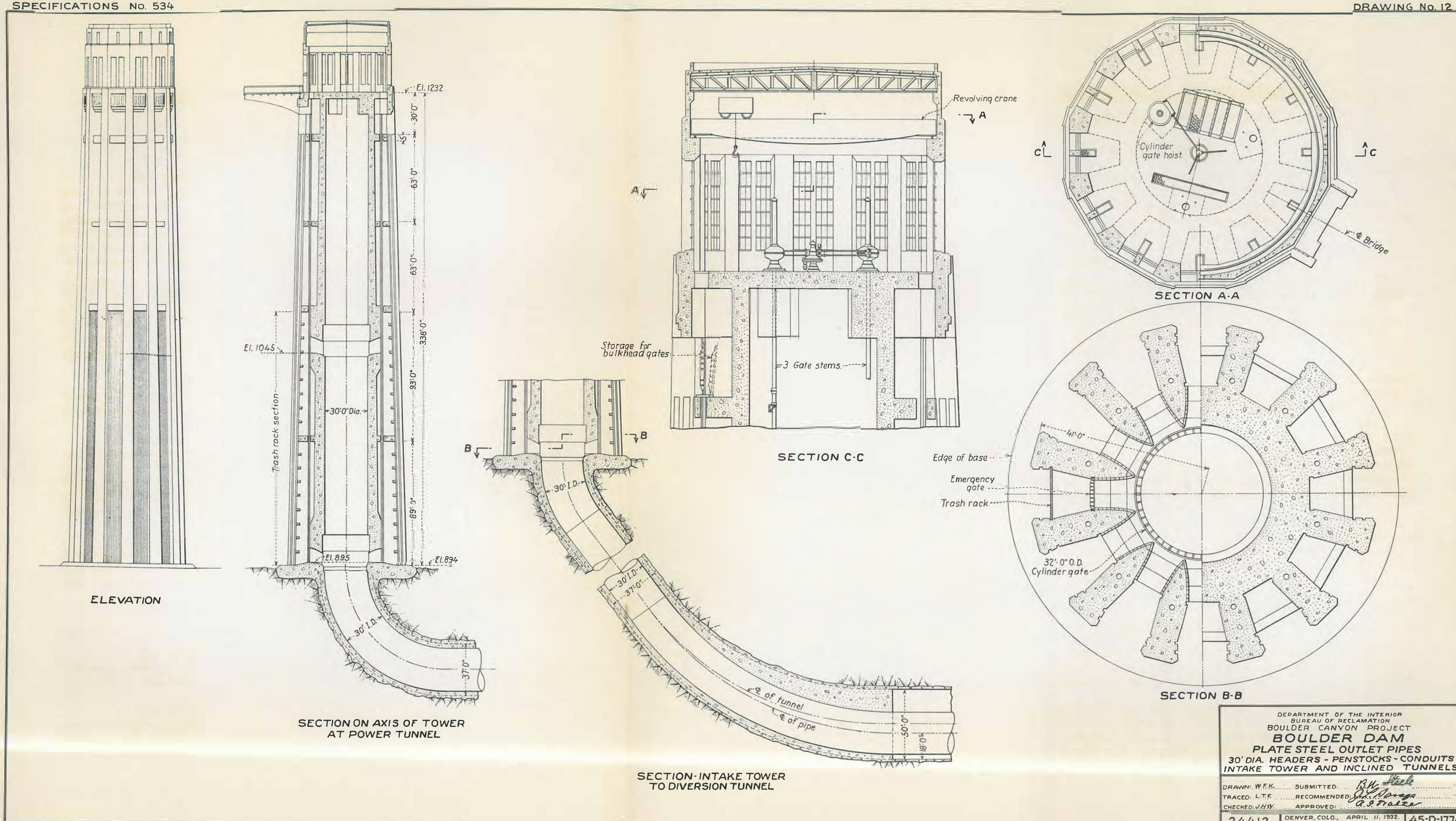
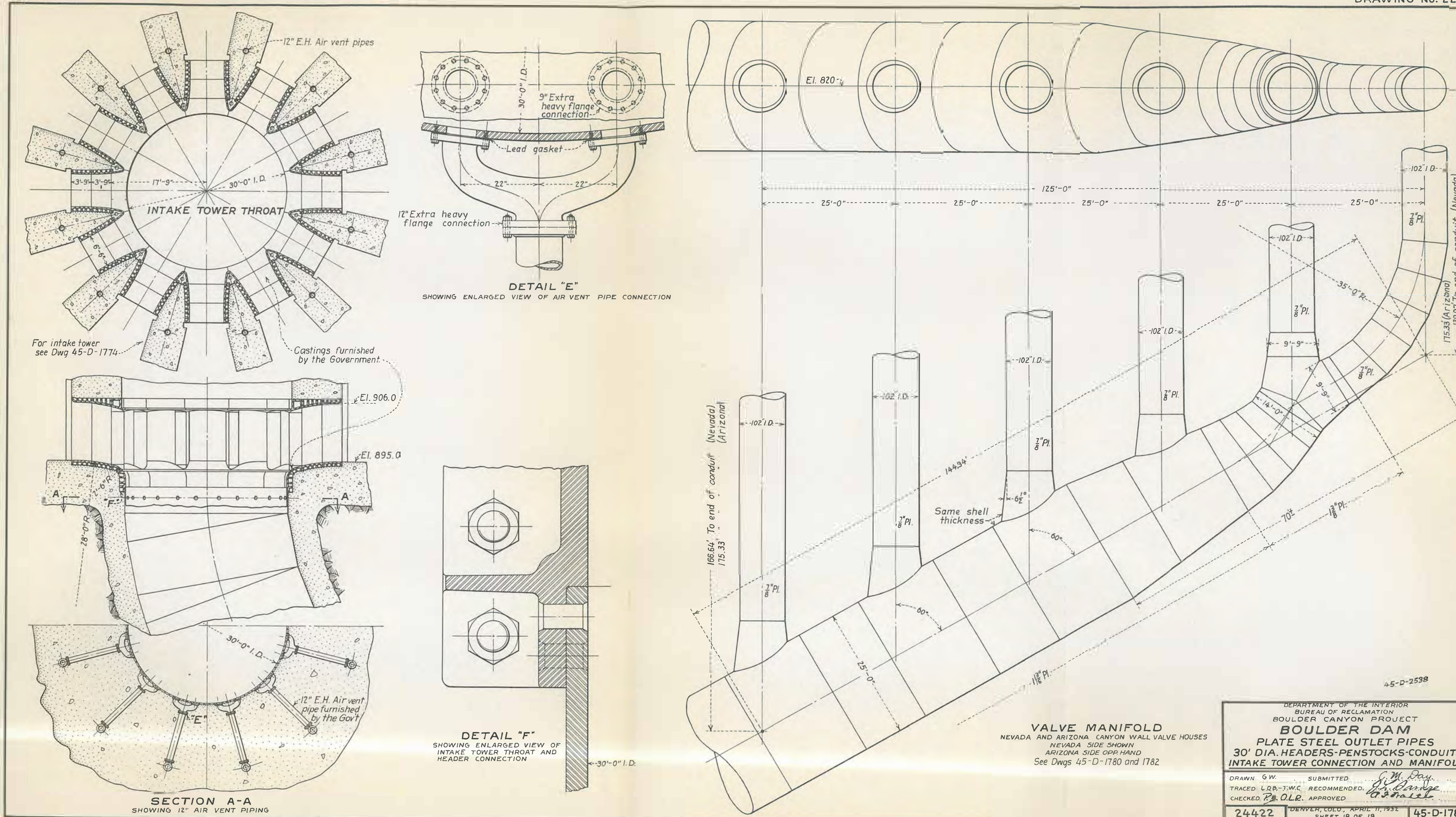
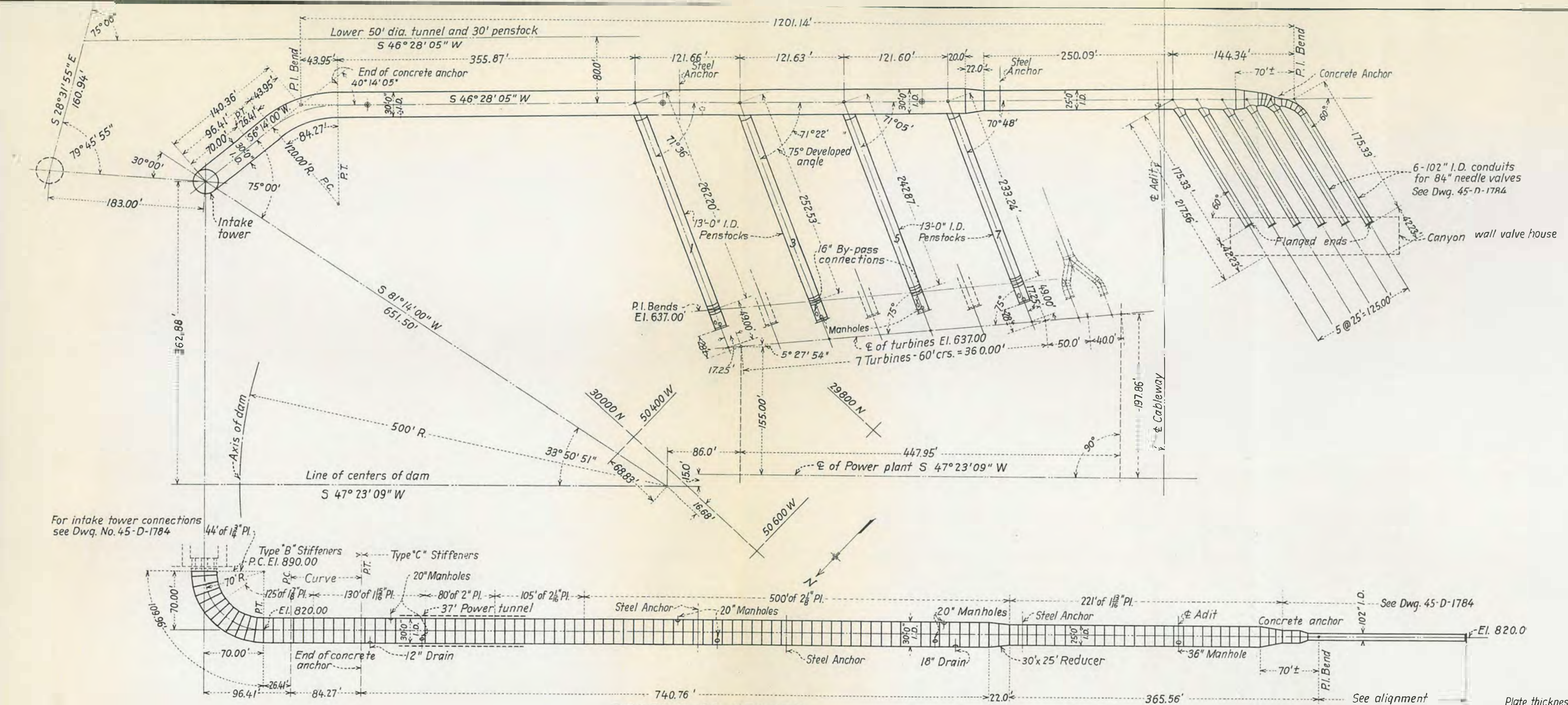

ELEVATION
SECTION ON AXIS OF TOWER AT POWER TUNNEL
El. 1232
30'-0"
5'
63'-0"
63'-0"
338'-0"
93'-0"
89'-0"
El. 1045
Trash rack section
30'-0" Dia.
El. 895
El. 894
30' I.D.
37'-0"
SECTION-INTAKE TOWER TO DIVERSION TUNNEL
30' I.D.
37'-0"
℄ of tunnel
℄ of pipe
50'-0"
18'-0"
SECTION C-C
Revolving crane
Storage for bulkhead gates
3 Gate stems
SECTION A-A
Cylinder gate hoist
℄ Bridge
SECTION B-B
41'-0"
Edge of base
Emergency gate
Trash rack
32'-0" O.D. Cylinder gate
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM
PLATE STEEL OUTLET PIPES
30' DIA. HEADERS - PENSTOCKS - CONDUITS
INTAKE TOWER AND INCLINED TUNNELS
DRAWN: W.F.K. SUBMITTED: B.W. Steele
TRACED: L.T.F. RECOMMENDED:
CHECKED: J.H.W. APPROVED:
24412
DENVER, COLO., APRIL 11, 1932. SHEET 8 OF 19
45-D-1774

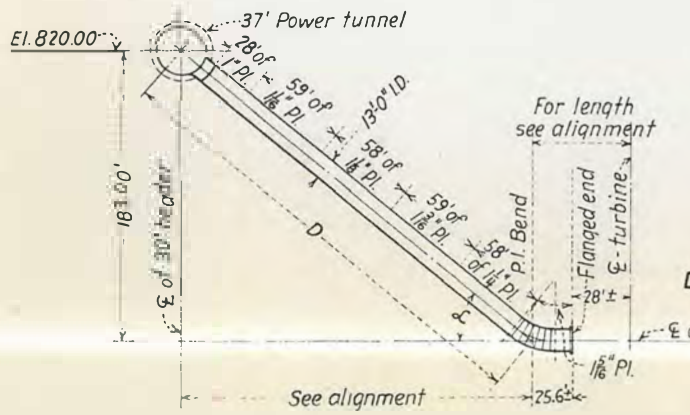
FIG. 31

DRAWING No. 22

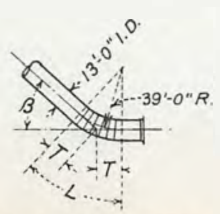




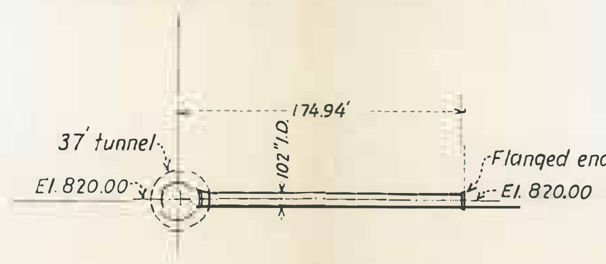
PROFILE OF HEADER



PROFILE OF 4 POWER PENSTOCKS



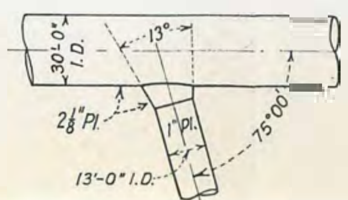
DEVELOPED BEND



PROFILE OF NEEDLE VALVE CONDUITS

PENSTOCK BRANCHES

Penstock No.	1	3	5	7
Horiz. angle	1°09	0°54'	0°38'	0°21'
Vert. angle α	34°55'	35°56'	37°00'	38°07'
Dev. angle β	34°56'	35°57'	37°00'	38°07'
D	319.74'	311.87'	304.10'	296.46'
T	12.27'	12.65'	13.05'	13.47'
L	23.78'	24.47'	25.19'	25.95'



DEVELOPED PENSTOCK CONNECTION

NOTE

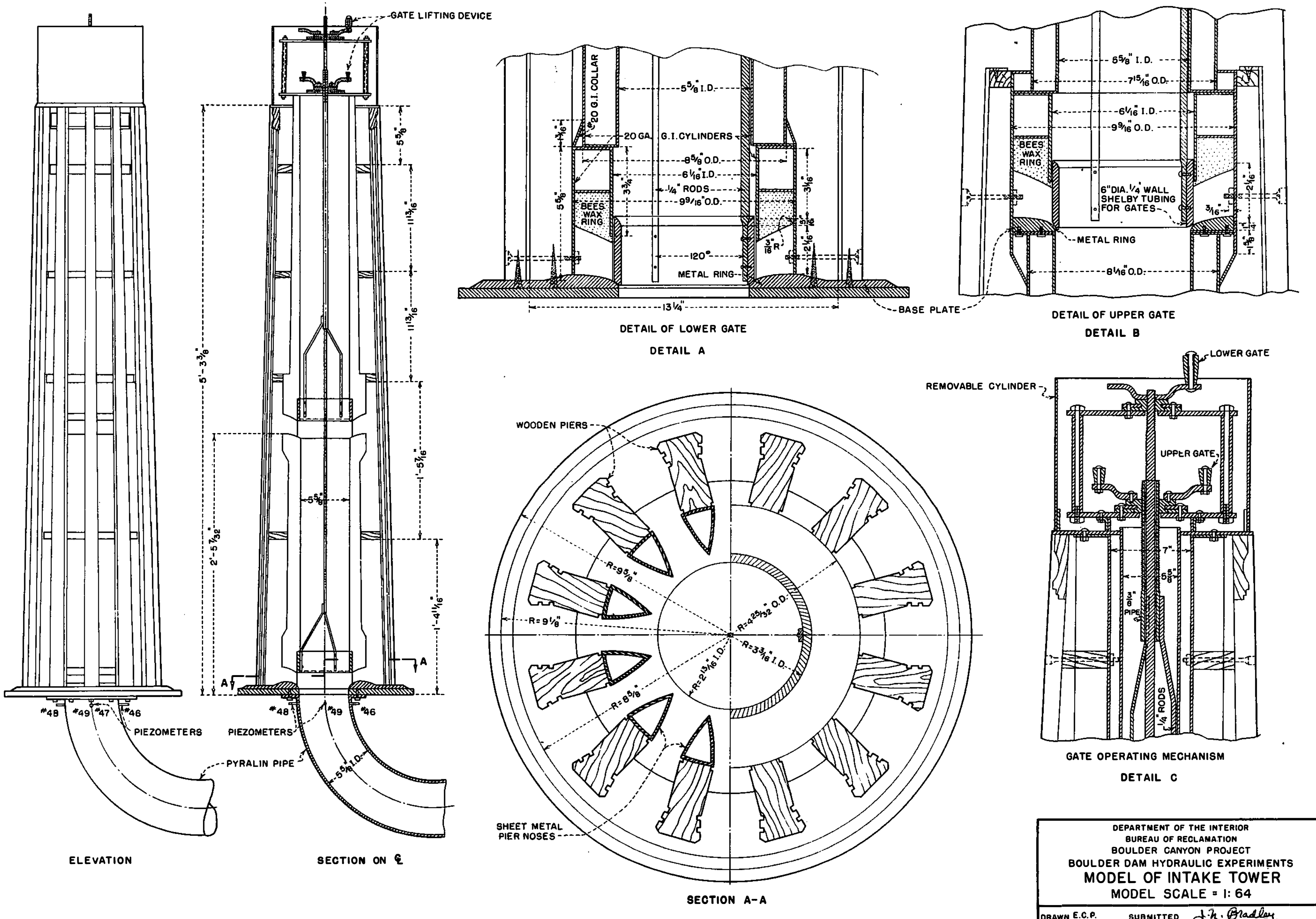
Plate thicknesses are minimum; changes in thickness may be made at the nearest joint See paragraph 55, spec. No. 534 and dwg. No. 45-D-1766.

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM
PLATE STEEL OUTLET PIPES
30' DIA. HEADERS - PENSTOCKS - CONDUITS
IN UPPER ARIZONA TUNNEL

DRAWN: C.M.B.-J.P. SUBMITTED: C. W. Day
TRACED: C.A.M. RECOMMENDED: J. L. Savage
CHECKED: T.D. O.L.R. APPROVED: R. F. Walter

Rev. 6-10-32

24418 DENVER, COLO., APRIL 11, 1932. SHEET 14 OF 19 45-D-178



DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS

MODEL OF INTAKE TOWER

MODEL SCALE = 1:64

DRAWN E.C.P.	SUBMITTED J. N. Bradley
TRACED M.C.G.-O.W.S.	RECOMMENDED J. E. Warnock
CHECKED J.O.M.	APPROVED J. L. Savage
DENVER, COLO., NOV. 25, 1935	45-D-9723

FIGURE 33

joints, forming a true representation of the prototype in detail and dimension. Two independently operated cylinder gates were installed in the tower, similar to those in the prototype. The gates were made of 12-gage seamless steel tubing with the lower outside edges beveled sufficiently to insure a close fit with the base plates and form a satisfactory water seal (fig. 33A and B). The gates were raised and lowered by three small rods attached to each gate at the third points on the circumference. The rods from the lower gate converged into one main rod which extended up the center of the tower to the hoisting apparatus. The upper end of the rod was threaded to permit raising and lowering the gate by means of a small crank in the upper part of the tower (fig. 33C). The rods to the upper gate were connected in a similar manner to a sleeve over the main rod leading to the lower gate. This sleeve was also threaded at the upper end, and was actuated by a second crank located in the upper part of the tower.

The lower portions of the gate entrances were machined metal plates, the lower one forming a base plate as well. The upper portions of the entrances were molded using a mixture of beeswax and paraffin (fig. 33A and B). With the exception of the piers and pier spacers, which were of wood, and the gate entrances, which were of wax and paraffin, the tower was constructed entirely of metal. The piers and spacer blocks were protected with two coats of aluminum paint to alleviate as much as possible any swelling of those parts of the model. The topography around the tower was constructed from contour maps of the canyon wall at the dam site, and consisted of a lean mixture of cinder concrete. Trash racks (plate VIB) were installed on the tower during a portion of the tests. These were built to scale, but due to their miniature size, there is some doubt as to the advisability of relying too closely upon the results obtained with them. Small edges and burrs which were practically impossible to remove, surface tension, and traces of grease and oil on the racks all probably had their effect upon the results. The racks (182 feet in length, prototype) were constructed of very thin strips of sheet metal set with the thickness of the metal normal to the direction of flow and held in place by cross pieces to which the strips were soldered. Upon completion of the various parts, the tower was assembled on its base plate and bolted to a corresponding plate located in the floor of the model tank to which the 30-foot diameter penstock header was connected. In making alterations in the model, it was only necessary to unbolt the upper plate from the lower and remove the intake tower as a unit.

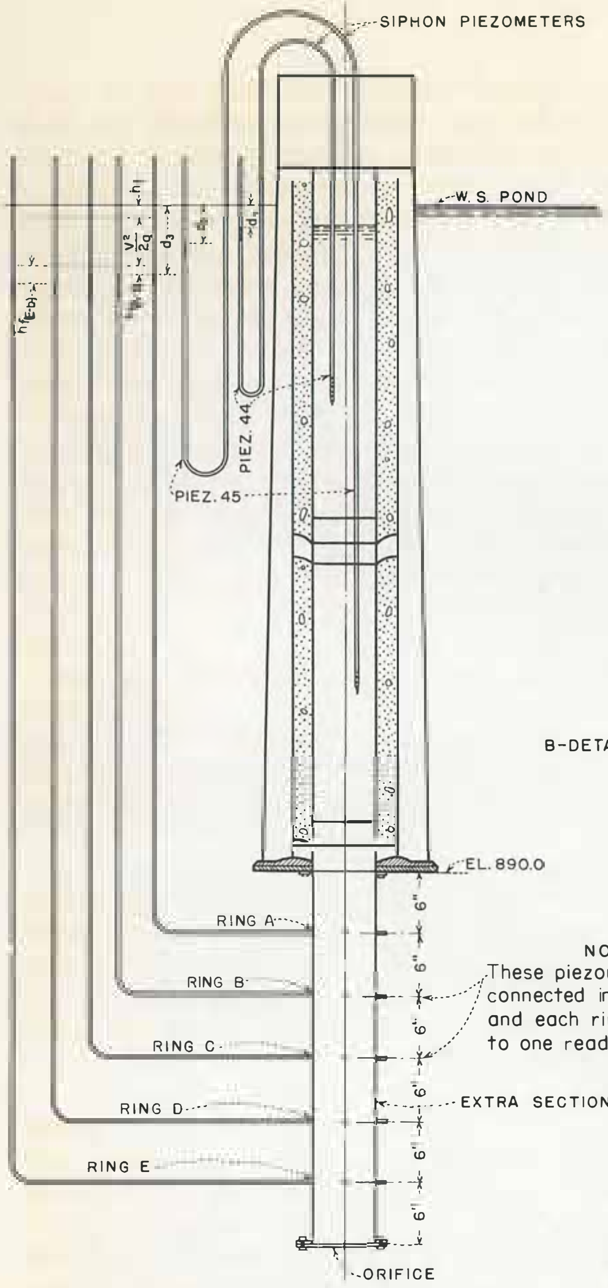
C. Method of Analysis of Losses in Tower

To analyze the losses in the intake tower, a ring of piezometers was installed in the base at elevation 890.0 (fig. 33). It was anticipated, and later confirmed, that the vertical 90-degree bend in the penstock header immediately below the tower would produce an unbalanced effect upon the flow in this portion of the model, causing a variation in pressure at the four piezometers. As the losses to be measured in the model were small, averaging the readings of this ring of piezometers would have been inaccurate. Furthermore, the computation of the velocity of the water leaving the tower would have been in error due to the unequal velocity distribution in this region. To avoid complicating the loss computations with these sources of error, the penstock header was disconnected from the base of the tower and a straight section of pipe three feet in length connected in its place (fig. 34A). The lower end of this auxiliary section was fitted with a flange to which orifices of different size were fastened to regulate the discharge through the tower. Five rings of piezometers designated as A to E, inclusive, (fig. 34A) were installed at 6-inch intervals along the auxiliary section of pipe. Each ring consisted of four piezometers spaced 90 degrees apart, connected with rubber hose to a single glass manometer tube. An average value for the pressure at the base of the tower at elevation 890.0 was computed from these five rings of piezometers by adding the pipe friction computed from each ring to elevation 890.0, corresponding to each observed gage reading. As this section of pipe was similar in construction to those used in the penstock experiments, the pipe friction was obtained from the curves plotted on figure 10.

Observed pressures at the five piezometer rings below the tower are plotted on figure 35 for a few runs with both gates open. Rings A and E are consistent with the other three, ring A appearing not to be effected by the lower gate, and ring E not being influenced by the orifice. The data for each run plotted as a straight line, thus readings from all five rings were used in the loss computations.

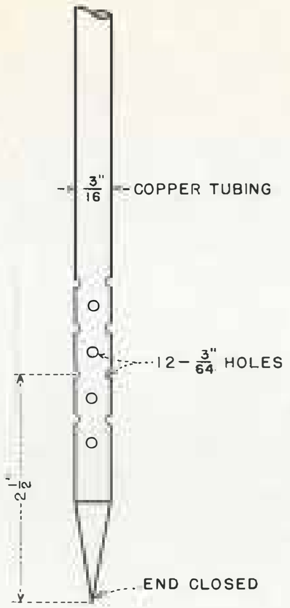
Two siphon piezometers were installed in the center of the tower (fig. 34A), one midway between the upper and lower gates, and the other about a foot above the upper gate. Each consisted of a piece of 3/16-inch copper tubing closed at the lower end by drawing it to a point as shown on figure 34B. Twelve 3/64-inch holes were drilled in each tube, perpendicular to the walls, and the two piezometers were suspended vertically in the center of the tower.

In all tests, one or both gates were always fully open, as they are intended to be so operated on the prototype. These gates are provided for unwatering purposes and not as regulators. The total discharge was measured over the laboratory 90-degree V-notch weir, the magnitude being determined by the size of the orifice below the tower and the elevation of the water surface in the model. The model discharge ranged from 0.15 to 0.90 second-feet, which corresponds to approximately 5,000 and 30,000 second-feet, respectively, in the prototype.

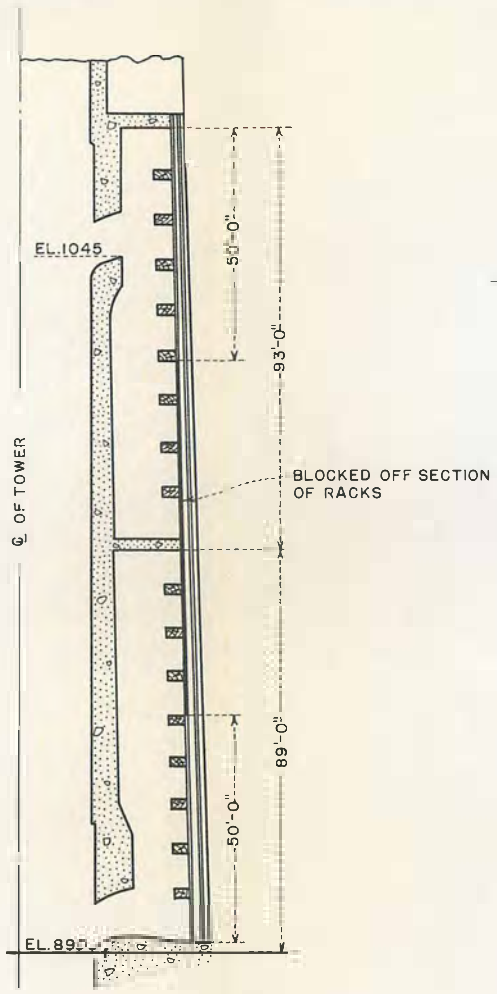


NOTE
These piezometers are connected in rings of four and each ring is attached to one reading gloss.

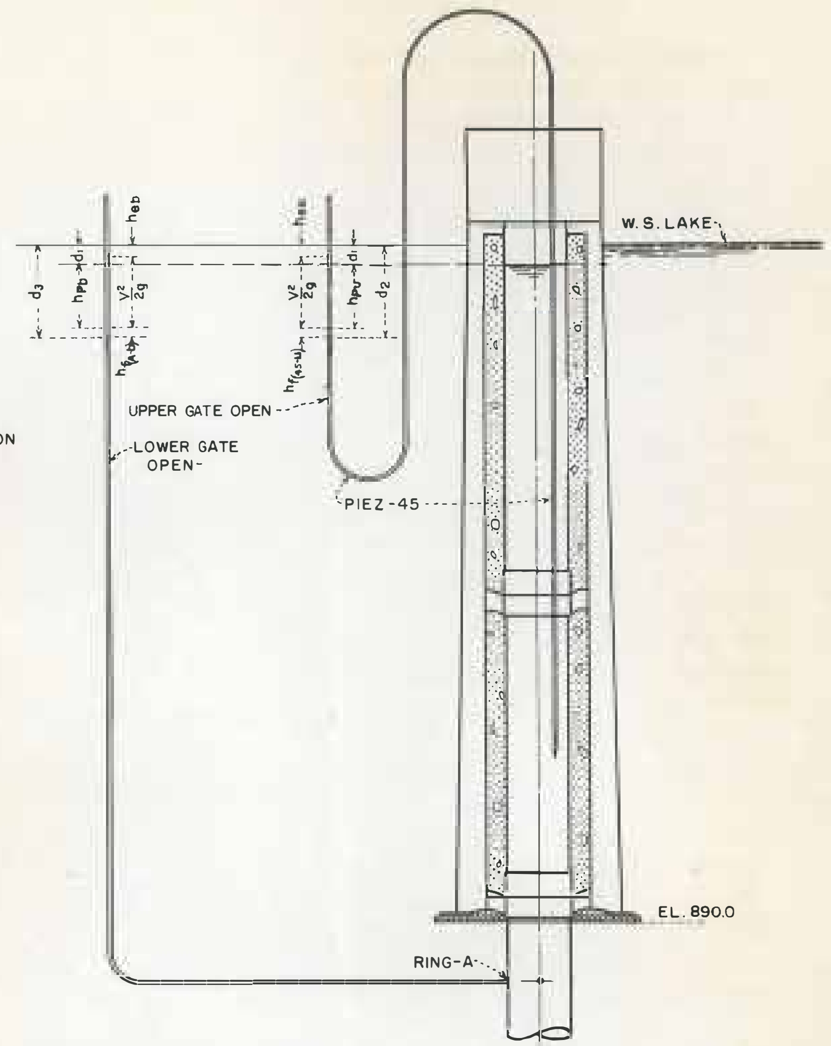
A-DETAIL OF PIEZOMETER CONNECTIONS FOR INVESTIGATING TOWER



B-DETAIL OF END OF SIPHON PIEZOMETER



C-DETAIL OF BLOCKED OFF TRASH RACK
LENGTH OF OPEN RACKS = 50 FEET FOR EACH RACK
(PROTOTYPE)

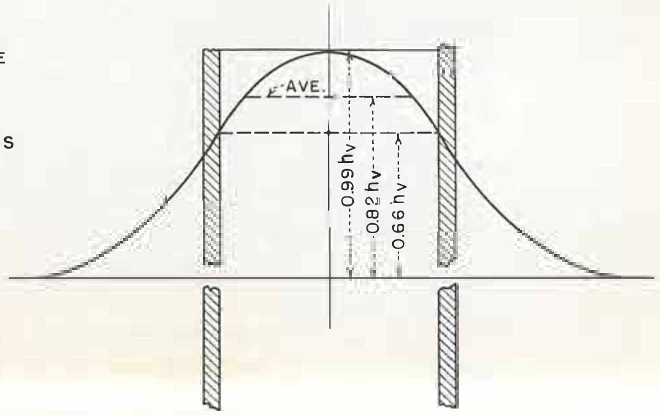


D-DIAGRAMMATIC SKETCH OF ENTRANCE LOSS, PRESSURE LOSS, TOTAL LOSS, ETC. WHEN EITHER THE UPPER OR LOWER GATE IS OPEN

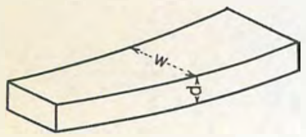
EXPLANATION

h_{eu} = Entrance loss at upper gate.
h_{eb} = Entrance loss at lower gate.
d_2 = Difference in elev. between w.s. in pond and piez. 45
d_3 = Difference in elev. between w.s. in pond and gage for ring A
d_1 = Difference in elev. between w.s. in pond and w.s. inside tower.

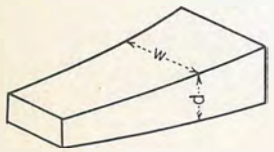
$h_{f(45-u)}$ = Pipe friction from piez. 45 to upper gate.
$h_{f(A-b)}$ = Pipe friction from ring A to lower gate.
h_{pu} = Head required to change direction of flow at upper gate.
h_{pb} = Head required to change direction of flow at lower gate.



E-PRESSURE DOME NECESSARY TO CHANGE DIRECTION OF FLOW



F-STREAM TUBE
ONE VARIABLE

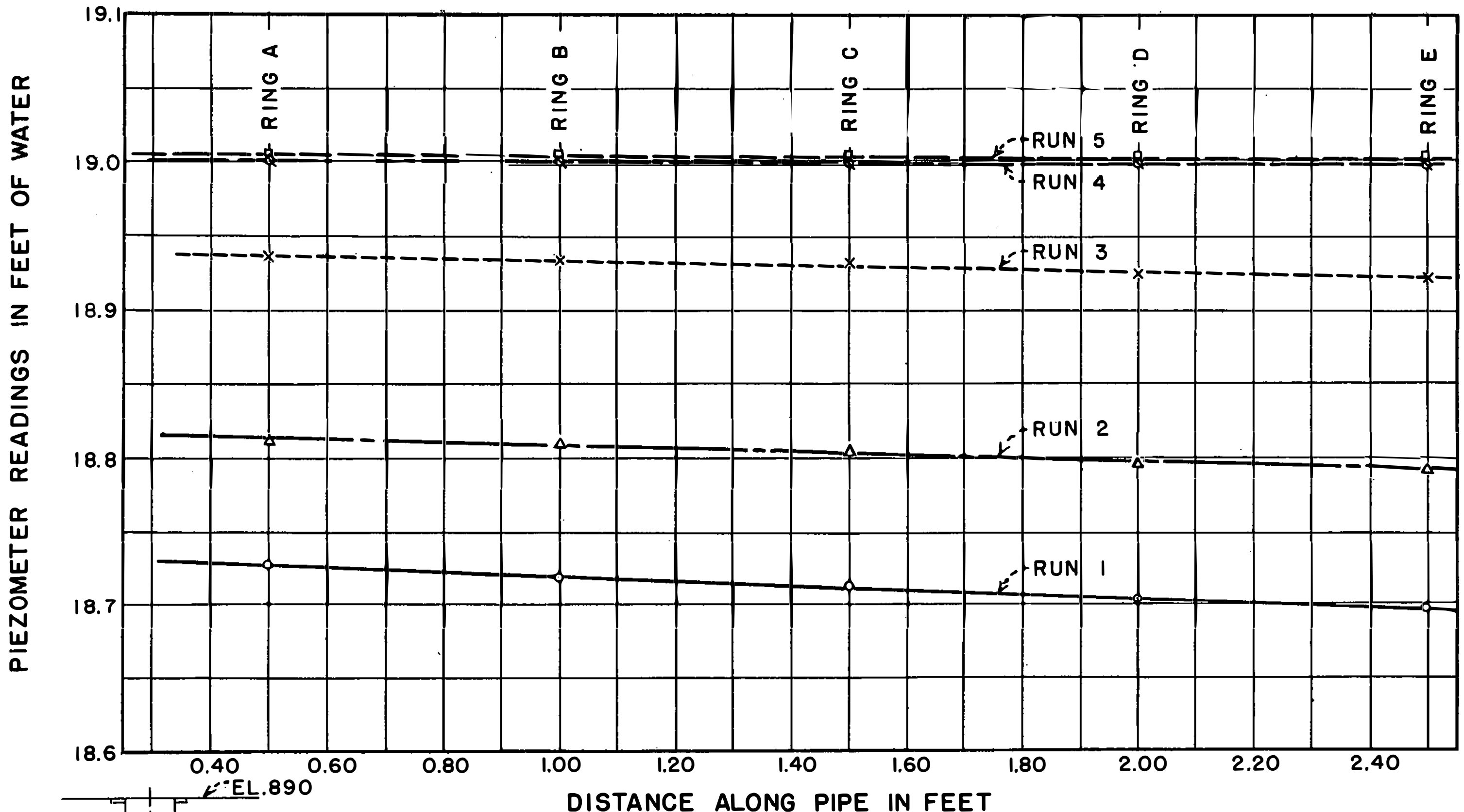


G-STREAM TUBE
TWO VARIABLES

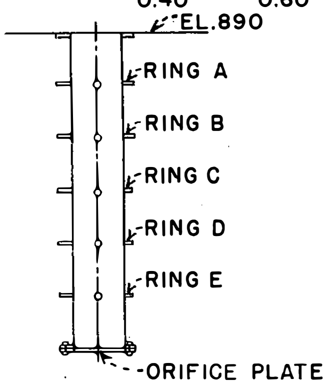
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS

INTAKE TOWER STUDIES
MISCELLANEOUS DETAILS

DRAWN. H.W.B. SUBMITTED J. N. Bradley
TRACED. O.R.S. RECOMMENDED
CHECKED. J.D.M. APPROVED
DENVER, COLO., DEC 16, 1935 45-D-9724



PRESSURE VARIATIONS FOR PIEZOMETER RINGS A, B, C, D AND E
TEST 13 BOTH GATES OPEN



DETAIL OF PIEZOMETER CONNECTIONS
FOR INVESTIGATING TOWER LOSSES

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS
INTAKE TOWER STUDIES
COMPARISON OF PIEZOMETER
PRESSURES BELOW TOWER

DRAWN. H.W.B. J.D.M. SUBMITTED. J. N. Bradley
TRACED. E.E.B. RECOMMENDED. J. E. Warnock
CHECKED. J.N.B. APPROVED. J. L. Savage

DENVER, COLO., JAN. 16, 1936. 45-D-9765

FIGURE 35

D. Total Losses Through Tower

Tests were made to determine losses through the tower with and without trash racks, with either the upper, the lower, or both gates open, and with a range of discharges. For all conditions, the total loss through the tower (fig. 34A) was computed using the expression

$$h_t = d_3 - h_{f(A-b)} - \frac{V^2}{2g}$$

where h_t = total loss through the tower to elevation 890

d_3 = difference in elevation between the water surface in the reservoir and the elevation of the water surface in the manometers connected to piezometer rings A to E, inclusive

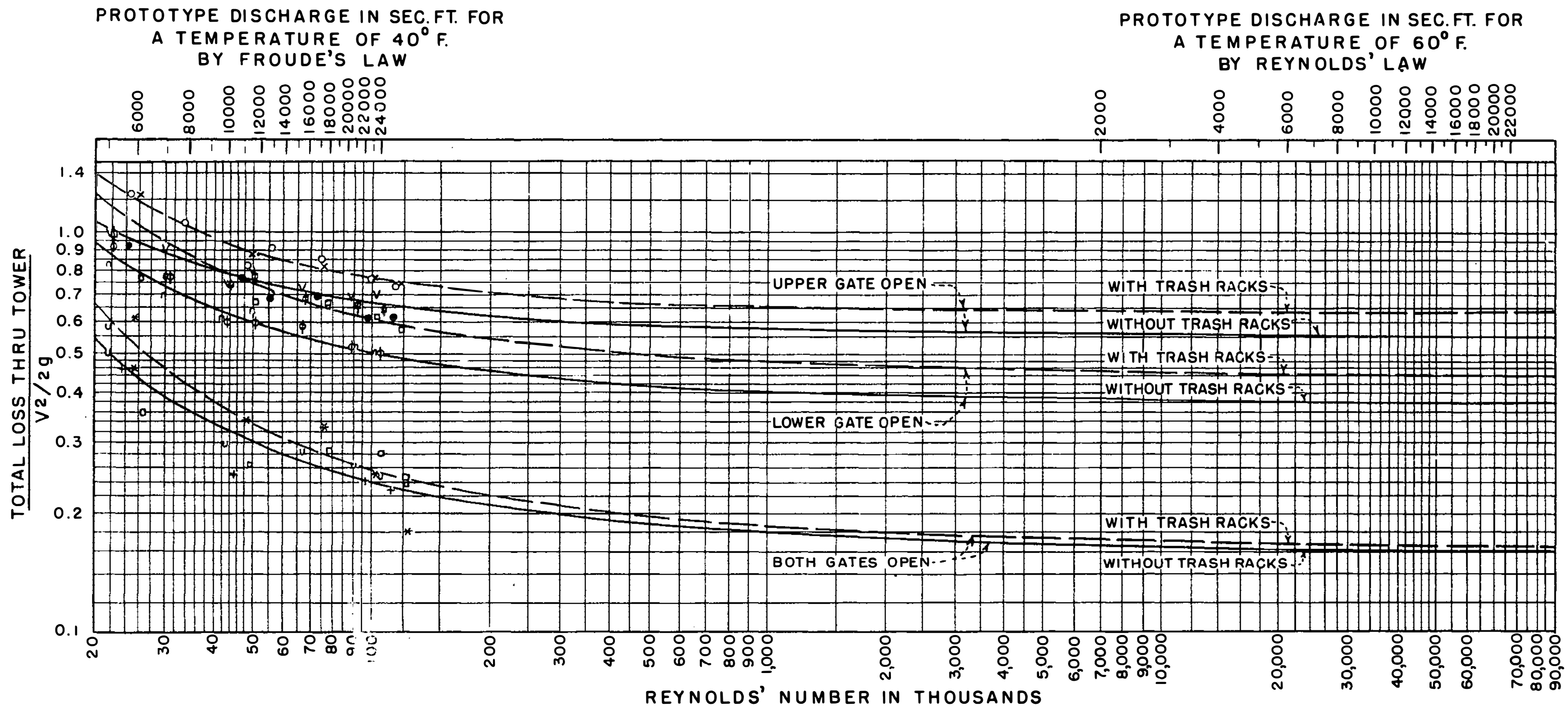
$\frac{V^2}{2g}$ = velocity head at elevation 890

$h_{f(A-b)}$ = pipe friction from elevation 890 to piezometers below.

The pipe friction was computed using the combined observations at the piezometer rings below the tower. The results were plotted logarithmically on figure 36. The total loss through the tower, expressed in terms of the velocity head in the tower at elevation 890.0, is plotted with respect to Reynolds' number rather than the model discharge, as in the former, the temperature of the water could be included while in the latter it was impossible to consider this factor. During the tests, the temperature of the water varied from 33 to 50 degrees Fahrenheit. The velocity and diameter of the pipe used to compute Reynolds' number was in all cases that existing in the tower at elevation 890.0.

It was desired to express the model losses with respect to the prototype structure as this was one of the ultimate objectives in performing the model experiments. It has been customary in the past to convert from one to the other by means of Froude's law, but this is not a valid method of conversion when small models are used and viscous effects are appreciable. In performing this transfer, it is assumed that the losses expressed in terms of velocity head are the same in model and prototype for the same value of Froude's number. With viscous effects present in the model, it can readily be seen that this assumption is not entirely correct.

Reynolds' law, on the other hand, considers viscosity in prototype and model, but is independent of the force of gravity and



EXPLANATION

WITH TRASH RACKS	WITHOUT TRASH RACKS
UPPER GATE OPEN	UPPER GATE OPEN
TEST 8 - X 2-50' RACKS	TEST 14 - ф
" 11 - O 1-182' RACK	" 17 - V
LOWER GATE OPEN	LOWER GATE OPEN
TEST 9 - Δ 2-50' RACKS	TEST 15 - ϕ
" 12 - o 1-182' RACK	" 18 - ∩
BOTH GATES OPEN	BOTH GATES OPEN
TEST 7 □ 2-50' RACKS	TEST 13 - +
" 10 - ✳ 1-182' RACK	" 16 - U

TOTAL LOSSES THRU TOWER TO ELEV. 890.0

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TOTAL LOSSES THRU TOWER WITH AND WITHOUT TRASH RACKS INSTALLED

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DENVER, COLO., JAN. 22, 1936 45-D-9775

FIGURE 36

consequently centrifugal effects. If the model of the intake tower were exactly similar to the prototype in dimension and roughness for all heads, and centrifugal effects were exactly similar, it would be possible to extend the model curves as shown on figure 36 to prototype values of Reynolds' number. This would be a similar procedure to plotting the friction factor, f, for closed conduits against Reynolds' number (fig. 10). In the latter case, all straight circular pipes are similar except for roughness, and Reynolds' number offers the correct model-to-prototype conversion. In the case of the intake towers, dissimilar centrifugal effects resulting from geometric dissimilarity also enter into the problem so that the Reynolds' number interpretation is not directly applicable.

Up to the present time, little success has been attained in expressing the relationship of all physical factors in a manner which would permit accurate extrapolation. At the present time, therefore, to correctly interpret prototype losses from models, it is necessary to resort to either of two methods, (1) to build one model large enough to reduce viscous effects to a negligible quantity, or (2) to construct three or more smaller models to different scales. In the first, the transformation could be made directly by Froude's law. In the second, the data from three or more models could be plotted with respect to Reynolds' number. There would result three or more sets of curves (one set for each model) on the graph instead of continuous curves as shown on figure 36. Each would resemble the set preceding it, but would be flatter and located lower on the graph. Lines representing similar heads could be drawn through the three or more sets of curves and extrapolated to include the desired prototype range.

As only one small model was used for the intake tower experiments, the model-to-prototype conversion, for lack of a better means, was made for this and the following experiments by the use of Froude's law. Due to viscous effects in the model, the corresponding actual losses in the prototype will undoubtedly be smaller than those indicated by the graphs. The actual amount of deviation can only be approximated by plotting the data by the two methods shown on figure 36 and interpolating between the results. It can be stated with assurance that the actual prototype losses will not exceed those obtained on the model when the conversion is made according to Froude's law.

E. Trash-rack Losses

The loss in head through the trash racks, with the different gate conditions, is represented by the differences between the three pairs of curves shown on figure 36. These differences are plotted separately on figure 37 and show the trash-rack losses directly for the various conditions of flow.

It is suggested that only reasonable reliance be placed upon the results of the trash-rack tests. As stated previously, the racks were constructed to scale and exhibited a commendable piece of workmanship, but due to their miniature size, small imperfections, surface tension due to natural causes as well as that due to traces of grease on the racks or oil in the water probably produced a combination of effects upon the results. Another difficulty was that it was physically impossible to construct the trash-rack bars in the model with a degree of roughness similar to the prototype.

Two lengths of trash rack were used in the above tests, (1) a rack 182 feet in length extending continuously over both gates, and (2) two racks 50 feet in length, each of which protected one gate. A detailed inspection of the points from which the lines were drawn on figure 36 shows that the total losses through the tower for the two lengths of trash rack were quite analogous for the same conditions of flow, which indicates that above a certain length, factors other than the length of the racks determine the losses through them.

During the tests, the only place where trash collected on the racks was directly in front of the gates. The remainder of the racks were continuously clean. Trash racks are further discussed in this report under the section "Intake Tower Electric-Analogy Studies."

F. Gate Entrance Losses

The entrance loss through the upper gate is the difference between the energy head at the water surface of the reservoir and at the bottom of the upper gate. The trash racks were removed before the individual losses in the gate tower were measured.

The entrance loss through the upper gate (fig. 34D) was computed using the equation

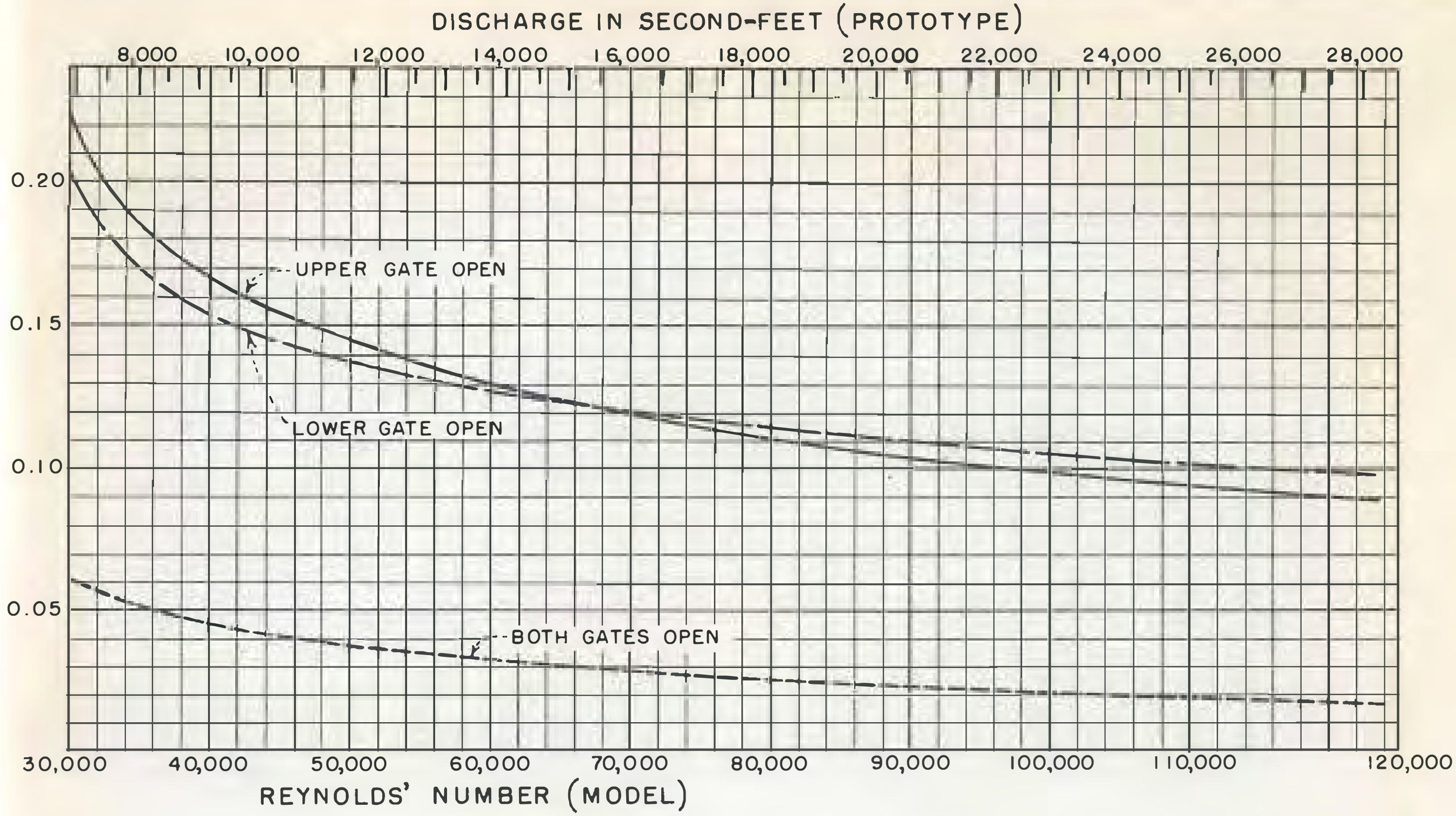
$$h_{eu} = d_2 - h_{f(45-u)} - \frac{V^2}{2g}$$

where h_{eu} = entrance loss through the upper gate

d_2 = difference in elevation between the water surface in the reservoir and the water surface in piezometer 45

$h_{f(45-u)}$ = pipe friction from piezometer 45 to the bottom of the upper gate

$\frac{V^2}{2g}$ = velocity head at piezometer 45.



TRASH RACK LOSSES

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INTAKE TOWER STUDIES

LOSSES THRU TRASH RACK

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DENVER, COLO., JAN. 20, 1936 | 45-D-9761

FIGURE 37

The entrance loss through the lower gate is the difference between the energy head at the water surface of the reservoir and at the bottom of the lower gate. This loss (fig. 34D) was computed using the equation

$$h_{eb} = d_3 - h_{f(A-b)} - \frac{V^2}{2g}$$

where h_{eb} = entrance loss through the lower gate

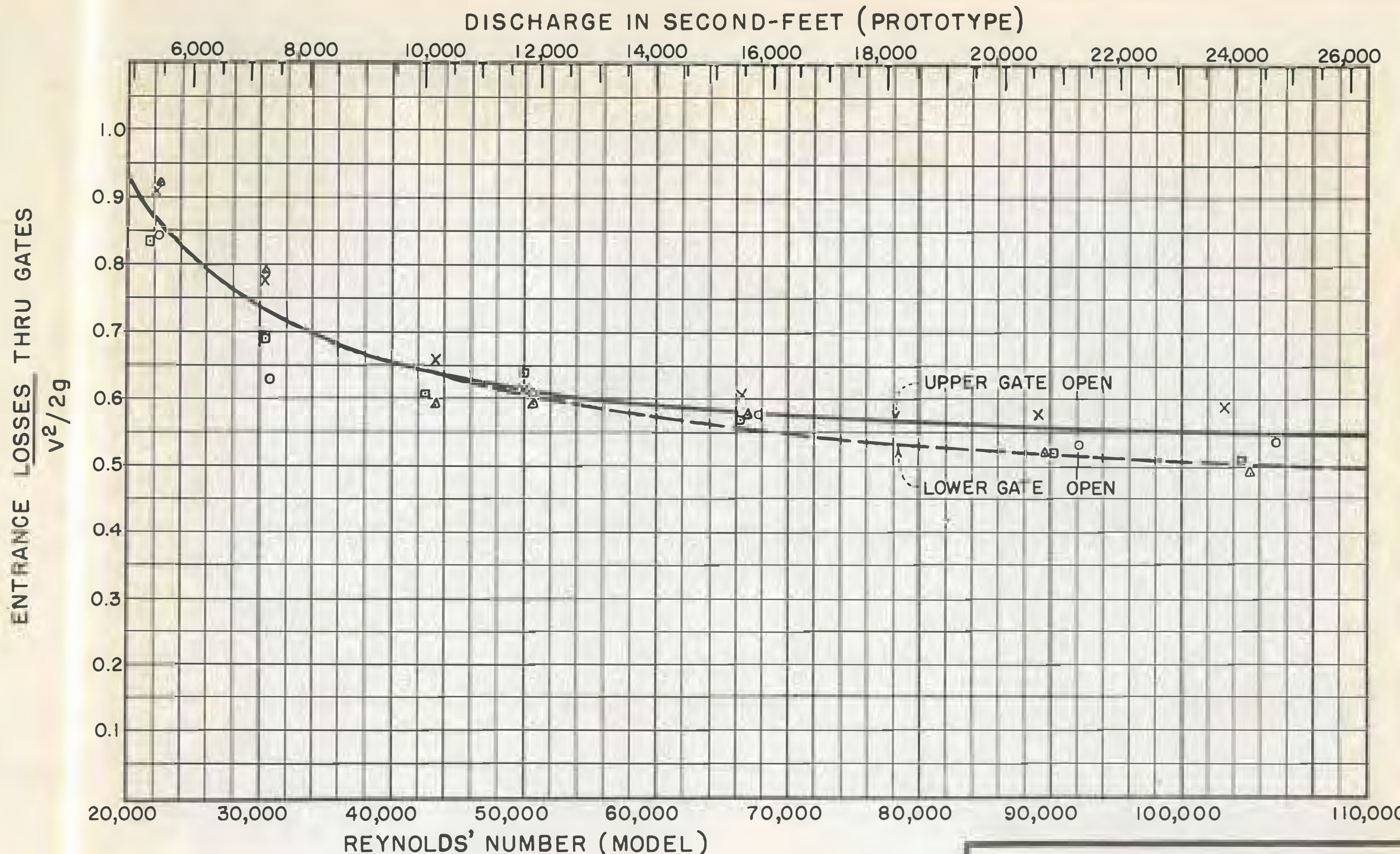
d_3 = difference in elevation between the water surface in the reservoir and the water surfaces in piezometer rings A to E, inclusive

$h_{f(A-b)}$ = pipe friction from elevation 890 to the piezometers below

$\frac{V^2}{2g}$ = velocity head computed at elevation 890.

The curves on figure 38 show the entrance losses computed for the two conditions of flow. These losses are expressed in terms of the velocity head computed in the tower at elevation 890.0, and are plotted with respect to Reynolds' number for the model computed at the same point. An additional scale has been superimposed on this graph so that the entrance losses can also be expressed in terms of the prototype discharge computed according to Froude's law. The entrance conditions to the lower gate are slightly superior to those at the upper gate. It is evident from the preceding explanation that the entrance loss curve for the lower gate, shown on figure 38, is the same as that for the total loss curve without trash racks, shown on figure 36, as in this case, the entrance loss is the total loss. In the case of the upper gate, the entrance loss is smaller than the total loss through the tower as it is necessary to deduct pipe friction.

In this analysis, it was necessary to make two assumptions, (1 that the velocity used in computing the velocity head in the tower was computed in each case by dividing the discharge by the cross-sectional area of the tower. It is known that the velocity distribution in the tower is of a complex nature and that the coefficient of velocity is somewhat above unity. This is probably the greatest source of error in these computations, and it is estimated that the difference between the mean and true velocity head does not exceed 8 percent. (2). It was necessary to compute the pipe friction in the tower in several instances and this was done by reference to the curves on figure 10. The pipes from which these curves were obtained were similar to the inner portion of the tower and any error that might



TRASH RACKS REMOVED

EXPLANATION

TEST 14 ○ } UPPER GATE OPEN
" 17 ×

TEST 15 △ } LOWER GATE OPEN
" 18 □

ENTRANCE LOSSES FOR ONE GATE OPEN

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BOULDER DAM HYDRAULIC EXPERIMENTS
INTAKE TOWER STUDIES
ENTRANCE LOSSES THRU GATES

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CHECKED J.D.M. APPROVED J. L. Savage

DENVER, COLO., JAN. 17, 1936 | 45-D-9762

FIGURE 38

arise from this source is small compared to the total losses in the tower, as in no case did the length of pipe considered in the friction computations exceed 2.5 feet or 5.3 diameters. It is well to mention again that the prototype losses as shown on the graphs are only approximate.

G. Head Required to Change Direction of Flow

As water enters the gates in nearly a horizontal direction, a definite force is required to produce a vertical acceleration. By measuring the pressures directly above and below each gate, it was possible to compute the magnitude of the force acting within the tower. It was originally intended to obtain the hydraulic losses through the tower by observing the difference in elevation of the water surface outside and within the tower. The existence of this force, however, produced a rise in the water level in the tower and made this method of computation impractical.

The head required to change the direction of flow at the upper gate (fig. 34D) with the lower gate closed was found experimentally by using the following equation,

$$h_{pu} = d_2 - d_1 - h_{f(45-u)}$$

where h_{pu} = Hydrostatic head in feet of water required to change the direction of flow at the upper gate.

d_2 = Difference in elevation between the water surface in the reservoir and the water surface in piezometer 45.

d_1 = Difference in elevation between the water surface in the reservoir and the water surface in the tower.

$h_{f(45-u)}$ = Pipe friction from piezometer 45 to the bottom of the upper gate.

The head required to change the direction of the flow at the lower gate (fig. 34D) with the upper gate closed was computed in a similar manner, using the equation

$$h_{pb} = d_3 - d_1 - h_{f(A-b)}$$

where h_{pb} = Hydrostatic head in feet of water required to change the direction of flow at the lower gate.

d_3 = Difference in elevation between the water surface in the reservoir and the water surface in piezometer rings A to E, inclusive.

d_1 = Difference in elevation between the water surface in the reservoir and the water surface in the tower.

$h_{f(A-b)}$ = Pipe friction from elevation 890 to the piezometers below.

The head required to change the direction of flow at the upper and the lower gate, expressed in terms of the velocity head, is plotted with respect to Reynolds' number for the model on figure 39.

A consideration of the principles of impulse and momentum provides a means of deriving theoretically the force required to change the direction of flow at the gates. It is not possible to obtain a rational theoretical solution from the forces acting entirely within the tower as the conditions of flow at the gates are too indefinite. Another logical method of attack is available, however, which is based on experimental data.

A free jet of water directed against a flat plate exerts a total force on the plate in the direction of the main jet equal to that given by the following expression:

$$F = wA \frac{V^2}{g}$$

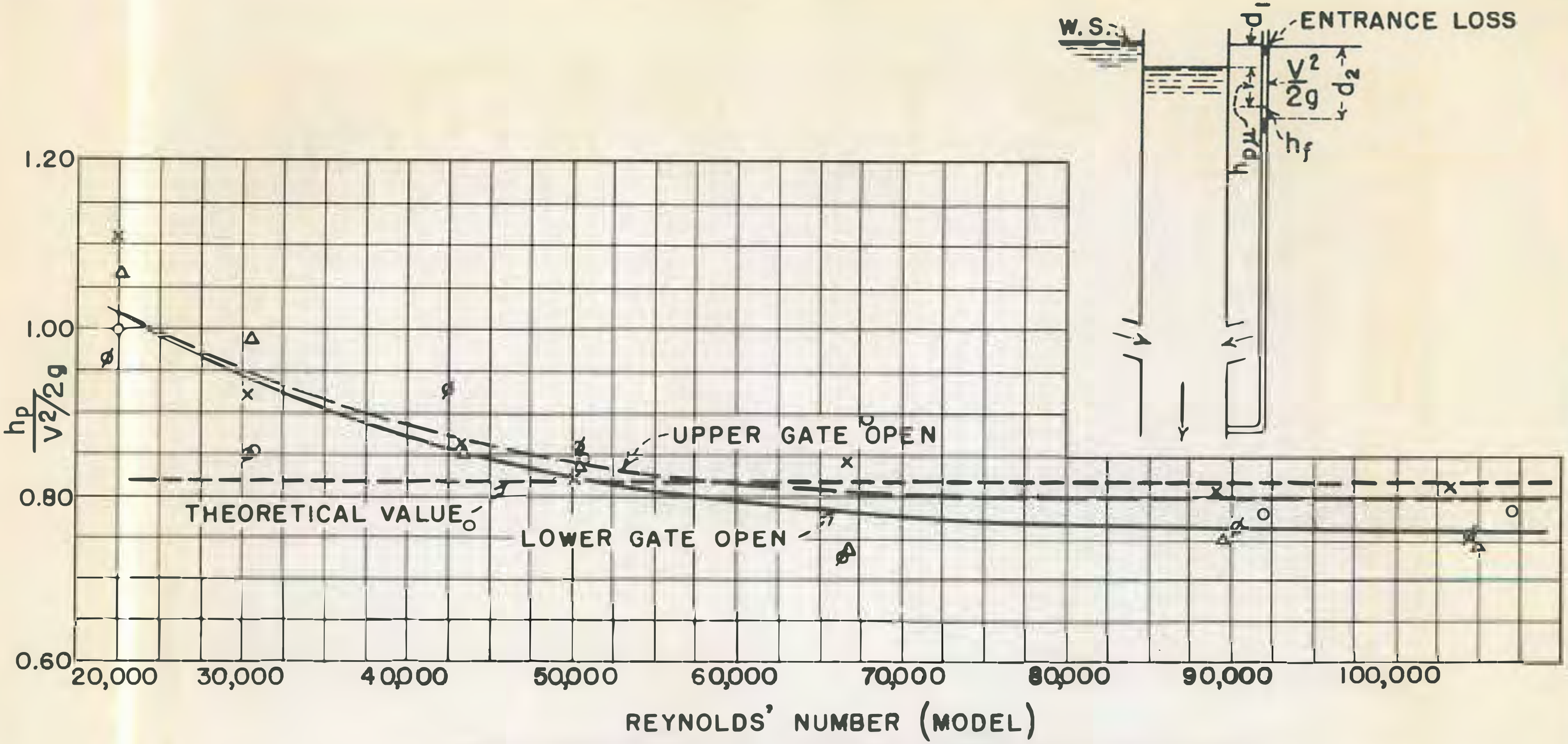
where F = Total force exerted on plate.
w = Unit weight of water.
A = Area of main jet.
V = Velocity measured in jet, and
g = Acceleration of gravity.

By analogy, if the direction of flow is reversed and water is assumed to flow radially inward toward the center of a circular plate and then turn downward away from the plate to form a solid column as in the case of the intake tower, the same law is assumed to apply. Professor A. H. Gibson[10] shows the bulb of pressure developed on a flat

[10] "Hydraulics and its Applications", by A. H. Gibson, pp. 368 and 371.

plate. The analogy with the intake tower is illustrated on figure 34E. If it were possible to measure the pressure at a number of points within the tower, at one of the gates, the pressure distribution might very easily resemble that shown on figure 34E. From experiments by Gibson it seems reasonable to assume that the maximum pressure head in the center of the tower is practically the velocity head while the pressure at the inside face of the tower is approximately 0.66 h_v. By integrating the pressures acting on individual small areas within the tower, an average pressure head of 0.82 h_v is obtained. This is the average height at which the water must stand within the tower in order to deflect the entering jets to a direction vertically downward.

The component in any direction of the total change in momentum between two points per unit time is the component of force in that direction required to produce this change. If the velocity of the



HEAD REQUIRED TO CHANGE DIRECTION OF FLOW AFTER ENTERING GATE

EXPLANATION

TEST 14 ⊙ UPPER GATE OPEN } — — —
TEST 17 × UPPER GATE OPEN
TEST 15 △ LOWER GATE OPEN } ———
TEST 18 Φ LOWER GATE OPEN
THEORETICAL - - - - - - - - - - - -

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INTAKE TOWER STUDIES
HEAD REQUIRED TO CHANGE
DIRECTION OF FLOW AT GATES

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TRACED. J.R.B. E.E.B. . RECOMMENDED. R. F. Blanks
CHECKED. J.N.B. . . . APPROVED. J. L. Savage
DENVER COLO., MAY 22, 1936 45-D-10201

FIGURE 39

water at the edges of the pressure bulb, outside of the tower, is assumed to be parallel to the direction of the intake (that is, horizontal), the vertical component of the momentum of the water is zero, and the change of momentum per unit time or the total force in the vertical direction required to produce this change between the point outside the tower and a point within the tower in the vertical jet below the intake is,

$$F = wA\frac{V^2}{g}$$

This expression is identical with that for the total pressure on a flat plate.

If the pressure distribution required to produce the total force, F, is assumed to be similar to that of the flat plate, then the maximum pressure head is $\frac{V^2}{2g}$ and the distribution is as shown on figure 34E. With this distribution, the average pressure head within the tower should be about

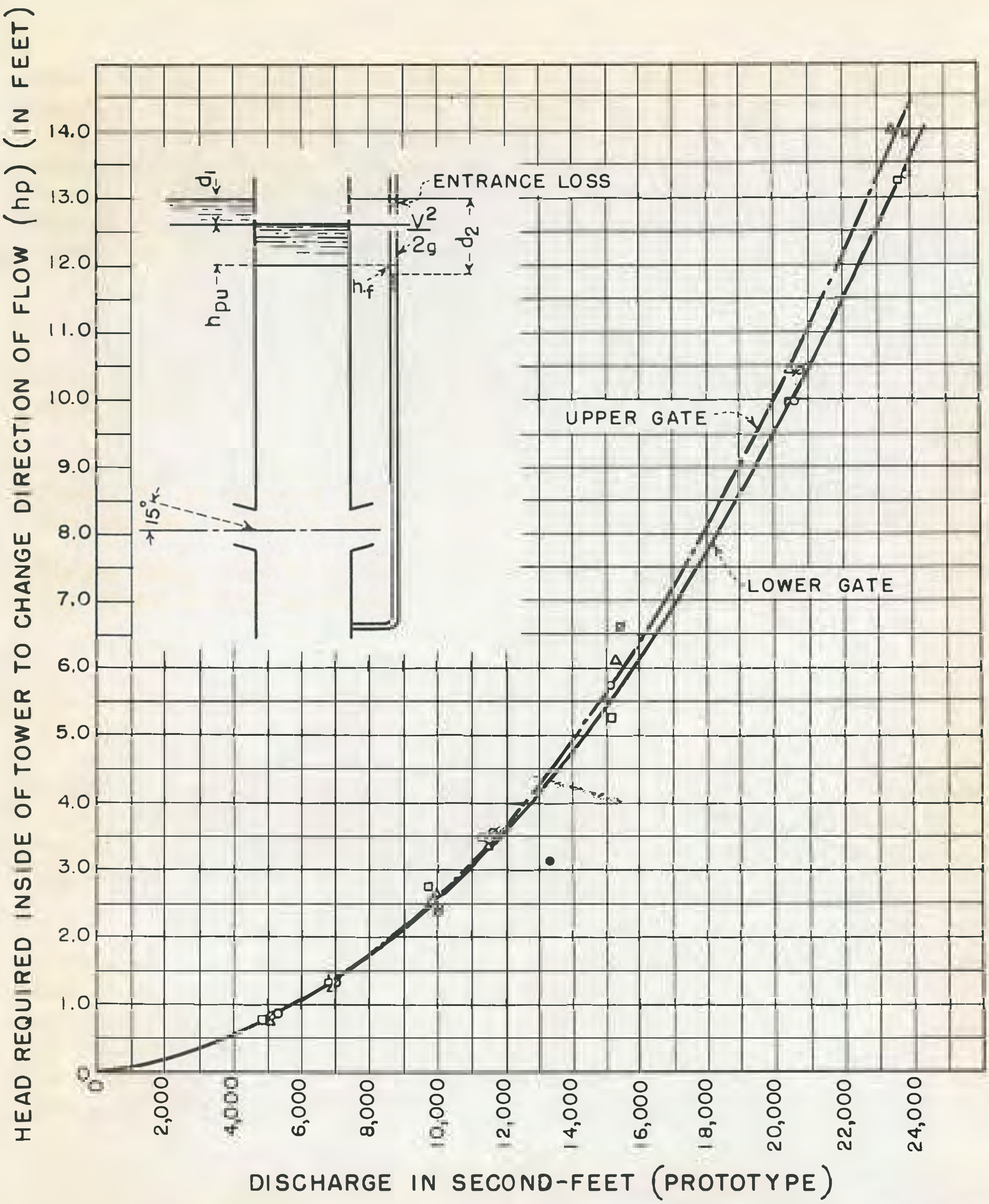
$$h_p = 0.82\,\frac{V^2}{2g}$$

This theoretical value has been plotted on figure 39 together with the actual values obtained on the model. The agreement is reasonably close and indicates that the analysis is a logical one. From the point of view of structural design, it is of interest that substantial upward reactions exist in the converging intake passages which, under some circumstances, might require structural provisions in the design.

To display the information obtained on the model in a more practical form, the model data were transferred into prototype values according to Froude's law and are shown plotted on figure 40. In this case, the average head of water required inside the tower to change the direction of flow of the water entering is plotted against the prototype discharge. These curves apply only for the upper and lower gate, operating separately.

H. Distribution of Total Discharge Passing Through the Upper and Lower Gate

It was desired to determine what portion of the total discharge passed through each gate when both gates were fully open. The piezometric drop, d_2, (fig. 34D) bears a definite relationship to the discharge passing through the upper gate. The piezometric drop, d_1, also shows a relationship to the discharge through the upper gate, and the two sets of data when plotted logarithmically with respect to the model discharge result in two straight lines



TEST 14 Ø } UPPER GATE OPEN
" 17 Δ }

TEST 15 ○ } LOWER GATE OPEN
" 18 □ }

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INTAKE TOWER STUDIES

HEAD REQUIRED TO CHANGE
DIRECTION OF FLOW AT GATES

DRAWN J.N.B. SUBMITTED J. N. Bradley
TRACED S.P.C.B. H.S. RECOMMENDED E. Waynock
CHECKED J. N. B. APPROVED J. L. Savage

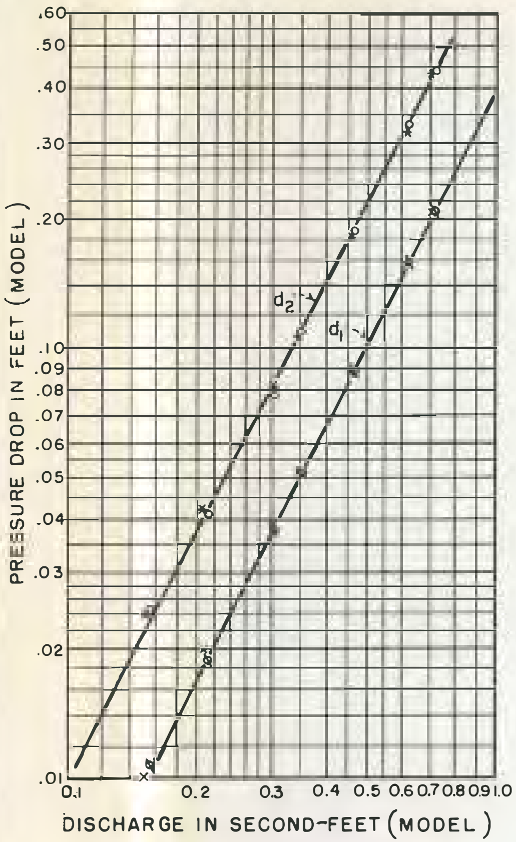
DENVER, COLO., JAN. 21, 1936 | 45-D-9707 (fig. 41A). Using these relationships, it was possible to compute the discharge through the upper gate when both gates were in operation. In other words, the pressure drops, d_1 and d_2, continue to be proportional to the discharge through the upper gate when both gates are in operation. They will be much smaller when both gates are open, however, as in this case the total discharge will be divided between two gates instead of one. For runs in which both gates were operating, the discharge through the upper gate was read from figure 41A for values of both d_1 and d_2, and the two averaged for each run. This average value was then considered as the discharge through the upper gate, while the remainder of the total discharge flowed through the lower gate. The curve on figure 41B shows the percentage of the total discharge flowing through each gate. The distribution of flow is expressed with respect to Reynolds' number computed for the total flow at elevation 890.0 for the model, and a second scale above the graph shows the approximate distribution with respect to the total prototype discharge. Figure 41B indicates that the flow through the upper gate will be approximately 40 percent of the total, regardless of the lake elevation and magnitude of the discharge providing the lake elevation is above the upper gate.

I. Relation of d_1 and D_1 to the Discharge

As the difference in elevation of the water surface in the reservoir and inside of the tower, d_1, is proportional to the discharge, this relationship may have a practical value. Values of d_1 have been plotted with respect to Reynolds' number computed at elevation 890.0 for the model for the upper, the lower, and both gates open, on figure 42. The drop, d_1, is the same for either the upper or the lower gate operating.

Although the difference in elevation, d_1, (fig. 34D) consists of the gate entrance loss plus a small portion of the velocity head, it is a linear measurement and by Froude's law can be converted into an approximate prototype value by multiplying it by the model scale, which is 64. This drop on the model, d_1, will be designated by the symbol, D_1, when converted to the prototype. Curves similar to those on figure 42 are plotted for the prototype on figure 43. This relation may prove useful for measuring the discharge through the prototype intake towers. The value of D_1 could be obtained by installing two float gages, one on the outside and one on the inside of each tower, and the curves on figure 43 used to determine the approximate discharge flowing through them.

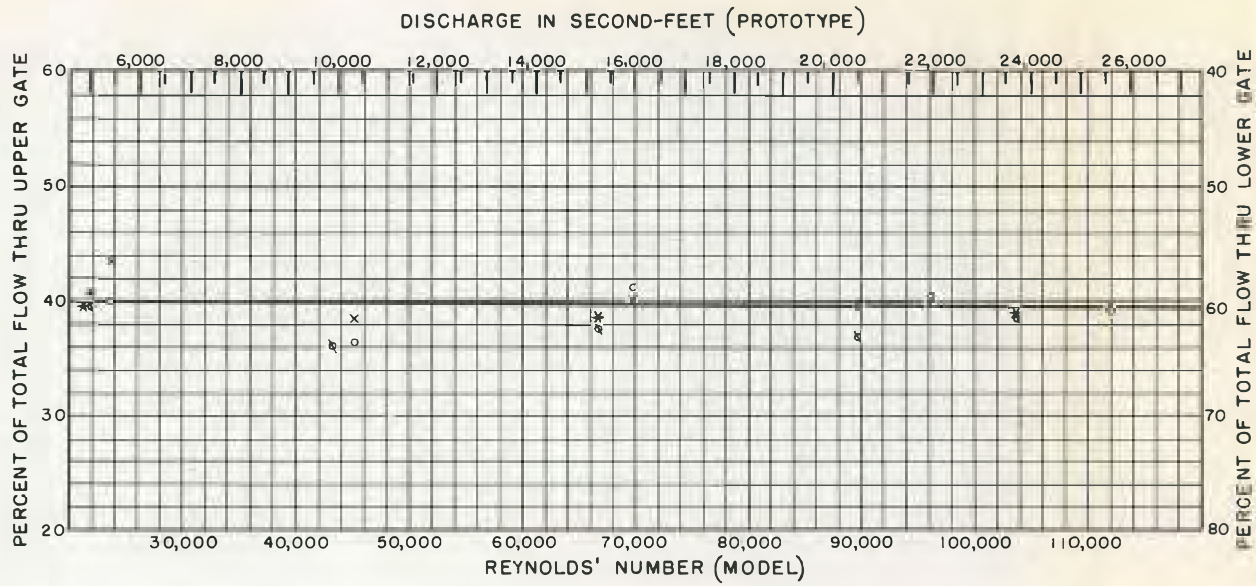
The relation of D_1 to the discharge has been plotted on figure 43 for both, with and without the trash racks in place. It is felt that the curves shown for the condition without the racks are essentially correct. It has been previously stated that the



A—RELATION OF d_1 AND d_2 TO THE DISCHARGE (MODEL)

EXPLANATION

TEST 14 - ø - d_1 UPPER GATE OPEN
" 14 - ⊙ - d_2 " " "
" 17 - X - d_1 " " "
" 17 - ✱ - d_2 " " "



B—DISTRIBUTION OF FLOW BETWEEN UPPER AND LOWER GATE WHEN BOTH GATES ARE OPEN

EXPLANATION

TEST 13 - ⊙ - d_1 BOTH GATES OPEN
" 13 - X - d_2 " " "
" 16 ø d_1 " " "
" 16 ✱ d_2 " " "

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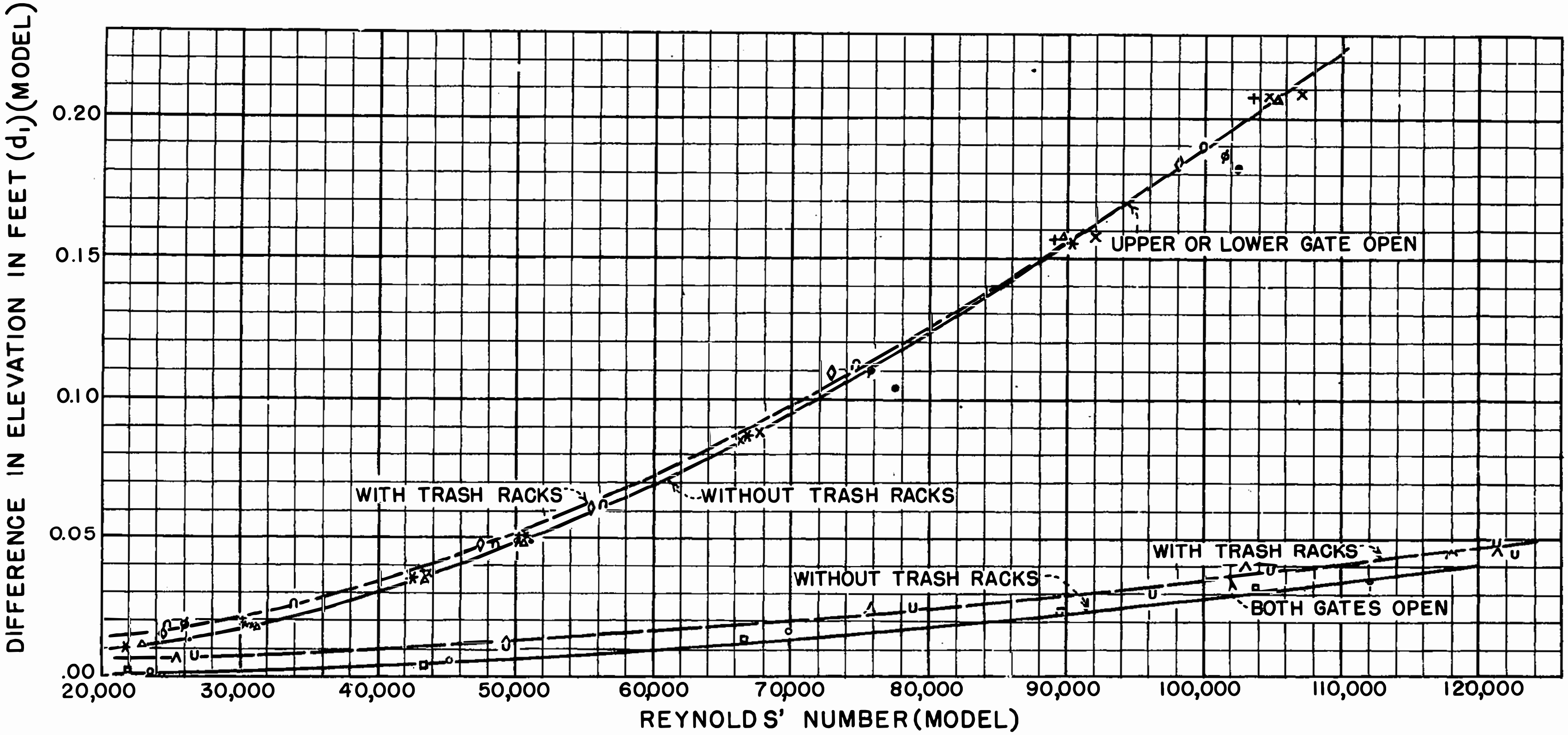
INTAKE TOWER STUDIES

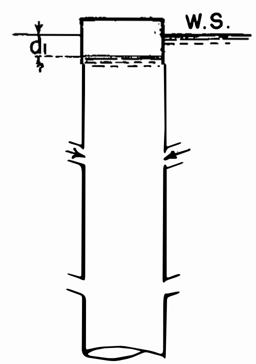
DISTRIBUTION OF FLOW THRU UPPER AND LOWER GATE (BOTH GATES OPEN)

DRAWN J.N.B. SUBMITTED J. N. Bradley
TRACED H.S. RECOMMENDED
CHECKED J.D.M. APPROVED

DENVER, COLO., JAN. 20, 1936 45-D-9768

FIGURE 41





EXPLANATION

TRASH RACKS IN PLACE — TRASH RACKS REMOVED

Trash racks in place			Trash racks removed	
TEST 8	Ø	UPPER GATE OPEN	TEST 14	×
" 11	∩		" 17	+
" 9	•	LOWER GATE OPEN	" 15	△
" 12	◊		" 18	*
" 7	∪	BOTH GATES OPEN	" 13	○
" 10	∧		" 16	□

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INTAKE TOWER STUDIES

RELATION OF d_1 TO REYNOLDS' NUMBER (MODEL)

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TRACED J.V.M. O.R.S. RECOMMENDED E. Wanock
CHECKED J. N. B. APPROVED J. L. Savage

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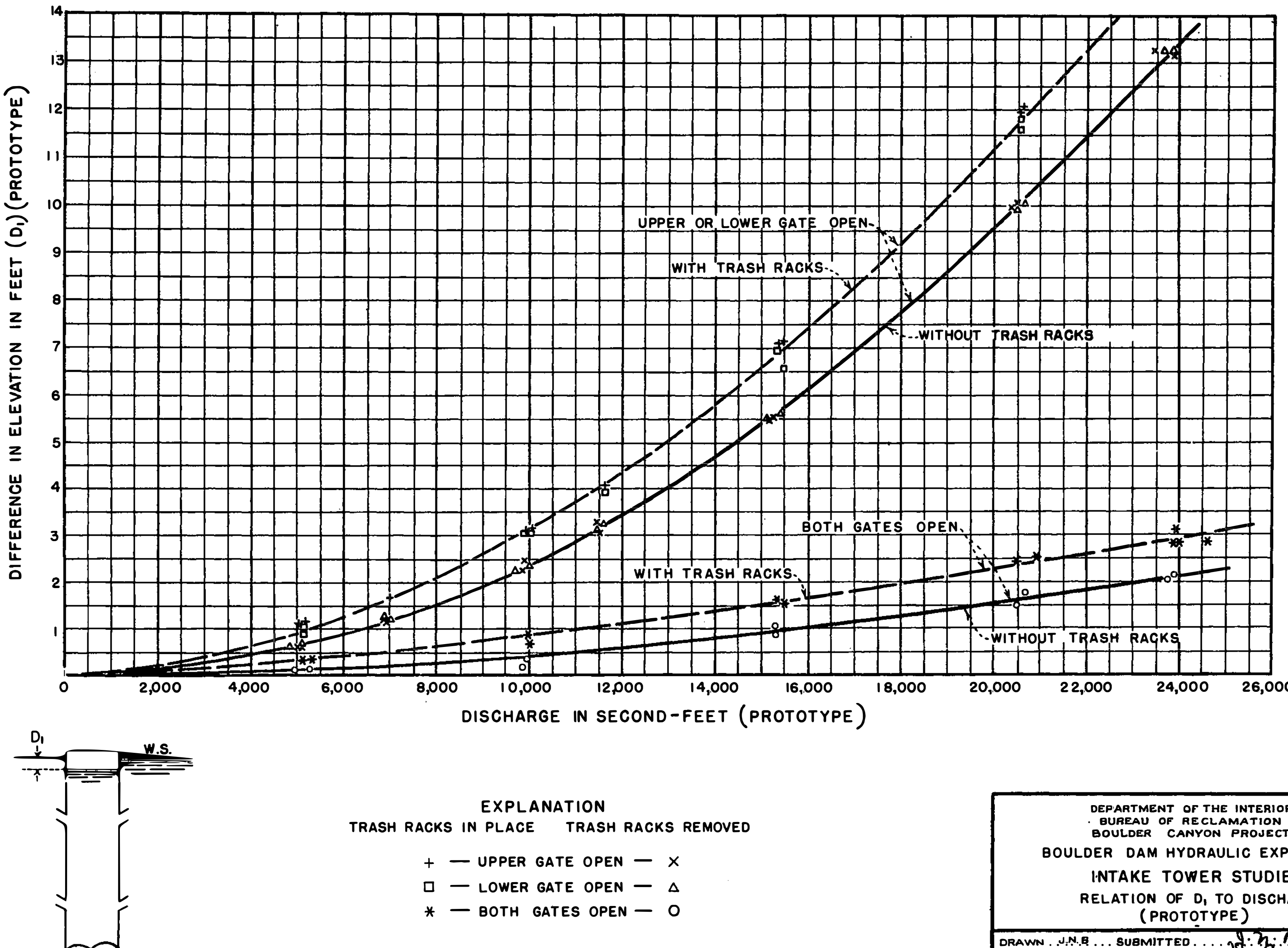
FIGURE 42



EXPLANATION

TRASH RACKS IN PLACE		TRASH RACKS REMOVED
+	UPPER GATE OPEN	X
□	LOWER GATE OPEN	△
*	BOTH GATES OPEN	O

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BOULDER DAM HYDRAULIC EXPERIMENTS

INTAKE TOWER STUDIES

RELATION OF D_1 TO DISCHARGE
(PROTOTYPE)

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TRACED J.R.B. O.R.S. RECOMMENDED E. Weymouth
CHECKED J.D.M. APPROVED J. L. Savage

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FIGURE 43

friction loss through the model trash racks was excessive due to a number of factors that were physically uncontrollable. Therefore, it can quite definitely be stated that the curves on figure 43 for the model trash racks installed are higher than would be similar curves made from actual results on the prototype.

It would be possible to actually calibrate the prototype intake towers by the Gibson method[11] at the time that the acceptance

[11] "Pressures in Penstocks Caused by the Gradual Closing of Turbine Gates", by Norman R. Gibson, Trans., Am. Soc. of Civil Engineers, vol. LXXXIII, page 707.

tests are made on the turbines. With the information already available on figure 43 only a few points would be necessary to establish a calibration curve for the prototype intake towers.

It is recommended that at least a portion of the foregoing experiments be repeated on the prototype intake towers in the future, after the appurtenant structures are in operation, to establish a check of actual results against those obtained on the model, which should definitely determine the merits of a model of this type. The experimental equipment required would be simple and inexpensive, and the testing would require a relatively short time.

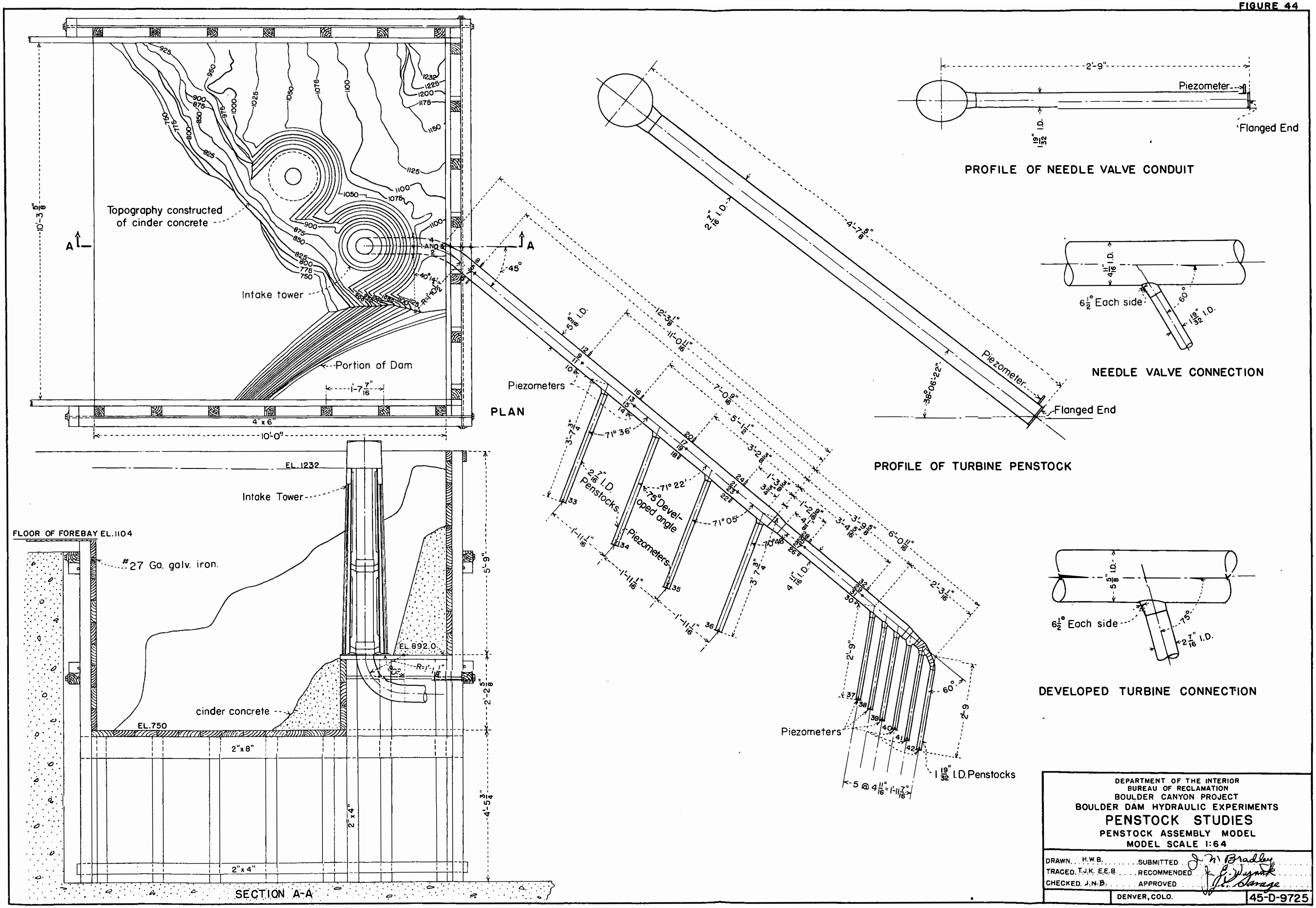
2. THE PENSTOCK ASSEMBLY

A. The Apparatus

At the conclusion of the tests on the intake tower, the 3-foot vertical section of straight pipe below the tower was removed and the penstock assembly model, constructed on a scale of 1:64, connected in its place (fig. 44). The assembly consisted of accurate, butt-jointed, smooth, sheet-metal pipe, constructed similarly to that used in the penstock quantitative tests except for a 90-degree 5-5/8-inch diameter vertical bend directly below the tower and a 40-degree horizontal bend of the same diameter located a short distance downstream, both of which were made of transparent pyralin pipe. The four turbine penstocks (figs. 32, 44, and plate VII) were 2-7/16 inches in diameter and were connected to the penstock header at a developed angle of 105 degrees in a downstream direction as shown. The six 1-19/32-inch diameter branches leading to the canyon wall outlets were connected to the downstream end of the penstock header box by a reduction manifold.

The discharge through each turbine branch was controlled and measured by a sharp-edged circular orifice with a diameter of

FIGURE 44



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BOULDER DAM HYDRAULIC EXPERIMENTS

PENSTOCK STUDIES

PENSTOCK ASSEMBLY MODEL
MODEL SCALE 1:64

DRAWN H.W.B. SUBMITTED J. N. Bradley
TRACED T.J.K. E.E.B. RECOMMENDED J. E. Warnock
CHECKED J.N.B. APPROVED J. L. Savage

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5 or 0.875 inches. The flow through the needle-valve branches was trolled and measured in a like manner by orifices with a diameter either 1.006 or 1.250 inches. The head on the orifices was measd by a piezometer located in the branch immediately above each fice. These orifices were calibrated by weight in similar posians previous to the penstock experiments (plate VII-B). The rat- curves obtained from the calibration runs for the four sizes of fices are shown on figure 45.

Piezometers for measuring the pressure intensities were talled in rings at intervals down the main header (fig. 44). A g consisted of four piezometers and each was connected to an inidual reading glass on the manometer board.

B. Investigation of Pressures at Base of Tower

In the original design of the intake towers, air vents were vided in the region below the lower gate (fig. 31) to relieve any ative pressure which might be created by the conditions of flow. uestion arose as to the necessity of these vents and piezometers to 49, inclusive, (figs. 33 and 46D and E) were installed to study pressure conditions at that point.

Tests were made with the upper, the lower, and both gates n and the differences in elevation, d, between the water surface the reservoir and the water surfaces in the four piezometers were orded. These differences, converted into prototype values, are otted against the discharge for the individual piezometers for the ee conditions of gate opening on figure 46A, B, and C. For the per gate open and lower gate closed (fig. 46), the drop, d, recorded piezometer 46 consists of the following:

$$d = h_{f(b-u)} + \frac{V^2}{2g} + h_{eu} + h_s$$

re d = Difference in elevation between the water surface in the reservoir and the water surface in piezometer 46.

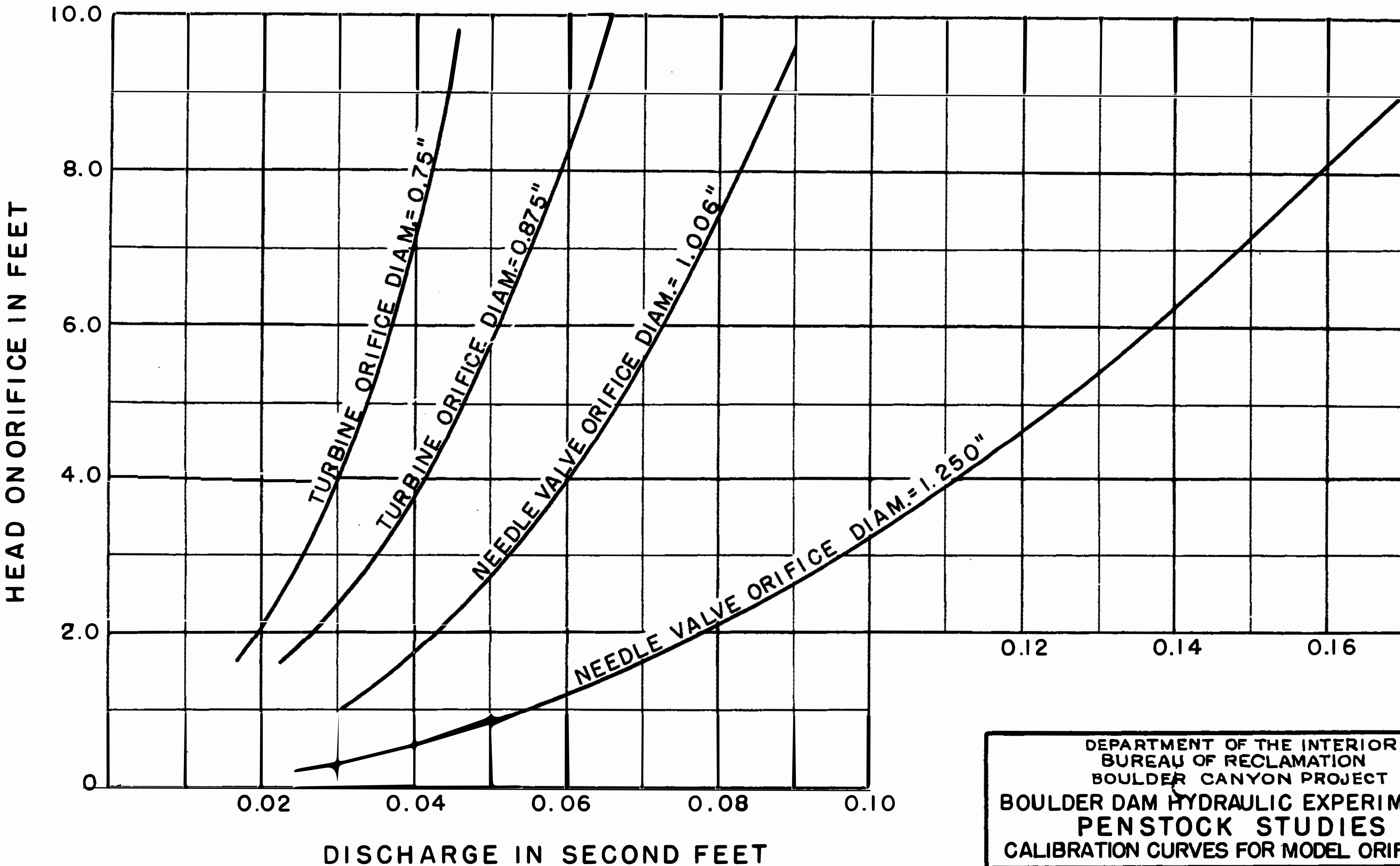
$h_{f(b-u)}$ = Pipe friction from piezometer to bottom of upper gate.

$\frac{V^2}{2g}$ = Velocity head at piezometer 46.

h_{eu} = Entrance loss at upper gate.

h_s = Drop in pressure at the piezometer caused by irregularity of flow and suction effect.

The curves for piezometer 46(fig. 46) were replotted on gure 47 for the three conditions of gate opening. In addition, two er curves have been superimposed on this graph, which indicate the



DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT

BOULDER DAM HYDRAULIC EXPERIMENTS

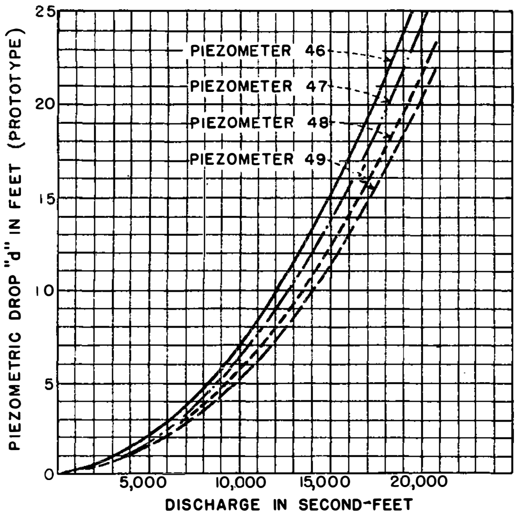
PENSTOCK STUDIES

CALIBRATION CURVES FOR MODEL ORIFICES

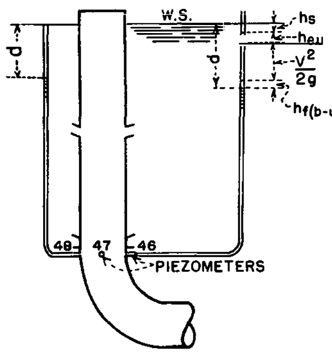
DRAWN J.N.B. .... SUBMITTED J. N. Bradley
TRACED E.F.J. H.S. RECOMMENDED J. E. Warnock
CHECKED J.D.M. APPROVED J. L. Savage

DENVER, COLO., APR. 20, 1936 | 45-D-9737

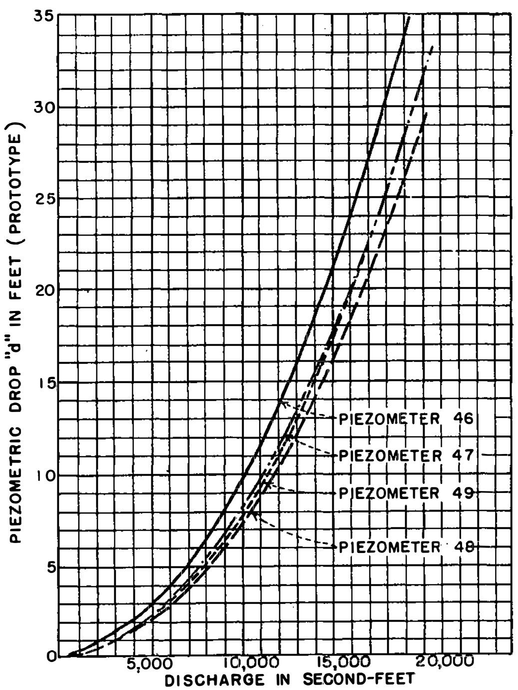
FIGURE 45



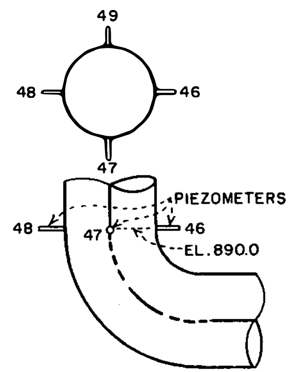
A-UPPER GATE OPEN



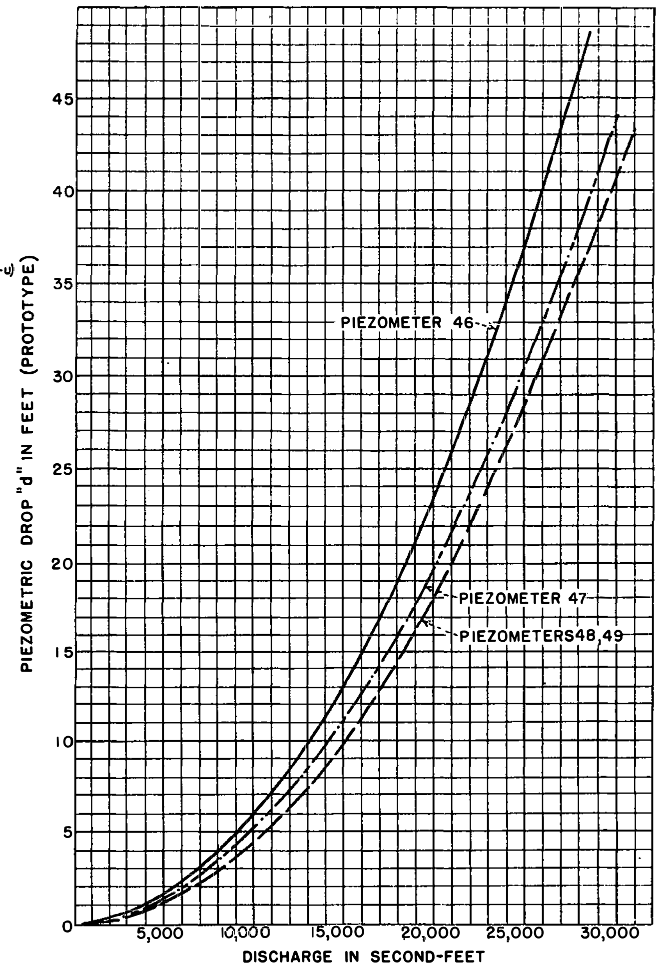
D- UPPER GATE OPEN



B-LOWER GATE OPEN



E-DETAIL AT BASE OF TOWER SHOWING LOCATION OF PIEZOMETERS



C-BOTH GATES OPEN

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BOULDER DAM HYDRAULIC EXPERIMENTS

INTAKE TOWER STUDIES

RELATION OF PRESSURE DROPS AT PIEZOMETERS 46, 47, 48, AND 49 TO DISCHARGE

DRAWN. H.W.B. J.D.M. SUBMITTED. J. N. Bradley
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FIGURE 46

maximum approximate discharge that can be obtained with all needle valves operating and with all needle valves and turbines operating for different elevations of the water surface in the reservoir. From these two sets of curves the pressure at piezometer 46 may be obtained for any particular discharge. For example, with the water surface in the reservoir at elevation 1200 and a discharge of approximately 21,000 second-feet through the lower gate (fig. 47), the water surface in piezometer 46 expressed in prototype would stand at 42 feet below the reservoir water surface, or elevation 1158 and a pressure of 268 feet of water would exist at this piezometer. For the same discharge passing through the upper gate, the water surface in piezometer 46 would stand at elevation 1170. For a discharge of 21,000 second-feet flowing through both gates, the water surface in piezometer 46 would be at elevation 1174. The water surface in the other three piezometers would be above this elevation. From these results it is evident that a vacuum cannot exist at the base of the tower and air vents are unnecessary.

C. Bend Losses

The combined loss for the two bends directly below the tower (a 90-degree vertical bend and a 40-degree horizontal bend interconnected by a straight section 0.88 diameters long, figure 32) was obtained from runs in which only the upper gate was open. Flow through the lower gate would have materially affected the velocity distribution in the bends. This loss was obtained by the equation

$$h_b = d_x - \left(\frac{V^2}{2g} + h_f + h_t\right)$$

where h_b = Combined bend loss.

d_x = Difference in elevation between the water surface in the reservoir and the average water surface elevation in the manometer tubes connected to piezometers 9 to 12, inclusive (figure 44).

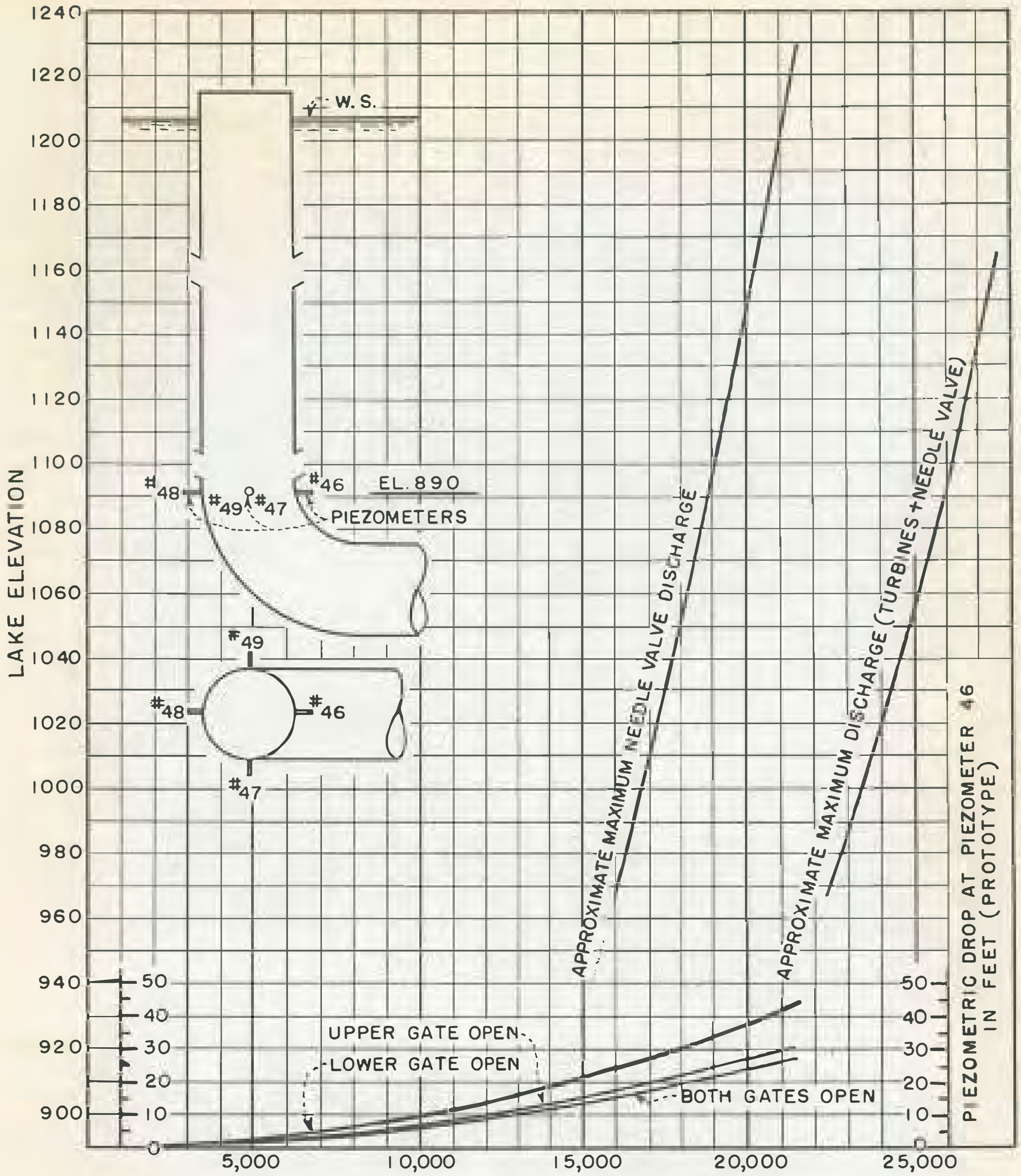
$\frac{V^2}{2g}$ = Velocity head at base of tower (elevation 890).

h_f = Pipe friction from piezometers 9, 10, 11, and 12 to the downstream edge of the horizontal bend, and

h_t = Total loss in intake tower.

A better velocity distribution existed at piezometers 9 to 12 so these were used in the computations rather than piezometers 5 to 8.

The combined loss for the two bends expressed in terms of the velocity head at the base of the tower is plotted with respect to



DISCHARGE IN SECOND-FEET (PROTOTYPE)
MINIMUM PRESSURES RECORDED AT ELEVATION 890.0
TRASH RACKS REMOVED

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BOULDER DAM HYDRAULIC EXPERIMENTS

INTAKE TOWER STUDIES

MINIMUM PRESSURES AT BASE OF TOWER

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Reynolds' number on figure 48. As the line is practically horizontal, the loss in the prototype would not be expected to differ much from this value.

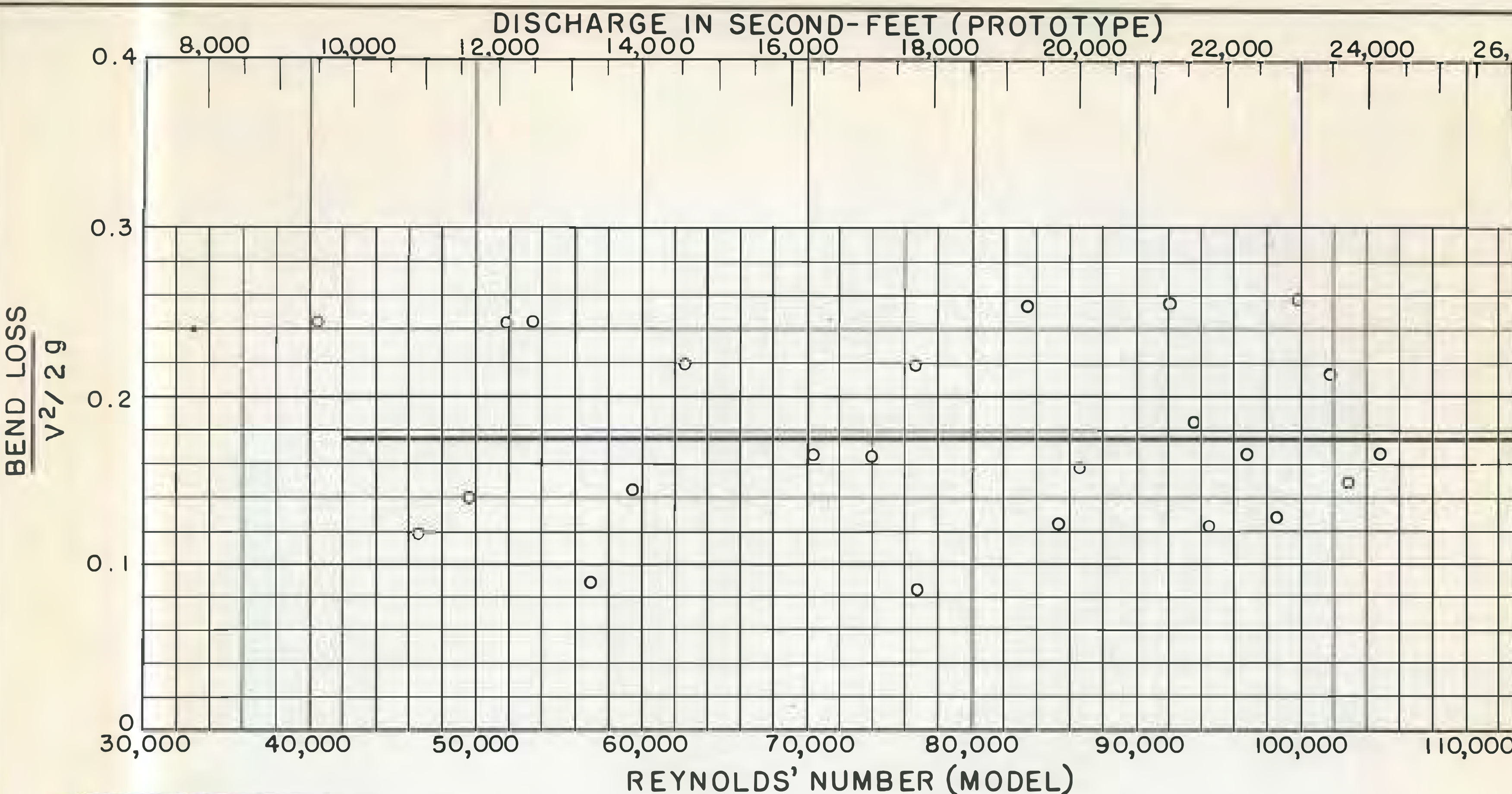
D. Tests on the Penstock Assembly

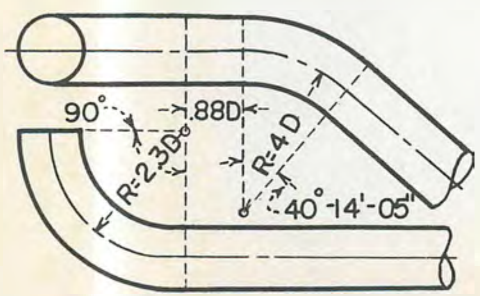
Tests were made on the complete penstock assembly during which the discharge, head, and piezometer readings were recorded. From these tests the energy head at each piezometer ring (pressure head plus mean velocity head) was computed and plotted as far down the penstock as possible. The energy heads for a few runs are shown on figure 49. The lines indicate a gain in energy head in the main penstock immediately downstream from the first junction. A gain in energy head is evident at the next junction, but less than that at the first. The third junction shows a slight gain in energy head in most cases and a loss was recorded at the fourth. This gain was usually recorded even when there was no flow through the branch. It will be recalled that similar results were obtained on the penstock junction quantitative tests described in chapter IV, section 3D. This apparent gain is undoubtedly due to an error in computing the velocity head. Due to the irregular velocity distribution at these points, the mean velocity head should be increased by multiplying it by a velocity head coefficient, α , which presumably lies within the limits of 1.03 to 1.15 and probably varied at each junction with each condition of discharge.

Curves representing the average recorded loss or gain in energy head in the main pipe at junctions 1, 2, and 3 in terms of the velocity head above each junction are plotted with respect to the discharge ratios $\frac{Q_s}{Q_a}$ on figure 50. To compare the curve obtained for junction 1 with the results obtained from experiments on the large penstock model previously described in chapter IV, data from the latter are also plotted on figure 50. The indicated gain in energy at the first junction agrees fairly well with the curve taken from the larger model up to a discharge ratio of 0.35. Above this ratio, the points are few and scattered and it is difficult to attempt a comparison. The curves for junctions 2 and 3 are also unreliable above the discharge ratio of 0.35. The results confirm those of Thoma, whose work was on small pipes, with respect to the recorded apparent gain in energy head in the main pipe downstream from the junctions, and indicate that this gain is due presumably to an error in computing the velocity head and not due to viscous effects produced only in small pipes.

An attempt was made to obtain the junction losses chargeable to the branch but the number of piezometers was insufficient to secure accurate measurements.

As an interesting comparison, pressure gradients for the same runs shown on figure 49 are plotted on figure 51. There is a





FOR DETAILS OF BENDS SEE FIGURE 32.

NOTE

Loss shown above is for both bends plus the short section of interconnecting straight pipe.

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BOULDER DAM HYDRAULIC EXPERIMENTS

PENSTOCK STUDIES

LOSS IN BENDS IN UPPER ARIZ. PENSTOCK

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TRACED E.F.J. H.S. RECOMMENDED
CHECKED J.D.M. APPROVED

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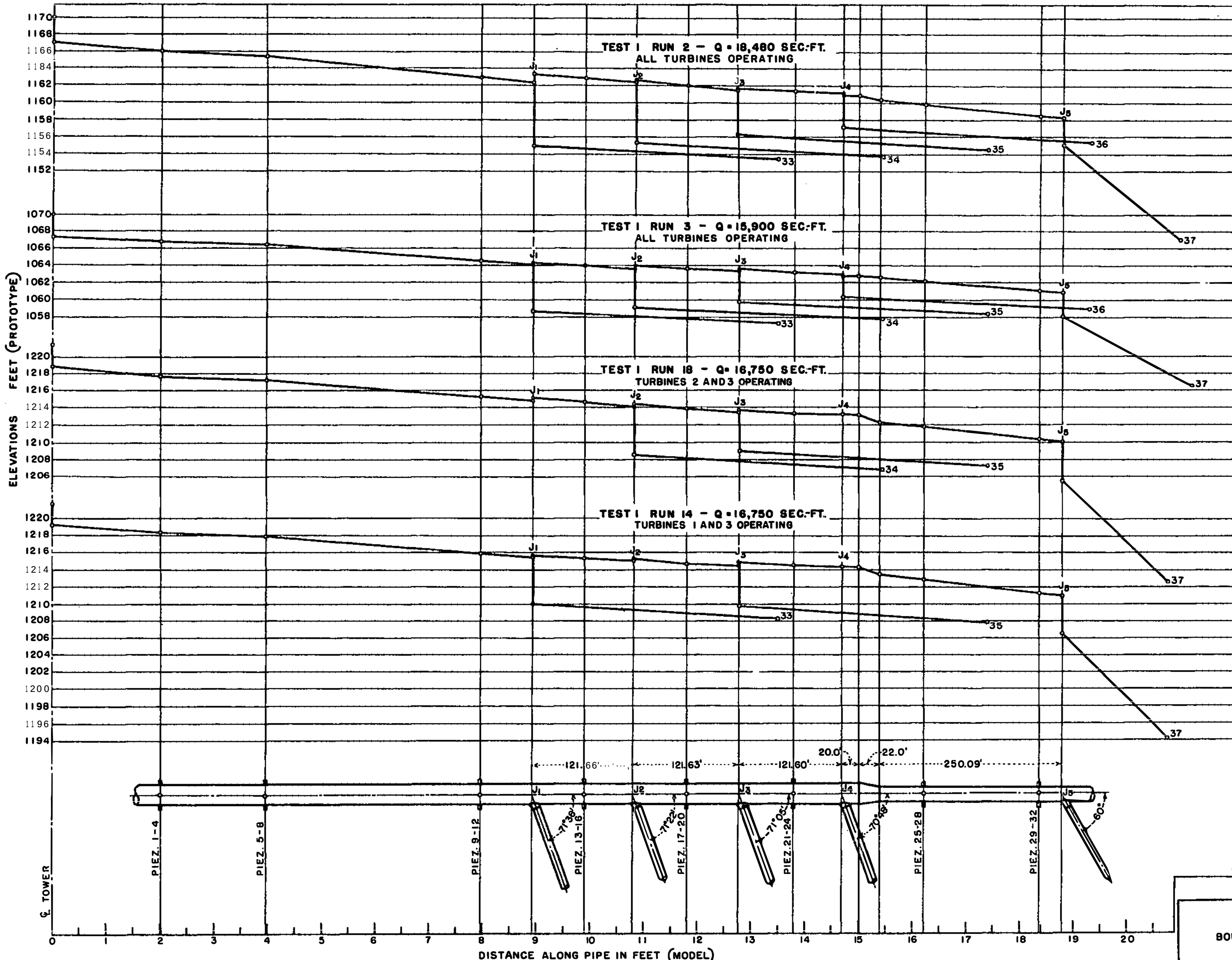
FIGURE 48



NOTE

Needle valve No. 1 joined to the main pipe at junction 5, is the only needle valve for which the energy gradient can be drawn.

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BOULDER DAM HYDRAULIC EXPERIMENTS

PENSTOCK STUDIES

PRESSURE PLUS VELOCITY HEAD GRADIENT

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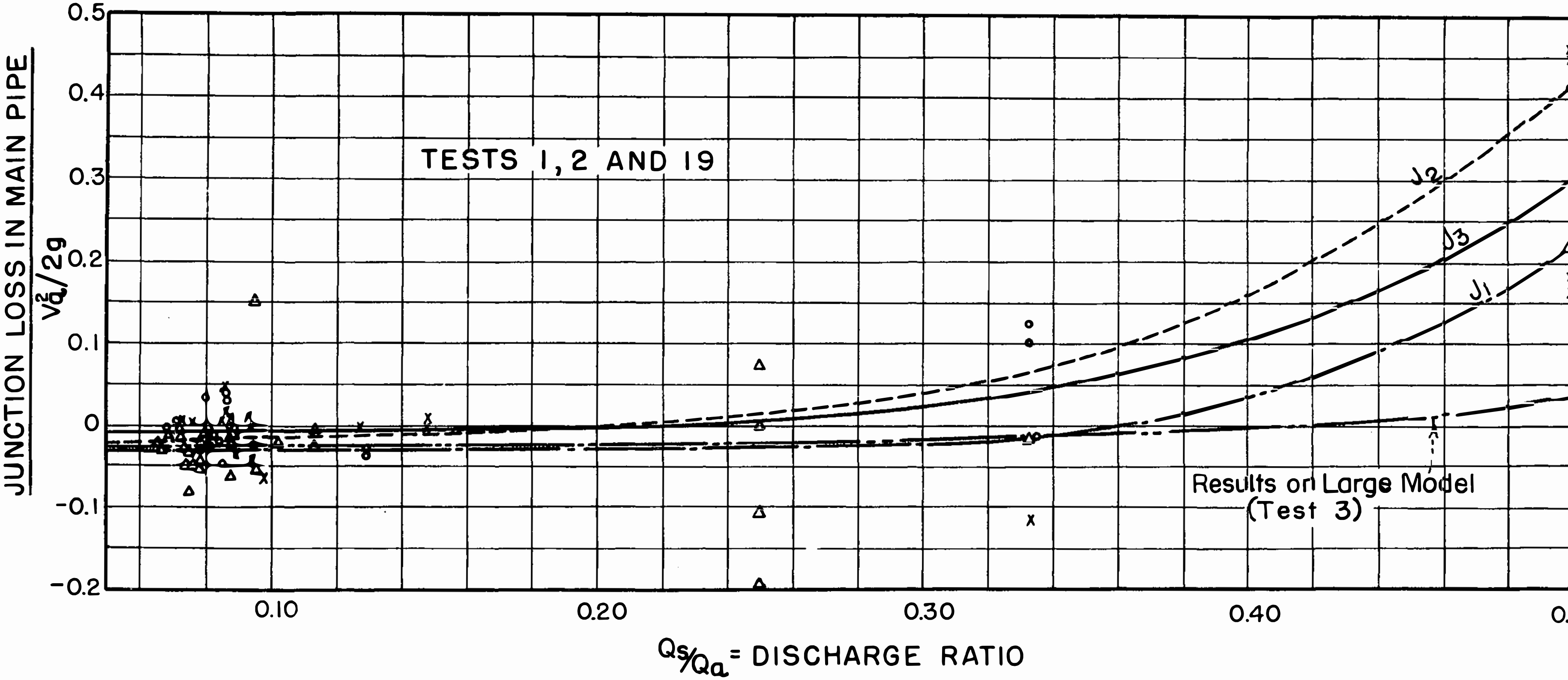
DENVER, COLO., JAN. 8, 1936 45-D-9726

FIGURE 49



EXPLANATION

J1 △——-·——△

J2 o- - - - - - -o

J3 x————x

LARGE JUNCTION MODEL ——-·-——
(TEST 3)

NOTE

V_a is measured above the junction in each case.

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BOULDER DAM HYDRAULIC EXPERIMENTS
PENSTOCK STUDIES
JUNCTION LOSSES IN MAIN PENSTOCK

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TRACED O.R.S.	RECOMMENDED E. W. Lane
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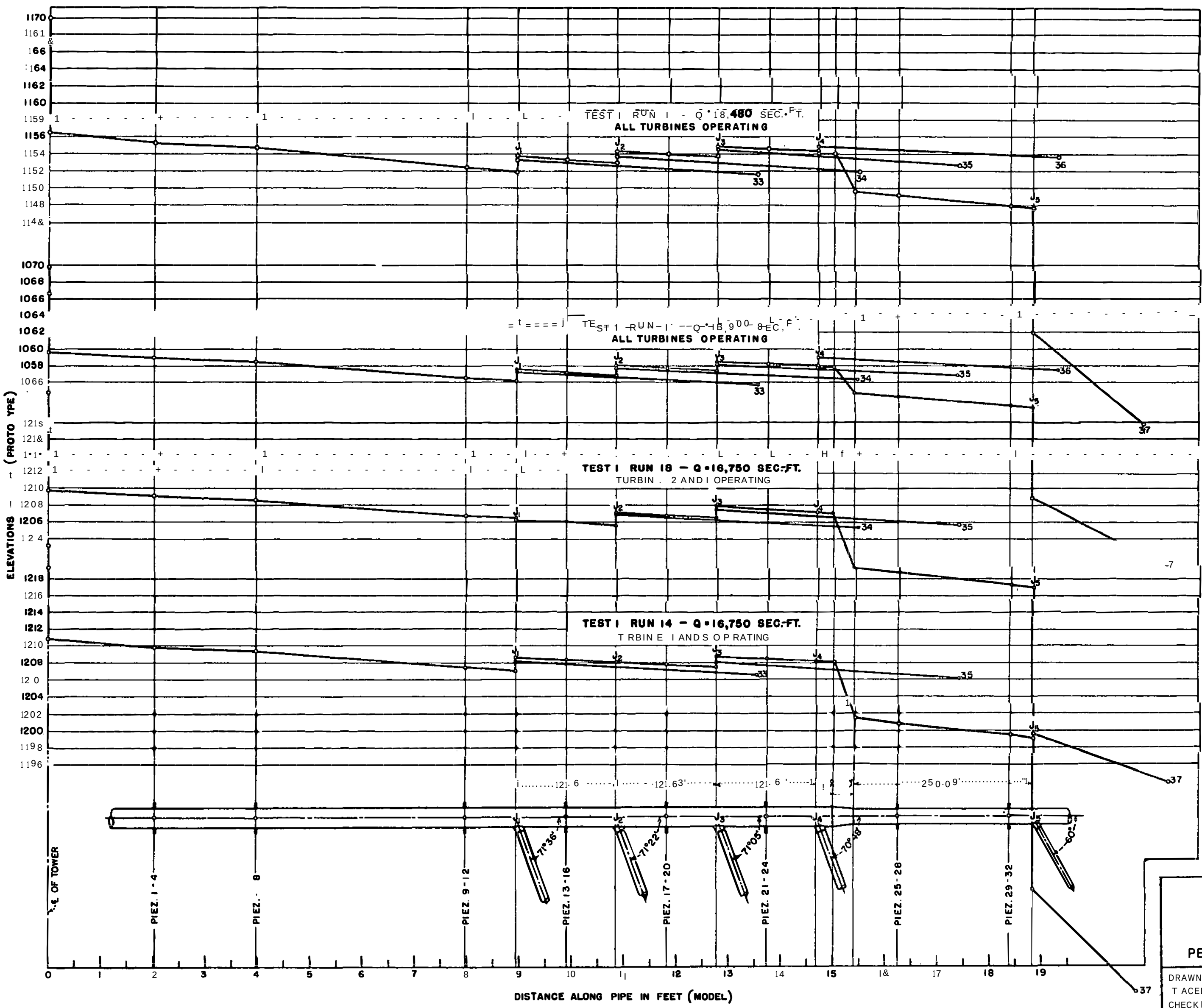
DENVER, COLO., 1-2-36	45-D-9744

FIGURE 50



NOTE

Needle valve number 1 joined to the main pipe at junction 5 is the only needle valve for which the pressure gradient can be shown.

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BOULDER DAM HYDRAULIC EXPERIMENTS

PENSTOCK STUDIES

PENSTOCK PRESSURE GRADIENT

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TRACED....H.S.... RECOMMENDED....
CHECKED....J.P.... APPROVED....

DENVER, COLO., JAN. 2, 1936 | 45-D-9727

FIGURE 5

decided increase in pressure at each turbine junction which is due to the reconversion of a portion of the velocity into pressure head. As the number of piezometers was limited, it was only possible to plot these pressures down to junction 5.

E. Needle Valve Calibration

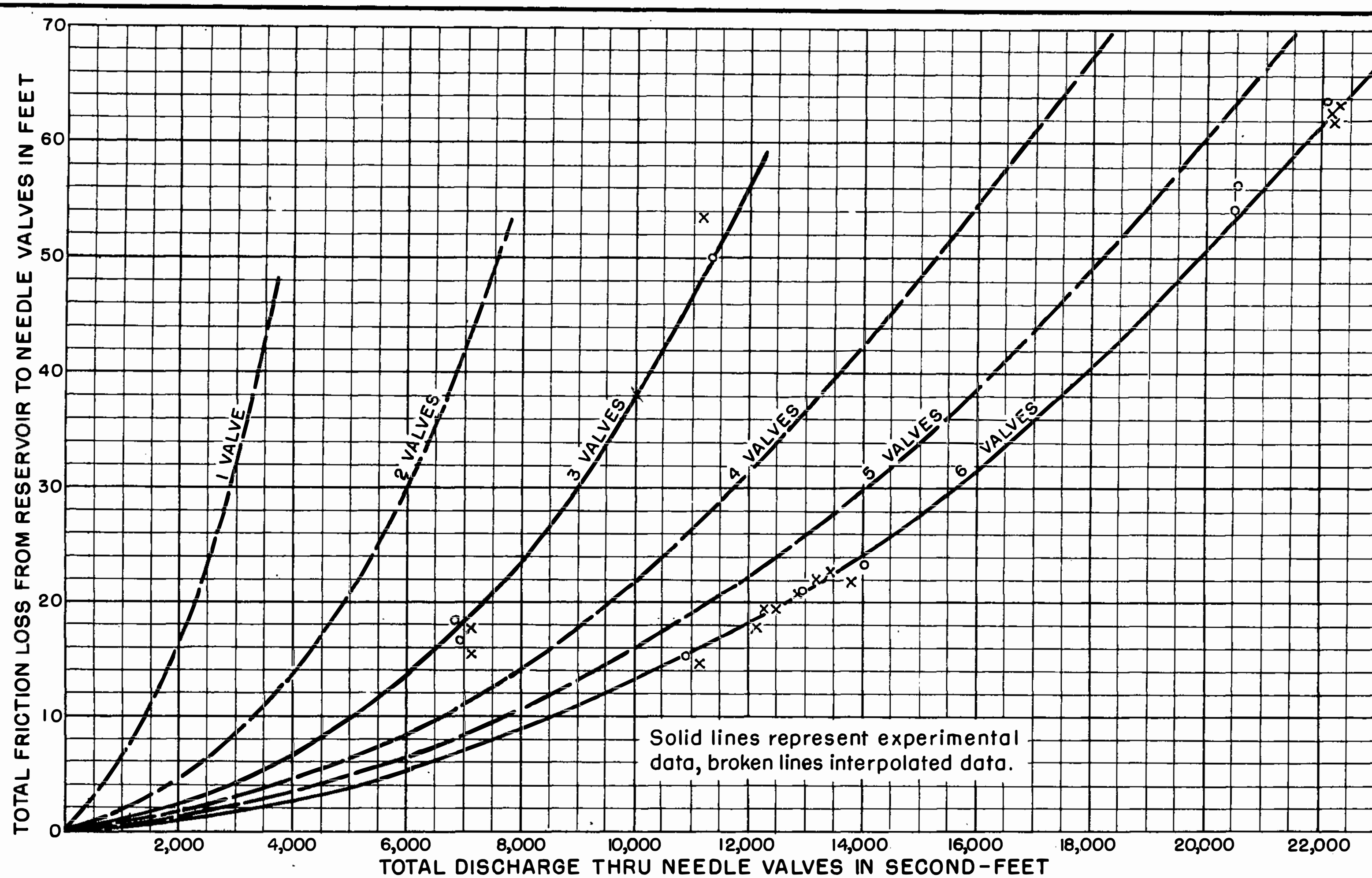
As the energy head at the six needle valves was found to be practically constant for an equal discharge through each valve, it was possible to plot a set of curves giving the loss from the reservoir to a point immediately above the needle valves for any discharge and valve combination (figure 52). The friction loss is plotted against the total discharge through the needle valves for one to six valves operating regardless of the combination. In other words, the loss for any discharge is practically the same for valves 1 and 2 operating as for 3 and 4 or 5 and 6. To use the curves on figure 52, it is necessary to comply with two limitations, first, that there is no flow through the turbines, and second, that the needle valve openings are the same for any one discharge combination. Without these limitations, the variables would become so numerous that plotting of the results would be impossible. The curves for three and six needle valves operating (figure 52) were obtained from the model experiments and were converted into prototype values according to Froude's law. The remaining curves were obtained by interpolation as experimental data were not taken for these combinations. As friction drop constitutes the major portion of these losses, the extrapolation from model to prototype could more accurately be made according to Reynolds' law, but insufficient data prevented the application of this method. The values shown by the curves are therefore approximate. With the Arizona canyon wall outlet works discharging 10,000 second-feet with six valves opened the same amount, the friction loss from the reservoir to any one of the six valves would be 13.2 feet. This discharge through any five valves would encounter a loss of 16.0 feet, and through any four valves, 21.8 feet.

The curves on figure 53A were plotted from the data on figure 52. From these curves, the required pressure head in feet of water immediately above any needle valve in operation can be computed for various discharges through any combination, providing each valve in operation is opened an equal amount and no flow is passing through the turbines. Referring to figure 53B, the pressure head at any needle valve operating

$$h_p = h_E - (h_f + \frac{V^2}{2g} + 820.0)$$

where,

h_p = Pressure head immediately above the discharging needle valves in feet of water.



TOTAL FRICTION LOSS FROM RESERVOIR TO
NEEDLE VALVES FOR ANY DISCHARGE
COMBINATION – NO FLOW THRU TURBINES
ARIZONA CANYON WALL OUTLET WORKS

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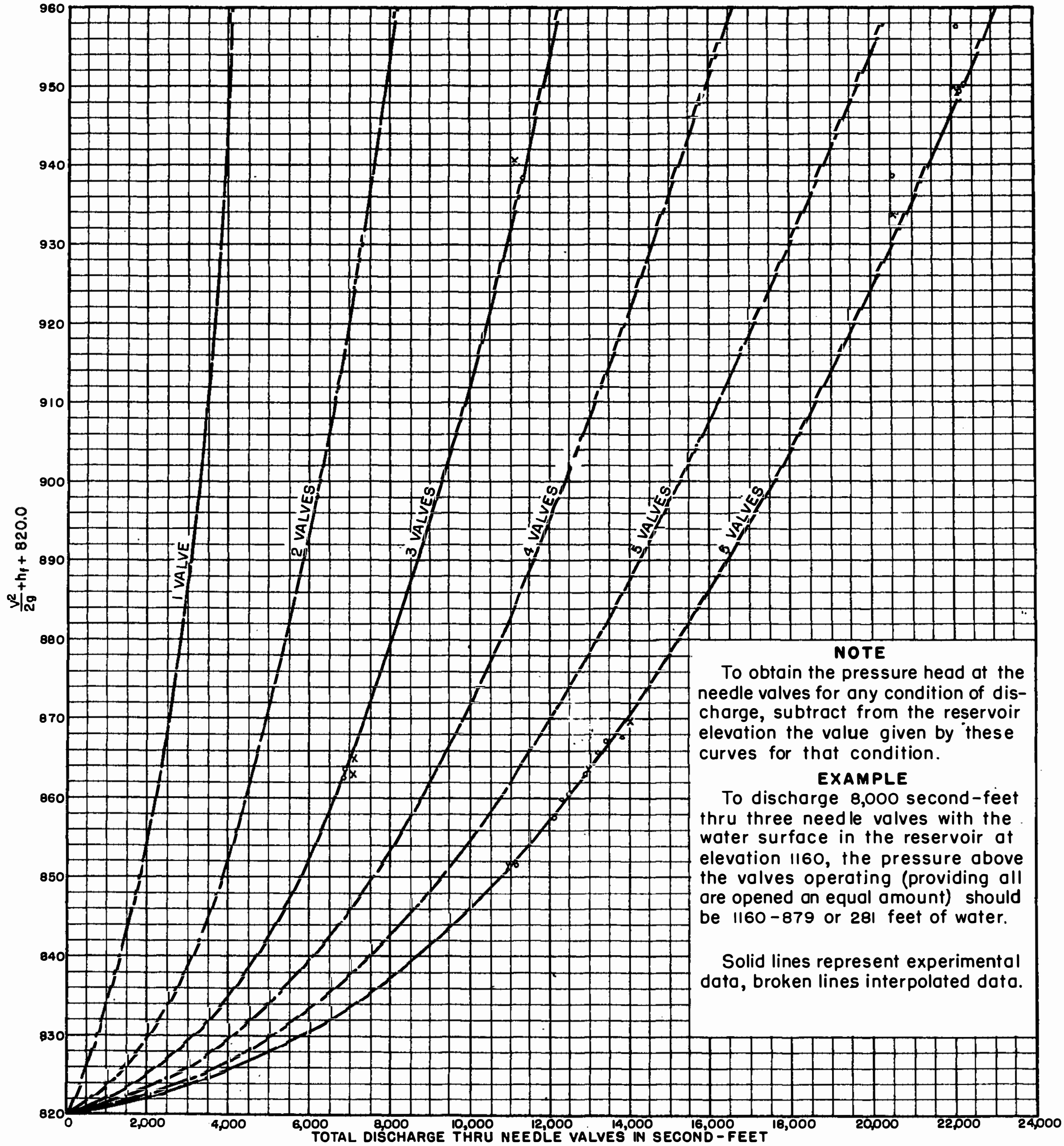
BOULDER DAM HYDRAULIC EXPERIMENTS

FRICTION LOSS FROM
RESERVOIR TO NEEDLE VALVES
NO FLOW THRU TURBINES

DRAWN J.D.M.	SUBMITTED J. N. Bradley
TRACED J.V.M.-D.W.S.	RECOMMENDED J. G. Warnock
CHECKED J.N.B.	APPROVED J. L. Savage
	DENVER, COLO., FEB. 10, 1936 — 45-D-9890

FIGURE 52



NOTE

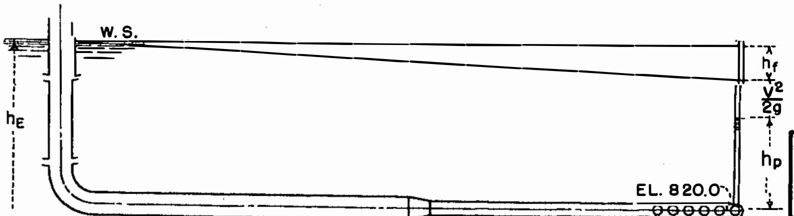
To obtain the pressure head at the needle valves for any condition of discharge, subtract from the reservoir elevation the value given by these curves for that condition.

EXAMPLE

To discharge 8,000 second-feet thru three needle valves with the water surface in the reservoir at elevation 1160, the pressure above the valves operating (providing all are opened an equal amount) should be 1160-879 or 281 feet of water.

Solid lines represent experimental data, broken lines interpolated data.

A-PRESSURE HEAD ABOVE NEEDLE VALVES FOR ANY DISCHARGE COMBINATION — NO FLOW THRU TURBINES ARIZONA CANYON WALL OUTLET WORKS



B-DIAGRAMMATIC SKETCH OF PRESSURE HEAD AT NEEDLE VALVES

EXPLANATION

h_P = Pressure head above discharging needle valves in feet of water.
$\frac{V^2}{2g}$ = Velocity head in branch penstock leading to needle valve in feet.
h_f = Total friction loss from reservoir to needle valve in feet.
h_E = Elevation head in reservoir in feet above sea level.

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BOULDER DAM HYDRAULIC EXPERIMENTS
PRESSURE HEAD FOR DISCHARGE THRU CANYON WALL NEEDLE VALVES NO FLOW THRU TURBINES

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TRACED E.M.W. D.W.S. RECOMMENDED J. L. Savage
CHECKED J.N.B. APPROVED R. F. Walter

DENVER, COLO., FEB. 10, 1936 45-D-9891

h_R = Elevation head in reservoir in feet above sea level.

h_f = Total friction loss from reservoir to needle valve, in feet of water.

$\frac{V^2}{2g}$ = Velocity head in branch penstock leading to needle valve, in feet.

820.0 = Elevation of center of needle valves above sea level.

Values of $(h_f + \frac{V^2}{2g} + 820.0)$ are plotted against the total discharge on figure 53A for any needle valve combination. The pressure head immediately above the needle valves for a particular discharge and valve combination can be obtained by subtracting the value given on figure 53A from the elevation of the water surface in the reservoir. For example, the pressure immediately upstream from the needle valves for a flow of 10,000 second-feet through six valves with the reservoir at elevation 1180 would be 334 feet of water, or for this same discharge through four needle valves, the pressure above these valves would be 308 feet of water.

Pressure gages will probably be installed above the prototype needle valves as a means of determining the discharge through them before the turbines are placed in operation. One gage in each outlet house would be sufficient providing the valve to which it was attached was always one of the combination in operation. With the adjacent valves open the same amount, the pressure at each would be the same for all practical consideration. By operating the valves according to the diagram shown on figure 53A, it should be possible to obtain the desired discharge through the valves with reasonable accuracy.

It was not possible to make the surface roughness in the model penstocks to scale so the friction loss in the model was probably somewhat higher than the corresponding friction in the prototype. This means that the pressure obtained from figure 53A will be slightly lower for a given discharge than that which will actually exist on the prototype. It is immaterial whether the flow is through the upper, the lower or both gates of the intake tower as the losses from this source are practically negligible when compared with the total loss through the penstock.

The above procedure offers a method for measuring the flow through the Arizona canyon wall needle valves before the turbines are operated. This calibration will not be correct when the turbines are in operation. A method previously described in chapter V, section 1-I offers a method for measuring the total discharge through each intake tower, and as it is intended to install flow measuring devices in each turbine penstock, the needle valve discharge will consist of the difference of the two for the condition in which both turbines and needle valves are operating simultaneously. The discharge through the tunnel plug outlet works can be determined in the same manner.

Flow combinations with both turbines and needle valves operating were tried on the model but the variables were so numerous that there was no way of presenting them in a concise form in this report.

F. Conclusions

It is felt that the method employed in analyzing the model results on the intake tower and penstock assembly was accurate and the results dependable insofar as the model data were concerned. It was desired, however, to extrapolate the model results to apply for prototype values as this was one of the objects in performing the experiments. With a single small model of this type it is not possible to accurately interpret the conditions that will prevail in the prototype.

The extrapolation from model to prototype has, in the foregoing graphs, been made according to Froude's law for lack of a more accurate and definite method. As a result, the losses, as plotted with respect to the prototype discharge, are larger than those that will actually exist on the prototype. It is predicted that the losses in the full-size structure will probably range from 5 to 20 percent less than those indicated by the graphs. For this reason, it is advisable to refer to the prototype values as shown on the graphs, not as absolute, but approximate values.

3. INTAKE TOWER ELECTRIC-ANALOGY STUDIES

A. Introduction

The electric-analogy method previously used in the analysis of problems of seepage through earth dams and under masonry dams on porous foundations[12] was applied as a means of determining the direction

[12] "The Flow Net and the Electric Analogy" by E. W. Lane, F. B. Campbell, and W. H. Price, U.S.B.R. Tech. Memo. No. 388, or Civil Engineering, October 1934.

of flow of water entering the intake tower, and incidentally, to study the merits of the method itself. In this particular problem, it was necessary to obtain a certain amount of information from the hydraulic model before the analogy model could be properly set up.

B. The Apparatus

The apparatus consisted of a shallow glass tray 39 inches long, 18 inches wide, and 3 inches deep equipped with a leveling screw at each corner. A radial section of the tower constructed of redwood on a scale of 1:100 was cemented to the bottom of the tray. Copper plate electrodes were placed in position in the tray and a solution of sodium chloride was used as a conductor. A drawing of the apparatus is shown on figure 54 and a photograph of the tray in a vertical position is shown on plate VIII. The current used in the experiments was obtained from a 110-volt, 60-cycle source. By inserting a bank of lamps in series with the apparatus, as shown on figure 54, the potential across the electrodes was reduced to about 20 volts. On one side of the tray, a high-resistance wire, one meter in length, was stretched on a meter stick and connected in parallel with the circuit to constitute a Wheatstone bridge. A spring was attached to one end of the high-resistance wire to keep it taut at all times, as the current raised the temperature of the wire and lengthened it. A metal terminal on one end of the meter stick made a continuous contact with the stretched wire and kept the length connected in the circuit at exactly one meter, regardless of expansion or contraction. A circuit was established from a sliding contact on the high-resistance wire to the liquid conductor by a wire which had connected in series with it a set of head phones and a probing pencil. All connections in the apparatus were made with heavy copper wire of very low resistance.

The radial section of the tower was represented by setting the tray on a slope with the electrolyte at zero depth on the center line of the tower, as shown in section A-A, figure 54. The variables in the experiment were the length of the trash racks, the gate combinations, and the positions of the electrodes. The apparatus on figure 54 represents a section of the tower nearest the river with both gates open. The long electrode was connected to one side of the circuit and the two small electrodes in the tower were connected to the other side. The lines resembling contours represent points of equal potential. The purpose of the apparatus was to determine the position of these lines for different percentages of the total potential drop across the electrodes.

The position of any particular potential line was determined by setting the sliding contact on the resistance wire at the point giving the desired potential drop, and moving the probing pencil about in the salt solution until a point was reached at which the absence of a hum in the head phones indicated that no current was flowing through them. To find the position of the potential line representing two percent of the drop between electrodes, the sliding contact would be set at a point on the resistance wire representing two percent of its length, or two centimeters on the meter stick, and the probing pencil moved about in the electrolyte until the alternating-current hum in the head phones faded. This indicated that the probing

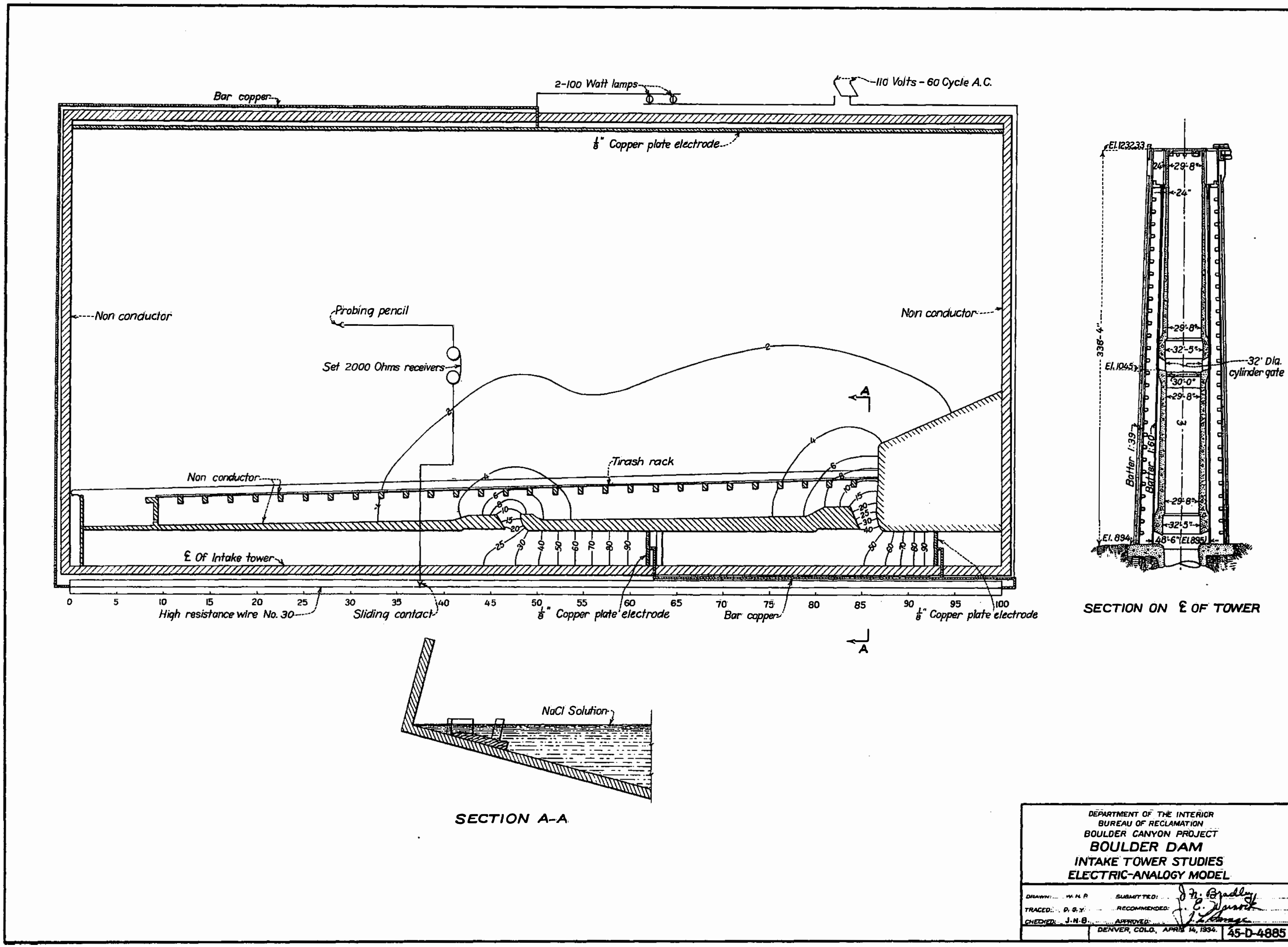

2-100 Watt lamps
110 Volts - 60 Cycle A.C.
Bar copper
1/8" Copper plate electrode
Non conductor
Probing pencil
Set 2000 Ohms receivers
Non conductor
Trash rack
Non conductor
℄ Of Intake tower
0 5 10 15 20 25 30 35 40 45 50 55 60 65 70 75 80 85 90 95 100
High resistance wire No. 30
Sliding contact
1/8" Copper plate electrode
Bar copper
1/8" Copper plate electrode
A
A
NaCl Solution
SECTION A-A
El.1232.33
24"
29'-8"
24"
338'-4"
29'-8"
32'-5"
El.1045
32' Dia. cylinder gate
30'-0"
29'-8"
Batter 1:39
Batter 1:60
29'-8"
32'-5"
El.894
48'-6"
El.895
SECTION ON ℄ OF TOWER
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM
INTAKE TOWER STUDIES
ELECTRIC-ANALOGY MODEL
DRAWN: W.H.R.
TRACED: D.G.Y.
CHECKED: J.N.B.
SUBMITTED: J. N. Bradley
RECOMMENDED: E. W. Lane
APPROVED: J. L. Savage
DENVER, COLO., APRIL 14, 1934.
45-D-4885

FIGURE 64

pencil was at a point in the solution where the potential drop was two percent of the total drop across electrodes. Other points would be located in a similar way with the same setting of the sliding contact until a sufficient number was found to draw the two percent line. Other potential lines would be located in a similar manner.

C. Tests on a Section of Tower Located Nearest the River

The first tests were made on a radial section of the tower located nearest the river with the electrodes in the positions shown on figure 54. These tests were made with three different lengths of trash rack: 306 feet, 182 feet, and two 50-foot racks. The resulting equipotential lines are shown on figures 55 and 56.

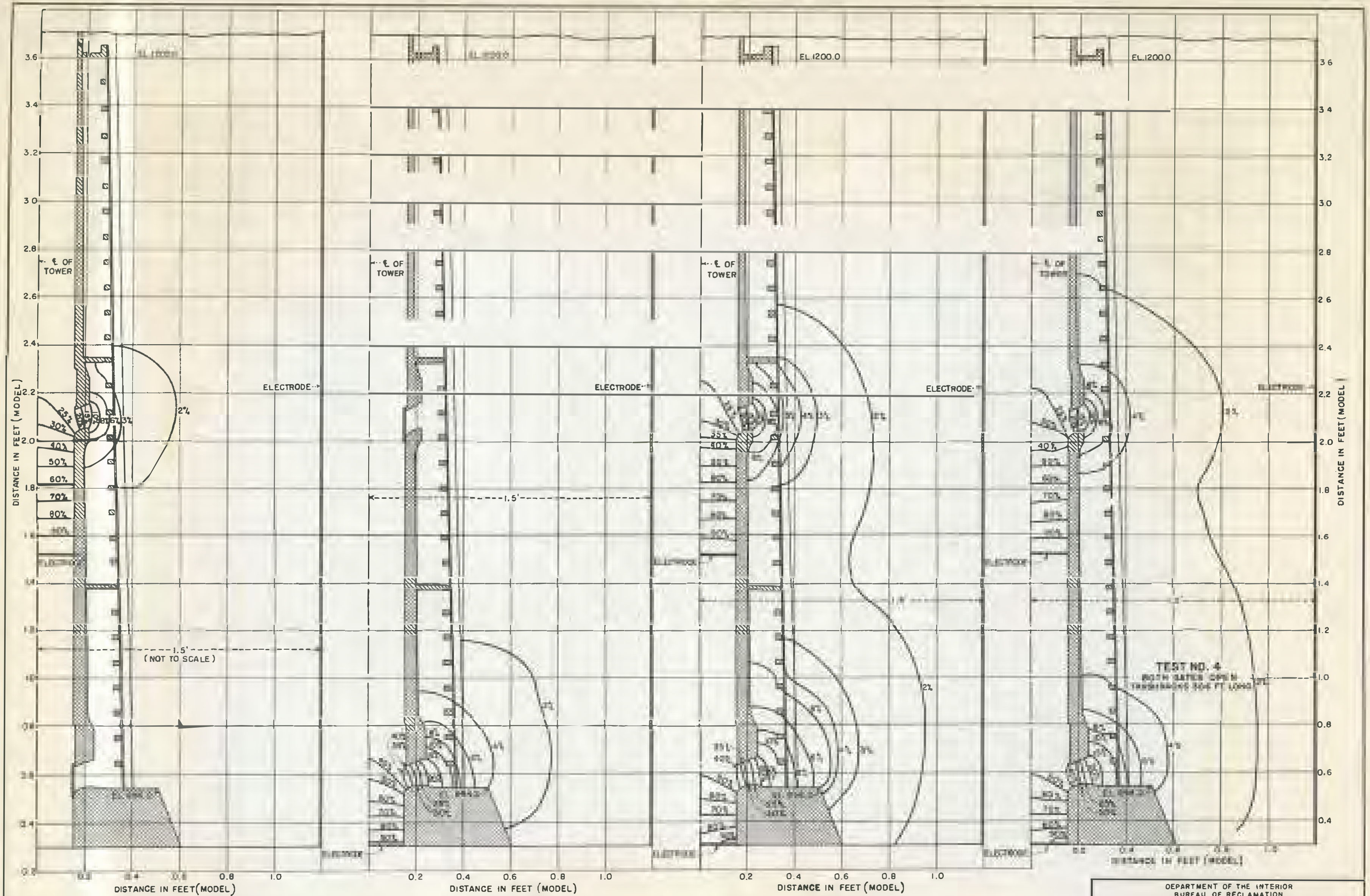
Comparing tests 2 and 6, (figures 55 and 56), in which the upper gate was closed and the trash-rack lengths were 50 feet and 306 feet, respectively, the corresponding equipotential lines practically coincide, which indicates that the total potential drop between electrodes agree closely. This means that the resistance to flow through the 50-foot rack was no greater than that through the 306-foot rack.

Tests 1 and 5 (figures 55 and 56) were made under the same conditions, except that the lower gate was closed and the upper gate open. Again the potential lines show a very close agreement, which indicates that the loss was no greater through the 50-foot rack than through the 306-foot rack.

Tests 3 and 4 (figure 55) were made on the same segment of the tower with both gates open. The potential lines with the two 50-foot racks agree very well with those for the 306-foot rack. The agreement of the potential lines is not as close for the range from one to five percent because the points for these lines were more difficult to locate than those for the larger potential drops.

The experiments thus far indicate that with the racks free from trash, a length of 50 feet in front of each gate would provide sufficient rack area on the river side of the tower.

Tests 7 and 8 (figure 56) were made on the same radial section of the tower with a 182-foot trash rack as in the final design. These, however, cannot be directly compared with the tests previously described, as the positions of the electrodes were shifted and the tilt of the tray was changed. They are included in this report as a matter of record.



TEST NO. 1
LOWER GATE CLOSED
50 FT SECTION OF TRASHRACK OVER UPPER GATE

TEST NO. 2
UPPER GATES CLOSED
50 FT. SECTION OF TRASHRACK OVER LOWER GATE

TEST NO. 3
BOTH GATES OPEN
50 FT. SECTION OF TRASHRACK OVER EACH GATE

SCALE OF MODEL = 1:100
DEPTH OF ELECTROLYTE AT 1.0 FOOT FROM ℄ = 0.138 INCHES

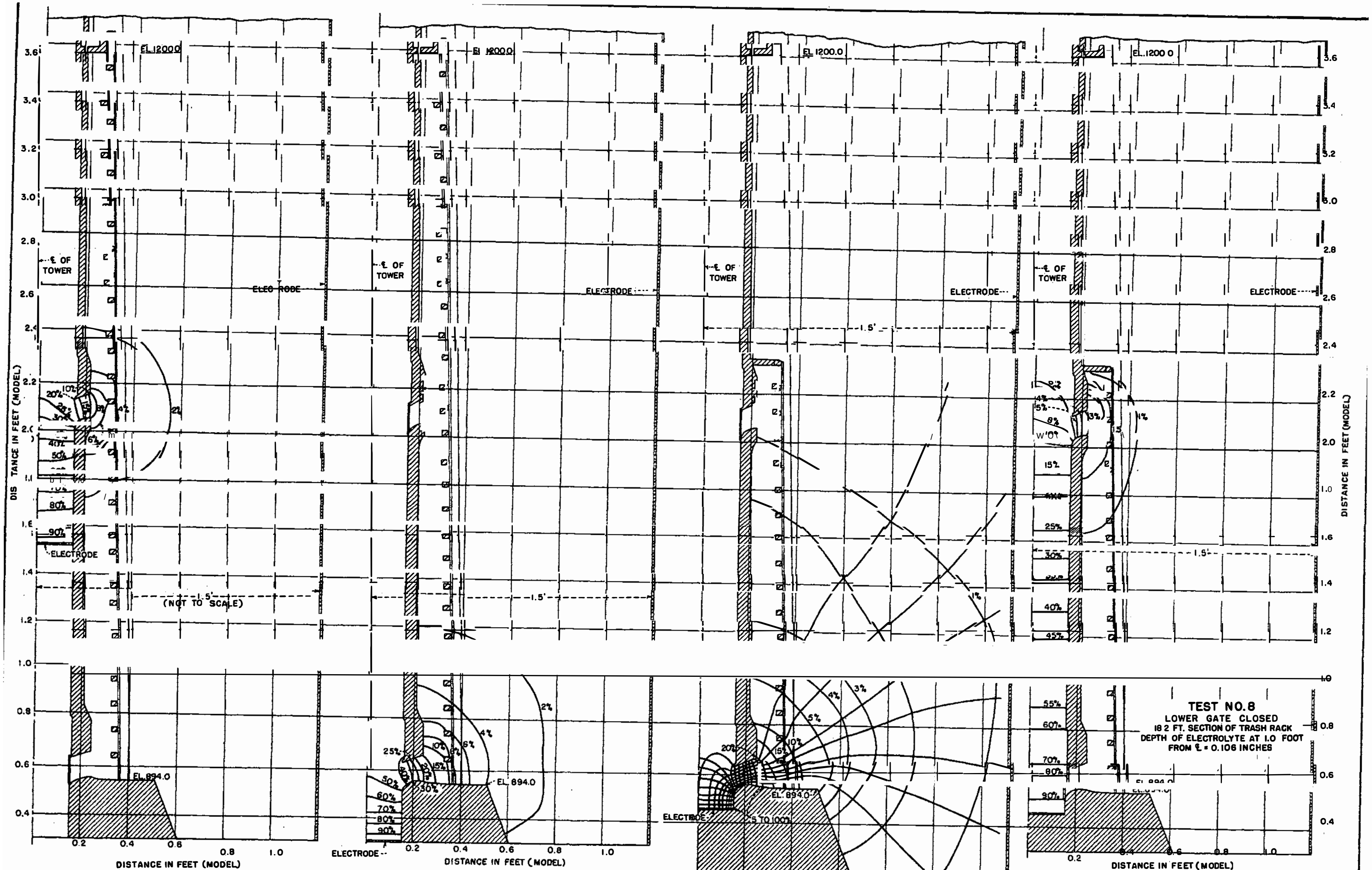
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS

INTAKE TOWER STUDIES

ELECTRIC ANALOGY RESULTS ON RADIAL SECTION OF TOWER NEAREST THE RIVER

DRAWN T.A.J. SUBMITTED
TRACED H.S. RECOMMENDED
CHECKED J.N.B. APPROVED
DENVER, COLO., APRIL 1934 45-D-9728

FIGURE 54



TEST NO. 5
LOWER GATE CLOSED
306 FT. SECTION OF TRASH RACK
DEPTH OF ELECTROLYTE AT 1.0 FOOT FROM ℄ = 0.138 INCHES

TEST NO. 6
UPPER GATE CLOSED
306 FT. SECTION OF TRASH RACK
DEPTH OF ELECTROLYTE AT 1.0 FOOT FROM ℄ = 0.138 INCHES

TEST NO. 7
UPPER GATE CLOSED
182 FT. SECTION OF TRASH RACK
DEPTH OF ELECTROLYTE AT 1.0 FOOT FROM ℄ = 0.106 INCHES

TEST NO. 8
LOWER GATE CLOSED
182 FT. SECTION OF TRASH RACK
DEPTH OF ELECTROLYTE AT 1.0 FOOT FROM ℄ = 0.106 INCHES

SCALE OF MODEL = 1:100

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS
INTAKE TOWER STUDIES
ELECTRIC ANALOGY RESULTS ON A RADIAL SECTION OF TOWER NEAREST THE RIVER

DRAWN T.A.J. SUBMITTED J. L. Bradley
TRACED S.P.C.B. H.G. RECOMMENDED J. E. Warnock
CHECKED J.H.D. APPROVED J. L. Savage
DENVER, COLO., APRIL 9, 1934
45-D-9729

FIGURE 56

D. Tests on a Section of Tower Located Nearest the Canyon Wall

A similar set of tests (figures 57 and 58) was made on a radial section of the tower located on the side nearest the canyon wall (plate VIII). Tests 11 and 14 (figures 57 and 58), made with the upper gate closed, using two 50-foot racks and one 306-foot rack, respectively, show a difference in the positions of the potential lines. This indicates that the total potential drop across the electrodes for the 50-foot racks was greater than that for the 306-foot rack. The increase is due to the new boundary conditions which reduce the area of approach and change the direction of the flow to the gates. With this decrease in approach area, there must be an increase of velocity to maintain a continuity of flow. As velocity is proportional to potential gradient, the gradient must increase as the trash racks and gate openings are approached. This increase was indicated by a reduction in the distance between equipotential lines. Closely spaced potential lines indicate flow concentrations and likewise high velocities.

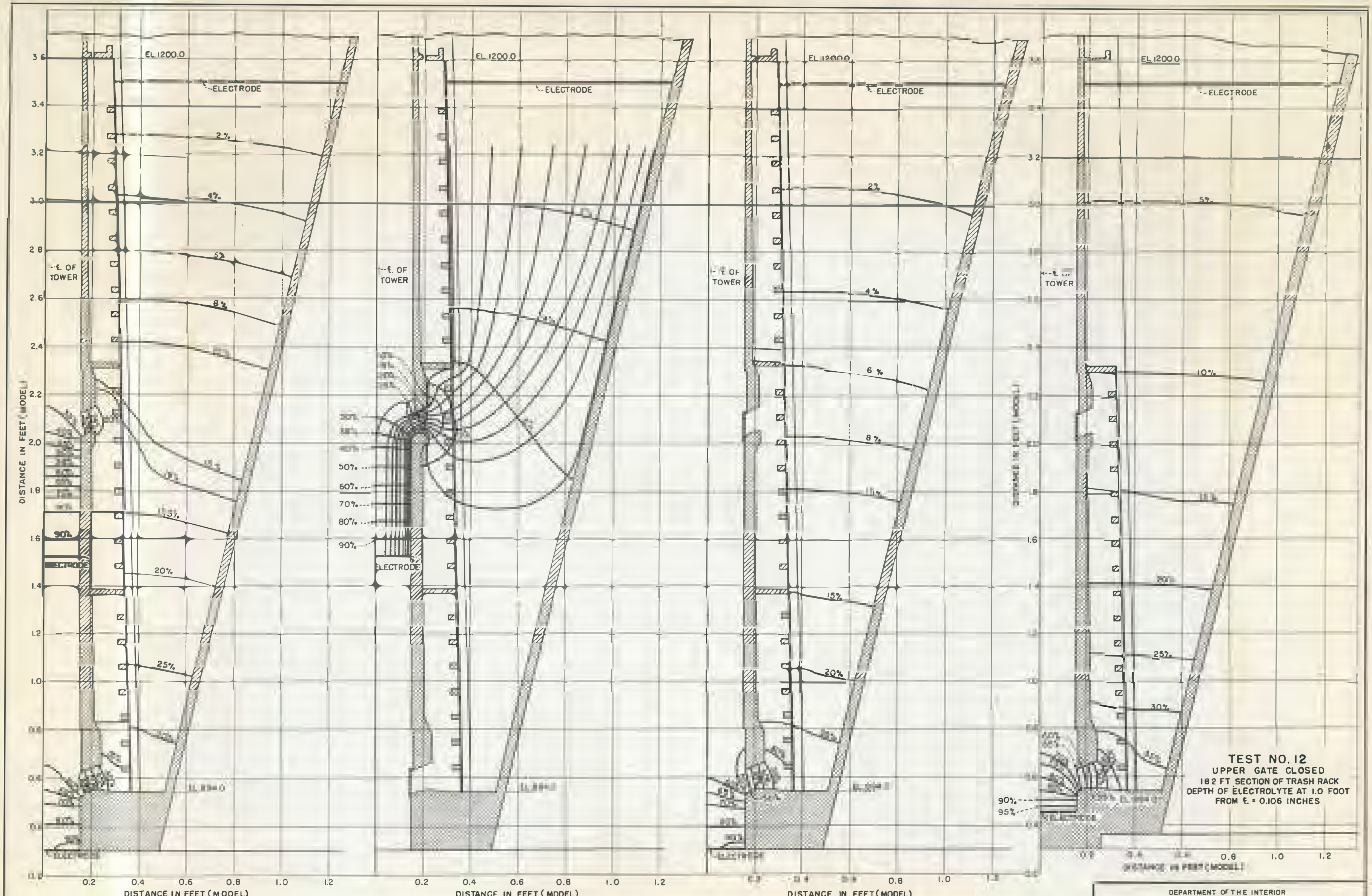
Tests 10 and 13 (figures 57 and 58) were made with the upper gate open and the lower gate closed, using two 50-foot racks and one 306-foot rack, respectively. A noticeable increase in the resistivity of the circuit was again witnessed when the 306-foot rack was replaced by the two 50-foot racks.

Tests 9 and 15 (figures 57 and 58) were made with both gates open, using two 50-foot racks and one 306-foot rack, respectively. There was little difference in the positions of the potential lines for the two setups. This could logically be expected as the same discharge was divided between the two gates and the effective rack area was doubled in the case of the two 50-foot racks. With one gate open, flow could occur only through one 50-foot rack.

Tests 12 and 16 (figures57 and 58) represent flow through the upper and lower gate, respectively, with 182-foot trash racks as finally designed. These results, however, cannot be directly compared with the others as the tilt of the tray was changed and the positions of the electrodes shifted.

The losses shown by the second set of experiments are exceptionally high, as flow conditions are not truly represented. In the experimental setup, all water was assumed to flow downward on the canyon-wall side of the tower, while actually a large portion will flow around the tower.

Flow nets have been drawn for tests 7, 10, and 15 (figures 56, 57, and 58). According to hydrodynamics, the flow lines should cross the potential lines at right angles. The volumes bounded by the



TEST NO. 9
BOTH GATES OPEN
50 FT LENGTH OF TRASH RACK OVER EACH GATE
DEPTH OF ELECTROLYTE AT 1.0 FOOT FROM ℄ = 0.138 INCHES

TEST NO. 10
LOWER GATE CLOSED
50 FT. LENGTH OF TRASH RACK OVER EACH GATE
DEPTH OF ELECTROLYTE AT 1.0 FOOT FROM ℄ = 0.138 INCHES

SCALE OF MODEL = 1:100

TEST NO. 11
UPPER GATE CLOSED
50 FT. LENGTH OF TRASH RACK OVER EACH GATE
DEPTH OF ELECTROLYTE AT 1.0 FOOT FROM ℄ = 0.138 INCHES

DEPARTMENT OF THE INTERIOR
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BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS

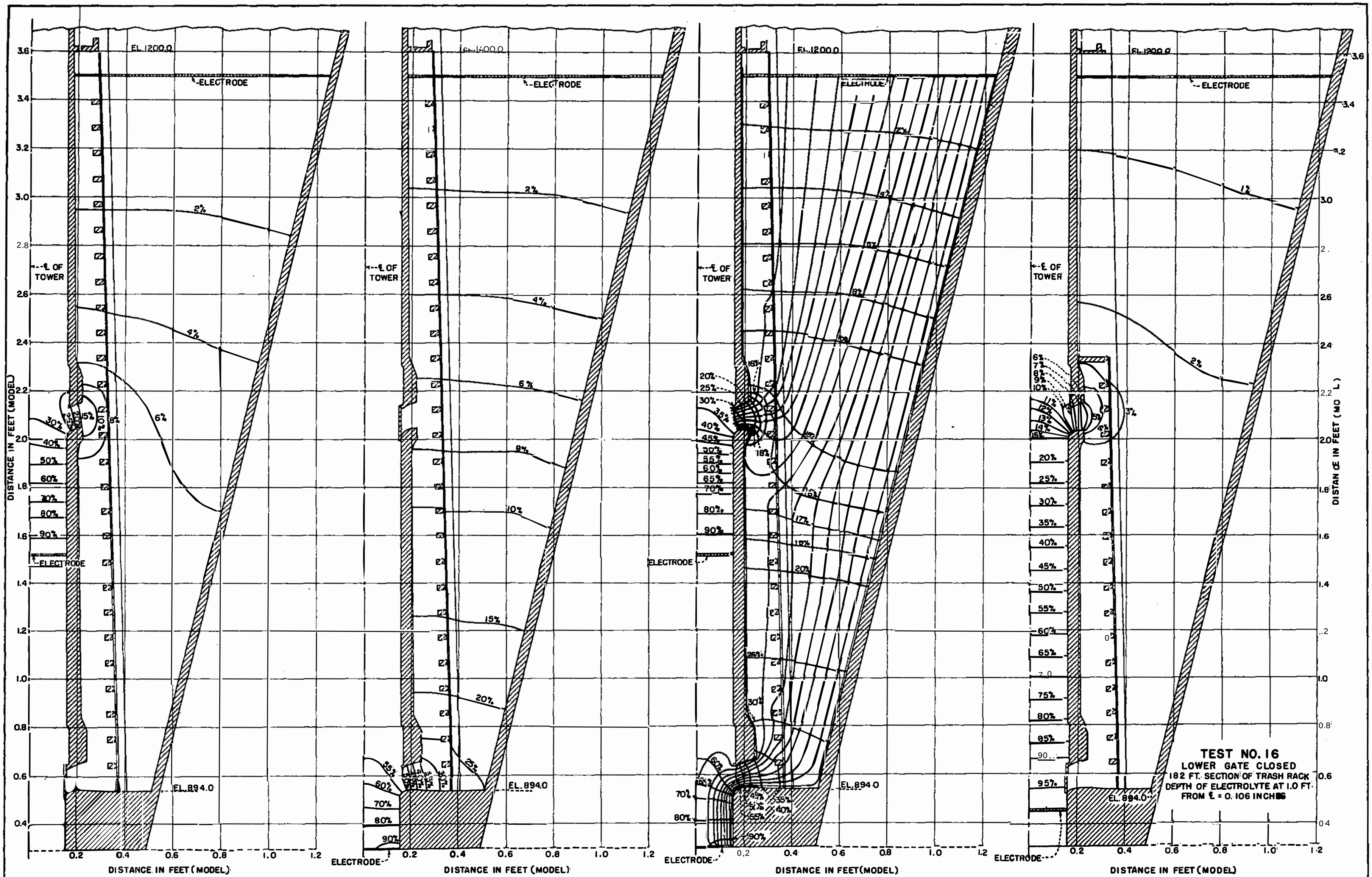
INTAKE TOWER STUDIES

ELECTRIC ANALOGY RESULTS ON A RADIAL SECTION OF TOWER NEAREST CANYON WALL

DRAWN T.A.J. SUBMITTED
TRACED R.E. RECOMMENDED
CHECKED I.H.B. APPROVED

45-D-9730

FIGURE 57



TEST NO. 13
LOWER GATE CLOSED
306 FT. LENGTH OF TRASH RACK
DEPTH OF ELECTROLYTE AT 1.0 FOOT FROM ℄ = 0.138 INCHES

TEST NO. 14
UPPER GATE CLOSED
306 FT. LENGTH OF TRASH RACK
DEPTH OF ELECTROLYTE AT 1.0 FOOT FROM ℄ = 0.138 INCHES

SCALE OF MODEL = 1:100

TEST NO. 15
BOTH GATES OPEN
306 FT. LENGTH OF TRASH RACK
DEPTH OF ELECTROLYTE AT 1.0 FOOT FROM ℄ = 0.138 INCHES

DEPARTMENT OF THE INTERIOR
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BOULDER CANYON PROJECT
BOULDER DAM HYDRAULIC EXPERIMENTS

INTAKE TOWER STUDIES

ELECTRIC ANALOGY RESULTS ON A RADIAL SECTION OF TOWER NEAREST CANYON WALL

DRAWN. T.A.J. SUBMITTED. J. N. Bradley
TRACED. G.P.C.B. H.S. RECOMMENDED. J. E. Warnock
CHECKED. J.M.B. APPROVED. [signature]
DENVER, COLO., APRIL 13, 1934 45-D-9731

FIGURE 58

flow lines may be considered as stream tubes, each carrying an equal quantity of water. To establish one end of each flow line, it was assumed that the velocity was constant across the electrode sections inside the tower. The electrodes were then divided into segments which, if revolved, would form annular rings. From these, the flow lines were projected and drawn perpendicular to the equipotential lines.

In the two-dimensional flow net where the electrolyte is constant in depth, the dimensions of a rectangle formed by the net bears a constant ratio to every other rectangle in the net by which the average velocity in each stream tube can be easily determined. For the flow nets drawn for tests 7, 10, and 15 (figures 56, 57, and 58), this ratio does not exist. With a sloping tray, the flow net is conditioned by a third dimension. The stream tubes in the three-dimensional net have no parallel sides and it is not possible to obtain the velocities directly from the length or breadth of the rectangles as in the two-dimensional system. The purpose in drawing the flow lines in tests 7, 10, and 15 was merely to indicate the direction of flow. It is possible, however, to obtain the velocities in these three-dimensional nets by a simple but laborious method.

There are eight stream tubes in test 7 (figure 56) each carrying an equal quantity of water. The discharge through each tube is equal to one-eighth of the total discharge flowing into this radial segment of the tower. Figure 34F shows a portion of a stream tube in a two-dimensional net, and figure 34G, a portion of a stream tube in a three-dimensional net. In the first, d is a constant value, while in the second, it is a variable. The velocity at any point in either stream tube is $V = \frac{q}{wd}$, where q is the discharge through the stream tube. If the velocity is known at any point in the two-dimensional net, it is only necessary to measure w to obtain the velocity at any other point. In the three-dimensional net, d, which is a variable, but is known at all points, must also be considered in computing the velocities.

The greatest source of error in obtaining velocities in a three-dimensional flow net of this type is not in the computation, but in the construction of the net. It is usually necessary to make one or two assumptions before attempting to draw a net, and after these are made and the net commenced, it is still necessary to use a certain amount of judgment as the potential lines are the only definite guides.

In test 15 (figure 58), there are seven stream tubes entering the upper gate and eight entering the lower, each carrying an equal discharge. This would indicate that 47 percent of the flow was passing through the upper gate and 53 percent through the lower. On the opposite side of the tower, where the area of approach is not restricted, the proportion of the total flow through the upper gate should be less, with the result that a greater percentage of the total flow should pass through the lower gate. To obtain the correct proportion of flow through the upper gate, the electrodes in the tower were shifted by trial until the indicated flow through each gate agreed with that measured on the hydraulic model. The above results show that the electric-analogy method is applicable for certain phases of the intake tower problem. Considerable information can be obtained from the flow lines regarding trash-rack areas and obstructions to flow.

E. Conclusions on Electric-Analogy Studies

The outstanding advantages in using the electric-analogy method, where applicable, is its simplicity, its speed in obtaining results, and its low cost. It is important, however, in interpreting the results of a study of this nature to keep in mind the limitations of the method. The symmetry and precision of the results are a temptation to extend the method at the expense of factors which cannot be considered in the apparatus.

- - -

VI. TESTS ON TUNNEL-PLUG OUTLET

1. TUNNEL-PLUG OUTLET MODEL

A. Introduction

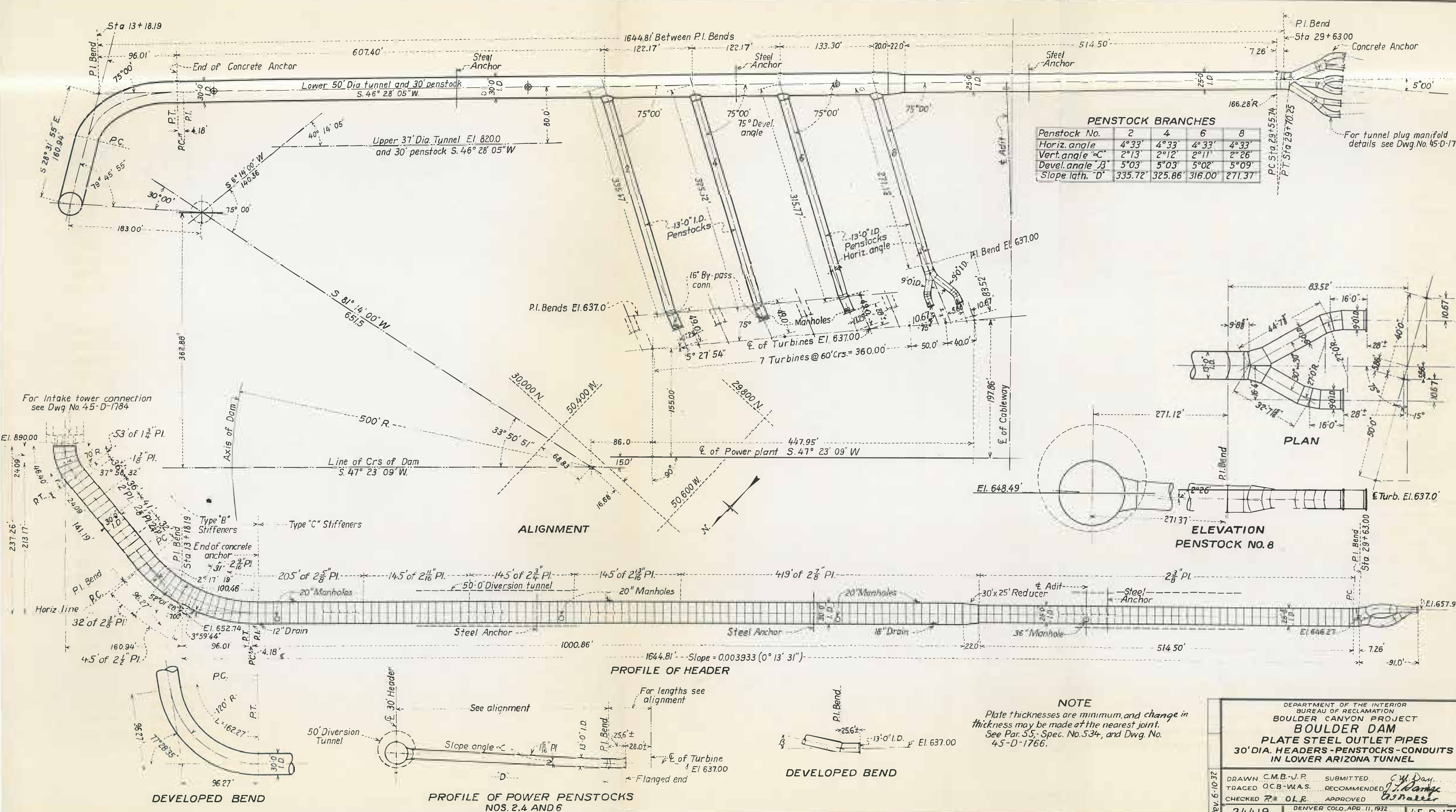
It is intended to discharge all of the surplus water at the Boulder Dam through the needle valves rather than to permit it to flow over the spillways. This is necessary since 9,000,000 acre-feet of storage is to be held in reserve for flood control, and water can be discharged over the spillways only when this space is, to a large extent, filled. The needle valves will at times, therefore, discharge large quantities of water over long periods. This will be particularly true after the reservoir is first filled and before the power demand requires the full stream flow.

The needle valves in the canyon-wall outlet works will discharge into the open air, while those in the diversion tunnels have been located a considerable distance from the tunnel outlets to effect appreciable economy in the cost of the steel-plate outlet pipe and to avoid the disadvantages of large quantities of spray in the vicinity of the powerhouse and the high voltage switching facilities.

Since the needle valves are located a considerable distance from the tunnel outlet, they will discharge into a closed space. As the combined discharge of the six valves in each tunnel would be as much as 21,400 second-feet with a maximum head of 454 feet, the energy of the water would reach 1,100,000 horsepower, and great care is necessary that no damage would result. Six 72-inch needle valves are required in each outlet (figs. 59 and 60), each valve being capable of discharging a maximum of 3,670 second-feet at a velocity of about 175 feet per second. It is important that the six valves in each outlet be arranged so that the jets will not be concentrated at one point and so that disturbances in the tunnel will be avoided as far as possible.

It was at first expected that tunnels would be necessary to supply air behind the valves to replace that ejected by the streams of water from the valves. Experience with other installations of the bureau had demonstrated that in certain cases this was absolutely necessary.

An extensive series of tests were conducted on three different model scales to determine the action of the jets from the needle valves in the tunnels and to determine the feasibility of air vent tunnels. The action of the three models was very similar and the results of the tests permitted the selection of an arrangement that



PENSTOCK BRANCHES

Penstock No.	2	4	6	8
Horiz. angle	4°33'	4°33'	4°33'	4°33'
Vert. angle "∝C"	2°13'	2°12'	2°11'	2°26'
Devel. angle "β"	5°03'	5°03'	5°02'	5°09'
Slope lgth. "D"	335.72'	325.86'	316.00'	271.37'

NOTE

Plate thicknesses are minimum, and change in thickness may be made at the nearest joint. See Par. 55, Spec. No. 534, and Dwg. No. 45-D-1766.

DEPARTMENT OF THE INTERIOR
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BOULDER CANYON PROJECT

BOULDER DAM

PLATE STEEL OUTLET PIPES

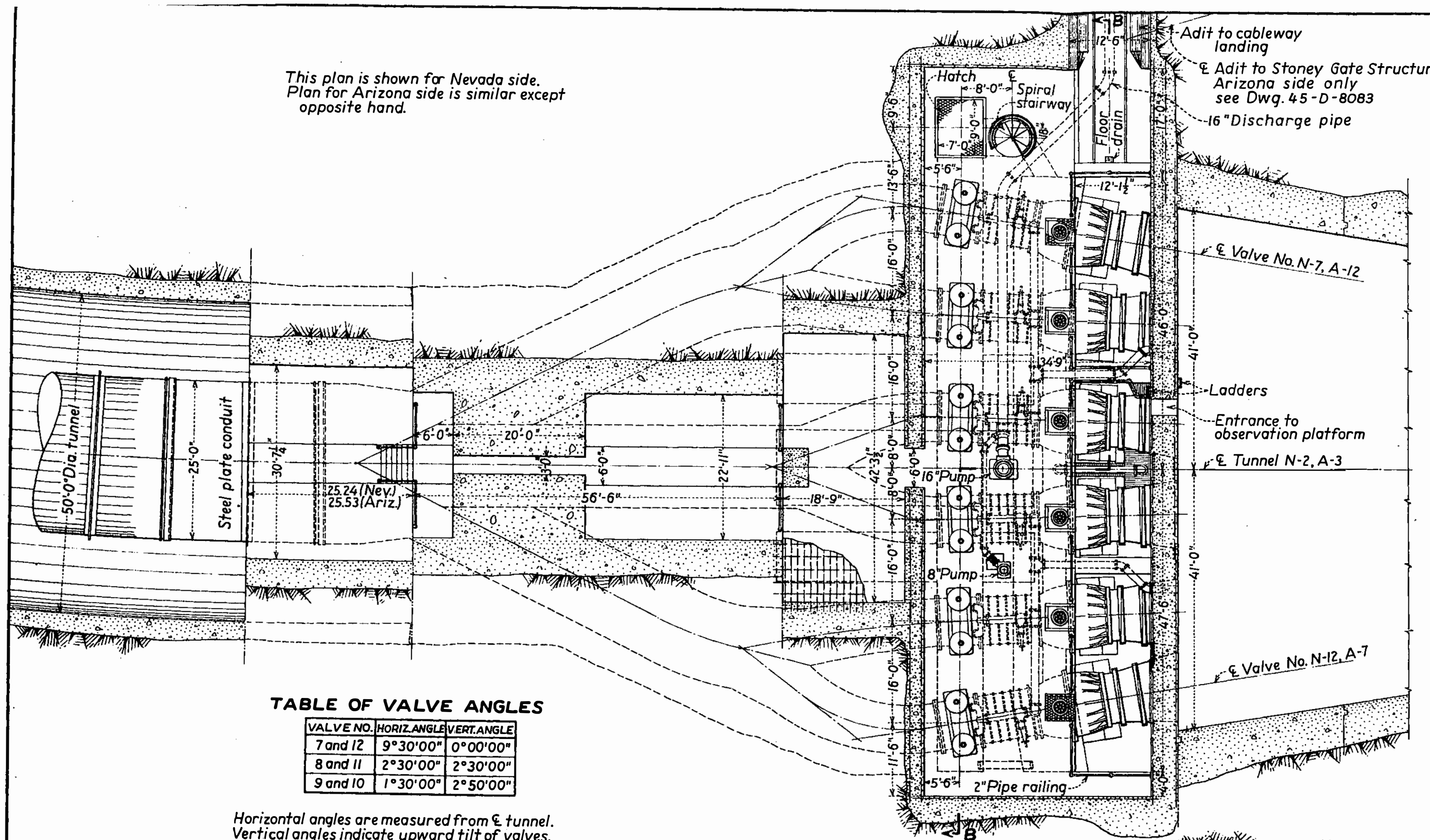
30' DIA. HEADERS - PENSTOCKS - CONDUITS IN LOWER ARIZONA TUNNEL

DRAWN C.M.B.-J.P. SUBMITTED C. W. Day
TRACED O.C.B.-W.A.S. RECOMMENDED J. L. Savage
CHECKED P. O.L.R. APPROVED A. J. Walter

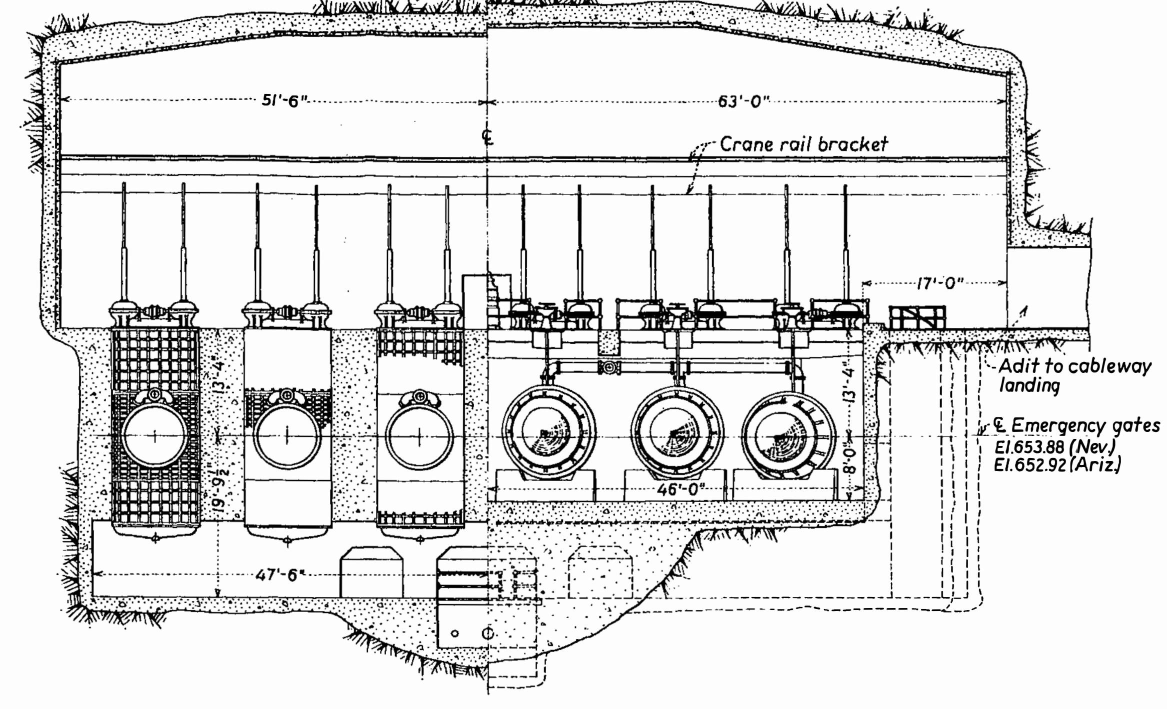
Rev. 6-10-32

24419 | DENVER COLO., APR. 11, 1932 SHEET 15 OF 19 | 45-D-1781

DRAWING No. 1



SECTION A-A



SECTION B-B

FOR ARIZONA VALVE CHAMBER - SEE DWG. 45-D-3473

TABLE OF VALVE ANGLES

VALVE NO.	HORIZ. ANGLE	VERT. ANGLE
7 and 12	9°30'00"	0°00'00"
8 and 11	2°30'00"	2°30'00"
9 and 10	1°30'00"	2°50'00"

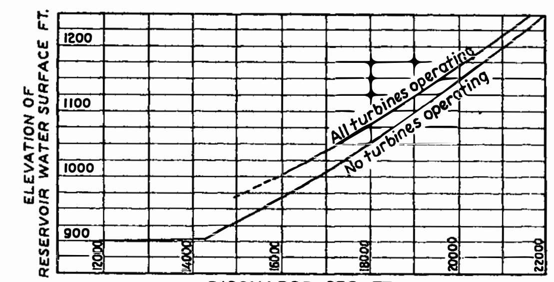
Horizontal angles are measured from ℄ tunnel. Vertical angles indicate upward tilt of valves. Angular adjustments to be accomplished by rotating adjusting wedges inserted between inlet flange of needle valve and flange of conduit lining.

NOTE

Emergency gate air vent piping except that shown as embedded in concrete shall not be installed at this time.

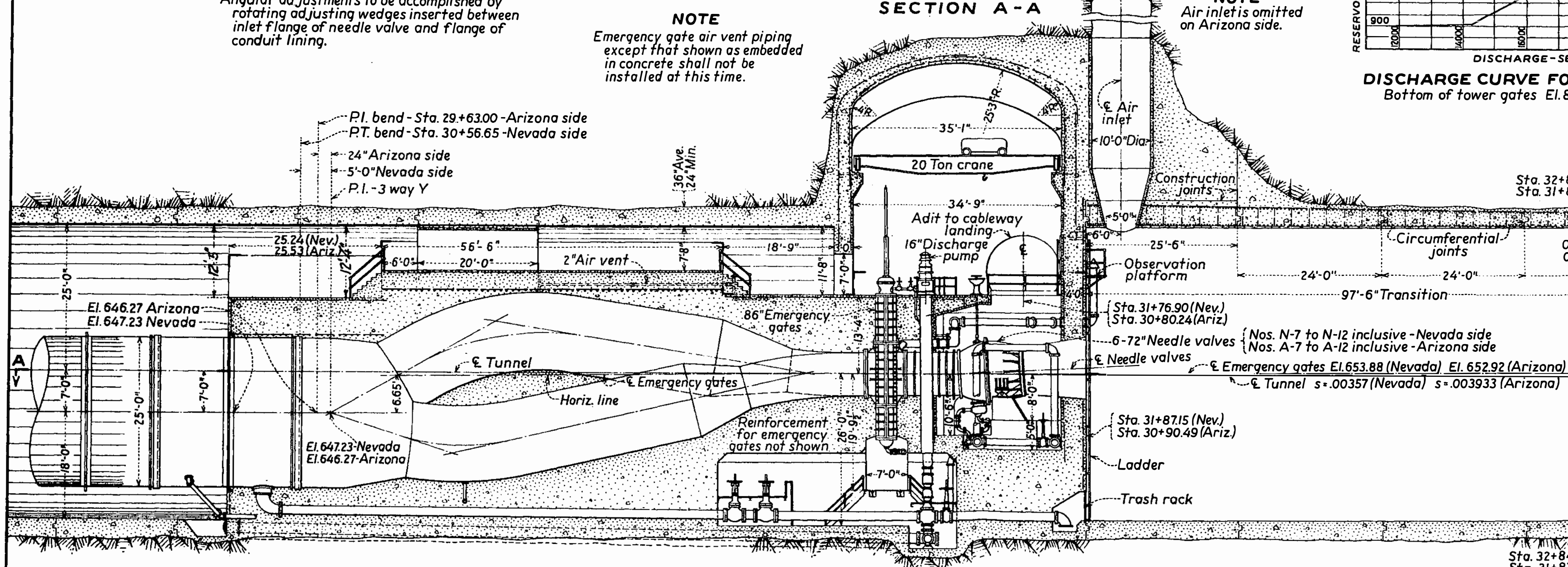
NOTE

Air inlet is omitted on Arizona side.



DISCHARGE CURVE FOR 6-72" VALVES

Bottom of tower gates El. 895.00 - 0 Discharge



SECTIONAL ELEVATION

REFERENCE DRAWINGS

DOWNSTREAM TUNNEL PLUG OUTLETS - GENERAL PLAN NEVADA POWER INSTALLATION	45-D-3806
DOWNSTREAM TUNNEL PLUG OUTLETS - GENERAL PLAN ARIZONA POWER INSTALLATION	45-D-3810
NEVADA DOWNSTREAM PLUG OUTLETS - TUNNEL ENLARGE. DETAILS	45-D-3470
NEVADA DOWNSTREAM PLUG OUTLETS - VALVE CHAMBER EXCAV. DETAILS	45-D-3471
ARIZONA DOWNSTREAM PLUG OUTLETS - TUNNEL ENLARGE. DETAILS	45-D-3472
ARIZONA DOWNSTREAM PLUG OUTLETS - VALVE CHAMBER EXCAV. DETAILS	45-D-3473
DOWNSTREAM PLUG OUTLETS - TRANSITION ARCH - CONCR. & REINF. DETAILS	45-D-4163
DOWNSTREAM PLUG OUTLET WORKS - ROCK GROUTING & DRAIN. SYSTEM	45-D-4168
DOWNST'M PLUG OUTLET WORKS - VALVE CHAMBER - CONCRETE DETAILS BELOW PEDESTAL FLOOR	45-D-4169
DOWNST'M PLUG OUTLET WORKS - VALVE CHAMBER - CONCRETE DETAILS AT PEDESTAL FLOOR	45-D-4170
DOWNSTREAM PLUG OUTLET WORKS - VALVE CHAMBER INTERNAL DRAINAGE DETAILS	45-D-4171
DOWNSTREAM TUNNEL PLUG OUTLETS - REINF. FOR 86" EMERGENCY GATES	45-D-4172 & 45-D-4173
DOWNSTREAM TUNNEL PLUG OUTLETS - VALVE	45-D-4174
DOWNSTREAM TUNNEL PLUG OUTLET WORKS - ARIZONA VALVE CHAMBER - DETAILS OF ROOF, BACK AND END WALLS	45-D-4175
DOWNSTREAM TUNNEL PLUG OUTLET WORKS - NEVADA VALVE CHAMBER - DETAILS OF ROOF, BACK AND END WALLS	45-D-4176
DOWNSTREAM PLUG OUTLET WORKS - ARIZONA VALVE CHAMBER - DETAILS OF FRONT WALL	45-D-4177 & 45-D-4178
DOWNSTREAM PLUG OUTLET WORKS - NEVADA VALVE CHAMBER - DETAILS OF FRONT WALL	45-D-4179 & 45-D-4180
DOWNSTREAM PLUG OUTLET WORKS - CONCRETE ANCHORAGE FOR WYES - ARIZONA SIDE	45-D-4181
DOWNSTREAM PLUG OUTLET WORKS - CONCRETE ANCHORAGE FOR WYES - NEVADA SIDE	45-D-4182
DOWNSTREAM PLUG OUTLET WORKS - SUMP AND PUMP PIT - EXCAVATION DETAILS	45-D-7704
NEVADA DOWNSTREAM PLUG OUTLET WORKS - CONSTRUCTION PROGRAM	45-D-7705
DOWNSTREAM PLUG OUTLET WORKS - REINF. BENDING DIAGRAM	45-D-7707
ARIZONA DOWNSTREAM PLUG OUTLET WORKS - CONSTRUCTION PROGRAM	45-D-7708
CONTROL PIPING - GENERAL ARRANGEMENT - ARIZONA SIDE	45-D-8050
CONTROL PIPING - GENERAL ARRANGEMENT - NEVADA SIDE	45-D-8051
CONTROL PIPING - GENERAL ARRANGEMENT - SECTIONAL ELEVATIONS	45-D-8052
DOWNSTREAM PLUG OUTLET WORKS - UPSTREAM BULKHEAD DOOR AND PASSAGEWAY - CONCRETE AND REINFORCEMENT DETAILS	45-D-7709

THIS DRAWING SUPERSEDES DWG. 45-D-3469

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT

BOULDER DAM
DOWNSTREAM PLUG OUTLET WORKS
GENERAL PLAN

DRAWN: A.F.W. SUBMITTED: W. E. Blomgren
TRACED: V.D.-C.C.H. RECOMMENDED: J. L. Savage
CHECKED: W.J.K. APPROVED: R. F. Walter

REV. 5-13-35 9-27-35

26264 DENVER, COLO. JAN 21, 1935 45-D-7706 minimized the flow disturbance and the erosive tendencies. The results also showed that the comparatively large air-vent tunnels that had been planned were not necessary, thus saving an appreciable amount of the initial cost.

B. Results on the 1:106.2 Model

The first model on a scale of 1:106.2 was constructed and tested in the Fort Collins laboratory with the arrangement of the needle valves as proposed by the design department. The model (plate 9A and B) was built so that the needle valves could be adjusted both horizontally and vertically, and the tunnel was made of pyralin to allow visual study of the flow conditions in the tunnel. The spacing and angularity of the needle valves as originally proposed was as follows:

Valve Number	Distance of Center Line of Emergency Gates Above Tunnel Invert	Spacing of Emergency Gates	Valve Angle in Relation to Tunnel Centers: Horizontal	Valve Angle in Relation to Tunnel Centers: Vertical
1 and 6	30 feet	16 feet	11° 57'	0° 00'
2 and 5	30 feet	16 feet	7° 14'	0° 00'
3 and 4	30 feet	16 feet	2° 25'	0° 00'

The needle valves were represented in this model by conical nozzles that produced smooth jets which gave little indication as to the action of the jets as aspirators. However, the impact of the jets in the 50-foot diversion tunnel and the resulting turbulence was unsatisfactory. The flow conditions in the tunnel with the original design (plate 9C and D) were such that a large fin formed in the center of the tunnel and the wave on each side carried up the side practically to the top. It was easy to picture the action that could occur in the prototype where a dense spray would be formed by the mixture of air and water.

Slight changes made in the angles of the needle valves with relation to the center line of the 50-foot diversion tunnel produced much better flow conditions (plate 9E and F). The angles and spacings of the valves in the most satisfactory set-up in the initial tests on the 1:106.2 model were as follows:

Valve Number	Distance of Center Line of Emergency Gates Above Tunnel Invert	Spacing of Emergency Gates	Valve Angle in Relation to Tunnel Centers	
			Horizontal	Vertical
1 and 6	20.45 feet	16.041 ft.	10° 25'	0° 00'
2 and 5	20.18 feet	16.041 ft.	4° 44'	0° 08'
3 and 4	21.28 feet	16.041 ft.	2° 26'	1° 48'

The vertical angle indicates that those valves were tilted upward with relation to the center line of the tunnel. The improvement obtained was so encouraging that the decision was made to continue the studies in a similar manner on a larger scale model where the details of the prototype could be duplicated to better advantage.

C. Results on the 1:20 Model

The large model was built on a scale of 1:20 (plate 10) at the outdoor laboratory of the Bureau of Reclamation at Montrose, Colorado. The 50-foot concrete-lined tunnel was represented by a 30-inch wood-stave pipe and the needle valves were duplicated to scale in their full-open position and mounted on frames which permitted horizontal and vertical adjustments as in the 1:106.2 model. The tunnel-plug outlet transition was built as an airtight compartment fitted with a hinged cover.

The jets from the 1:20 model needle valves were rougher with respect to the scale ratio than those in the prototype due to the model needle valves being iron castings with a higher degree of roughness in proportion than the prototype. With the present state of development of model science, it cannot be definitely determined just how close the prototype was simulated, but the stream from the model valves appeared from visual observations to be quite similar to those from large needle valves now installed and in operation.

As in the 1:106.2 model, the action in the tunnel with the original position of the valves was extremely turbulent (plate 11). By adjusting the position of the needle valves, the conditions of the flow in the tunnel were improved (plate 12) and the results agreed closely with those on the 1:106.2 model. The positions of the valves with the most satisfactory set-up on the 1:20 model were as follows:

Valve Number	Distance of Center Line of Emergency Gates Above Tunnel Invert	Spacing of Emergency Gates	Valve Angle in Relation to Tunnel Centers	
			Horizontal	Vertical
1 and 6	25 feet	16 feet	9° 35' 39"	0° 00' 00"
2 and 5	25 feet	16 feet	2° 30' 27"	2° 28' 56"
3 and 4	25 feet	16 feet	1° 2' 41"	4° 19' 24"

The vertical angles indicate that those valves were tilted upward with relation to the center line of the tunnel.

In the most desirable set-up, the jets of water fall to the bottom of the tunnel at different locations so that the action is not concentrated at any one point. The jets combine and flow along the invert of the tunnel in a smooth stream with surprisingly little turbulence, leaving a free air space of about 85 percent of the cross-sectional area of the tunnel above the surface of the flowing water when only the tunnel-plug outlets are discharging. This action is not changed by completely closing the cover of the tunnel transition.

During the course of the experiments on both the 1:106.2 and the 1:20 models, considerable data were obtained. Eight different set-ups were studied in the initial tests of the 1:106.2 model in the Fort Collins laboratory and 14 on the 1:20 model at the Montrose laboratory. More or less complete data were taken on each set-up with the idea that it might be the most satisfactory. In this report only the initial, semifinal, and final set-ups are discussed.

After a satisfactory set-up was found for all six needle valves operating at full capacity, studies were made of the flow conditions in the tunnel with different combinations as illustrated on plate 13. Very poor conditions prevailed when one, two, three, or four valves on one side were discharging, while good conditions prevailed when either the two center or four center valves were discharging.

When the tests on the 1:20 model had been completed, the results were checked by duplicating the set-up on the 1:106.2 model as shown on plate 14.

D. Results on the 1:60 Model

Neither the 1:106.2 nor the 1:20 model had approach conditions similar to the prototype (fig. 60). Both had been built with a canvas hose connected to each needle valve and to a common water supply. The manifold and emergency gates were not incorporated in the smaller model because of its minute size and because of the preliminary nature of the tests. It had not been incorporated in the 1:20 model on account of the high cost of construction. As a result, a model on a scale of 1:60 (plate 15) was constructed and tested in the Fort Collins laboratory to study the effect of the manifold and emergency gates on the flow conditions.

The most satisfactory set-up derived from the 1:20 model tests was first studied (plate 16) and indications were that improvement could be accomplished by further adjustment of the horizontal and vertical angles. As a result, the following combination of valve positions was recommended as a final design (plate 17),

Valve Number	Distance of Center Line of Emergency Gates Above Tunnel Invert	Spacing of Emergency Gates	Valve Angle in Relation to Tunnel Centers	
			Horizontal	Vertical
1 and 6	25 feet	16 feet	9° 30'	0° 00'
2 and 5	25 feet	16 feet	2° 30'	2° 30'
3 and 4	25 feet	16 feet	1° 30'	2° 50'

The vertical angle indicates an upward tilt of the valves. It was recommended that a beveled washer be inserted between the inlet flange of the needle valve and the flange of the conduit lining to provide for the adjustment of the vertical and horizontal angles.

In evolving the correct position of the tunnel-plug outlet needle valves from a hydraulic standpoint, it was necessary to consider the mechanical limitations of any proposed arrangement. In the original design, all of the needle valves were level, in the same plane with their center lines 30 feet above the invert and 5 feet above the center line of the 50-foot tunnel, and spaced with a distance of 16 feet between center lines of the emergency gates.

The distance of 16 feet between center lines of emergency gates was the minimum because of the mechanical difficulties of assembly. In certain tests, very good hydraulic conditions in the tunnel were obtained with the outer valves inclined downward, but that was objectionable on account of the lifting reaction conditions, and in the recommended design practically the same hydraulic action was obtained by inclining the center valves upward, thereby eliminating the objectionable reaction.

E. Tunnel-Plug Needle Valve Operating Program

Since there will be a total of twelve needle valves in the tunnel-plug outlets in tunnel 2 (Nevada) and tunnel 3 (Arizona), it was believed to be desirable to determine the best procedure in opening those valves to obtain the most satisfactory flow conditions.

The good flow conditions in the 1:20 model with two and four center valves discharging (plate 13E and F) were checked on the 1:60 model (plate 18B, C, and D). The first combination was still satisfactory, but the second was too turbulent, and the combination of valves 1, 3, 4, and 6 operating together was tried and found to be better.

Other combinations were studied and the following order of operating the needle valves in the tunnel-plug outlets is suggested (the valves in each outlet are numbered as shown on figure 60):

a. One tunnel-plug outlet only in operation:
   1. Open two center valves (9 and 10)
   2. Open two outside valves (7 and 12)
   3. Open other two valves (8 and 11)

b. Both tunnel-plug outlets in operation:
   1. Open two center valves on Nevada side (N-9 and N-10)
   2. Open two center valves on Arizona side (A-9 and A-10)
   3. Open two outside valves on Nevada side (N-7 and N-12)
   4. Open two outside valves on Arizona side (A-7 and A-12)
   5. Open two remaining valves on Nevada side (N-8 and N-11)
   6. Open two remaining valves on Arizona side (A-8 and A-11)

The procedure of closing the valves should be the reverse of that of opening.

F. Coefficient of Discharge of Needle Valves

In the design of the needle valves for the canyon-wall and tunnel-plug outlets, the discharge was computed using the equation

$$Q = CA_2 \left(2gh_1 + V_1^{2}\right)^{\frac{1}{2}} \qquad (1)$$

where, Q = discharge quantity, second-feet

C = coefficient of discharge

A_2 = nominal area of discharge outlet, square feet

h_1 = pressure head immediately above valve (measured above center line), feet

$\frac{V_1^2}{2g}$ = head on valve due to velocity of approach, feet.

Equation (1) was obtained[13] from the equation for discharge

$$Q = A_2 V_2 \qquad (3)$$

[13] Schoder and Dawson "Hydraulics", pp. 136-139.

by applying Bernouilli's theorem between the approach and the outlet sections, in which case

$$h_1 = \frac{V_2^2}{2g} - \frac{V_1^2}{2g}$$

and
$$V_2 = \left[2g\left(h_1 + \frac{V_1^2}{2g}\right)\right]^{\frac{1}{2}}$$

or,
$$V_2 = (2gh_1 + V_1^2)^{\frac{1}{2}} \qquad (4)$$

Since there is a loss between the two points, the actual velocity, V_2, is less than the theoretical velocity. To correct for that loss a coefficient of discharge, C, must be applied to equation (4)

Then, $V_2 = C\ (2gh_1 + V_1^2)^{\frac{1}{2}}$ (5)

and,
$$Q = CA_2\ (2gh_1 + V_1^2)^{\frac{1}{2}} \qquad (6)$$

A value of the coefficient of discharge was assumed to be 0.725 in the design of the valves and a check was made of this value on the model of the tunnel-plug outlet built on a scale of 1 to 20 at the Montrose laboratory.

A mercury manometer was connected above each valve on the model to register the pressure head. The manometer was read simultaneously with the head on the 12-foot sharp-crested suppressed weir used to measure the flow through the model.

Representative runs for low, medium, and high heads were selected from tests 12 and 14 to determine the coefficient of discharge. Since the experimental needle valves were constructed in the full open position, the data will apply only to that condition. The values of the coefficient computed from the runs as shown on table IV are plotted on figure 61.

TABLE IV

Coefficient of Discharge for Needle Valves Based on Data from 1 to 20 Model

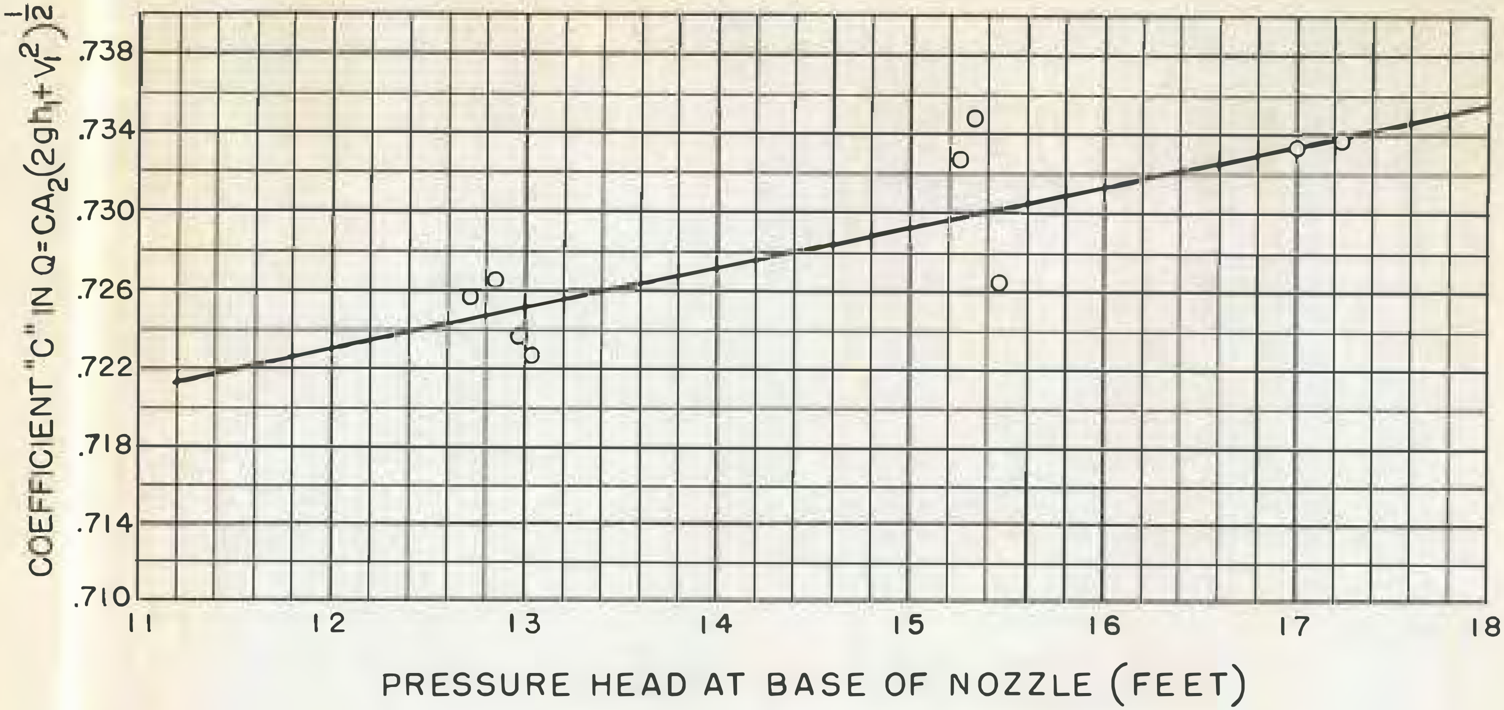
Test No.	Head Feet	A_1, Sq. Ft.	A_2, Sq. Ft.	Q_1, Sec.-Ft.	C
14-69	15.254	0.10083	0.07149	1.9052	0.7328
12-68	13.011	0.10083	0.07149	1.7269	0.7228
14-54	12.828	0.10083	0.07149	1.7269	0.7266
12-78	12.972	0.10083	0.07149	1.7270	0.7236
14-44	12.711	0.10083	0.07149	1.7163	0.7257
12-1	17.243	0.10083	0.07149	2.0291	0.7337
14-2	17.007	0.10083	0.07149	2.0138	0.7334
12-56	15.447	0.10083	0.07149	1.8950	0.7266
14-59	15.309	0.10083	0.07149	1.9160	0.7348

G. Air Demand Tests

One of the primary problems to be studied on the working models of the tunnel-plug outlets was the necessity of an air vent immediately above the needle-valve jets to relieve any low pressure in the tunnel which might develop, due to the high velocity jets acting as aspirators and withdrawing the air. In fact, besides the excessive cost of the initial installation, the air vents would be objectionable due to the large amount of spray or mist formed by the high velocity jets which would be carried out into the canyon near the powerhouses and the high-tension electrical facilities.

The model on a 1:20 scale at Montrose was completely enclosed and a vent installed which could be controlled. With different operating heads on the valves, the vacuum conditions were determined with the vent open and with the vent closed.

With the improved conditions in the 50-foot tunnel obtained by the adjustment of the positions of the needle valves, a large space was available between the water surface and the roof of the tunnel to supply the air needed to replace that carried away by the jets of high velocity water. When no air was supplied by an air vent, under normal tail-water conditions, the air in the tunnel



DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM
TUNNEL PLUG OUTLETS
NEEDLE VALVE DISCHARGE COEFFICIENT

DRAWN D.C.W. SUBMITTED J. N. Bradley
TRACED H.A.L.-H.S. RECOMMENDED
CHECKED J.E.W. APPROVED J. L. Savage

DENVER, COLO., MAR. 27, 1938 45-D-10147

FIGURE 61

circulated moving downstream near the water surface and upstream near the roof of the tunnel.

The maximum capacity of the Boulder Dam outlet works with the reservoir at elevation 1221.4 or the spillway gates completely raised has been estimated at 91,000 second-feet for the 24 needle valves and 30,560 second-feet for the powerhouse operating at full capacity. A total discharge of 121,560 second-feet will produce a tail water at the diversion-tunnel portals of elevation 669 (fig. 62) with the present conditions of the river bed. The roof of the tunnel at the portal is elevation 676, so that with the possible maximum discharge of 121,560 second-feet from the outlet works, there will be a segment of the tunnel approximately 6.3 feet high with a cross-sectional area of 143 square feet available to relieve the low pressure created in the tunnel. The proposed air vent was to be 10 feet in diameter, or an area of 78 square feet.

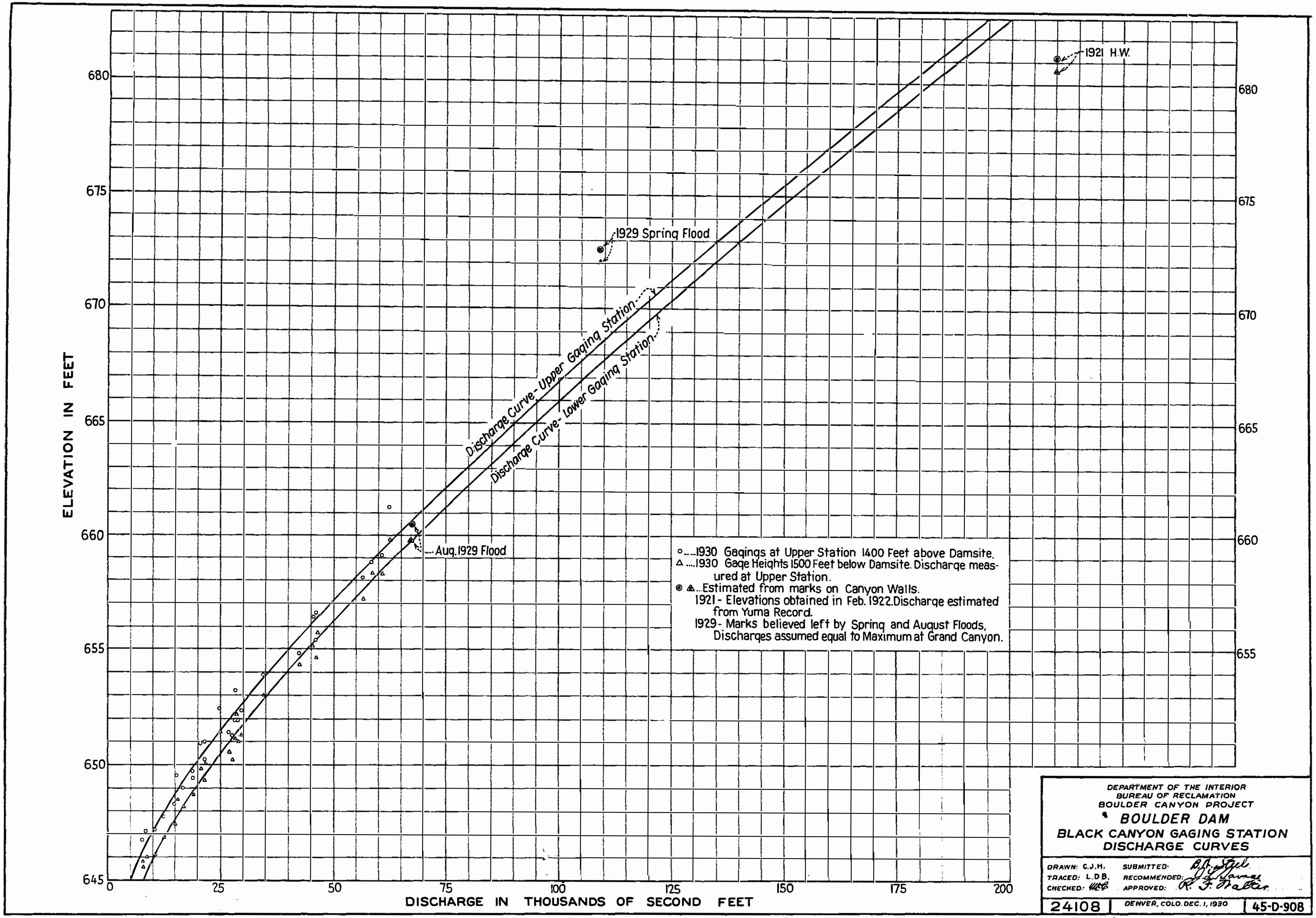
It is expected that the river bed below the dam will be gradually eroded by the clear water released from the reservoir and the loose material carried downstream so that in the future the tailwater at the portals will be lower than at the present time. With this possibility an even larger segment of the tunnel will be available for ventilation at a possible maximum discharge.

The only possibility of a larger flow in the river, and hence higher tail water, will be for the spillways to discharge. In such a case, the necessity of the operation of the tunnel-plug outlets will no longer exist and they can be closed.

The vacuum conditions on the 1:20 model related to the river elevation on the prototype are shown on figure 63. The curves show that a slight vacuum will exist for flow conditions in the river up to 70,000 second-feet regardless of the head on the valves and the installation of an air vent. The vacuum increases quite rapidly up to 130,000 second-feet, but not sufficiently to be of any serious consequence.

A slight increase of vacuum was noted with an increase of head on the valves, but the amount was so slight as to be negligible. The data plotted on figure 63 are from tests on the model using heads corresponding to 340 to 475 feet on the prototype.

The change of tail water is the controlling factor in the formation of the vacuum. When the tail water is low, there is sufficient room above the water surface in the tunnel for the air to flow into the space around the high-velocity jets and replace that being carried away. In that case, there would be very little demand for air through the vents. As the water reaches an elevation in the



DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT

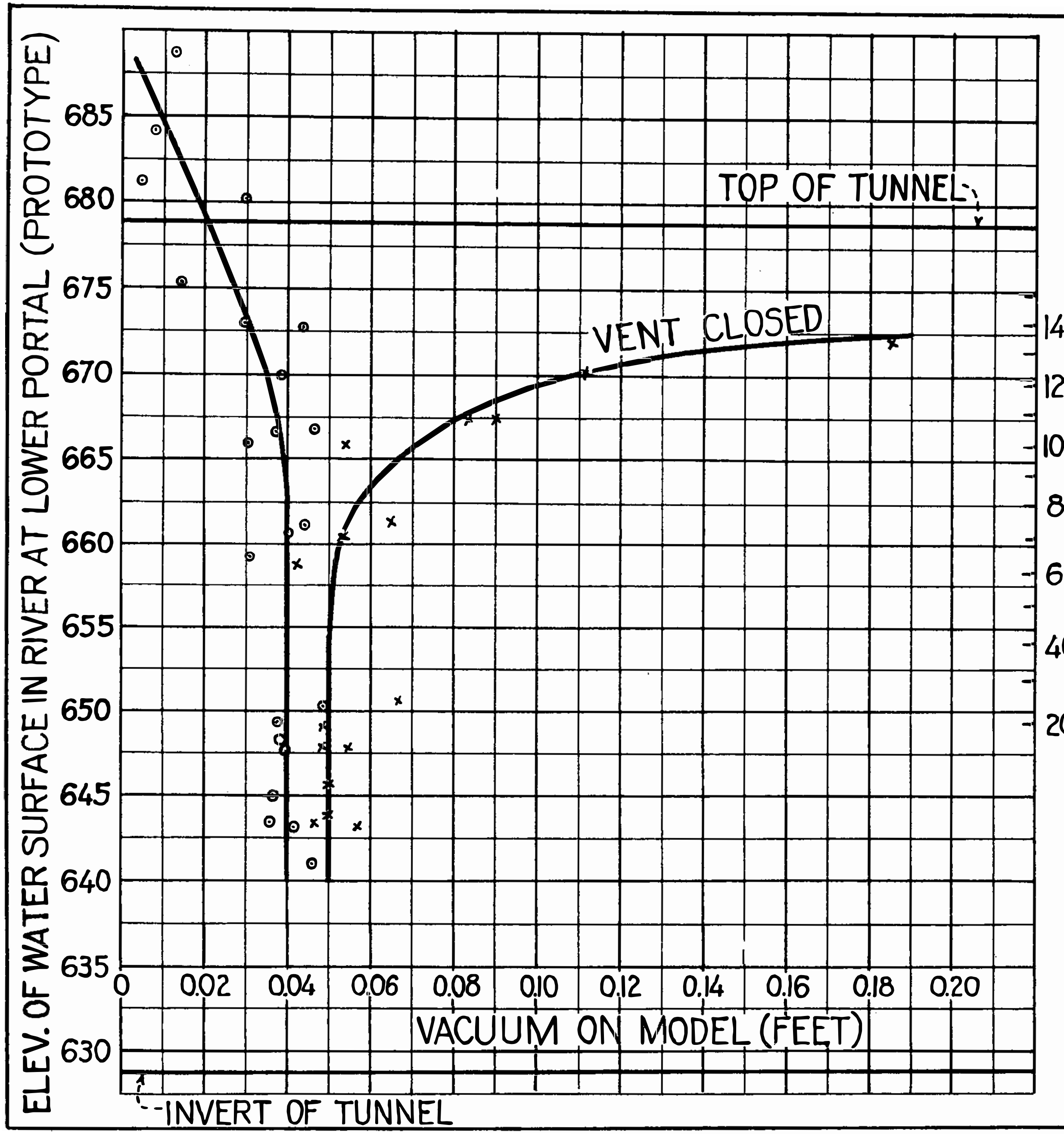
BOULDER DAM

BLACK CANYON GAGING STATION
DISCHARGE CURVES

DRAWN: C.J.H.	SUBMITTED:
TRACED: L.D.B.	RECOMMENDED:
CHECKED:	APPROVED: R. F. Walter

24108 | DENVER, COLO. DEC. 1, 1930 | 45-D-908

FIG. 62



DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT

BOULDER DAM
TUNNEL PLUG OUTLETS
RELATION OF RIVER ELEVATION TO PRESSURE IN TUNNEL

DRAWN J.D.M. SUBMITTED J. N. Bradley
TRACED J.V.M. RECOMMENDED J. E. Warnock
CHECKED J.E.W. APPROVED J. L. Savage

DENVER, COLO., NOV. 26, 1935 | 45-D-10148

FIGURE 63

tunnel where the space above is considerably diminished, either one of two things will occur. If no air vent is provided, the amount of the vacuum will be rapidly increased and the velocity of the in-bound air will be increased, or, if an air vent is provided, the demand for air will be supplied by it and the vacuum will not be increased. In the first case, as the air space in the tunnel decreases, the vacuum will increase because the amount of necessary air cannot be supplied fast enough. This condition will continue until the tunnel is completely filled at the end.

As previously mentioned, there are two movements of air flow in the tunnel. The boundary layer adjacent to the water surface will be moving in the same direction as the water and at the same or slightly less velocity, while the air near the roof of the tunnel will travel at a lower velocity in the opposite direction. The outward flow will be the last to disappear as the water surface approaches the tunnel roof at the portal, since the velocity of the water is high and would tend to draw the air out rather than let it in. As a result, the air demand on the vents would reach a maximum just before the water reached the top of the tunnel. As the tunnel is completely filled, the velocity of the water will be decreased and there would be a resulting decrease in the vacuum. This theory was actually substantiated by model observations. The most dangerous point then is when the tailwater is at or very near to the top of the tunnel.

Inasmuch as the present state of knowledge of the behavior of models in connection with the development of a vacuum and its resultant effects is as yet not sufficiently developed to be absolutely certain, it was decided to form the junction of the air tunnel in the concrete lining so that should a remote contingency arise that the air tunnels did prove to be necessary, they could be constructed without difficulty and at no greater cost than the original installation.

2. NEEDLE VALVE AND EMERGENCY GATE MODEL

During the course of experiments on the 1:60 model in the Fort Collins laboratory, the effect was studied of closing the paradox emergency gates with the needle valves completely open. It was discovered that when the paradox gates were nearly closed, a surging of the jets from the needle valves occurred, as shown on plate 18A.

To further study this phenomenon, one of the needle valves used on the Montrose 1:20 model of the tunne-plug outlet was mounted in its relative position with a model of an emergency gate and sufficient length of approach pipe.

The surging or pulsation on the 1:20 model was very similar to that encountered on the 1:60 model and is shown somewhat in detail on plate 19. These photographs disclose only instantaneous conditions. Actually, the jet oscillated with a fairly definite cycle in a vertical plane, and the results were best recorded by motion pictures.

Operation of the emergency gate without the needle valve in place showed that the stream of high-velocity water expanded and completely filled the end of the pipe. Critical examination disclosed low-velocity areas on each side and a high-velocity area in the center such that with the needle valve in place, a body of water would collect in the low-velocity pockets and be carried out by the high-velocity jet, causing surges or pulsations. At an opening of approximately 15 percent of the paradox gate, this surging caused a vibration of the entire model which could be felt distinctly at the end of the 20-foot approach pipe.

VII. CHANNEL CONDITIONS IN RIVER BELOW BOULDER DAM

1. RIVER MODEL

A. Introduction

In the early stages of design of the Boulder Dam with its various appurtenant structures, it was felt that a scale model of the river channel below the proposed site would be of value in determining the effect of the discharges from the powerhouses, the canyon-wall outlets, the tunnel-plug outlets and the spillway tunnels on the water surface elevations in front of the powerhouses and hence the effect on the operating head of the turbines.

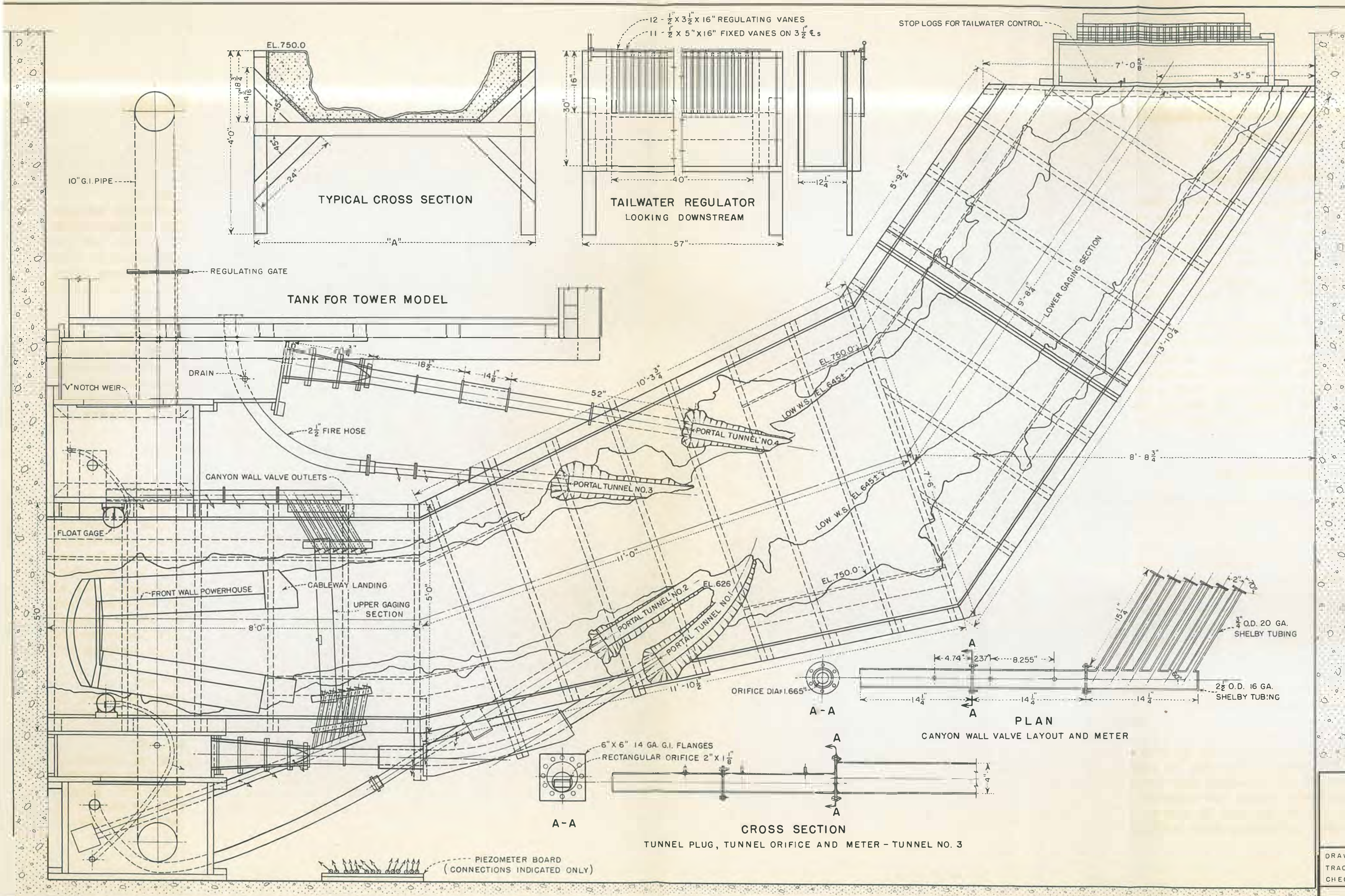
Unfortunately, the urgency for other more detailed studies and the lack of space in the hydraulic laboratory made it necessary to postpone those studies until the design and construction had progressed to such an extent that major changes were not possible. It is believed, however, that sufficient information of value to the operating staff was gained to justify the expense of the studies. The results so obtained are being presented with this in view.

B. Apparatus

The model (fig. 64) on a scale ratio of 1:150 was constructed in the hydraulic laboratory of the Colorado Agricultural Experiment Station, Fort Collins, Colorado. The outlet works were constructed in detail, except that the discharge from the powerhouse was introduced through a 90-degree V-notch weir placed between the upstream end of the powerhouses. Diaphragm orifices were installed in the conduits leading to the canyon-wall and tunnel-plug outlets for measuring the water through those structures, while 90-degree, V-notch weirs were so placed as to discharge into the models of the side-channel spillways, thereby introducing the correct quantities at the correct heads.

The diaphragm orifices and the V-notch weirs were calibrated in place by comparison with the master 90-degree V-notch weir in the measuring channel.

The topography of the river bed (plate 20) was constructed in detail by the use of galvanized iron cut to conform to the cross sections of the river and fastened in the wooden tank holding the model. The space between the cross sections was filled with damp sand to within an inch or two of the top of the metal and the remainder of the space was filled with concrete, into which the configuration



SECTION	"A"	REMARKS
1	60"	NO BATTER ON RT. SIDE
2	"	" " " " "
3	"	" " " " "
4	"	TYPICAL SECTION
5	"	" "
6	"	" "
7	"	RIGHT LEG 2"X 6"
8	64¼"	LT. LEG 2X6 NO BATTER RT.
9	68½"	RT. " " " " "
10	73"	LT. " " " " "
11	77¼"	" " " " " LT.
12	81½"	TYPICAL SECTION
13	86"	" "
14	90"	2"X 8" CROSS MEMBER
15	8¾"	NO LEG OR BRACES LT. SIDE
16	17½"	" " " " " "
17	73½"	TYPICAL SECTION
18	"	" "
19	"	" "
20	"	" "
21	57"	NO LEG OR BRACES LT. SIDE
22	31½"	" " " " " "

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
HYDRAULIC MODEL EXPERIMENTS

BOULDER DAM

RIVER MODEL — SCALE RATIO 1 TO 150

PLAN AND DETAILS OF MODEL

DRAWN. C.W.T. SUBMITTED
TRACED. H.A.L. H.S. RECOMMENDED
CHECKED. J.O.M. APPROVED

DENVER, COLO., JUNE 1, 1933 45-D-9732

of the river bed was detailed by hand, utilizing the metal cross sections as guides and the detail topography sheets as references. The bottom of the channel was filled with sand to conform to the elevation of the movable river bed. Stop-logs adjustable in height were placed at the lower end of the channel to hold the bed material to any given elevation, and a regulator consisting of fixed and movable vanes was constructed at the lower end to regulate the elevation of the water surface to any predetermined elevation. Between the stop-logs and the regulator, a sand box was built to trap the majority of the sand eroded from the river bed to prevent its being carried into the laboratory recirculating system.

Two gaging stations were installed on the model for regulating the river flow and measuring the slope of the water surface. One, known as the upper section, was installed to measure the elevation of the water surface directly upstream from the point at which the jets from the canyon wall outlets will impinge on the river bank, while the other station, known as the lower section, was located sufficiently far below the portals of the spillway tunnels to avoid interference from the tunnel jets.

C. River Conditions for Flow Combinations

During the course of these experiments, the flow data and conditions of the river bed were recorded by three different methods:

(1) Point gage readings were made at the stations above and below the outlet works to record the effect of the anticipated "ejector" action produced by the outlet works discharging into the river at an angle. These data have been consolidated and tabulated in table V where any flow combination and its action can be determined at a glance. The data in the columns entitled, "Rise and Drop" have been plotted on figure 65 against the total flow in the river for any specific run. Three conditions of tail water elevation were used, namely: normal, 10 feet below normal, and 20 feet below normal. In all of these tests, the lower gaging station was used as a control and the gage height for any given discharge was that obtained from the rating curve (fig. 66). constructed from gagings made prior to the start of the construction. That discharge-elevation relationship is referred to as "normal".

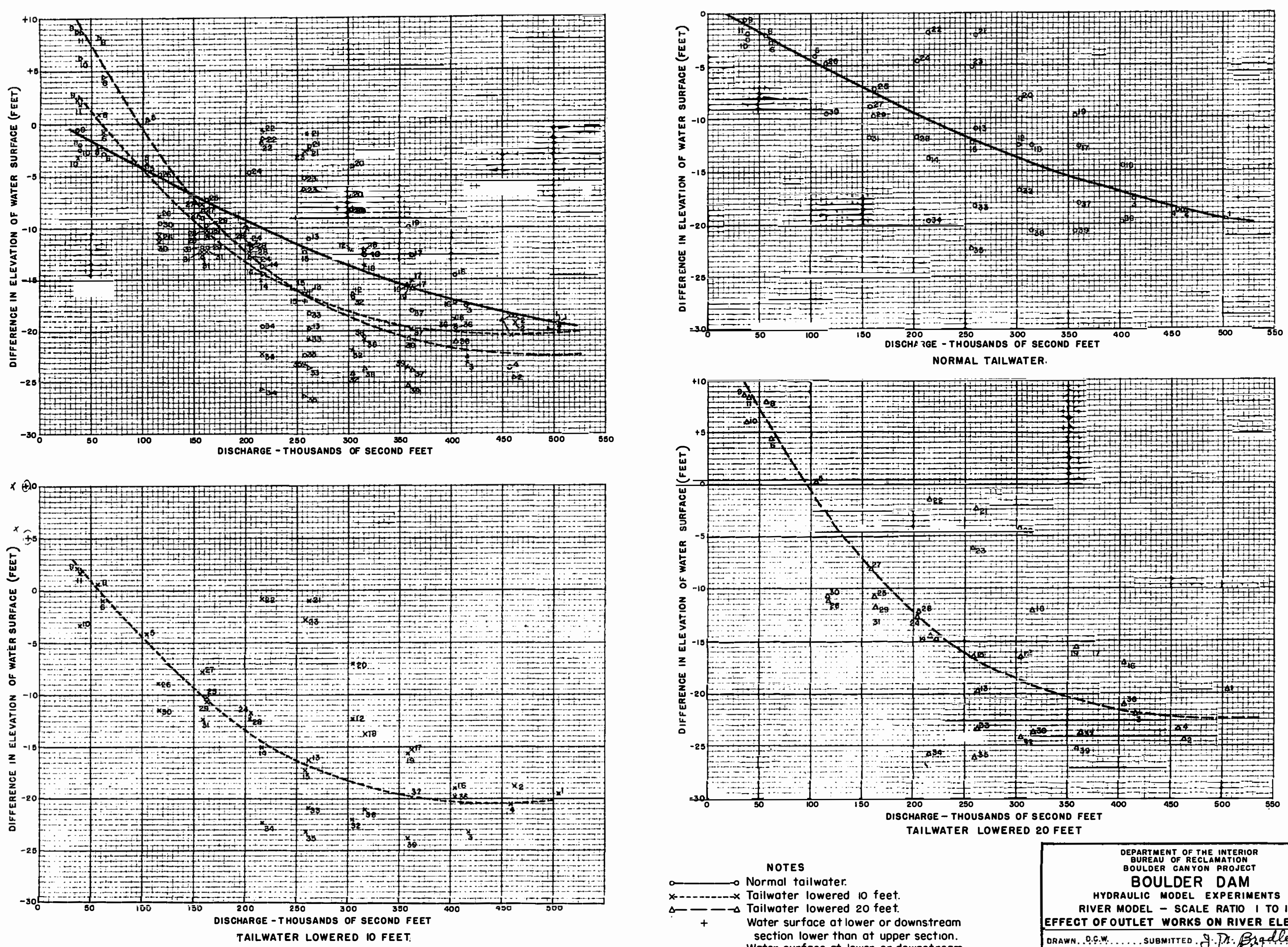
(2) Pictorial observations were made with still and motion picture equipment of the conditions for each typical flow combination. A portion of the still pictures have been included as plates 21 to 32, inclusive, to better illustrate the results.

RELATIONSHIP OF ELEVATION OF WATER SURFACE TO DISCHARGE WITH DIFFERENT OUTLET WORKS OPERATING COMBINATIONS AS DETERMINED ON 1:150 MODEL OF CHANNEL OF RIVER BELOW BOULDER DAM

Run No.	Location of Outlets	Discharge Second-Feet: Power House	Canyon Wall Outlets	Tunnel Plugs	Spillway	Total	Elevation of Water Surface in River: Upper Section: Normal	10 Feet Below Normal	20 Feet Below Normal	Rise	Drop	Lower Section: Normal	10 Feet Below Normal	20 Feet Below Normal
1	Arizona	15,000	21,000	23,000	200,000	503,000	708.47	698.95	690.23	19.26 19.23 19.77		727.73	718.18	710.00
	Nevada		21,000	23,000	200,000									
2	Arizona	15,000	0	23,000	200,000	461,000	704.22	693.71	676.67	18.66 18.99 24.54		722.88	712.70	701.21
	Nevada		0	23,000	200,000									
3	Arizona	15,000	0	0	200,000	415,000	694.75	677.17	667.71	17.46 23.13 21.99		712.21	700.30	689.70
	Nevada		0	0	200,000									
4	Arizona	15,000	21,000	0	200,000	457,000	702.98	689.81	678.88	18.54 20.49 23.40		721.52	710.30	702.28
	Nevada		21,000	0	200,000									
5	Arizona	15,000	21,000	23,000	0	103,000	662.74	652.18	649.30	4.08 3.99	0.21	666.82	656.17	649.09
	Nevada		21,000	23,000	0									
6	Arizona	15,000	0	23,000	0	61,000	655.64	647.92	643.50	2.88 0.87	4.38	658.22	648.79	639.12
	Nevada		0	23,000	0									
8	Arizona	15,000	21,000	0	0	57,000	655.37	648.78	645.87	2.13	0.75 8.07	657.50	648.03	637.80
	Nevada		21,000	0	0									
9	Arizona	15,000	10,500	0	0	36,000	652.92	645.82	639.76	0.57	2.19 8.58	653.49	643.63	631.18
	Nevada		10,500	0	0									
10	Arizona	15,000	0	23,000	0	38,000	651.30	637.39	637.27	2.49 3.24	6.09	653.79	640.63	631.18
	Nevada		0	0	0									
11	Arizona	15,000	0	0	0	38,000	652.39	645.30	642.31	2.01	1.77 8.52	654.40	643.63	633.79
	Nevada		0	23,000	0									
12	Arizona	15,000	21,000	23,000	100,000	303,000	686.67	677.63	662.87	12.60 12.24 16.53		699.27	689.87	679.40
	Nevada		21,000	23,000	100,000									
13	Arizona	15,000	0	23,000	100,000	261,000	681.42	666.13	653.23	11.01 16.23 19.77		692.43	682.36	673.00
	Nevada		0	23,000	100,000									
14	Arizona	15,000	0	0	100,000	215,000	671.84	661.15	651.09	13.77 14.91 14.52		685.61	676.06	665.61
	Nevada		0	0	100,000									
15	Arizona	15,000	21,000	0	100,000	257,000	679.92	664.72	655.59	12.36 17.19 16.32		692.28	681.91	671.91
	Nevada		21,000	0	100,000									
16	Arizona	15,000	21,000	23,000	200,000	403,000	698.84	684.19	677.33	14.55 18.84 17.22		713.39	703.03	694.55
	Nevada		21,000	23,000	100,000									
17	Arizona	15,000	0	23,000	200,000	361,000	694.65	682.38	665.10	12.63 15.12 15.93		707.28	697.50	681.30
	Nevada		0	23,000	100,000									
18	Arizona	15,000	0	0	200,000	315,000	687.86	676.78	668.32	12.60 13.68 12.18		700.46	690.46	680.50
	Nevada		0	0	100,000									
19	Arizona	15,000	21,000	0	200,000	357,000	697.25	680.64	671.52	9.87 15.69 15.60		707.12	696.33	687.12
	Nevada		21,000	0	100,000									
20	Arizona	15,000	21,000	23,000	200,000	303,000	690.72	681.65	675.31	8.28 6.99 4.24		699.00	688.64	679.55
	Nevada		21,000	23,000	0									
21	Arizona	15,000	0	23,000	200,000	261,000	690.72	681.37	670.66	2.19 0.99 2.34		699.00	688.64	673.00
	Nevada		0	23,000	0									
22	Arizona	15,000	0	0	200,000	215,000	683.96	674.86	664.23	1.80 0.75 1.53		685.76	675.61	665.76
	Nevada		0	0	0									
23	Arizona	15,000	21,000	0	200,000	257,000	686.02	679.66	665.29	5.19 2.70 6.21		691.21	682.36	671.50
	Nevada		21,000	0	0									
24	Arizona	15,000	21,000	23,000	100,000	203,000	679.15	661.60	650.61	4.68 11.79 12.72		683.83	673.39	663.33
	Nevada		21,000	23,000	0									
25	Arizona	15,000	0	23,000	100,000	161,000	669.41	655.98	656.23	7.26 10.23 10.83		676.67	666.21	667.06
	Nevada		0	23,000	0									
26	Arizona	15,000	0	0	100,000	115,000	663.51	649.56	637.02	4.98 8.97 11.01		668.49	658.53	648.03
	Nevada		0	0	0									
27	Arizona	15,000	21,000	0	100,000	157,000	666.91	657.40	647.55	9.00 7.65 8.19		675.91	665.05	655.74
	Nevada		21,000	0	0									
28	Arizona	15,000	21,000	23,000	0	203,000	671.92	661.85	649.73	11.76 12.12 12.09		683.68	673.97	661.82
	Nevada		21,000	23,000	100,000									
29	Arizona	15,000	0	23,000	0	161,000	667.10	656.26	644.68	9.87 10.41 11.94		676.97	666.67	65[illegible].62
	Nevada		0	23,000	100,000									
30	Arizona	15,000	0	0	0	115,000	658.98	647.22	636.72	9.51 11.31 10.86		666.49	658.53	647.58
	Nevada		0	0	100,000									
31	Arizona	15,000	21,000	0	0	157,000	664.09	653.37	643.37	11.82 12.29 12.81		675.91	665.61	656.18
	Nevada		21,000	0	100,000									
32	Arizona	15,000	21,000	23,000	0	303,000	682.26	666.25	655.55	16.71 21.93 24.15		698.97	688.18	679.70
	Nevada		21,000	23,000	200,000									
33	Arizona	15,000	0	23,000	0	261,000	674.10	662.12	649.57	18.33 20.97 23.52		692.43	683.09	673.09
	Nevada		0	23,000	200,000									
34	Arizona	15,000	0	0	0	215,000	665.13	654.11	640.16	19.62 22.26 25.74		684.75	676.37	665.90
	Nevada		0	0	200,000									
35	Arizona	15,000	21,000	0	0	257,000	669.97	658.90	645.87	22.47 23.16 26.34		692.42	682.06	672.21
	Nevada		21,000	0	200,000									
36	Arizona	15,000	21,000	23,000	100,000	403,000	694.15	683.99	673.34	19.68 19.65 21.03		713.83	703.64	694.37
	Nevada		21,000	23,000	200,000									
37	Arizona	15,000	0	23,000	100,000	361,000	689.50	677.79	663.94	18.00 19.86 23.94		707.50	697.65	687.88
	Nevada		0	23,000	200,000									
38	Arizona	15,000	0	0	100,000	315,000	680.91	669.00	656.35	20.61 20.97 23.70		701.52	689.97	680.05
	Nevada		0	0	200,000									
39	Arizona	15,000	21,000	0	100,000	357,000	686.50	673.10	661.31	20.70 23.70 25.26		707.20	696.80	686.67
	Nevada		21,000	0	200,000									

TABLE V

45-D-10151



NOTES

o———o Normal tailwater.

x- - - - -x Tailwater lowered 10 feet.

△— — —△ Tailwater lowered 20 feet.

\+ Water surface at lower or downstream section lower than at upper section. Water surface at lower or downstream section higher than at upper section.

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT

BOULDER DAM

HYDRAULIC MODEL EXPERIMENTS

RIVER MODEL – SCALE RATIO 1 TO 150

EFFECT OF OUTLET WORKS ON RIVER ELEVATION

DRAWN. D.C.W. SUBMITTED J. N. Bradley

TRACED. E.F.J. O.R.S. RECOMMENDED

CHECKED. J.E.W. APPROVED.

DENVER, COLO., FEB. 24, 1936

45-D-10149

FIGURE 65

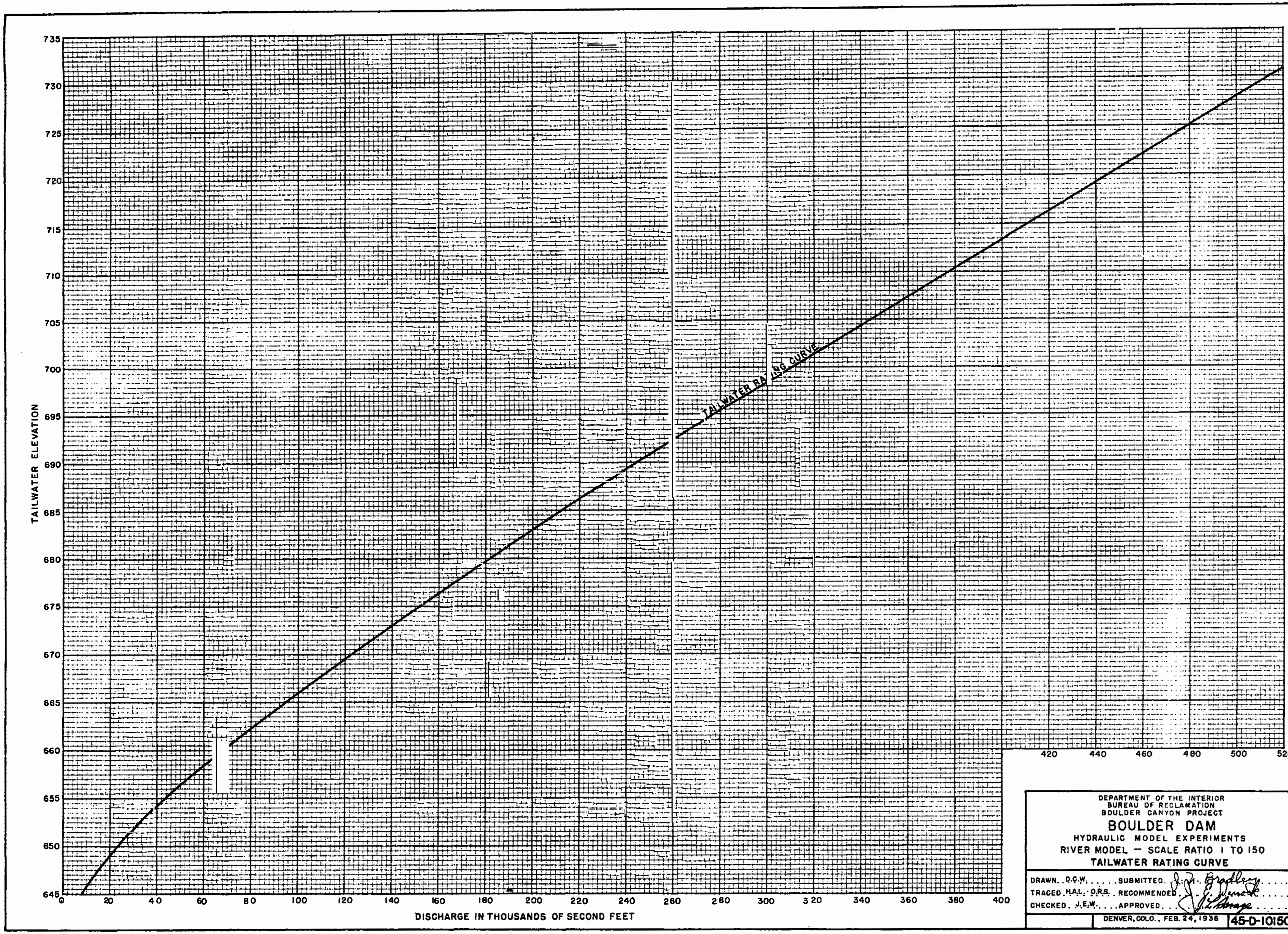

TAILWATER RATING CURVE
TAILWATER ELEVATION
735
730
725
720
715
710
705
700
695
690
685
680
675
670
665
660
655
650
645
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340
360
380
400
420
440
460
480
500
520
DISCHARGE IN THOUSANDS OF SECOND FEET
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
BOULDER DAM
HYDRAULIC MODEL EXPERIMENTS
RIVER MODEL — SCALE RATIO 1 TO 150
TAILWATER RATING CURVE
DRAWN. D.C.W. SUBMITTED. J. N. Bradley
TRACED. H.A.L. O.R.S. RECOMMENDED.
CHECKED. J.E.W. APPROVED.
DENVER, COLO., FEB. 24, 1936
45-D-10150

FIGURE 66

(3) Visual observations were made by the observer and recorded in the form of notes on each run. These deal primarily with the erosion in the river bed and with the flow conditions in the river. They have been digested and condensed for the sake of brevity and are included in this discussion as table VI. These data are self-explanatory and cross-referenced to the plates.

In analyzing the results of the point gage observations as shown on table V, they were divided into three classes:

(a) Equal inflow from both sides of river (runs 1 to 9, and 12 to 15, inclusive).

(b) Larger quantity of water from the Arizona side of the river (run 10 and runs 16 to 27, inclusive).

(c) Larger quantity of water from the Nevada side of the river (run 11 and runs 28 to 39, inclusive).

The data from the first class were plotted (fig. 65) with all three conditions of tail water; namely, normal, 10 feet below normal, and 20 feet below normal. Curves were drawn through each group. The data from the other classes were then added for comparative purposes.

In table V, the data are presented in the order in which they were actually taken, while in table VI, they have been rearranged in the order of increasing total flow in the river below the outlet works to allow comparison of the effects of a similar flow from either or both sides of the river.

D. Conclusions

It was expected, and the expectation has since been justified, that retrogression would occur such that the movable material in the river bed would be transported downstream and a new discharge-elevation relationship would result. In studying the effect of this retrogression as indicated on the 1:150 model, it was only possible, with the information available, to move the rating curve down as a unit 10 and 20 feet, respectively, preserving its original shape. Actually, the rating curve for a condition of retrogression of 20 feet may have a slightly different characteristic shape, depending upon the cross section of the nonerodible material in the river bed. Furthermore, there are two variables at work which have a tendency to counteract each other and effect the tail-water relationship curve. As the river retrogresses, the slope will decrease and the velocity in the cross sections will decrease for a given quantity.

CHANNEL CONDITIONS IN RIVER BED BELOW BOULDER DAM

Run No.	Location of Outlets	DISCHARGE SECOND-FEET Power House	Canyon Wall Valves	Tunnel Plug Valves	Spillway	Total	FLOW CONDITIONS IN RIVER BED
9	Arizona	15,000	10,500	0	0	36,000	(a) Flow in river was very quiet with three upstream valves on Nevada side and three downstream valves on Arizona side discharging. Sand was eroded where jets impinge. (b) The velocity was increased and sand bar flattened and washed farther downstream. (c) The velocity was further increased and sand washed farther downstream. Some sand was moving all along the river bed. More was moving at narrow point below the Arizona spillway exit.
	Nevada		10,500	0	0		
	PLATE 21						
10	Arizona	15,000	0	23,000	0	38,000	(a) Flow in river was smooth. A slight return flow occurs upstream from the Arizona tunnel plug stream and there was slight erosion where stream entered river bed. (b) The stream tended to flow along the Arizona bank and form a decided whirl along the Nevada bank. The sand was washed farther downstream and the erosion was greater. (c) The flow and erosion increased and sand was washed along the river from tunnel plug downstream.
	Nevada		0	0	0		
	PLATE 22						
11	Arizona	15,000	0	0	0	38,000	(a) Flow in the river was smooth. Stream from tunnel plug hits Nevada bank above the end of Nevada spillway tunnel cut. The water was deflected into the river and eroded considerably. A sand bar was formed below the Arizona spillway exit. (b) Velocity increased and more sand was piled across the river below the Arizona spillway exit. (c) Velocity increased and sand bar was flattened and washed downstream. Sand was washed from the Arizona spillway exit on down the river.
	Nevada		0	23,000	0		
	PLATE 22						
8	Arizona	15,000	21,000	0	0	57,000	(a) Flow conditions very smooth except where jets hit river. Slight erosion occured and sand piled downstream from where jets impinged. (b) Sand was spread and washed farther downstream. (c) Sand was washed down to the spillway exit on the Arizona side.
	Nevada		21,000	0	0		
6	Arizona	15,000	0	23,000	0	61,000	(a) Flow in river was very smooth, a hydraulic jump formed at the exit of the Nevada tunnel and about 75 feet downstream from exit of Arizona tunnel. Sand erosion was slight below the tunnel plug exits. Slight whirl formed upstream from junction of tunnel plug streams. Erosion occured opposite Arizona spillway exit. (b) Sand erosion increased and a sand bar was formed across the stream below Arizona spillway exit. Flow conditions were practically the same except the hydraulic jump had moved downstream slightly on the Nevada side and about 75 feet on the Arizona side. (c) The sand bar was washed downstream beyond the gaging station.
	Nevada		0	23,000	0		
	PLATE 23						

CHANNEL CONDITIONS IN RIVER BED BELOW BOULDER DAM

Run No.	Location of Outlets	DISCHARGE SECOND-FEET Power House	Canyon Wall Valves	Tunnel Plug Valves	Spillway	Total	FLOW CONDITIONS IN RIVER BED
5	Arizona	15,000	21,000	23,000	0	103,000	(a) Flow in river was very smooth. The hydraulic jump from tunnel plug was at the exit of the tunnels. The sand erosion by the tunnel plug streams was very slight. A bar of sand was thrown across the river downstream from the canyon wall jets. The sand bar started about 225 feet below the downstream valve on Arizona side and continued about 300 feet downstream.
	Nevada		21,000	23,000	0		(b) Flow conditions become a little rough and the sand bar below the canyon wall valves was flattened and moved downstream nearly to the tunnel plug exits. The tunnel plug streams eroded the river bed slightly. Hydraulic jump was just outside tunnels.
	PLATE 24						(c) The sand bar below the canyon wall valves was washed still farther downstream. The streams from the tunnel plugs eroded considerable and a sand bar was thrown across the stream about 750 feet downstream on the Nevada side and about 150 feet on the Arizona side.
26	Arizona	15,000	0	0	100,000	115,000	(a) Flow upstream from spillway exit was very smooth.
	Nevada		0	0	0		(b) Hydraulic jump was downstream from portal. Flow below spillway exits was a little rough. Sand bar was washed to a point below lower gaging station.
							(c) Velocity of power house discharge increased. Spillway stream impinged on projecting point on Nevada side. The stream was split and two whirls were formed, one on the upstream side near the center of the stream and the other on the downstream side along the Arizona shore. The sand bar below the lower gaging station was completely washed downstream.
30	Arizona	15,000	0	0	0	115,000	(a) Similar to conditions in Run 29-b.
	Nevada		0	0	100,000		(b) Similar to conditions in Run 29-c.
							(c) Flow was fairly rapid, more erosion occurred and sand was washed full width of river bed downstream from spillway exits.
27	Arizona	15,000	21,000	0	100,000	157,000	(a) Similar to conditions in Run 25-a.
	Nevada		21,000	0	0		(b) Similar to conditions in Run 25-b.
							(c) Similar to conditions in Run 25-c.
31	Arizona	15,000	21,000	0	0	157,000	(a) Similar to conditions in Run 29-a.
	Nevada		21,000	0	100,000		(b) Similar to conditions in Run 29-b.
							(c) Sand washed downstream from the canyon wall valves and formed a bar across the Nevada tunnel plug exit. The return flow on the Arizona side with the lower tailwater washed sand into the Arizona spillway exit.

CHANNEL CONDITIONS IN RIVER BED BELOW BOULDER DAM

Run No.	Location of Outlets	DISCHARGE SECOND-FEET Power House	Canyon Wall Valves	Tunnel Plug Valves	Spillway	Total	FLOW CONDITIONS IN RIVER BED
25	Arizona	15,000	0	23,000	100,000	161,000	(a) Flow conditions were very quiet and similar to Run 24-a, except flow through the tunnel plugs was more noticeable.
	Nevada		0	23,000	0		(b) Similar to conditions in Run 24-c. Hydraulic jump was outside of tunnel plug portals.
							(c) Flow from Nevada tunnel plug hit wall of cut and deflected into river. Hydraulic jump was below the tunnel exit. Flow below the spillway exits was a little rough. Sand bar was washed mostly below the lower gaging station. The stream from the Arizona spillway slipped off into the river.
29	Arizona	15,000	0	23,000	0	161,000	(a) Similar to conditions in Run 28, except the tunnel plug flow was a little more noticeable.
	Nevada		0	23,000	100,000		(b) Flow was nearly the same except more rapid. Spillway stream did not climb wall of cut as high as with normal tailwater. Hydraulic jump moved out of the tunnel plug exits.
							(c) Velocity was increased and spillway stream hits wall of cut and was deflected across the river, striking the Arizona side just below the spillway exit. Spillway stream climbed wall of cut very slightly and hydraulic jumps from tunnel plugs moved out into the stream. Sand bar was flattened and washed farther downstream.
24	Arizona	15,000	21,000	23,000	100,000	203,000	(a) Flow conditions were quiet above the spillway exits. Flow from the tunnel plugs was slightly noticeable. The stream from the Arizona spillway flowed across the river to the projecting point on the Nevada side. There was a return flow along the Nevada side above the stream and on the Arizona side below the stream. The sand bar near the lower gaging station was not quite so far downstream.
	Nevada		21,000	23,000	0		(b) Flow was a little more rough and very rapid through the tunnel plugs. Erosion was slightly more with the sand bar washed just a little more parallel with the Arizona bank.
	PLATE 25						(c) Hydraulic jumps were at the tunnel plug portals. Impingement of spillway on projection very severe.
28	Arizona	15,000	21,000	23,000	0	203,000	(a) Flow conditions were the best obtained by any combination of spillways discharging. Flow from the tunnel plugs was noticeable. Stream from Nevada spillway shot along cut and climbed wall almost to Elevation 750 but flowed into and down the center of the river. Erosion was very slight compared with other runs. There was a very slight return flow along each bank below the tunnel exits.
	Nevada		21,000	23,000	100,000		(b) Velocity and roughness was increased. Hydraulic jump was inside of the tunnel plug portals. The stream from the Nevada spillway rose higher on the wall of the cut. Erosion slightly increased with a sand bar being formed upstream from the lower gaging station.
	PLATE 26						(c) Hydraulic jump moved below tunnel plug exits. Spillway stream did not climb wall as high. Erosion was slightly increased.

CHANNEL CONDITIONS IN RIVER BED BELOW BOULDER DAM

Run No.	Location of Outlets	DISCHARGE SECOND-FEET					FLOW CONDITIONS IN RIVER BED
		Power House	Canyon Wall Valves	Tunnel Plug Valves	Spillway	Total	
14	Arizona Nevada	15,000	0 0	0 0	100,000 100,000	215,000	(a) Similar to conditions in Run 12-a. The return flow along the Nevada bank was a little more noticeable and the hydraulic jump was at the exit of each spillway tunnel. (b) The hydraulic jump moved about 75 feet from the exit of the Arizona spillway tunnel and slightly outside of the Nevada tunnel. (c) The hydraulic jump moved out into the stream on the Arizona side and about 150 feet downstream from the exit of the tunnel on the Nevada side.
22	Arizona Nevada	15,000	0 0	0 0	200,000 0	215,000	(a) Similar to conditions in Run 20-b. (b) Similar to conditions in Run 20-c. (c) Arizona spillway stream shot across the river and climbed to Elevation 750. Whirl upstream became smaller. Flow downstream was very rapid.
34	Arizona Nevada	15,000	0 0	0 0	0 200,000	215,000	(a) Similar to conditions in Run 33-b. (b) Similar to conditions in Run 33-c. (c) Flow from spillway hit wall of cut and was deflected across the river where it impinged on the Arizona bank near the spillway exit.
15	Arizona Nevada	15,000	21,000 21,000	0 0	100,000 100,000	257,000	(a) Similar to conditions in Run 12-a. Hydraulic jump was slightly inside of the spillway tunnels. Nevada tunnel plug submerged occasionally. Arizona tunnel plug exit was never completely submerged. (b) Hydraulic jump moved 75 feet downstream on the Arizona side and slightly below the tunnel exit on the Nevada side. (c) Hydraulic jump moves about 150 feet from the end of the Arizona spillway tunnel and about 75 feet downstream on the Nevada side.
23	Arizona Nevada	15,000	21,000 21,000	0 0	200,000 0	257,000	(a) Similar to conditions in Run 20-b. (b) Similar to conditions in Run 20-c. (c) Similar to conditions in Run 22-c.
35	Arizona Nevada	15,000	21,000 21,000	0 0	0 200,000	257,000	(a) Similar to conditions in Run 32-a. (b) Similar to conditions in Run 33-b. (c) Similar to conditions in Run 33-c, except the canyon wall valves wash sand along each bank and a sand bar is formed below the exit to the Nevada tunnel plug. Sand is not washed so far downstream on the Arizona side. Flow from the canyon wall valves is slightly faster on the Nevada side due to the unbalanced flow from the spillways.
21	Arizona Nevada	15,000	0 0	23,000 23,000	200,000 0	261,000	(a) Similar to conditions in Run 20-a. (b) Similar to conditions in Run 20-b. (c) Similar to conditions in Run 20-c, except flow through the tunnel plugs is more noticeable.

CHANNEL CONDITIONS IN RIVER BED BELOW BOULDER DAM

Run No.	Location of Outlets	DISCHARGE SECOND-FEET Power House	Canyon Wall Valves	Tunnel Plug Valves	Spillway	Total	FLOW CONDITIONS IN RIVER BED
13	Arizona	15,000	0	23,000	100,000	261,000	(a) Similar to conditions in Run 12-a. Flow from tunnel plug exits was noticeable.
	Nevada		0	23,000	100,000		(b) Similar to conditions in Run 12-b. Hydraulic jump is slightly outside of the Nevada spillway tunnel and about 50 feet below the Arizona spillway exit.
							(c) Hydraulic jumps move out of all the tunnels. Flow is quite rapid from spillway exits on down the river. The hydraulic jump from the Arizona spillway moved out into the river.
33	Arizona	15,000	0	23,000	0	261,000	(a) Similar to conditions in Run 32-a, except the flow from the tunnel plugs was more noticeable because of slightly lower tailwater.
	Nevada		0	23,000	200,000		(b) Hydraulic jump is at the exit of the Arizona tunnel plug and slightly inside the tunnel on the Nevada tunnel plug portal.
							(c) Hydraulic jump from tunnel plug moved almost into the river. Stream from Nevada spillway did not climb as high on the wall of the cut. Sand was washed the entire width of the river and downstream into the tailwater regulator box. A more decided whirl was formed on the upstream side of the spillway jet.
12	Arizona	15,000	21,000	23,000	100,000	303,000	(a) Flow above spillway streams was very quiet. Spillway streams meet in center of river and directly out from Arizona exit. Streams from Nevada side climb wall of cut to about Elevation 750. Whirl is noticeable on Nevada side above the point where Arizona stream hits Nevada wall. A sand bar was thrown across the river. There was considerable erosion where streams meet, more so along the Nevada wall. Tunnel plugs were completely submerged.
	Nevada		21,000	23,000	100,000		(b) Sand is washed farther downstream and more parallel to Arizona bank. Hydraulic jump was inside Nevada tunnel and just outside Arizona tunnel. Tunnel plugs still submerged.
	PLATE 27						(c) Sand bar was washed farther downstream. Flow from tunnel plugs was noticeable. Upper end of sand bar remained about the same.
20	Arizona	15,000	21,000	23,000	200,000	303,000	(a) Flow above spillway exits was very smooth. The stream from the Arizona spillway shot across the river, impinged on the projecting point on the Nevada side and a large return flow formed which eroded at the lower end of the Nevada spillway cut. Considerable erosion occurred under the spillway stream. A bar of sand was thrown across to the Nevada side about 600 feet downstream. There was a return flow downstream from spillway stream and on the Arizona side.
	Nevada		21,000	23,000	0		(b) Flow from tunnel plugs was slightly noticeable.
	PLATE 28						(c) Erosion was greater and the sand bar near the lower gaging station was flattened. Tunnel plug flows were slightly noticeable.

CHANNEL CONDITIONS IN RIVER BED BELOW BOULDER DAM

Run No.	Location of Outlets	DISCHARGE SECOND-FEET Power House	Canyon Wall Valves	Tunnel Plug Valves	Spillway	Total	FLOW CONDITIONS IN RIVER BED
32	Arizona Nevada	15,000	21,000 21,000	23,000 23,000	0 200,000	303,000	(a) Flow conditions were as satisfactory as in Run 28, being very smooth above the spillway stream. Flow from the tunnel plugs was slightly noticeable. Stream from spillway climbed wall to Elevation 750 and was deflected into the river but flowed more along the Nevada bank than in Run 28. A whirl was formed on the upstream side of the spillway stream opposite the Arizona spillway exit. There was considerable erosion and a sand bar was formed clear across, starting directly opposite the Arizona spillway exit and continuing downstream and across to the Nevada side about 300 feet below the gaging station. A small amount of sand was washed in the exit of the Arizona spillway tunnel. (b) Not much change in flow conditions, except for being a little more rapid and rough. Flow from the tunnel plugs was not noticeable. Sand was washed farther downstream. (c) Hydraulic jump was in the exit of the tunnel plugs. Spillway stream did not climb so high on the wall of the cut. Flow was much more rapid and rough. Sand was washed the entire width of the river and almost into the tailwater control box.
	PLATE 29						
18	Arizona Nevada	15,000	0 0	0 0	200,000 100,000	315,000	(a) Flow above the spillway exits was very quiet. A whirl upstream from spillway streams piled sand in front of the tunnel plug exit. (b) Arizona spillway stream tended to shoot across on river water surface and climb the projecting point on the Nevada side. The sand bar above the spillway streams was washed a little farther upstream. (c) Stream from Nevada spillway climbed wall of cut higher than at normal tailwater or 10 feet below. Sand bar above streams was washed nearly to exit of Arizona tunnel plug outlet.
38	Arizona Nevada	15,000	0 0	0 0	100,000 200,000	315,000	(a) Similar to conditions in Run 37-b. (b) Similar to conditions in Run 37-c. (c) Nevada spillway stream hit wall of cut and was deflected into the river. It shot into the river under the Arizona stream and allowed the Arizona stream to shoot across and climb the projecting point on the Nevada side. Flow was more rapid along the Nevada bank. The erosion was increased and sand was carried downstream.
19	Arizona Nevada	15,000	21,000 21,000	0 0	200,000 100,000	357,000	(a) Similar to conditions in Run 16-a. (b) Similar to conditions in Run 16-b. (c) Similar to conditions in Run 16-c.
39	Arizona Nevada	15,000	21,000 21,000	0 0	100,000 200,000	357,000	(a) Similar to conditions in Run 37-a. (b) Similar to conditions in Run 37-b. (c) Similar to conditions in Run 37-c.

CHANNEL CONDITIONS IN RIVER BED BELOW BOULDER DAM

Run No.	Location of Outlets	DISCHARGE SECOND-FEET Power House	Canyon Wall Valves	Tunnel Plug Valves	Spillway	Total	FLOW CONDITIONS IN RIVER BED
17	Arizona	15,000	0	23,000	200,000	361,000	(a) Similar to conditions in Run 16-a. (b) Similar to conditions in Run 16-b. (c) The flow becomes very rough and the hydraulic jump from the Arizona spillway was in midstream. The stream seems to shoot across the river on top of the water surface. Hydraulic jump on the Nevada spillway was about 50 feet downstream from the exit. The flow from tunnel plugs was very noticeable and the jump was at the exit of the tunnels. Sand piled downstream from the Arizona tunnel plug exit was washed nearer the center of the stream.
	Nevada		0	23,000	100,000		
37	Arizona	15,000	0	23,000	100,000	361,000	(a) Similar to conditions in Run 36-b. (b) Flow from tunnel plugs became noticeable. Spillway jumps were outside of tunnel exits. (c) The sand bar was washed farther downstream and flattened. Hydraulic jump was at the tunnel plug portals.
	Nevada		0	23,000	200,000		
16	Arizona	15,000	21,000	23,000	200,000	403,000	(a) Flow above the spillway exits was very quiet. The velocity of the Nevada spillway stream was greatly reduced before it reached the stream from the Arizona spillway which allowed the Arizona stream to shoot across the river and climb the projection on the Nevada side. Part of the stream flows upstream causing a whirl on the Nevada side. The flow was mostly along the Nevada side below the spillway exits. Tunnel plugs were completely submerged. Erosion at canyon wall valves was very slight. (b) Very little change in flow conditions or erosion. Hydraulic jump outside the Arizona tunnel and inside the Nevada tunnel. Tunnel plugs still submerged. (c) The tunnel plug flow was noticeable. Erosion downstream was increased. Arizona spillway hydraulic jump was in the river. The Nevada spillway jump was at the tunnel exit.
	Nevada		21,000	23,000	100,000		
	PLATE 30						
36	Arizona	15,000	21,000	23,000	100,000	403,000	(a) Flow was fairly smooth. Arizona spillway stream did not shoot across the river and climb the projecting point on the Nevada side. Streams from the spillways met in the center of the river and continued down the river with the water along the Nevada bank having a slightly higher velocity. Tunnel plugs were completely submerged. The erosion was to rock in center of river bed opposite the Arizona spillway exit. Sand was piled upstream from junction of spillway streams. The sand bar below was nearly straight across the stream with the Nevada side extended a little farther downstream. (b) The junction of the spillway streams was such that the flow is more evenly distributed through the width of the river. More erosion occurred downstream and the sand bar was carried to about 300 feet below the gaging station where it extended the full width of the stream. (c) Flow from the tunnel plugs becomes noticeable. More erosion occurred downstream similar to (b).
	Nevada		21,000	23,000	200,000		
	PLATE 31						

CHANNEL CONDITIONS IN RIVER BED BELOW BOULDER DAM

Run No.	Location of Outlets	DISCHARGE SECOND-FEET: Power House	Canyon Wall Valves	Tunnel Plug Valves	Spillway	Total	FLOW CONDITIONS IN RIVER BED
3	Arizona	15,000	0	0	200,000	415,000	(a) Water above tunnel exits was smooth with exception of whirl immediately above spillway streams. Hydraulic jump was formed at exit of Arizona tunnel. Exit of the Nevada tunnel was covered and jump was inside tunnel. Streams met at a point in center of river directly opposite Arizona spillway exit. Some of the velocity of Nevada stream was dissipated before it reached the Arizona stream allowing the Arizona stream to climb point on Nevada side. (b) The hydraulic jump moved to the exit of the Nevada spillway tunnel and downstream from the Arizona tunnel. The return flow on the Nevada side and upstream from the spillway streams increased. (c) Velocity and flow increased.
	Nevada		0	0	200,000		
4	Arizona	15,000	21,000	0	200,000	457,000	(a) Tunnel plug exits were completely submerged. Considerable disturbance occurred at the spillway tunnel exits caused by entrained air escaping. The stream from the Nevada spillway hit the canyon wall a short distance below the tunnel exit, climbed above Elevation 750 and followed the canyon wall. The stream from the Arizona spillway shot across the river, met the Nevada stream and impinged on the projection on the Nevada side. A decided whirl occurred on the Arizona side downstream from the streams and directly across from the projection. The flow from the canyon wall valves caused a slight disturbance above the tunnel plug portals. (b) Roughness below spillway tunnels increased, and a decided whirl appeared upstream from point at which Arizona spillway stream struck Nevada bank. Streams met nearer center of river. Hydraulic jumps formed slightly above spillway tunnel portals. (c) Hydraulic jump in Nevada spillway tunnel was at portal and 75 feet below Arizona exit.
	Nevada		21,000	0	200,000		
2	Arizona	15,000	0	23,000	200,000	461,000	(a) Similar to conditions in Run 4-a. (b) Similar to conditions in Run 4-b. Flow from tunnel plugs was not noticeable. (c) Similar to conditions in Run 4-c. Flow from tunnel plugs was noticeable.
	Nevada		0	23,000	200,000		
1	Arizona	15,000	21,000	23,000	200,000	503,000	(a) Water upstream from spillway tunnels was fairly smooth except for slight disturbance from canyon wall outlet jets. A slight whirl formed upstream from spillway streams. Tunnel plug outlets were completely submerged. Surging and bubbling occurred at exit of each spillway tunnel. Sand was washed completely away from cut on Nevada side to a point 900 feet downstream. A sand bar was formed across river bed above spillway tunnels and below tunnel plugs. A large bar of sand was formed across river starting above the gaging station on Arizona side and ending on Nevada side about 900 feet downstream. A small sand bar was formed across the river below the canyon wall valves. (b) Sand erosion occured more quickly and flow below spillway tunnels was rougher. Tunnel plug exits were still submerged. (c) Flow condition still rougher.
	Nevada		21,000	23,000	200,000		

PLATE 32

On the other hand, as the river retrogresses, the cross-sectional area will decrease with a resultant influence on the relationship.

As a result of these studies, while they can only be considered as qualitative or indicative trends, it was definitely shown that there are operating combinations of the various outlet works which are far superior from the standpoint of increasing the effective head on the turbines when the load reaches a point where power is at a premium, and superior from the standpoint of flow conditions in the channel, particularly below the portals of the spillway tunnels.

Since the results obtained from these studies can only be considered as relative and indicative and since the retrogression of the river bed as a variable makes absolute results on models extremely difficult to secure, it is believed desirable that sufficient gaging stations be established in positions downstream from the powerhouse and the spillway outlets to permit data to be collected, compiled, and analyzed as a definite aid to the power plant operating engineer in determining the most effective operating combination.

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